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Flood Analysis of the Clare River Catchment Considering Traditional Factors and Climate Change

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SUPERVISOR

Sean Moloney

Submitted to the Galway Mayo Institute of Technology,
September 2010

DECLARATION OF ORIGINALITY

September 2010

The substance of this thesis is the original work of the author and due reference and acknowledgement has been made, when necessary, to the work of others. No part of this thesis has been accepted for any degree and is not concurrently submitted for any other award. I declare that this thesis is my original work except where otherwise stated.

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Date: 17-09-10

Abstract

The main objective of this thesis on flooding was to produce a detailed report on flooding with specific reference to the Clare River catchment. Past flooding in the Clare River catchment was assessed with specific reference to the November 2009 flood event. A Geographic Information System was used to produce a graphical representation of the spatial distribution of the November 2009 flood. Flood risk is prominent within the Clare River catchment especially in the region of Claregalway. The recent flooding events of November 2009 produced significant fluvial flooding from the Clare River. This resulted in considerable flood damage to property. There were also hidden costs such as the economic impact of the closing of the N17 until floodwater subsided.

Land use and channel conditions are traditional factors that have long been recognised for their effect on flooding processes. These factors were examined in the context of the Clare River catchment to determine if they had any significant effect on flood flows. Climate change has become recognised as a factor that may produce more significant and frequent flood events in the future. Many experts feel that climate change will result in an increase in the intensity and duration of rainfall in western Ireland. This would have significant implications for the Clare River catchment, which is already vulnerable to flooding.

Flood estimation techniques are a key aspect in understanding and preparing for flood events. This study uses methods based on the statistical analysis of recorded data and methods based on a design rainstorm and rainfall-runoff model to estimate flood flows. These provide a mathematical basis to evaluate the impacts of various factors on flooding and also to generate practical design floods, which can be used in the design of flood relief measures.

The final element of the thesis includes the author's recommendations on how flood risk management techniques can reduce existing flood risk in the Clare River catchment. Future implications to flood risk due to factors such as climate change and poor planning practices are also considered.

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The Office of Public Works, especially, Mr. Tim Pasley, for providing hydrometric data and significant knowledge on the Clare River catchment.

My family for their continued encouragement and support.

Glossary

AEP	Annual Exceedance Probability
AOGCM	Atmospheric and Oceanic Global Climate Model
APSR	Areas with Potentially Significant Risk
AR4	IPCC's Fourth Assessment Report on Climate Change
CFRAMS	Catchment Flood Risk Assessment and Management Study
CO_2	Carbon Dioxide
DoEHLG	Department of Environment, Heritage and Local Government
EIA	Environmental Impact Assessment
FRA	Flood Risk Assessment
FSR	Flood Studies Report
GCM	Global Climate Model
GHG	Greenhouse Gas
GIS	Geographic Information System
GSI	Geographical Survey Ireland
HEFS	High-End Future Scenario
IPCC	Intergovernmental Panel on Climate Change
LAP	Local Area Plan
MRFS	Mid-Range Scenario
M-5	M5 rainfall is the rainfall depth with a return period of 5-years
mAod	metres Above ordnance datum. This will be taken as mAod Malin for the purpose of this report unless otherwise stated
NSS	National Spatial Strategy
OPW	Office of Public Works
PFRA	Preliminary Flood Risk Assessment
ppb	parts per billion
ppm	parts per million
PSFRM	Planning System and Flood Risk Management Guidelines
RFRA	draft Regional Flood Risk Appraisal for the west region
RPG	draft Regional Planning Guidelines for the west region
SAC	Special Area of Conservation
SEA	Strategic Environmental Assessment
SFRA	Strategic Flood Risk Assessment

s.m.d.	Soil moisture deficit
SRES	Special Report on Emissions Scenario
SUDS	Sustainable Urban Drainage System
TAR	IPCC's Third Assessment Report on climate change
utc	coordinated universal time
WRFB	Western Regional Fisheries Board

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Chapter 1

Introduction

There has been an increased interest in the interaction between people and their environment. People are becoming more aware of the influence they have on the environment and the effects it can have on them. This observation is particularly prevalent to flood risk. Advances in flood management in the mid-20th century have resulted in improvements in the manner in which flood risk is managed. The introduction of the Arterial Drainage Act in 1945 provided the legal format that enabled the OPW to undertake catchment wide flood alleviation works that were aimed at reducing flooding of agricultural land. Ireland's landscape has changed significantly in the past 50 years. Ireland's population has benefited from a period of economic growth that has seen an economy focused primarily on agriculture begin to diversify into many different sectors of business. This has brought with it population growth, increased wealth and an increase in flood risk due to a poorly informed approach to planning. The Amendment Act of 1995 recognised the need for increased consideration of flood risk due to the increased potential for flood damage.

Flood risk is an issue that is particularly relevant to the west of Ireland due to its wet climate. There have been significant flood events in recent times that have resulted in considerable flood damage. Flood damage can take the form of direct economic damage (e.g. property), indirect economic damage (traffic disruption) or intangible damages (e.g. stress to owner of flooded property). The socio-economic implications of flooding are considerable. Galway has not escaped such implication with significant flooding occurring in areas such as Gort in south Galway. The profile of flood risk management at national level gained significant importance subsequent to the flood events of November 2009, which produced flooding throughout the county. East county Galway was particularly affected with significant flood damage incurred in areas such as Ballinasloe, Gort and Ardrahan. The flooding was not just confined to Galway. The effects of the November 2009 floods were experienced in Cork when a decision by the ESB to release floodwaters from Iniscarra dam produced considerable damage in Cork city. England also experienced considerable flooding due to the intensity of rainfall that was experienced at the time. These events highlight the importance of adequate consideration of the effects of flooding and understanding of the potential causes of flood events. A clear understanding of the reasons behind flooding can greatly assist planning and implementation of flood relief measures. It is essential that flood risk management adopt a proactive approach. Many towns throughout Ireland have to experience significant flood damage before necessary action is taken.

This report will look at the different aspects of flooding from the perspective of the Clare River catchment. This is located in east Galway and did not escape the effects of the November 2009 flooding. Claregalway is located towards the outfall of the catchment and experienced some of the worst flooding as a result. Other areas such as Tuam and a significant amount of agricultural land in the vicinity of Corofin were also inundated by floodwaters. This report will identify the characteristics of the catchment that are relevant to the drainage pattern. This includes the topography, hydrology and geology of the catchment. An analysis of historical flood events will be provided to help to put the severity of the 2009 flooding in context. This analysis will include the 2009 event for which a flood extent map will be generated using GIS software. This will provide a visual representation of the spatial distribution of the floods experienced.

A statistical analysis will be carried out on hydrometric data obtained for the Clare River. The objectives of this analysis will be to identify the frequency and magnitude of events along the Clare River. It will also provide an indication of the frequency and magnitude of historical flood events up to and including November 2009. Analysis of this kind will help to identify if the magnitude of recent events is considered extreme in a climatic context. It will also identify design flows upon which future flood risk consideration should be based.

The report will analyse the projected impact of climate change in relation to increased flood flows. This is of particular importance in ensuring that present day decisions provide adequate protection for a sufficient length of time. These allowances should be incorporated into design flows identified by the frequency analysis stage to produce a scenario that provides an adequate allowance or adaptation capacity to react to future increases in flood magnitude. The report will also identify the role that rainfall took in historical flooding within the Clare River catchment and if there has been any significant increase in the duration or severity of rainfall in recent times that would support the opinion of climate change advocates.

Land Use can play a significant role at a number of stages in the flooding process. This may be due to a change in agricultural practices or increased development. The impacts at each stage of the flooding process will be discussed. This will look at implications of land use change on flood risk and also look at the impact of flooding on such land use changes. Flood risk is a function of flood magnitude and potential flood damage. Therefore an increase in flood risk can be just as significant due to an increase in potential flood damage. The potential implications to the Clare River catchment will also be analysed. While consideration will be given to changes in land use practices it is felt that the most significant effect on flood risk within the catchment would be as a result of improper planning of development. The report will evaluate current zoning and planning practices and also analyse the potential implications of the urban fraction on the synthetic flood hydrograph for the Clare River catchment at its outfall in the vicinity of Claregalway.

Channel conditions can also take a significant role in defining floodwater levels. The implications of channel conditions on the different processes that contribute to the net effect of flood damage will be discussed. These potential implications will then be applied to the Clare River catchment to determine if the condition of the Clare River channel has made any contribution to fluvial flooding. Due to the potential influence of high water levels in Lough Corrib contributing to flooding in the lower reaches of the Clare River this will also be assessed.

The report will also evaluate potential flood risk management procedures and works that could provide a reduction in flood risk. The benefit of these measures in relation to the flooding problems experienced in the Clare River catchment will be considered and a list of potential actions will be proposed to reduce flood risk to an acceptable level.

The purpose of this report is to gain a greater understanding of flooding mechanisms and the impacts of flood risk. It also aims to assess whether current conditions and practices are sufficient to cope with the flood risk within the Clare River catchment and to make suggestion as to how flood risk can be more efficiently managed. There are a number of key objectives of this report which are outlined below:

- Provide a comprehensive report on flood mechanisms and the implications of improper consideration of flood risk

- Generate useful data that has a practical application to flood risk management through the application of statistical analysis techniques to relevant hydrometric data
- Generate an indicative floodplain map for the Clare River that will provide a visual representation of the spatial extent of flooding and can be applied to decision-making that requires consideration of flood risk
- Identify the effect of climate change on future flood risk
- Identify the impact that land use within the Clare River catchment has on flood flows
- Provide an indication of the influence of the existing urban fraction on the synthetic flood hydrograph using a method which converts the design rainfall into a design flood to provide a comparison between existing conditions and a 'no development' scenario
- Identify the impact that surface water management within the Clare River catchment has on flood flows
- Identify whether current methods are sufficient to manage flood risk effectively within the Clare River catchment
- Evaluate potential flood relief measures and potential constraints to their application
- Provide a list of potential flood alleviation measures that should be considered when addressing flood risk within the Clare River catchment

Chapter 2

Clare River

Catchment Characteristics

2.1 Topography and Hydrology of Catchment

The River Clare is a major tributary of the River Corrib. The Dalgan and Sinking River combine to form the main Clare River channel. Tributaries, such as the Grange and the Abbert join the Clare River to form the Clare River Drainage District. The Clare River catchment is situated in the eastern part of the Corrib catchment. It has an area of approximately 1,078 km^2. This equates to approximately 30% of the Corrib catchment area, which covers an area of 3,056 km^2. The catchment is bound by the Suck catchment to the east and the Moy catchment to the north. The Dunkellin/Craughwell River and Clarinbridge River catchments are adjacent to the south. It is bound to the west by other tributaries of the Corrib River such as the Cregg and Black River which discharge farther up Lough Corrib and the Robe River that flows to Lough Mask.

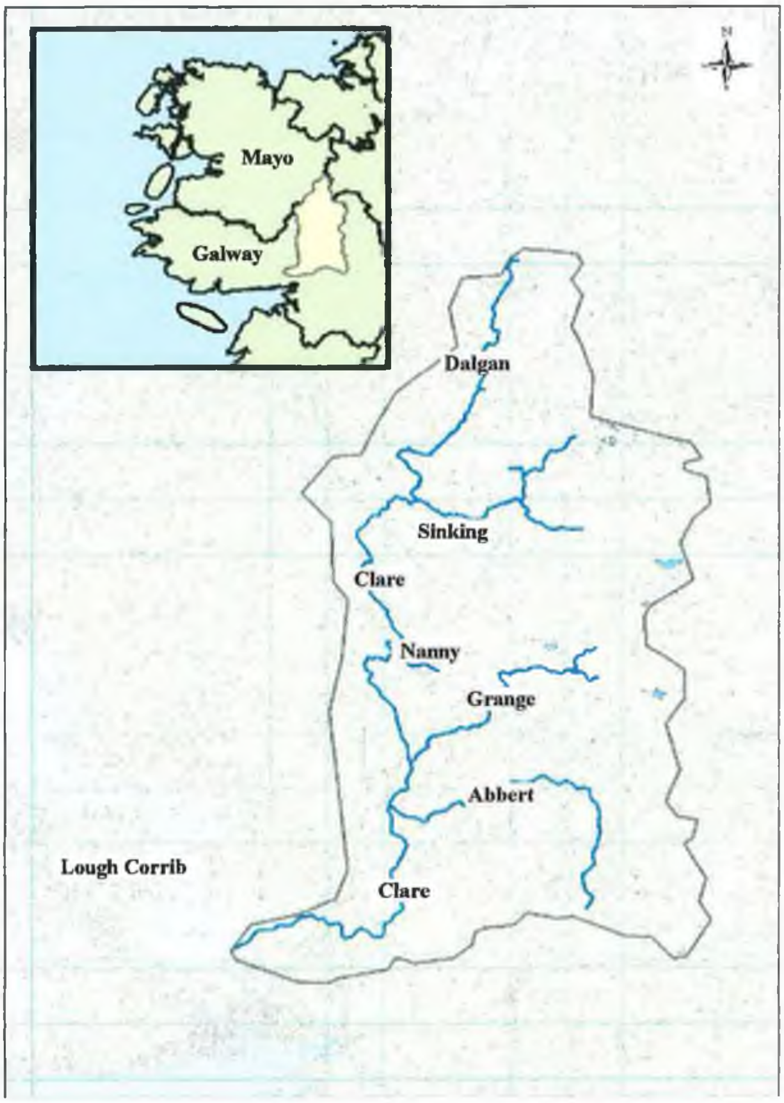

Figure 2.1 – Clare River Catchment with Watercourses Labelled

The topography of the Clare River catchment is predominantly even. The most hilly ground within the catchment is located just southeast of Ballyhaunis and varies between 120 mAod and 160 mAod. This high ground forms the border between the catchment and that of the River Suck. The Clare River is located close to the western boundary of the catchment with the majority of the land drained from the east. The eastern boundary of the catchment is predominantly in the region of 100 mAod falling gradually in the direction of the Clare River. Large networks of tributaries service the flow of water across the catchment (Appendix A–1). The most significant watercourses draining the central and eastern portion of the catchment are the Sinking, Nanny, Grange and Abbert River. The ground level in the upper portion of the catchment is approximately 100 mAod. The catchment is predominantly even with gentle slopes. Ground levels are in the region of 60 mAod in the east of the catchment and 50 mAod in the west of the catchment. Ground level is at its lowest (approximately 10 mAod to 20 mAod) in the southwest corner of the catchment just above where the river discharges to Lough Corrib.

Figure 2.2 – Dalgan River upriver from Ballyhaunis

Figure 2.3 – Sinking River upriver from confluence with Dalgan River

The Clare River system begins as the Dalgan River (Figure 2.2). The source of the river is located north of Ballyhaunis in an area approximately 100 mAod. The river is approximately 3.5 m wide upstream of Ballyhaunis with low dry weather flows. The Clare River system drains a number of population centres over its course. The first notable population centre through which it passes is Ballyhaunis. Gradients are at their greatest in the upper reaches of the Clare River system. The gradient in the upper reaches is still reasonably shallow at approximately 1/1300 to 1/900. The Dalgan River flows in a southerly direction through Ballyhaunis and increases in width to approximately 8 m before joining the Sinking River (Figure 2.3) to form the Clare River north of Milltown. Below the confluence of these two rivers the main channel width becomes more pronounced and is typically 10 m to 12 m. The channel depth in the mid section of the Clare River was designed to cope with approximately 2.9 m flow depth as per the 1 in 3 year design carried out in the 1950's. The channel depth is much greater than this in some areas due to new cuts through high ground that were required to improve the conveyance of water through the catchment. From Milltown the river flows on to Tuam where the River Nanny discharges into the main channel just outside the town. The gradient of the main channel can be as low as 1/3000 in some places. The gradient of the river from Tuam to Lough Corrib is most often in the region of 1/1200 although the gradient becomes shallower as it nears Lough Corrib. This shallow gradient is due to the even topography of the catchment. The Grange River (Figure 2.4) discharges to the main channel approximately 1.5 km upriver of Corofin before flowing on to be joined by the Abbert River (Figure 2.5) approximately 2 km south of Corofin. 9 km south of the confluence of the Clare River and the Abbert River the main channel turns to flow westerly through Claregalway (Figure 2.6). Downriver of the Headford Rd (N84) crossing the channel widens to approximately 30 m (Figure 2.7). This lower reach of the river has an extremely low gradient with surface water levels similar to those recorded on Lough Corrib in times of steady flow. The Clare River discharges to Lough Corrib approximately 10 km north of Galway city.

Figure 2.4 – Grange River Looking Downriver from R347
(approx. 2.6km Upriver from Confluence with Clare River)

Figure 2.5 – Abbert River Looking Upriver from Bridge near Bullaun (approx. 2.5km Upriver from Confluence with Clare River)

Figure 2.6 – Clare River Looking Downriver from Claregalway Bridge

Figure 2.7 – Clare River Looking Downriver from N84 (Headford Rd.) Bridge

2.2 Geology and Hydrogeology of the Catchment

Figure 2.8 shows the bedrock formations present in the Clare River Catchment. The vast majority of the catchment is underlain by undifferentiated Visean Limestones. The formations that make up the remainder of the catchment are included in table 2.1. The catchments bedrock is composed primarily of Dinantian Pure Bedded Limestone's. This is equivalent to Burren Limestone, which is pale grey, clean, medium to coarse-grained, bedded limestone. There are few faults mapped in the area. This is due to the lack of any major variation in the rock lithology [1].

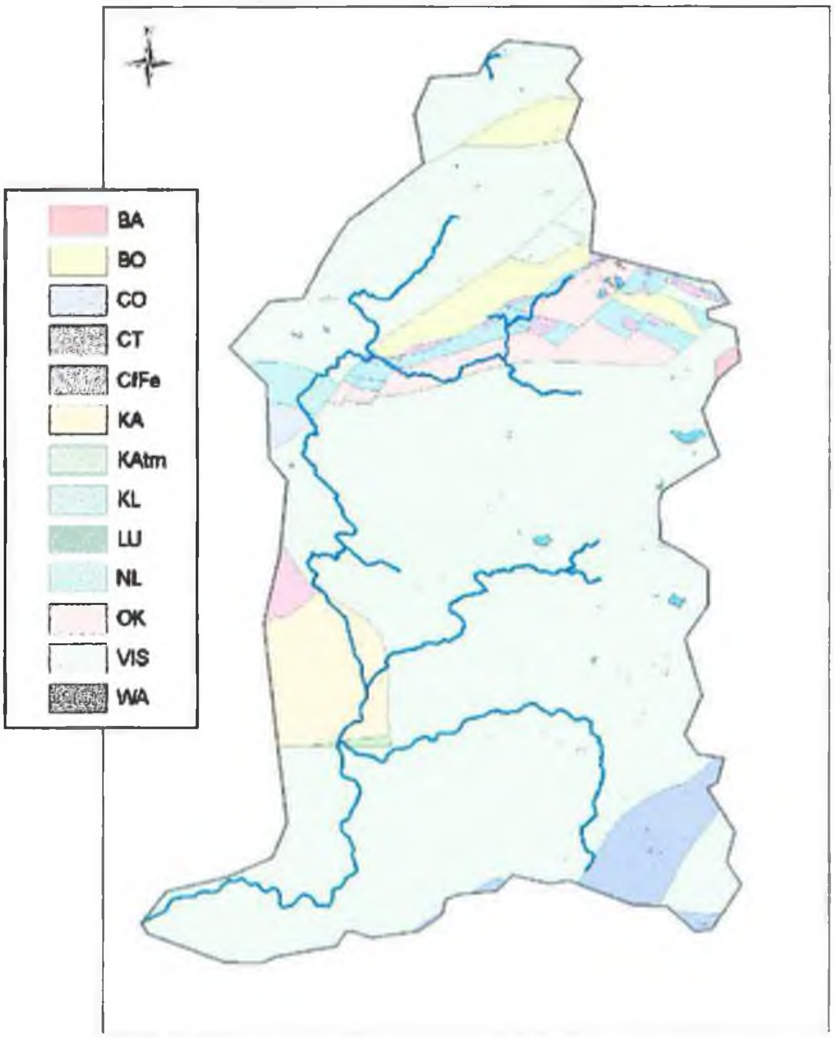

Figure 2.8 – Bedrock Formations in Clare River Catchment

Table 2.1 – Description of Bedrock Units in Catchment

Code	Rock Unit Name	Description
BA	Ballysteen Formation	Dark Muddy Limestone, Shale
BO	Boyle Sandstone Formation	Sandstone, Siltstone, Black Mudstone
CO	Cong Limestone Formation	Thick Bedded Pure Limestone
CT	Coranellistrum Formation	Medium to Thick-Bedded Pure Limestone
CfFe	Caledonian Cloonfad Felsite	Felsite
KA	Knockmaa Formation	Thick Bedded Pure Limestone
Katm	Two Mile Ditch Member	Thick-Bedded Limestone Clay Wayboards
KL	Kilbryan Limestone Formation	Dark Nodular Calcarenite & Shale
LU	Lucan Formation	Dark Limeston & Shale ('calp)
NL	Cong Canal Formation	Medium to Thick-Bedded Pure Limestone
OK	Oakport Limestone Formation	Pale Grey Massive Limestone
VIS	Visean Limestones (undifferentiated)	Undiffeerentiated Limestone
WA	Waulsortian Limestones	Massive Unbedded Lime-Mudstone

Pure bedded limestone is susceptible to karstification. This is the process whereby fissures and cracks in the rock are widened due to the rock being dissolved by mildly acidic rain. As rain passes through the atmosphere it absorbs carbon dioxide (CO_2) forming carbonic acid, ($H_2O + CO_2 = H_2CO_3$). Sulphuric acid and hydrosulfuric acid may also contribute to karstification. The rain permeates the soil layer absorbing further CO_2. The water permeates the bedrock through fissures and bedding planes. The weak acidic solution dissolves the calcium carbonate present in the limestone. The fractures in the rock enlarge over time significantly enhancing the permeability of the rocks. A large number of karst features are present throughout the catchment highlighting the karstified nature of the bedrock. This is also realised by the layout of the Clare River system in the 1700's prior to arterial drainage works. The river system ended at Turloughmore and was connected to Lough Corrib via underground flows (see section 7.2). Karstified catchments can experience flooding arising from insufficient capacity and collapse of these underground channels. It was therefore beneficial to link the drainage network with Lough Corrib via a surface water channel.

The bedrock is generally over 100m thick [1]. Groundwater flows in an epikarstic layer a few metres thick. This is an upper layer of karstified carbonate rock situated in the unsaturated zone just below the soil layer. The groundwater also extends approximately 30 m below this layer in a zone of interconnected fissures and conduits that have been enlarged due to chemical erosion of the rock by the groundwater [1]. The karstified nature of the catchment facilitates movement of groundwater. The vast majority of the catchment is a regionally important karstified aquifer (Rkc) as shown in figure 2.9. It is most probably dominated by conduit flow due to the rapid groundwater velocities present. A conduit is an underground stream completely filled with water and under hydrostatic pressure. The remainder of the catchment consists of mainly locally important aquifer (LI).

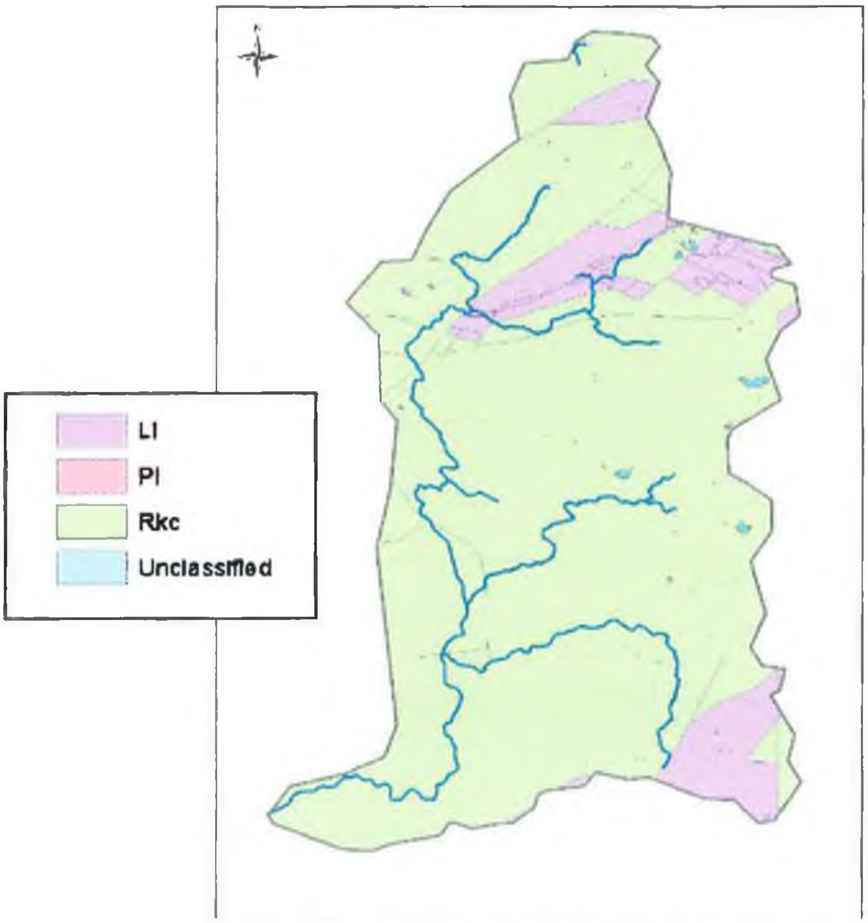

Figure 2.9 – Aquifers in Clare River Catchment

Large springs such as Ballyhaunis WSS (1200 m^3/day) and Barnaderg Group Scheme (5000 m^3/day) indicate the large amount of groundwater available in the catchment [1]. Water tables produce high annual variations. There is also fluctuation in spring flows as they respond quickly to rainfall events. This indicates a low level of storativity. A number of tracer tests carried out by the GSI indicate variability's in groundwater movements. It was found that the catchment displayed anisotropy in the transmissivity of groundwater. A higher east-west transmissivity was observed with groundwater velocities between 100 and 450 m/hr. North-south velocities were considerably lower in the region of 6 to 35 m/hr [1]. The direction of groundwater flow is predominantly in a southwesterly direction similar to surface water flow. All groundwater from the catchment discharges to Lough Corrib. Groundwater flow through karst is complex and difficult to predict due to flow paths being determined by established fissures and conduits in the rock. Tracer test data indicates that the pathway taken by the groundwater in reaching Lough Corrib may involve traversing catchment boundaries. Water sinking at Ballyglunin Cave in the Abbert River catchment emerges at Auclogeen Spring near the source of the Cregg River to the west [1]. This is a distance of approximately 10 km and involves passing beneath the Clare River. This groundwater moves at a velocity of 200 m/hr. Water sinking along a losing stretch of the Sinking River re-emerges to join the Clare River. Water sinking along a losing stretch of the Clare River re-emerges at the source of the Black River. These observations demonstrate the large degree of interconnection between surface and groundwater within the catchment. Factors such as this are expected to contribute to the diminished flows that are experienced at Claregalway based on expected flows from hydrometric data at Corofin. Groundwater is also expected to be entering the catchment from the Shannon river basin district [1]. The effect of this on flooding is not investigated further due to the difficulty associated with estimating groundwater flows.

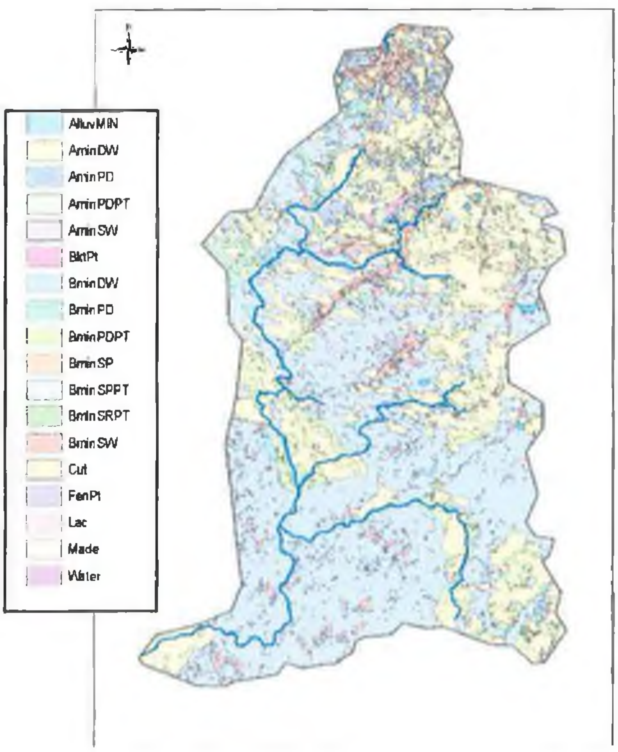

Figure 2.10 – Soil in Catchment

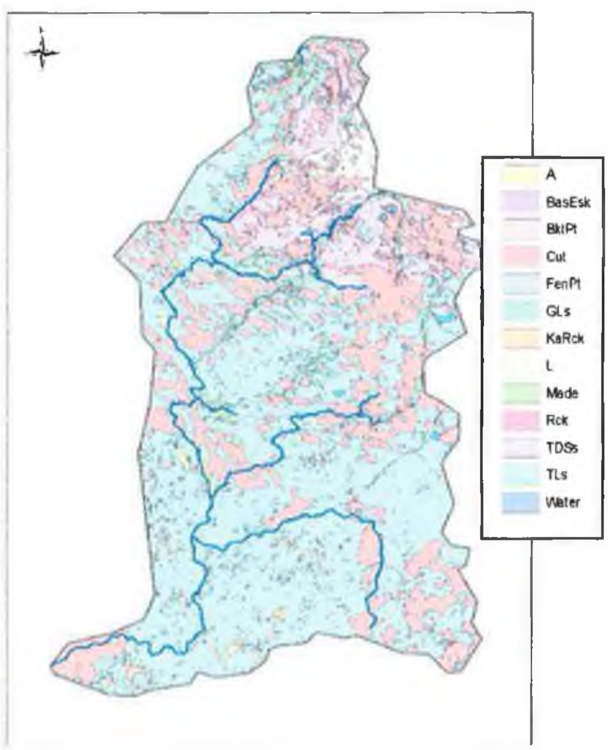

Figure 2.11 – Subsoil in Catchment

The vast majority of the subsoil in the region is limestone till, carboniferous (TLs) and cutover peat (Cut). These have a variable and peaty texture respectively. Soil consists of predominantly BminDW, which is derived from mainly basic plant materials and is a well-drained soil. Cutaway raised bog (Cut) is also a common soil type in the catchment and is poorly drained. Areas of till are considered to exhibit moderate to good permeability while areas consisting of peat are poor draining. There is a general increase in subsoil thickness from west to east with depth to bedrock increasing from 4 m to 9 m [1]. Surface rock outcrops are generally confined to areas closer to Lough Corrib.

The pure limestone bedrock results in the catchment being underlain by a karstified aquifer. Due to anisotropy in the transmissivity of this groundwater body higher velocities are observed along the east west axes in a westerly direction. The permeable nature of a significant portion of soil in the catchment along with the karstified bedrock provides a high degree of interconnection between surface and groundwater. This results in anomalies in surface water flows and allows for water to traverse catchment boundaries making it extremely difficult to predict the movement of water upon entering the catchment.

Chapter 3
Flood Events in the Clare River Catchment

3.1 Historical Flooding

The Clare River catchment is susceptible to experiencing considerable flood events. Recent floods such as those experienced in 2006 and 2009 have increased concern in relation to the vulnerability of certain areas within the catchment. There have been hydrometric records of flooding within the catchment as far back as 1968. Lake levels largely influence the river water level from Lough Corrib to Claregalway. Fluvial flooding is the main cause of flooding above Claregalway (map Appendix A–3). Groundwater can lead to flooding in areas removed from the Clare River system. This section will examine historical records of flooding experienced within the catchment. This will provide a basis for relating the magnitude of the 2009 event to past flooding within the catchment.

3.1.1 November 1968

A limited amount of hydrometric data is available regarding flows in the Clare River in 1968. The hydrometric station at Corofin is the only station to provide records as far back as 1964 for the Clare River. There was widespread flooding across Ireland. This led to pressure being placed on governmental representatives to take action in relation to flood protection such as in Bray, an area in which flooding had become more frequent and severe according to newspaper reports from the time [2].

Hydrometric records available from Corofin show that a maximum flow of 207 m^3/s was observed on November 2nd. This is of greater magnitude than any other annual maximum recorded at Corofin including the November 2009 events. This flow was statistically estimated to have a return period of 273 years. The maximum flow corresponded to a water level of 27.3 mAod. The effect the river levels had at population centres such as Claregalway and Tuam is not known, as the hydrometric stations at these locations do not provide information for this period. October 30th to November 1st produced a 3-day total rainfall of 78.9 mm (15% of total rainfall for winter 1968-1969) recorded at Glenamaddy. This is the 3rd highest 3-day total on record at Glenamaddy with an average recurrence interval of 22 years and has not been exceeded since 1968. 3-day rainfall such as this would therefore be less frequent in more recent times. The large 3-day total came in a particularly wet autumn that produced 452.7 mm rainfall (average 1944-2009 = 307 mm), the second highest autumn rainfall total on record. 60% of this had fallen prior to October 30th. This would have led to a low soil moisture deficit (smd) and possibly saturated soil conditions prior to the rainfall event thus reducing the attenuation capacity of the catchment. Figure 3.1 displays the rainfall depths at Glenamaddy and the water levels at Corofin recorded around the time of the flood.

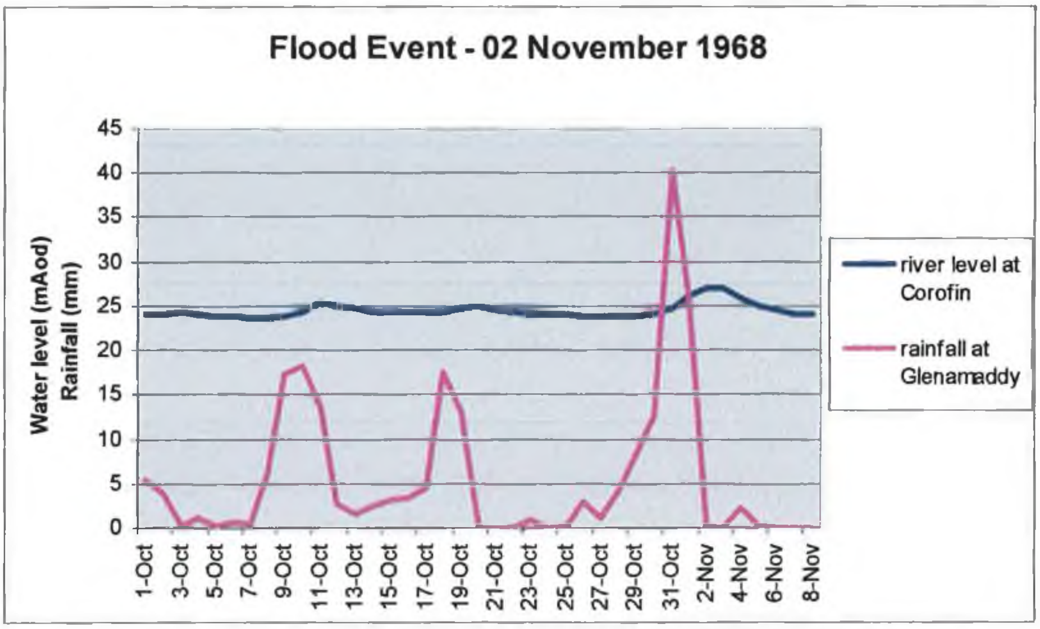

Figure 3.1 – Rainfall and River Level for the Flood Event of 1968

3.1.2 February 1990

Flooding experienced during the winter of 1989-1990 was as a result of considerable rainfall that fell mainly in the month of February. During the month of January the weather was unsettled with the area in the region of the Clare River catchment experiencing 150-175% normal rainfall of 1951-1980. The heavier rain fell in the latter stages of the month from the 22^{nd} – 25^{th} [3]. February was the wettest month on record for many stations. There was high depth and persistence of rainfall observed. Claremorris recorded the highest total rainfall for the month of 251 mm. Much of the rainfall occurred in the early part of February with the 6^{th} and 7^{th} producing the heaviest daily rainfall amounts for many stations [4]. This corresponds to the maximum water levels observed on the River Clare on the 7^{th} and 8^{th}. Figure 3.2 shows the rainfall at Glenamaddy and water levels at Ballygaddy and Corofin for the months of January and February.

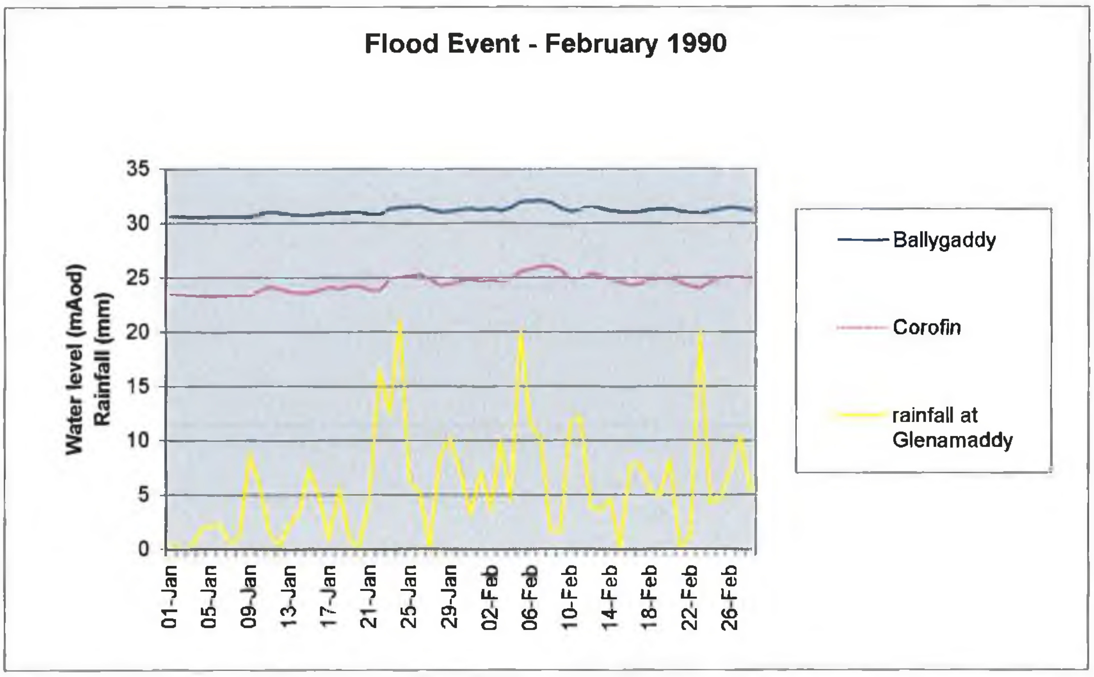

Figure 3.2 - Rainfall and River Levels for the Flood Event of 1990

Table 3.1 – Hydrometric Data for the Flood Event of 1990

Station No.	Location	Max. Water Level (mAod)	Max. Flow (m^3/s)	Date observed	Estimated Return Period
30007	Ballygaddy	32.24	96	7-Feb-1990	1 in 32 years
30004	Corofin	26.1	123	8-Feb-1990	1 in 6 years

Table 3.1 shows the maximum water levels and flows observed at the hydrometric stations at Ballygaddy and Corofin and their associated return periods. The statistical significance of the flood was greater at Ballygaddy than at Corofin. This is in some part due to the fact that records at Ballygaddy are only available from 1974 to 2009 and therefore do not include the extreme events of November 1968. However the flooding event of February 1990 produced the 2nd highest water levels and flows observed at Ballygaddy over its entire data series while the magnitude of the maxima data recorded at Corofin was exceeded at 5 other times during its partial distribution series (1968, 1994, 1999, 2006, 2009).

The winter of 1989-1990 produced rainfall of 416.5 mm. This is 35% greater than the average winter precipitation of 307.6 mm taken from the data set 1945-2009 at Glenamaddy. It is the 4th largest winter rainfall total for on record. The 3-day, 5-day and 10-day rainfall totals were all reasonably low for the station at Glenamaddy leading up to the flood event in February 1990. Each had an average recurrence interval of 1.5 years. It was the persistence of the rainfall that was the most obvious factor behind this flood event. Of the 416.5 mm of rain that fell in the winter 1989-1990, 40% fell in the 18 days leading up to the floods on the 7th and 8th of November. 17 of these days were wet days, 13 were very wet days and 8 were heavy precipitation days.

3.1.3 Winter 1990-1991

The flood events of the winter of 1990-1991 were as a result of heavy and persistent rainfall in the months of December and January. Most of the precipitation in December fell in the last 11 days with the 20th, 22nd, 25th and 26th producing the most significant quantities of rain across the country [5]. Claremorris recorded the highest monthly rainfall of 162 mm (130% of 1951-1980 normal) and also the highest daily total of the month of 28.5 mm on the 22nd [5]. The highest daily rainfall total at Glenamaddy for December was 30.9 mm, recorded on the 20th. January produced a monthly rainfall total similar to normal values. The majority of this rainfall occurred in the first 11 days of the month. This was a continuation of the heavy precipitation experienced in the closing days of December. Figure 3.3 shows the rainfall at Glenamaddy and the available daily mean water levels at Ballygaddy and Corofin. It shows that intense rainfall beginning on December 20th caused the initial rise in water levels. This initial rainfall produced the maximum water levels observed during the flood event. Persistent rainfall maintained high water levels for approximately 3 weeks until January 8th.

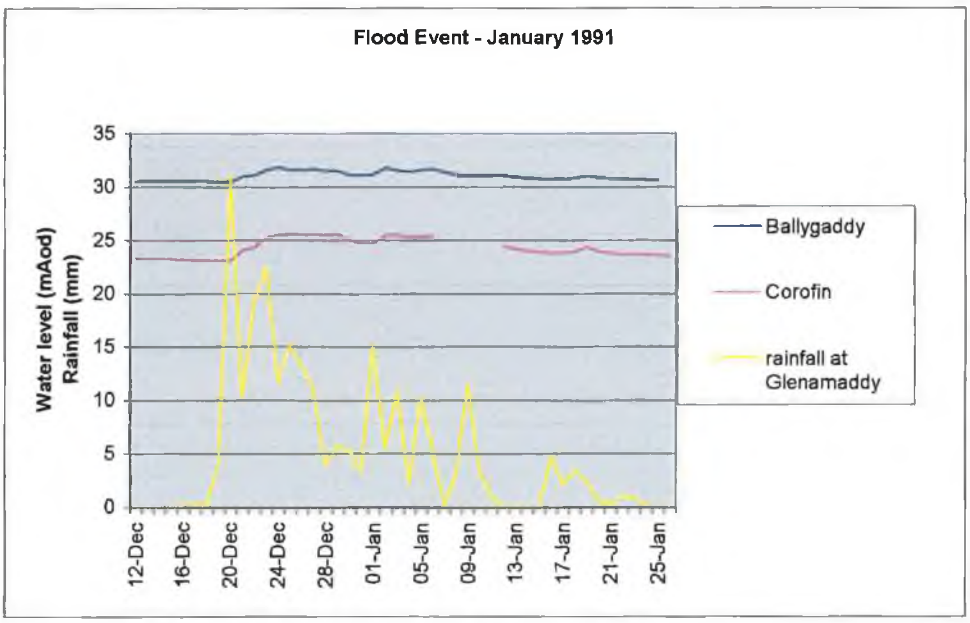

Figure 3.3 - Rainfall and River Levels for the Flood Event of 1991

The magnitude of the floods were not as severe as those observed during the winter of 1989-1990. The estimated return period for the flows at Ballygaddy and Corofin were 3 years and 2 years respectively. The max flow measured at Ballygaddy was not even the highest flow recorded for the hydrometric year of 1990. Corofin did experience an annual maximum flow for this event of 98.5 m^3/s measured on the December 29th. However this magnitude is still not particularly significant in relation to other extreme events observed at this station. The duration of this event was the most destructive aspect. The high water levels lasted for 3 weeks due to persistent rain. Table 3.2 shows the rainfall depths of varying duration during December compared to the long-term average. The 1-day total was less than average. The severity of the running totals increases with longer duration. The 10-day duration was in fact the largest 10-day total on record at Glenamaddy.

Table 3.2 – Rainfall Data for the Flood Event of 1990-1991

Station No.	Duration	Rainfall (mm)	Date ending	Average Totals for Each Duration (mm)
3127	1-day	30.9	20-Dec-1990	34.7
3127	3-day	60.3	22-Dec-1990	54.4
3127	5-day	94.4	24-Dec-1990	69.1
3127	10-day	143.4	29-Dec-1990	100.2

3.1.4 December 1999

The month of December was exceptionally wet in the northwest of the country [6]. There were reportedly 25 wet days observed in county Galway for the month [6]. Only 16 wet days were recorded at Glenamaddy. However, 11 of these were also very wet days with rainfall over 5 mm. Figure 3.4 shows rainfall at Glenamaddy and water levels for the Clare River. Water levels for Ballyhaunis are shown in figure 3.5 to emphasise the profile of water level, as fluctuations are small in comparison to height above ordnance datum at Malin. The profile of water levels shows that the hydrometric stations farther upstream experienced maximum water levels from November 28th – 30th. This was not the case at Claregalway where the maximum water level occurred on December 25th. Corofin experienced high water levels of equal magnitude in both months. These peaks are shown in table 3.3. 35.8 mm of rain fell on November 4th and 32.1 mm fell on the 27th to produce high water levels in the last few days of November. 31.2 mm of rain fell on December 21st. This daily total was slightly less than both November peaks.

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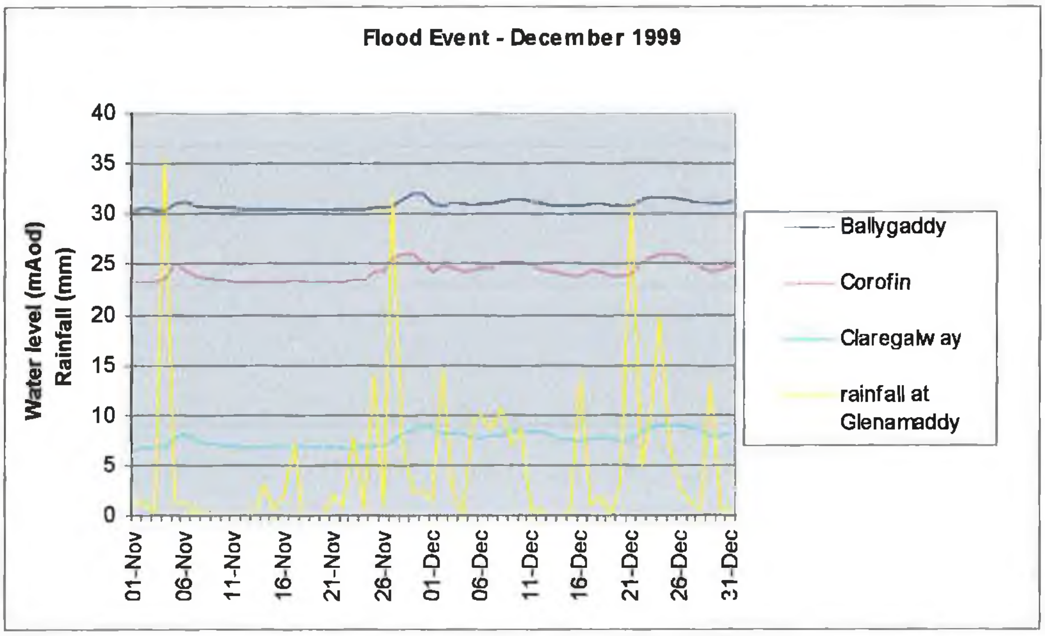

Figure 3.4 - Rainfall and River Levels for the Flood Event of 1999

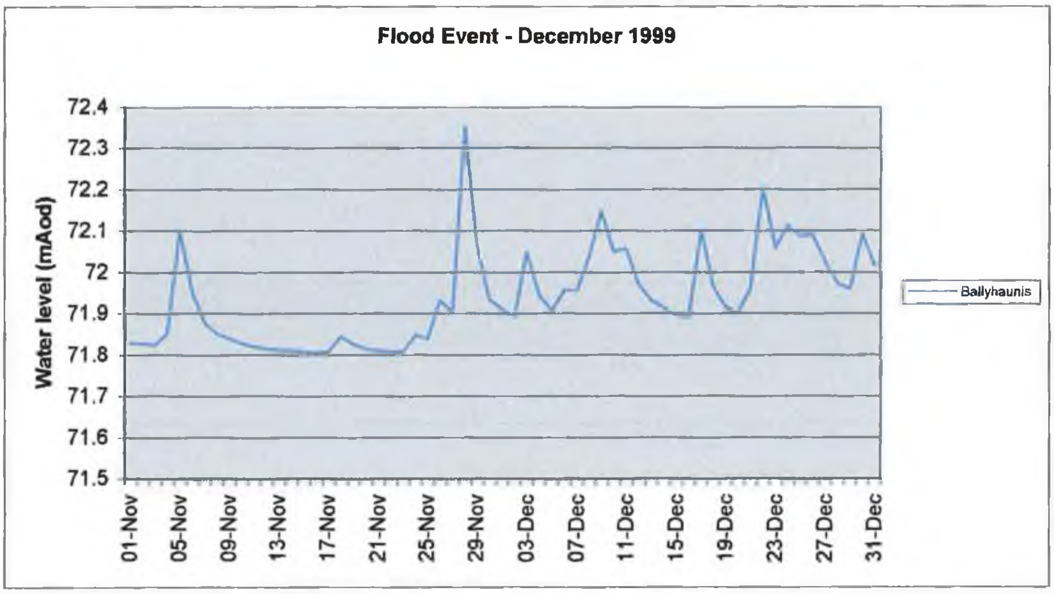

Figure 3.5 - River Level at Ballyhaunis for the Flood Event of 1999

Table 3.3 – Hydrometric Data for the Flood Event of 1999

Station No.	Location	November Peak (mAod)	Date ending	December Peak (mAod)	Date ending
30020	Ballyhaunis	72.354	28-Nov-1999	72.204	22-Dec-1999
30007	Ballygaddy	32.11	29-Nov-1999	31.73	24-Dec-1999
30004	Corofin	26.1	29-Nov-1999	26.1	25-Dec-1999
30012	Claregalway	8.754	30-Nov-1999	8.907	25-Dec-1999

The relationship between Lough Corrib water levels and river levels up as far as Claregalway may have had an affect on floodwaters at Claregalway hydrometric station. The rainfall during November could have served to raise water levels in Lough Corrib thus compounding the effect of the December rainfall on river flows at Claregalway. River flows above Claregalway would not have been affected by Lough Corrib water levels due to the difference in head increasing with an increase in distance from Lough Corrib. This may explain why upper reaches of the Clare River system did not experience water levels as large as those in November. Due to a lack of water levels from Lough Corrib for this period this cannot be assessed. The maximum December flow at Claregalway is estimated to have a return period of 6 years.

3.1.5 January 2005

The majority of January precipitation fell in the first 3 weeks of the month. Soil moisture deficits in the west were 0 mm (0 mm = field capacity) at the end of December. These had been saturated to –9 mm by January 10th [7]. Figure 3.6 shows the rainfall at Glenamaddy and water levels along the Clare River. Water levels at Ballyhaunis followed a similar pattern as the other hydrometric stations peaking one day before Ballygaddy and two days before Corofin and Claregalway. This is usual as the peak flows move downriver to Lough Corrib over a 2 to 3 day period.

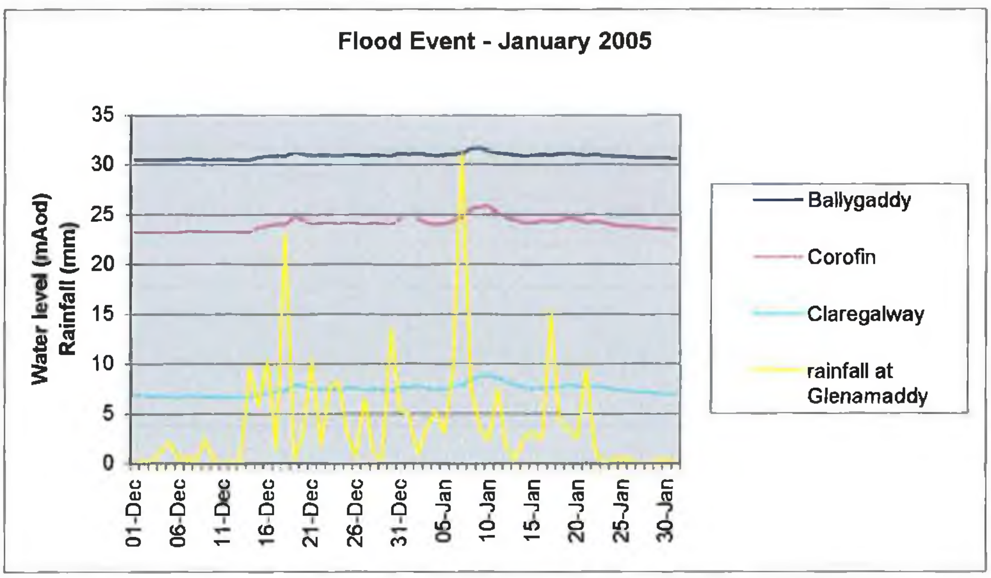

Figure 3.6 – Rainfall and River Levels for the Flood Event of 2005

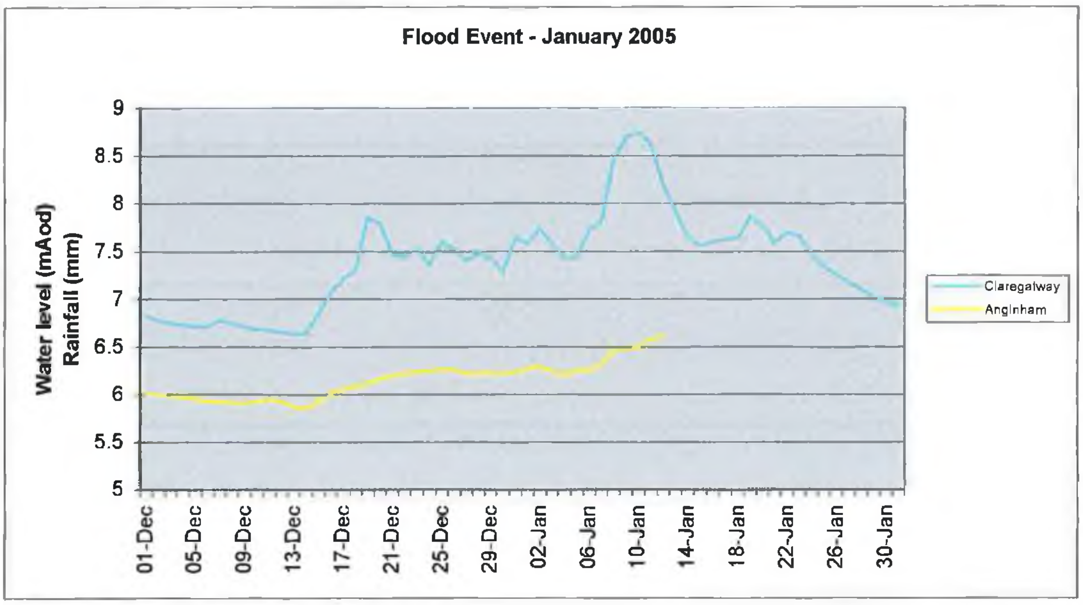

Figure 3.7 – Water Levels at Claregalway and Lough Corrib for the Flood Event of 2005

Figure 3.7 shows the water level at Claregalway in relation to water levels taken on Lough Corrib close to the Clare River outfall at Anglinham hydrometric station (map Appendix A–3). There is on average a 1.2 m difference in head between the two locations with a maximum head difference of 2.25 m on January 10th. Unfortunately records do not exist at Anglinham for the period after January 12th, as it appears lake levels continued to rise. The 1, 3, 5 and 10-day rainfall totals at Glenamaddy were not statistically significant in that they had average recurrence intervals of less than 1 year. Annual maxima were recorded at each hydrometric station during the month of January. The estimated return periods ranged from 2 to 4 years showing that this magnitude of flooding should not be treated as a rare occurrence. The work on the channel carried out by the OPW in the 1950's had been for a 1 in 3 year event. The 2005 flood had a statistical significance similar to the design capacity of the channel. However there was still a considerable amount of land flooded in areas such as Montiagh near Claregalway and the turlough at Cloonkeen North just upriver of Corofin.

3.1.6 December 2006

Table 3.4 shows the magnitude and return periods of maximum flows in the Clare River for December 2006. Maximum water level at Claregalway is known to have been 8.920 mAod measured on December 7th. The maximum flow at Claregalway was less statistically significant than at Ballygaddy and Corofin. Significantly maximum flow at

Claregalway is less than at Corofin. It is unlikely that floodwaters flowing around the bridge would have produced these diminished flows as the peak water level was below the bridge soffit of 9.085 mAod. These losses could potentially be attributable to temporary surface water storage or most probably groundwater leakage due to the karstified nature of the catchment.

Table 3.4 – Hydrometric Data for the Flood Event of 2006

Station No.	Location	Water Level (mAod)	Max. Flow (m^3/s)	Date observed	Estimated Return Period
30020	Ballyhaunis	72.31	4.16	3-Dec-2006	1 in 6 years
30007	Ballygaddy	32.08	84.5	5-Dec-2006	1 in 12 years
30004	Corofin	26.51	148	6-Dec-2006	1 in 18 years
30012	Claregalway	8.920	135.1	7-Dec-2006	1 in 7 years

Figure 3.8 shows the rainfall of 2006 at Glenamaddy plotted alongside the average daily rainfall totals for the period 1945-2009. 2006 rainfall total is shown as a 10-day running average to even out fluctuations in the data. The summer of 2006 was particularly dry with rainfall below average. An increase in rainfall was observed from September after which rainfall depths peaked at levels well above average, most notably in late September and early December. Figure 3.9 shows the soil moisture deficits for the west of Ireland from July to December. Soil moisture deficit (smd) values were available bimonthly on the 10^{th} and last day of each month. The corresponding rainfall shown is the total rainfall that fell at Glenamaddy during the intervals between soil moisture deficit readings. The graph shows that due to considerable rainfall in late September soil moisture deficits achieved –10 mm thus becoming saturated. They did not rise above field capacity for the remainder of 2006, achieving and maintaining a value of –10 mm throughout the month of December. This would have greatly reduced the attenuation capacity of the Clare River catchment during December 2006.

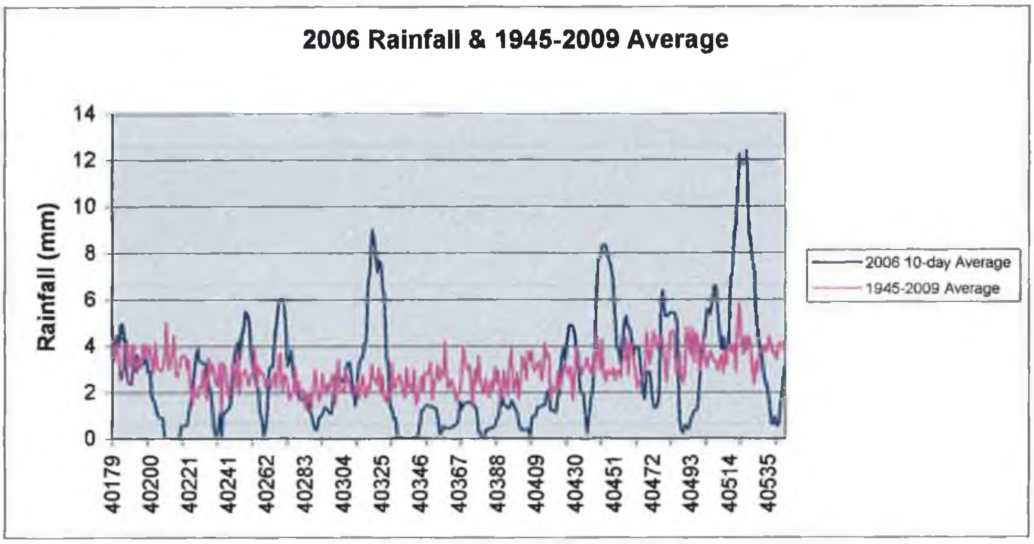

Figure 3.8 – 2006 Rainfall & 1945-2009 Average for Glenamaddy

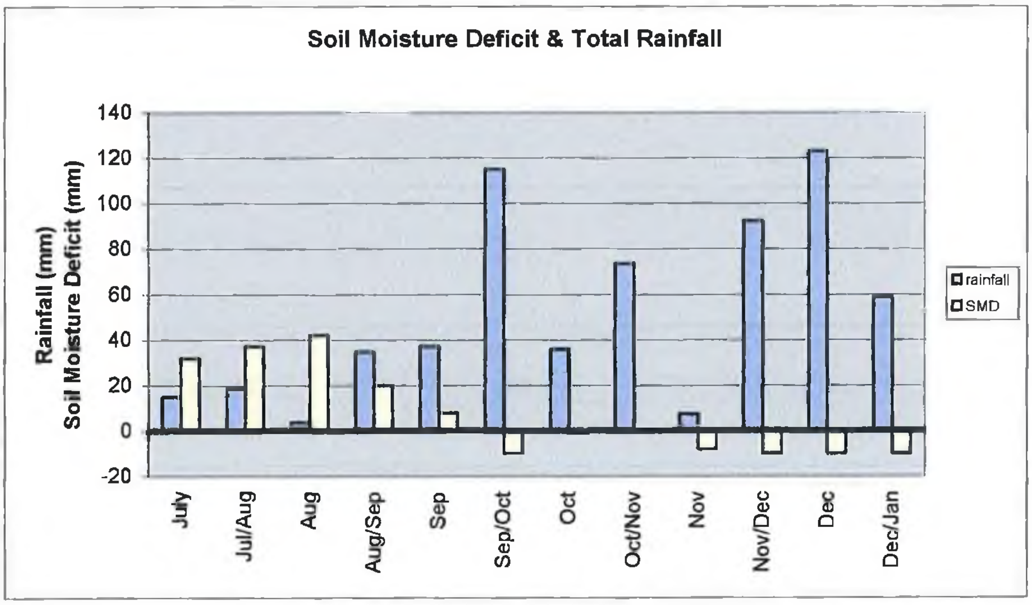

Figure 3.9 – Soil Moisture Deficit for West & Rainfall at Glenamaddy

Autumn rainfall at Glenamaddy was 361.4 mm. This was above average and the 13th highest autumn rainfall total for the data set 1945-2009. This was followed by a winter with a total rainfall of 399.9 mm. This was the 7th highest winter rainfall total. Significantly almost 30% of this winter rainfall fell in the first 8 days of December. Table 3.5 show the total rainfall of varying durations for the heavy rainfall in December. It also shows the average and maximum values for these durations. This serves to show that these rainfall levels, although above average, were not hugely significant.

Table 3.5 – Rainfall Data for the Flood Event of 2006

Station No.	Duration	Rainfall (mm)	Date ending	Average of Maxima 1945-2009 (mm)	Highest Maxima 1945-2009 (mm)
3127	1-day	38.2	2-Dec-2006	34.70	58.7
3127	3-day	66.3	4-Dec-2006	54.36	90
3127	5-day	88	6-Dec-2006	69.09	111.2
3127	10-day	123.7	11-Dec-2006	100.19	143.4

Weather throughout Galway was reported as particularly wet with September rainfall levels as high as 300 mm reported in Maam Valley. Therefore it is possible that spatial variations within the catchment may have resulted in other areas experiencing rainfall levels far greater than those recorded at Glenamaddy. Total autumn rainfall at Milltown was 491.4 mm. This was the second highest autumn precipitation total from available data. 509.7 mm fell in the winter of 2006-2007. This was the highest rainfall total at Milltown for the winter season. 50% of the total winter precipitation fell in December. This shows that available rainfall data is not a complete representation of the entire catchment due to variations within the catchment. The available rainfall data serves to provide a reasonably accurate indication of precipitation in the catchment area in a climatic context.

3.1.7 November 2009

The month of November was the wettest on record for many rainfall stations across the country. Rainfall stations such as Claremorris recorded highest ever monthly totals in over 50 years of operation [8]. Most of the country experienced over twice the normal rainfall for the month as shown in figure 3.10. The preceding months were not as wet as that of November. Most of October rainfall occurred in the latter half of the month. The summer of 2009 was particularly wet. Rainfall totals measured at Glenamaddy were the highest recorded since the summer of 1985. Total summer rainfall was recorded as 333.7 mm. This was almost 40% greater than the average summer rainfall. This could potentially have greatly reduced the soil moisture deficit of the catchment leading up to the flood event of November 2009. Figure 3.11 shows the soil moisture deficit in the region of the catchment from June to December for 2009 in comparison to the average of 2004-2009 records. This shows that summer rainfall did greatly reduce soil moisture deficit values. However, the relatively dry months of September and October resulted in soil moisture deficits for October being greater than average prior to the prolonged October and November rainfall.

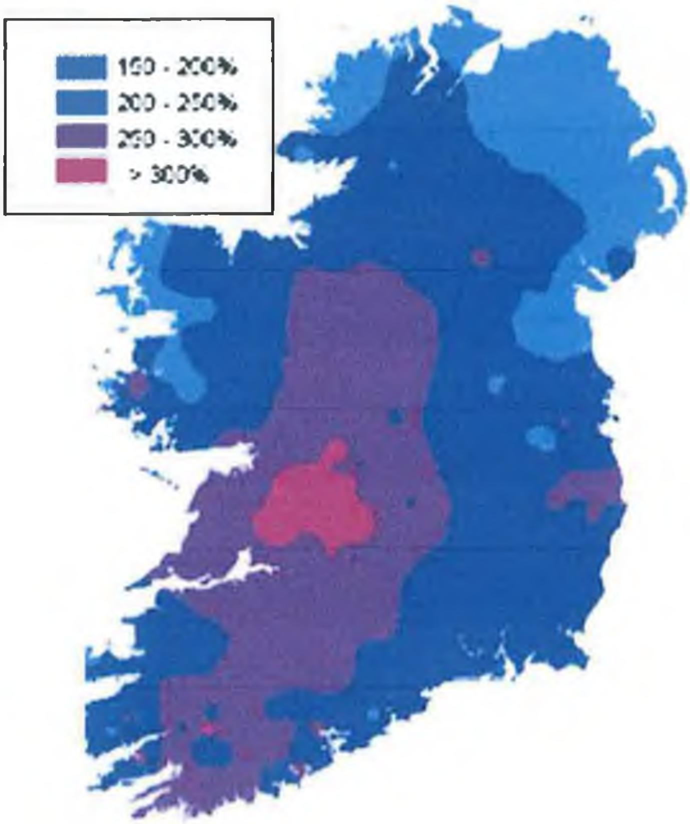

Figure 3.10 – November 2009 Rainfall as percentage of normal 1961-1990 [8]

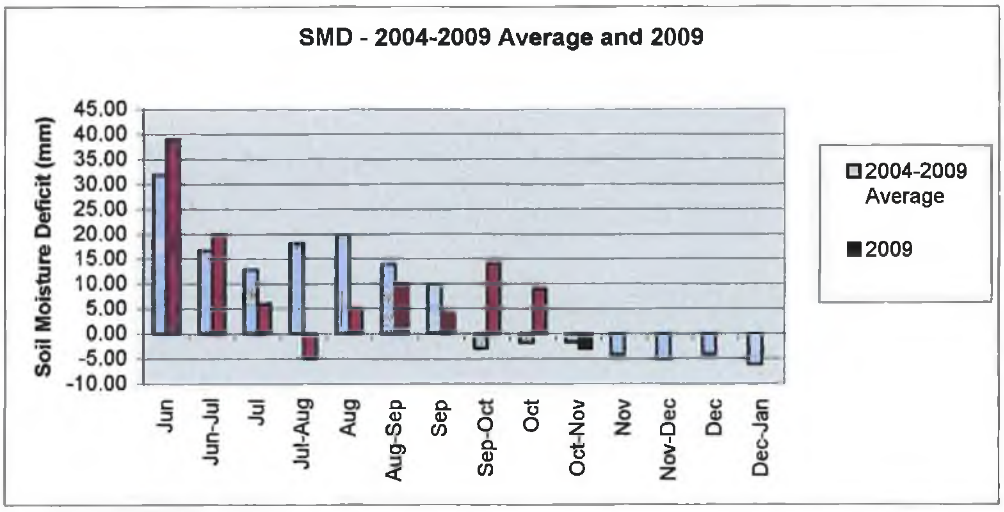

Figure 3.11 – Soil Moisture Deficit for 2009 and 2004-2009 Average

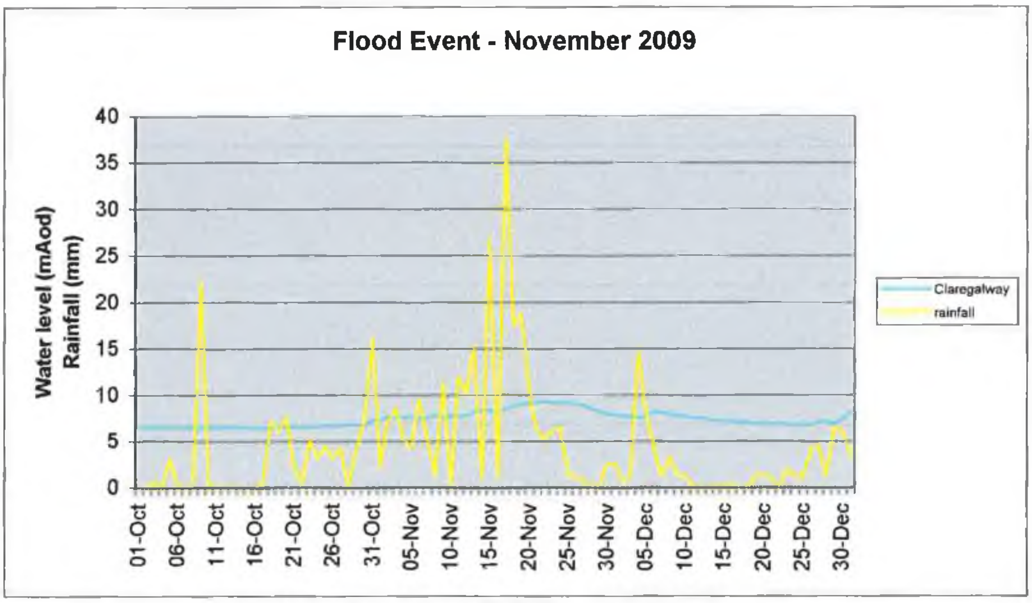

Figure 3.12 – Rainfall and River Level for the Flood Event of November 2009

Figure 3.12 shows the water level at Claregalway and the rainfall observed for October through December. Rainfall increased towards the end of October. Recorded rainfall for the 18^{th} – 30^{th} was 30% greater than average rainfall for this period. November 2009 produced the highest November rainfall total on record. Total November rainfall at Glenamaddy was 234.8 mm. This is 220% of normal November rainfall. The highest daily rainfall totals were experienced during mid-November producing a peak water level of 9.280 mAod at Claregalway on November 22^{nd}. The persistence of the rainfall was notable with 25 wet days and 9 heavy precipitation days recorded for the month of November. Table 3.6 show the magnitude of the 1, 3, 5, and 10-day totals and relates them to the maximum and average of these values from recorded data. The values recorded for the November 2009 event were above average for each of these totals. The magnitude of the 3, 5 and 10-day totals were more significant than that of the maximum daily total which was only slightly above average. The 5-day rainfall total was the 5^{th} highest 5-day total on record.

Table 3.6 – Rainfall Data for the Flood Event of November 2009

Station No.	Duration	Rainfall (mm)	Date ending	Average of Maxima 1945-2009 (mm)	Highest Maxima 1945-2009 (mm)
3127	1-day	37.6	17-Nov-2009	34.70	58.7
3127	3-day	73.8	17-Nov-2009	54.36	90
3127	5-day	101.7	19-Nov-2009	69.09	111.2
3127	10-day	129.9	21-Nov-2009	100.19	143.4

Table 3.7 – Return Periods for varying Rainfall Durations and Rainfall Stations in the Vicinity of the Clare River Catchment [9]

Location	Return Period					
	1-day (years)	2-day (years)	4-day (years)	8-day (years)	16-day (years)	25-day (years)
Galway (Univ.)	29	134	293	306	272	131
Ballygar	5.5	73	201	405	>500	251
Ballinasloe	2.4	20	159	>500	>500	>500

Table 3.7 shows the return periods for rainfall of varying durations at rainfall stations located near to the Clare River catchment. The maximum daily rainfall was not as statistically significant as rainfall totals for lager durations. The values of these return periods do not appear to reflect the magnitude of the rainfall at Glenamaddy. 1, 3, 5 and 10-day rainfall totals at Glenamaddy were not the largest on record and each were exceeded at least 4 times during the 1945-2009 data set. The duration of the return periods from Galway, Ballygar and Ballinasloe indicate the extreme rainfall that occurred in the west of Ireland.

Annual maximum flows were recorded along the Clare River for November 2009. The magnitude and return period of the event at hydrometric stations along the Clare River is shown in table 3.8. The floods of November 2009 were considerable. The flow at Corofin had a return period of 143 years. The most significant aspect of the flows in the river is that the maximum flow at Claregalway is over 10% less than that at Corofin. This is particularly significant due to the significant catchment area drained to the River Clare by the Abbert River between the two locations. The reason for the lower peak flow may be due to storage of water between both locations, most notably at Caherlea/Lisheenavalla. It is also felt that groundwater leakage is a significant contributory factor as highlighted in section 2.2. The fact that the gauge is located on the downstream face of the Claregalway Bridge, which acts as a hydraulic control during high flows such as this, would also contribute to these diminished flow measurements. During peak flow a certain amount of floodwaters would have escaped around the bridge and therefore escaped measurement.

Table 3.8 – Hydrometric Data from the Flood Event of November 2009

Station No.	Location	Water Level (mAod)	Max. Flow (m^3/s)	Date observed	Estimated Return Period
30020	Ballyhaunis	72.481	5.91	19-Nov-2009	1 in 51 years
30007	Ballygaddy	-	108.9	20-Nov-2009	1 in 97 years
30004	Corofin	27.14	193	21-Nov-2009	1 in 143 years
30012	Claregalway	9.280	163.2	22-Nov-2009	1 in 35 years

The location of the gauge at Claregalway is problematic due to the bridge constraining flow during peak flows. Therefore the gauge is not an accurate indication of hydraulic conditions on the upstream face of the bridge. A survey carried out during the peak water levels on November 21st indicates that water levels on the upstream face of the bridge were over 1 m above those on the downstream face. Also turbulence in the vicinity of the gauge produced water levels over 200 mm lower than those observed downstream of the bridge [10]. The lower water level at the gauge was due to supercritical flow at this point. Supercritical flow occurs when the flow velocity is greater than the wave velocity [13]. It can occur in channels with steep gradients or on the downstream face of a hydraulic constraint due to the build up of head on the upstream face. The difference in water levels corresponded to water levels of 9.536 mAod downstream and 10.336 mAod on the upstream face. The underside of the bridge span is 9.085 mAod. Therefore the bridge provided an insufficient height to cope with the discharge produced by the November 2009 events. Road level on the bridge is 10.5 mAod. This is above peak water level and was not flooded. The N17 was flooded either side of the bridge resulting in the road being closed for almost a week. Figure 3.13 – 3.16 shows flooding along the Clare River for November 2009.

Figure 3.13 – Flooding Along the N84 (Headford Rd), 22/Nov/2009

Figure 3.14 – Flooding Downstream of Claregalway at Montiagh, 22/Nov/2009

Figure 3.15 – Flooding in Claregalway, 22/Nov/2009

Figure 3.16 – Flooding at Confluence of Abbert and Clare River, 22/Nov/2009

The aerial photographs of the November 2009 flooding give an indication of the extent of flooding experienced. Flood extent maps were created from aerial photography and anecdotal evidence arising from meetings with the Office of Public Works (OPW) and the Western Regional Fisheries Board (WRFB) representatives who observed the extent of the flooding first hand. These flood extent maps were created to provide an accurate visual representation of the extent of the November 2009 flood event and are provided in Appendix A–2. A considerable amount of land, mainly peat land, was flooded in the vicinity of the N84 road crossing. Due to the low population density in this area, limited damage to property was experienced. The floods affected a number of population centres. Milltown and Tuam both experienced flooding. It was the areas located in the lower extremes of the catchment that experienced the worst of the flooding. Claregalway and surrounding town lands were impacted greatest by the events. Corofin avoided any flooding in the town itself mainly due to the depth of the channel at this point. The floods did lead to the formation of the Cloonkeen turlough upriver of Corofin at the confluence of the Grange and the Clare River. This flooded a considerable amount of agricultural land in the area. Historical information suggests that this turlough was a more permanent feature in the catchment in the 1700's prior to any channel works being carried out.

The November 2009 floods were as a result of above average and persistent rainfall. This produced river flows with return periods estimated as high as 143 years at Corofin and 97 years at Ballygaddy. The soil moisture deficit was at field capacity prior to the heavy rainfall of October and November. However this was no wetter than average and therefore the wet summer does not appear to have been a significant factor in the flood event. The most significant flooding occurred in the region of Claregalway due to the channels inability to cope with peak flows. Flooding just upstream of Claregalway Bridge would also have been contributed to due to the bridge acting as a hydraulic constraint when water levels rose above 9.085 mAod.

Chapter 4
Frequency Analysis

4.1 Hydrometric Stations and Data Used

A limited amount of hydrometric data is available in relation to the Clare River system. There are 4 hydrometric stations located within the Clare River drainage network. The locations of these are shown in figure 4.1 and details regarding available information provided in table 4.1. There is an absence of gauged data on the Sinking, Grange and Abbert Rivers. Flows are recorded at each of the hydrometric stations at 15-minute intervals. A complete record of daily flows and water levels was available for Ballyhaunis and Claregalway from the EPA. Annual maxima distribution series for flows and water levels was available from the OPW for Ballygaddy and Corofin. Records for flows and water levels at Corofin for October-December 2009 were also obtained to extrapolate maximum flows. The maximum flow for the 2009 flooding for Ballygaddy was obtained from published material [10]. The annual maxima distribution series are provided for each hydrometric station in Appendix C–1.

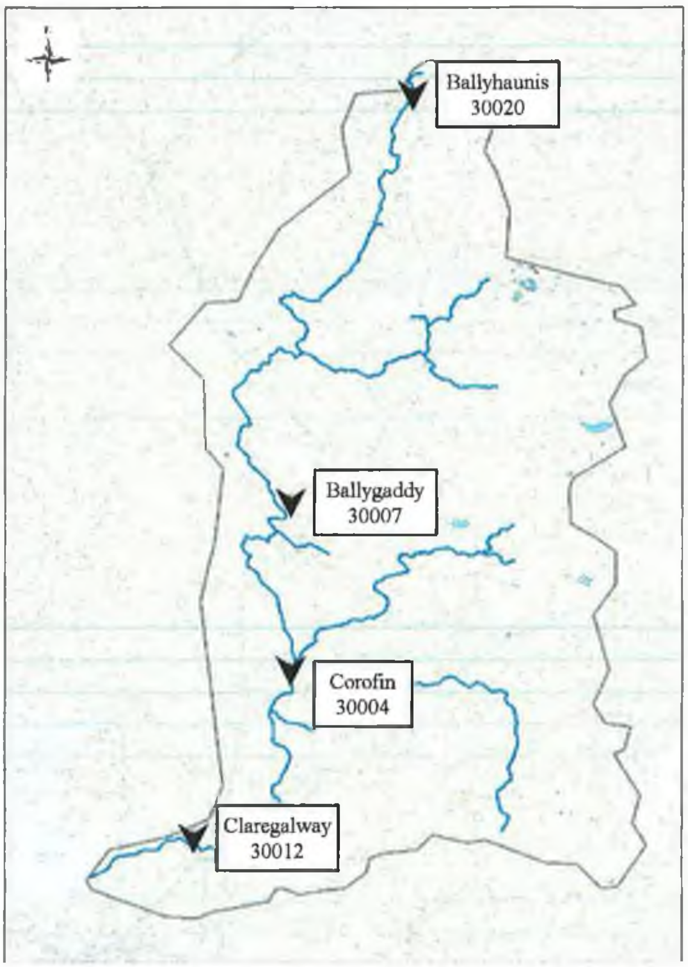

Figure 4.1 – Hydrometric Station Locations

Table 4.1 – Hydrometric Station Information

Station No.	30020	30007	30004	30012
Location	Ballyhaunis	Ballygaddy	Corofin	Claregalway
Waterbody	Dalgan River	Clare River	Clare River	Clare River
Body Responsible	Mayo Co. Council	OPW	OPW	Galway Co. Council
Station Status	Active Primary	Active Permanent	Active Permanent	Active Primary
Station Type	Data Logger	Logger/ Autographic Recorder	Logger/ Autographic Recorder	Data Logger
Catchment size (km^2)	21.4	469.9	699.9	1072.9
Records Available	1991 – Jul 2010 (19 yrs)	1974 – Dec 2009 (36 yrs)	1964 – Dec 2009 (46 yrs)	Aug 1996 – Jul 2010 (14 yrs)

4.2 Methodology

Statistical analysis of the flood peak data is used in estimating the design Annual Exceedance Probability (AEP) flow. This will be particularly effective for the stations at Ballygaddy and Corofin due to the long duration of available records. Analysis of an annual maxima distribution series is generally considered accurate to twice the length of the distribution series. Therefore the stations at Ballyhaunis and Claregalway are limited for the extrapolation of longer return periods due to the short duration of data, 19 years and 14 years respectively. Analysis of the annual maximum flows will be carried out using extreme value distributions. Hydrometric stations record data every 15 minutes. This equates to 35,040 readings every year. The annual maxima are located in the extreme tail of the probability distribution of this parent population and as such display a very different probability distribution. There are three asymptotic forms of extreme value distributions (type 1, type 2, type 3) of which type 1 (EV1) best suits the analysis of an annual maxima series in Ireland [11]. Frequency analysis by applying probability plotting using Gringorten and the method of moments (Gumbel) can both be used to fit the EV1 distribution to the annual maximum series. Both of these are used in the context of this study and their methods are outlined in this section.

Extreme value series consist of the most extreme values occurring within a predefined time interval. An annual maximum series includes the largest events occurring within each hydrometric year. This partial duration series provides a good degree of certainty in relation to the assumptions of independence and identical distribution of the hydrologic data required for the analysis. A hydrometric year is taken from October 1^{st} of the given year to September 30^{th} of the following year (i.e. hydrometric year 2009 runs from October 1^{st} 2009 to September 30^{th} 2010). This reduces the probability of interdependence between extreme events in consecutive years, as they are generally temporally located toward the central portion of this time period. The production of the hydrologic data is assumed to be produced by a system (rainfall event) that is stochastic space independent and time independent [11].

An extreme flood event is determined to have occurred when its magnitude is greater than a predefined value. The period of time between the occurrences of these events is the recurrence interval. The average value of the recurrence interval (τ) is known as the return period (T), which can be used to determine the probability of events occurring.

4.2.1 EV1 Method of Moments

EV1 method of moments is used in flood estimation to estimate the magnitude of a given return period. It is a commonly used flood flow estimation technique. It has been employed in studies carried out in conjunction with the OPW. The method assumes that the events being analysed are independent in space and time. The EV1 probability distribution function (Gumbel) is [11]:

$$F(x) = \exp[-\exp\{-(x-u)/\alpha\}] \quad (4.2.1.1)$$

The parameters included are given by equations 4.2.1.2 and 4.2.1.3:

$$\alpha = s * 6^{1/2} / \pi \quad (4.2.1.2)$$

where: α = scale parameter

s = standard deviation of the annual maxima series

$$u = x_{av} - 0.5772\,\alpha \quad (4.2.1.3)$$

where: u = location parameter

x_{av} = average of the annual maxima series

A reduced variate y can therefore be defined as:

$$y = (x-u) / \alpha \quad (4.2.1.4)$$

Subbing equation 4.2.1.4 into equation 4.2.1.1 gives:

$$F(x) = \exp [-\exp\{-y\}] \quad (4.2.1.5)$$

Solving equation 4.2.1.5 for y yields:

$$y = -\ln [\ln \{1/F(x)\}] \quad (4.2.1.6)$$

Assume an extreme event is defined to have occurred when the magnitude of a random variable X is greater than or equal to a predefined threshold x_T. The recurrence interval (τ) is the period of time (usually in years) between these extreme events occurring. The return period (T) of an event $X \geq x_T$ equates to the average/expected value of τ, $E(\tau)$, for the distribution series. The probability of an extreme event $X \geq x_T$ occurring is given by:

$$p = P(X \geq x_T) \quad (4.2.1.7)$$

For each observation in the distribution series there are two possible outcomes.

1. Success – $X \geq x_T$ – probability = p
2. Failure – $X < x_T$ – probability = 1-p

The observations are independent. Therefore the probability of experiencing a return period of duration τ is the sum of the probabilities of τ-1 failures followed by 1 success:

$$E(\tau) = \sum \tau (1-p)^{\tau-1} p$$

$$= p + 2 (1 - p) p + 3 (1 - p)^2 p + 4 (1 - p)^3 p + \ldots\ldots$$

$$= p [1 + 2 (1 - p) + 3 (1 - p)^2 + 4 (1 - p)^3 + \ldots\ldots] \quad (4.2.1.8)$$

Equation 4.2.1.8 takes the form of the power series expansion:

$$(1 + x)^n = 1 + nx + [n (n - 1) / 2] x^2 + [n (n - 1) (n - 2) / 6] x^3 + \ldots\ldots] \quad (4.2.1.9)$$

where: $x = -(1 - p)$ $\qquad n = -2$

Therefore equation 4.2.1.8 can be given by:

$$E(\tau) = p / [1 - (1 - p)]^2 = 1 / p \quad (4.2.1.10)$$

Since $E(\tau) = T$ and $p = P(X \geq x_T)$ then:

$$P(X \geq x_T) = 1 / T = 1 - P(X < x_T) \quad (4.2.1.11)$$

Now: $F(x_T) = P(X < x_T)$ (4.2.1.12)

Subbing equation 4.2.1.12 into equation 4.2.1.11 gives:

$$1 / T = 1 - F(x_T) \quad (4.2.1.13)$$

Solving equation 4.2.1.13 for $F(X_T)$ yields:

$$F(x_T) = (T - 1) / T \quad (4.2.1.14)$$

Subbing equation 4.2.1.14 into equation 4.2.1.6 yields:

$$y_T = -\ln [\ln \{T / (T - 1)\}] \quad (4.2.1.15)$$

Solving equation 4.2.1.4 for x relates x_T to y_T:

$$x_T = u + \alpha (y_T) \quad (4.2.1.16)$$

Equation 4.2.1.2 and equation 4.2.1.3 are used to determine the scale parameter and location parameter for the annual maxima series. Equation 4.2.1.15 is used to determine the reduced variate y_T for a defined return period (T). Equation 4.2.1.16 can then be used to determine the magnitude of the event by combining the results obtained from equations 4.2.1.2, 4.2.1.3 and 4.2.1.15.

4.2.2 Frequency Factor

For EV1 distribution Chow (1953) derived the expression to determine the frequency factor (K_T) for a return period (T) [11]:

$$K_T = -(6)^{1/2} \{0.5772 + \ln [\ln (T / (T-1))]\} / \pi \quad (4.2.2.1)$$

K_T can then be used to calculate the magnitude of such an event (x_T) for an annual maxima distribution series using:

$$x_T = x_{av} + K_T (s) \quad (4.2.2.2)$$

where: x_{av} = average of annual maxima series

s = standard deviation of annual maxima series

The return period of an event can also be determined by reversing the method. Let x_T be the event magnitude and solve for K_T. This value can then be used to solve for T, which by rearranging equation 4.2.2.1 is given by:

$$T = 1 / 1 - \exp \{ - \exp [- (0.5772 + \pi K_T / 6^{1/2})]\} \quad (4.2.2.3)$$

Equation 4.2.2.1 applies the same principles as those described for Gumbel and as such will provide the same answers and a check for calculations.

4.2.3 EV1 Distribution using Gringorten (Probability Plotting)

This statistical analysis method is suitable for applying to an annual maximum distribution series. Assumptions are made that the events being analysed are independent in space and time. This assumption is followed due to the start and end date of the hydrometric year as outlined in section 4.2. The method involves calculating the plotting positions. If n is the total number of values to be plotted and m is the rank of the value in a list ordered by descending magnitude then the exceedance probability of the m^{th} largest value for an extreme value distribution is given by the Gringorten formula given by equation 4.2.3.1:

$$P(X \geq x_m) = (m - b) / (n + 1 - 2b) \qquad (4.2.3.1)$$

where: $b = 0.44$ Gringorten (1963)

We know $P(X \geq x_m) = 1 / T$ from equation 4.2.1.11

Therefore:

$$T = 1 / P(X \geq x_m) = (n + 1 - 2b) / (m - b) \qquad (4.2.3.2)$$

The data can then be plotted in a variety of ways. The method chosen was to plot the actual discharges on a normal scale y-axis against the return periods as determined by Gringortens formula on a logarithmic scale x-axis to linearize the plot. The logarithmic relationship between the two was used to determine the magnitude of events of various return periods.

4.2.4 Comparison of Probability Plotting with Lognormal Distribution Fitted to them by Frequency Factor Method.

The comparison of the plotted data with the lognormal distribution fitted to them by the frequency factor method shows if the fitted line is consistent with observed data. The return periods estimated using Gringortens formula are converted to a frequency factor (K_T) using equation 4.2.2.1. The lognormal values of the actual discharges are calculated. The mean (y_{av}) and sample deviation (s_y) of this lognormal distribution series is estimated. Log Q (y_T) from the lognormal distribution is then estimated by using the following equation:

$y_T = y_{av} + K_T\,(s_y)$ (4.2.4.1)

The magnitude of the event x_T is given by:

$x_T = (10)\,^\wedge\,(y_T)$ (4.2.4.2)

The estimated flows and recorded flows can then be plotted on a logarithmic y-axis against the K_T values on the x-axis to linearize the plot. The plot shows whether the fitted line is consistent with the recorded data.

4.2.5 Standard Error and Confidence Limits

Confidence limits are the upper and lower limit of a confidence interval within which the true value of a statistical estimate can reasonably be expected to lie. A greater required confidence level (β) results in a wider confidence interval. The significance level (α) that corresponds to a given confidence level is given by:

$\alpha = (1 - \beta) / 2$ (4.2.5.1)

A 95% confidence level was chosen for this study corresponding to the following values:

$\beta = 0.95 \qquad \alpha = 0.025$

Therefore the required standard normal variable (z_α) has an exceedance probability of 0.025 and a cumulative probability of 0.975. The value of z_α is obtained from the table in Appendix D–1: $z_\alpha = 1.96$.

The standard error is a measure of the standard deviation of event magnitudes from samples about the true event magnitude. The standard error for EV1 distributions is given by [12]:

$s_e = [(1 / n)\,(1 + 1.1396\,K_T + 1.1\,K_T^2)]^{1/2}\,s$ (4.2.5.2)

where: s_e = standard error

n = number of observations (years) in series

K_T = frequency factor

s = standard deviation

The confidence interval can then be defined by the formula:

$x_T +/- s_e\,z_\alpha$ (4.2.5.3)

where: x_T = magnitude of event of return period T

By applying the 95% confidence level to each return period a confidence interval can be constructed for which there is a 95% confidence that the true magnitude of an event will lie within.

4.3 Results of Statistical Analysis

The results of the statistical analysis are shown in tables 4.2 to 4.5. Figure 4.2 to 4.5 are graphical representations of the confidence interval and estimated relationship between discharge and return period. A more extensive breakdown of calculations, results and graphs are provided in Appendix C–2, C–3, C–4 and C–6.

Table 4.2 – Statistical Analysis Results for Ballyhaunis

Ballyhaunis (30020)

Return Period T	EV1 (Gumbel MoM) (m^3/s)	EV1 (Frequency Factor) (m^3/s)	Prob. Plot (Gring) (m^3/s)	Standard Error (m^3/s)	Confidence Interval (m^3/s)
2	3.14	3.14	3.01	0.21	3.1 +/- 0.4
5	4.02	4.02	3.98	0.35	4.0 +/- 0.7
10	4.61	4.61	4.71	0.48	4.6 +/- 0.9
25	5.35	5.35	5.68	0.65	5.3 +/- 1.3
50	5.89	5.89	6.42	0.77	5.9 +/- 1.5
100	6.44	6.44	7.15	0.90	6.4 +/- 1.8
500	7.69	7.69	8.85	1.20	7.7 +/- 2.3

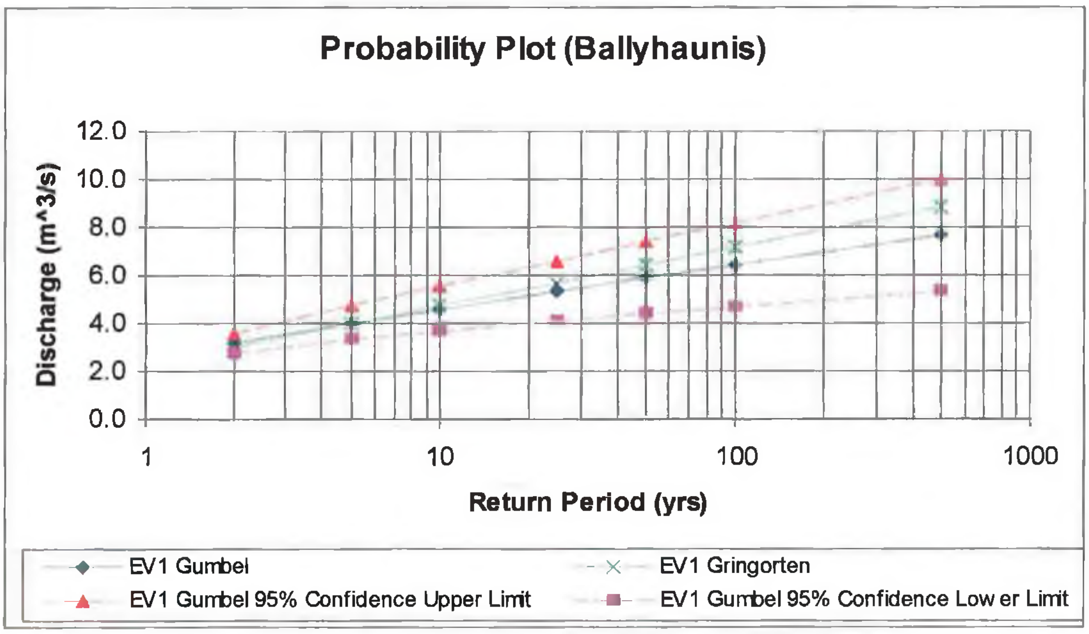

Figure 4.2 – Graphical Representation of Statistical Results from Ballyhaunis

Table 4.3 – Statistical Analysis Results for Ballygaddy

Ballygaddy (30007)

Return Period T	EV1 (Gumbel MoM) (m^3/s)	EV1 (Frequency Factor) (m^3/s)	Prob. Plot (Gring) (m^3/s)	Standard Error (m^3/s)	Confidence Interval (m^3/s)
2	60.63	60.63	58.63	2.26	60.6 +/- 4.4
5	73.65	73.65	72.52	3.80	73.7 +/- 7.4
10	82.28	82.28	83.02	5.13	82.3 +/- 10.1
25	93.18	93.18	96.91	6.92	93.2 +/- 13.6
50	101.26	101.26	107.42	8.28	101.3 +/- 16.2
100	109.28	109.28	117.92	9.64	109.3 +/- 18.9
500	127.83	127.83	142.32	12.83	127.8 +/- 25.1

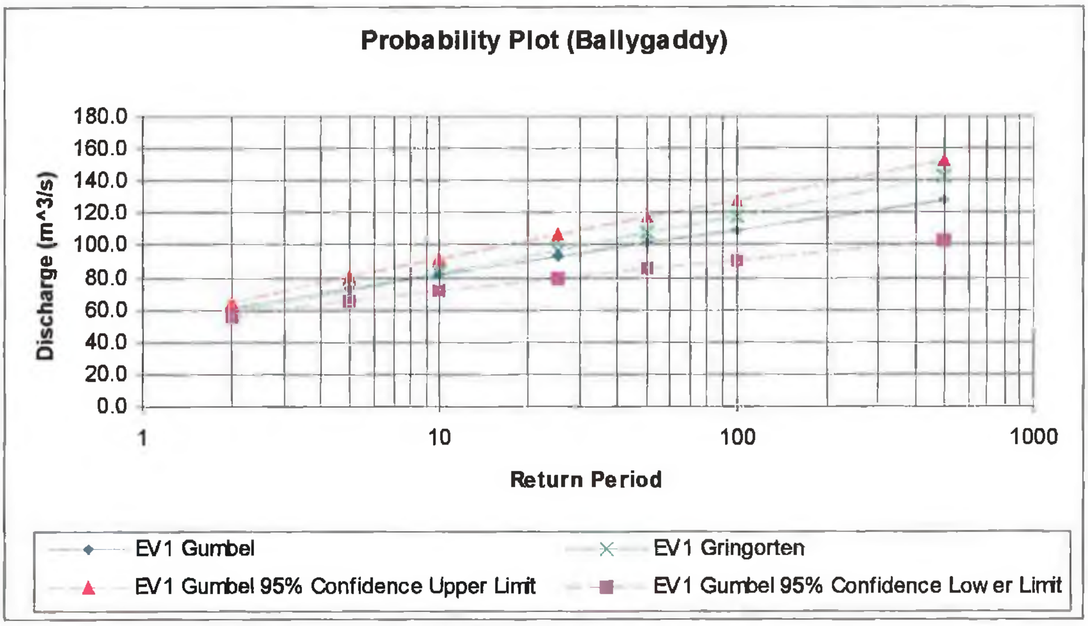

Figure 4.3 – Graphical Representation of Statistical Results from Ballygaddy

Table 4.4 – Statistical Analysis Results for Corofin

Corofin (30004)

Return Period T	EV1 (Gumbel MoM) (m^3/s)	EV1 (Frequency Factor) (m^3/s)	Prob. Plot (Gring) (m^3/s)	Standard Error (m^3/s)	Confidence Interval (m^3/s)
2	93.66	93.66	89.95	3.75	93.7 +/- 7.4
5	118.17	118.17	115.67	6.32	118.2 +/- 12.4
10	134.40	134.40	135.12	8.54	134.4 +/- 16.7
25	154.90	154.90	160.84	11.51	154.9 +/- 22.6
50	170.12	170.12	180.30	13.77	170.1 +/- 27.0
100	185.21	185.21	199.76	16.05	185.2 +/- 31.5
500	220.11	220.11	244.93	21.35	220.1 +/- 41.8

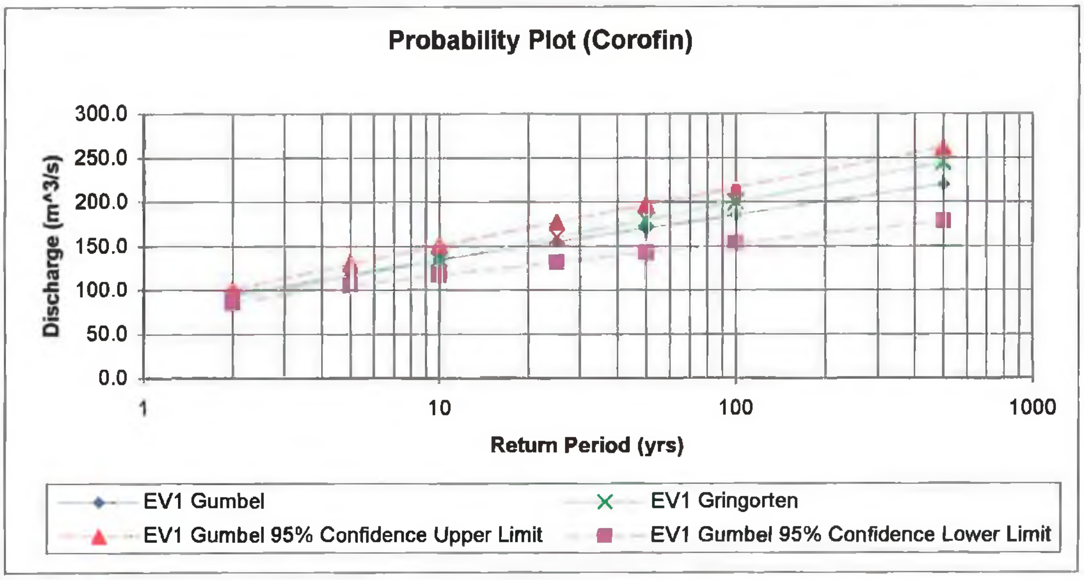

Figure 4.4 – Graphical Representation of Statistical Results from Corofin

Table 4.5 – Statistical Analysis Results for Claregalway

Claregalway (30012)

Return Period T	EV1 (Gumbel MoM) (m^3/s)	EV1 (Frequency Factor) (m^3/s)	Prob. Plot (Gring) (m^3/s)	Standard Error (m^3/s)	Confidence Interval (m^3/s)
2	110.29	110.29	107.56	5.26	110.3 +/- 10.3
5	129.24	129.24	128.62	8.86	129.2 +/- 17.4
10	141.78	141.78	144.56	11.97	141.8 +/- 23.5
25	157.64	157.64	165.62	16.13	157.6 +/- 31.6
50	169.40	169.40	181.55	19.30	169.4 +/- 37.8
100	181.07	181.07	197.48	22.49	181.1 +/- 44.1
500	208.05	208.05	234.47	29.92	208.0 +/- 58.6

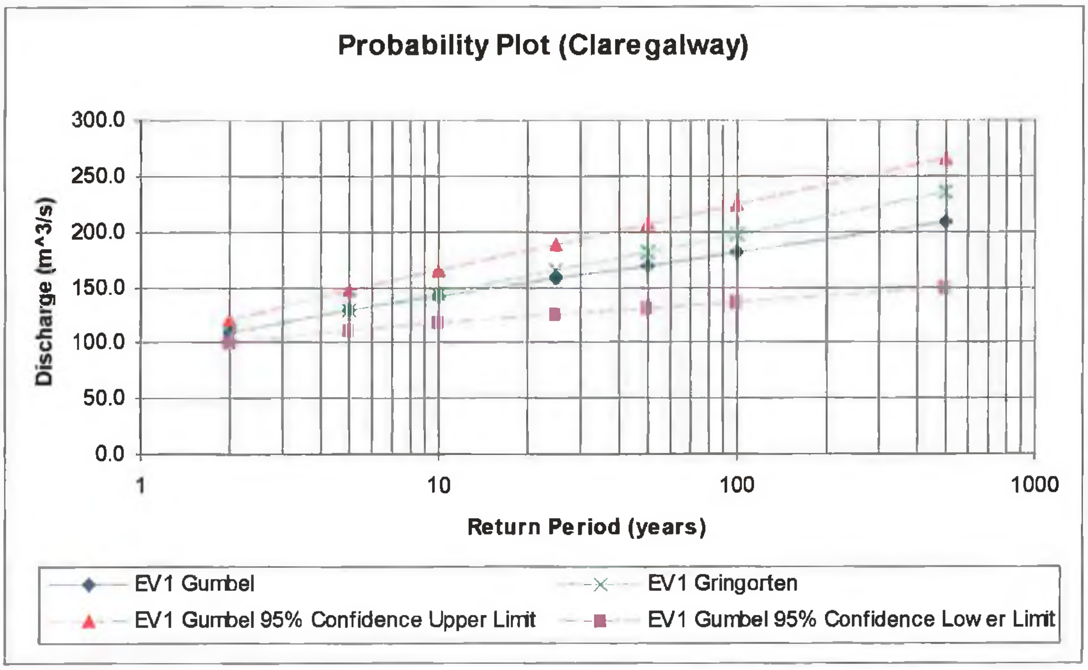

Figure 4.5 – Graphical Representation of Statistical Results from Claregalway

Subsequent to the estimation of the magnitude of events of defined return periods an analysis of the magnitudes of the flood events described in chapter 3 was carried out using EV1 distribution (Gumbel). The purpose of this was to ascertain whether there had been any increase in the return period of recorded extreme events in recent times. This could indicate an increase in the severity of flooding in the catchment. The results of this are provided in table 4.6 to 4.7 with a more detailed breakdown of calculations provided in Appendix C–7.

Table 4.6 – Return Period at Ballyhaunis and Ballygaddy of Historical Floods in Catchment

Event	Ballyhaunis		Ballygaddy	
	Discharge (Q) (m^3/s)	Return Period (T) (years)	Discharge (Q) (m^3/s)	Return Period (T) (years)
Nov.-1968	-	-	-	-
Feb.-1990	-	-	96.0	32
Winter 1990-91	-	-	64.6	3
Dec.-1999	4.7	11	94.5	28
Jan.-2005	3.0	2	58.9	2
Dec.-2006	4.2	6	84.5	12
Nov.-2009	5.9	51	108.9	97

Table 4.7 – Return Period at Corofin and Claregalway of Historical Floods in Catchment

Event	Corofin		Claregalway	
	Discharge (Q) (m^3/s)	Return Period (T) (years)	Discharge (Q) (m^3/s)	Return Period (T) (years)
Nov.-1968	207.0	273	-	-
Feb.-1990	123.0	6	-	-
Winter 1990-91	98.5	2	-	-
Dec.-1999	131.0	9	134.0	6
Jan.-2005	110.0	4	122.5	4
Dec.-2006	148.0	18	135.1	7
Nov.-2009	193.0	143	163.2	35

The November 2009 events are the most severe on record at every station except Corofin, which recorded November 1968 floods as the most severe on its records. At Ballyhaunis the return period of the 2009 event was estimated at 51 years. This was 4.5 times greater than the next most significant event that occurred in December 1999.

At Ballygaddy the return period of the 2009 event was estimated at 97 years. This was 65 years longer than the next most significant event that occurred in February 1990.

The November 1968 floods recorded at Corofin were by far the most significant with an estimated return period of 273 years while November 2009 events were estimated at 143 yrs. Apart from these 2 events the remainder were deemed relatively insignificant at Corofin with return periods of less than 10 years except for 2006 which was estimated at 18 years.

Claregalway had a relatively short distribution series compared to Ballygaddy and Corofin. The estimated return period of the November 2009 event was 35 years. The estimation of return periods is accurate out to approximately twice the duration of the distribution series, which is 14 years at Claregalway. 35 years is greater than this but indicates that the event was significant relative to other maximum events on record that displayed return periods of less than 10 years.

Due to the long distribution series available from both Ballygaddy and Corofin an analysis using EV1 distribution (Gumbel) was carried out on the first and second half of the distribution series for both hydrometric stations. The purpose of this was to ascertain whether there had been any increase in the magnitude of events, of defined return periods, estimated from more recent distribution series. This would indicate if there had been an increase in the severity of such events along the Clare River in more recent times. The results of this are provided in table 4.8 with calculations provided in Appendix C–8.

Table 4.8 – Magnitude of events from 1st and 2nd half of distribution series at Ballygaddy and Corofin

Return Period	Ballygaddy Discharge (Q)		Corofin Discharge (Q)	
T (years)	1974-1991 (m^3/s)	1992-2009 (m^3/s)	1964-1986 (m^3/s)	1987-2009 (m^3/s)
2	62.66	58.66	87.76	99.67
5	72.78	74.25	112.46	123.39
10	79.48	84.58	128.82	139.10
25	87.95	97.62	149.48	158.94
50	94.24	107.30	164.81	173.66
100	100.47	116.91	180.02	188.27
500	114.88	139.11	215.19	222.03

The results of this analysis show an increase in the magnitude in more recent times for every event except for the 2-year event at Ballygaddy. This estimation is lower for the period 1992-2009 than for 1974-1991. The remainder of events at Ballygaddy are greater for the more recent distribution series with a 100-year event being estimated as 116% of what a 100-year event would be defined as using the 1974-1991 series. The greatest increase in event magnitude for Corofin comes for more frequent events of shorter return period. For the period 1987-2009 a 2-year event increases by 14% from that estimated from the 1964-1986 series, a 5-year event increases by 10% and a 100-year event only increased by 5%.

4.3.1 Discussion of Results

Statistical analysis of the annual maximum distribution series at each hydrometric station showed that magnitudes estimated by Gumbel were slightly larger than those estimated by Gringorten for shorter duration return periods and slightly smaller for return periods of longer duration. The comparison of the plotted data with the lognormal distribution fitted to them by the frequency factor method shows that the fitted line is relatively consistent with observed data (Appendix C–5). The values estimated from EV1 Distribution using Gumbel are expected to be the most accurate and are the values used in estimating standard error and associated confidence intervals. This method estimated the 100-year event at Ballyhaunis as 6.44 m^3/s, Ballygaddy as 109 m^3/s, Corofin as 185 m^3/s and Claregalway as 181 m^3/s. The most notable aspect of the predictions is the decrease in the magnitude of a 100-year event from Corofin to Claregalway. This is particularly significant due to the large area drained to the Clare River by the Abbert River between the two locations. This area covers approximately 240 km^2 (22% of catchment). There are a number of factors that could potentially contribute to this anomaly in varying degrees. The data set at Claregalway is shorter and is not expected to provide a sufficient analysis of a 100-year event. However it is still expected to provide a reasonably good indication of flood magnitude. Also the fact that the November 2009 floods were estimated as being at least a 100-year event at Corofin and Ballygaddy suggest that the peak flow of November 2009 would have been a 100-year event at Claregalway also. This would imply that the 100-year flow estimated by statistical analysis is an overestimation by 18 m^3/s. The gauge at Claregalway is situated on the downstream face of the bridge. The bridge acts as a hydraulic constraint in times of high flow as explained in section 3.1.7. Some of the floodwaters also pass around the gauge on the floodplain in particularly extreme events. This reduces the peak flow estimate at Claregalway thus reducing the estimated magnitude of the 100-year event. However the most significant factor in producing lower flow estimates at Claregalway is due to the groundwater leakage from the catchment to the west as explained in section 2.2.

Estimates of the return period for the November 2009 event were lower at Ballyhaunis and Claregalway. This is most likely due to the short duration of the distribution series. Statistical analysis of data from Ballygaddy and Corofin suggest that the November 2009 event was a 100-year event. Therefore the maximum-recorded flow at Claregalway of 163.2 m^3/s is most probably a 1 in 100 year flow. However it is suggested that using the estimated 100-year flow of 182 m^3/s would provide a safety factor in flood defence design.

Following the analysis of the return periods of historical flood events the 2009 flooding does not appear to be part of a trend of more severe flooding in the Clare River catchment in recent times. The historical flood events described in chapter 3 are all far less statistically significant than the 2009 event apart from November 1968 floods recorded at Corofin. This would suggest that the November 2009 event was just an isolated extreme event. However analysis of the distribution series at Ballygaddy and Corofin suggest that there has been an increase in the magnitude of flood events in the latter half of both distribution series. The magnitude of a 100-year flow at Ballygaddy is 16% greater for the latter half of its distribution series while at Corofin it is 5% greater. It is thought that if the annual maximum series at Ballygaddy extended back to the 1968 flood event recorded at Corofin that the increases in the magnitude of the 100-year flow estimate would be considerably less. The increase of 5% at Corofin is also thought not to be significant enough to conclude that there is an increase in more severe events. Therefore, from analysis of the data, recent flood events do not necessarily indicate an increase in the frequency and magnitude of flood flows along the Clare River in more recent times.

Chapter 5
Climate Change and Rainfall

5.1 Climate Change

Global Warming is the phenomenon by which changes within the composition of the Earths atmosphere result in an increase in the Earths climatic temperature. The warming/cooling influence that a factor exhibits on climate is referred to as radiative forcing. These influencing factors include greenhouse gases (GHG's), aerosols, solar activity, land surface use etc. Greenhouse gases (GHG), such as CO_2 and methane, have been put forward as the main driver of climate warming in the last century. Carbon dioxide (CO_2) increases are mainly due to an increase in fossil fuel usage and altering land-use i.e. deforestation. CO_2 has risen from a pre-industrial level of 280 ppm to 379 ppm in 2005. This is higher than at any time in the past 650,000 years when levels remained between 180 ppm and 300 ppm. The natural range for methane over the past 650,000 years had been 320 ppb to 790 ppb. However methane levels have been seen to increase from pre-industrial levels of 715 ppb to 2005 levels of 1774ppb [14]. This is due to the influences of a number of factors such as industry and more intensive agriculture. These GHG's are the main driving force behind the subsequent greenhouse effect. This is the process by which atmospheric gases absorb radiative energy leaving a planetary surface. This results in the energy being retained within the atmosphere resulting in an increase in the average Earths air and ocean temperature. While this process is a necessary element of the Earths cycle an increase or decrease in its rate of operation can lead to extreme shifts in natural cycles and ecosystems with significant results for the effected population [14].

There are varying opinions relating to the causes and effects of climate change. The earth is constantly experiencing climatic cycles. These have resulted in periods of cooling and warming throughout history. Scientists have been able to determine the occurrence of such climatic cycles from the analysis of ice cores that contain air samples frozen within their voids. More recently experts have become increasingly worried about the impact of human actions on these natural cycles. A hypothesis raised by William F. Ruddiman suggests that our ancestors kicked off global warming thousands of years ago with CO_2 starting to rise 8,000 years ago and methane 5,000 years ago [15]. Agriculture, deforestation and crop irrigation were some of the most likely causes for this. The fact is if it were not for this rise in GHG levels when they should have been dropping due to orbital influences temperatures could have been 3°C to 4°C colder [15]. This may seem to promote the benefits of influencing the Earths climate. However the fact that our ancestors may have been able to affect the earth's climate so significantly with primitive technologies suggests that we have every reason to be concerned with the relatively recent and unprecedented increases in GHG's attributed to advances made since the industrial revolution. Sceptics of anthropogenic warming have proposed alternative theories as to the cause of the recent variation in the Earths climate such as the theory of cosmoclimatology that aims to attribute the increase to changes in the cosmic ray flux [16]. While such a theory appears to correlate to observations over a geological time scale it does not correlate with shorter millennial cycles. Anthropogenic forcing still appears to be the most logical explanation for the post industrial revolution increases in temperature.

The effects of these driving forces is a matter of increasingly urgent concern as increasing air and sea temperature, rise in sea levels and reduction in global ice and snow mass is being observed. There has been a notable upsurge in the rate at which climatic temperature is increasing since pre-industrial times. Since the beginning of temperature recording in 1850 eleven of the twelve warmest years on record fell within the twelve years preceding the IPCC's Fourth Assessment Report (AR4) [14]. There has been a total temperature increase of 0.76 °C since pre-industrial times, which will continue to rise even if we were to cease our fossil fuel consumption, due to the lagging effect of the world's oceans. Climatic changes such as higher temperatures and wind patterns have been linked to the increased intensity observed in droughts since 1970. A reduction in frost and cold weather and increase in warm weather has been observed. There has also been an increase in intense tropical cyclone activity. Tropical storm tracks are expected to migrate towards the poles as a result of climate change thus moving existing rainfall patterns away from the equator with obvious effects regarding drought, flooding etc. Increase in the frequency of heavy precipitation events has been observed corresponding to higher levels of atmospheric water vapour due to increased air temperature [14]. It is this aspect of climate change and its subsequent impact on flooding in Ireland that this report is concerned with.

5.1.1 Predicting Climate Change

The predictions of future climate changes are carried out using mathematical models. The Earths climate is modelled using Global Climate Models (GCM). These models are based upon physical principles including fluid dynamics, thermodynamics and radiative transfer. They are generated from physical laws such as Newton's second law of motion. These laws are subjected to physical approximations that are determined to be suitable for the global climate system [14]. These models attempt to consider all factors associated with future climate change. However due to technological constraints and limitations in knowledge relating to the climate system (i.e. future quantity and impact of climate driving factors) there are uncertainties relating to assumptions made. This section looks at the reliability of these climate models and their accuracy in predicting future changes in precipitation events and subsequent flooding.

Global Climate Models

Modern climate models combine models for different elements of the Earth. Atmospheric models predict future behaviour of atmospheric properties such as air movement, temperature and clouds. Ocean models predict ocean currents, salinity and temperature. Other models include models of ice cover and models of heat and moisture transfer from soil and vegetation to the atmosphere. The interconnection of these models produces a climate model. This climate model also considers effects from anthropogenic forcing. The production of GHG's is a key factor in determining future climate. The accuracy of these climate models relies upon the accuracy of each of these different elements.

The IPCC have played a large role in evaluating the performance of these climate models. Developments made relating to model formulation have resulted in improved consideration in relation to the effects of driving factors such as aerosols, terrestrial processes and oceanic interaction with climatic conditions. The analysis methods utilised by the IPCC in evaluating the models for the purpose of the AR4 report involve controlled experiments being carried out by eighteen modelling groups and the subsequent results being scrutinised by hundreds of researchers. Weather forecasts can be produced and assessed on a regular and relatively short time scale. This enables statistical analysis of the performance of forecasting models to determine their reliability relatively quickly. However climate change models aim to make projections about climatic trends over longer time scales that are at least decades in duration. It is therefore much more difficult and time consuming to evaluate their performance effectively. The methods of evaluating these models include model intercomparisons and testing models against past and present climate. Comparison with past and present climate enables a certain amount of confidence to be gained in climate models as they can be compared with observed data for wide variations in atmospheric and oceanic variables from both recent records and paleoclimatic data. However past climatic trends contain no precise correlation with future climate variables. This limits the reliability of evaluation of these climate models [14].

The prediction of future climatic trends also depends heavily on the levels of GHG's within the atmosphere. The variation between different scenarios can lead to large variations in climatic predictions. The IPCC's Special Report on Emission Scenarios (SRES) produced a range of alternative scenarios that may arise in the future depending upon a range of factors including economic, societal, legislation etc. The SRES was produced in conjunction with the IPCC's Third Assessment Report (TAR) and replaced the IS92 scenarios that accompanied the second assessment report. These scenarios aim to make predictions relating to human activity that greatly influences climate change. These predictions relate to technological and economic development and its impact on driving factors of climate change such as GHG's and land use.

There are 40 scenarios in total. These scenarios can be broken up into families that display common themes. The A1 scenarios are of a more integrated world in which there is widespread social and cultural interaction. They are described by rapid economic growth coupled with a rapid spread of efficient technology. The A1 family assumes a global population that reaches 9 billion in 2050 and then gradually declines. Subgroups include A1FI (focused on fossil-fuels), A1B (balanced use of all energy sources) and A1T (focused on non-fossil energy sources). The A2 scenarios are of a divided world. This is typified by the independent operation of natures that results in a slower uptake of new and efficient technology, and a constantly growing population with focus only put on economic development at a national level. B1 and B2 families are similar to A1 and A2 families respectively in their integrated and divided world assumptions. The B families are the same as the A families except that they also put an emphasis on more ecologically friendly methods. The generation of such a wide range of scenarios shows the inherent difficulties in predicting the impact that human activities will have on climate change due to the non-linear behaviour of human activities.

Climate models have been shown to reproduce observed climatic features. There is a high confidence in the ability of Atmospheric-Ocean General Circulation Models (AOGCM) to predict future climatic trends especially on a larger scale [14]. Confidence in these models predicting climatic variables such as temperature is greater than it is in predicting precipitation events. An investigation into the performance of 18 AOGCM's in predicting daily precipitation intensity was carried out [14]. This evaluation found that most models produced too many days of light precipitation (< 10 mm per day), too few heavy precipitation events (≥ 10 mm per day) and too little precipitation in these heavy events. The assessment found that these errors tended to cancel each other out to provide a relatively accurate average seasonal precipitation. As it is heavy precipitation events that are the key factor in flood events the evaluation of the climate models would suggest that in future extreme events could potentially be more significant than predicted. Simulation of extreme precipitation is heavily reliant upon the resolution of the model and parameters used. A higher resolution produces a more realistic prediction of daily precipitation. Evaluation of Global Climate Models contained within the IPCC's AR4 suggests that unreliability is still present throughout climate models in relation to predicting the magnitude and frequency of extreme precipitation events accurately. It seems that insufficient knowledge about variables such as human emissions and natural influences such as soil moisture feedback may be too complex to overcome in the foreseeable future [14].

5.1.2 Climate Change and Flooding

Organisations such as the OPW have become increasingly aware of the importance in understanding the influence of climate change on future flooding. Predictions made within the IPCC's AR4 report that are of particular importance to the OPW would be [14]:

- A rise in global mean sea level of between 0.18 m and 0.59 m over the 21st century, with further rises expected beyond this.
- More frequent heavy precipitation events, particularly in high-latitude areas, such as Ireland

The expectation is that even within a nation as small as Ireland there will be significant regional variations. The east and southeast are expected to experience the greatest effects of the drier and warmer summers while the most significant increases in winter precipitation are expected in the west and northwest. It is also the opinion of the OPW that the varying characteristics of different catchments cause them to respond differently to changes in rainfall intensity and frequency.

A review of the national flood policy was carried out in 2003 [17]. This review assessed varying aspects of the flood policy and included evaluation of the potential causes of flooding, suggestions of any improvements in methods and policy that should be implemented and also examination of the potential impact of climate change on flood events. The flood policy review group met on nine occasions throughout 2003 completing the report by December of that year. Within the scope of the report the issue of climate change is discussed. It is suggested that the compound effect of development and climate change both increasing flows by 20% could result in a 100-year event occurring approximately every 10 years and an increase in average annual flood damages by 20 to 30 times [17]. This scenario is thought to be extreme but highlights the compound effect that different factors could have on flooding. It follows that flood protection measures would be best suited to incorporate such effects into their initial design so as to prevent costly future investment to increase their storage capacity due to an increasing magnitude of high flows. The report suggests that should flows increase by 20% as a result of climate change that flood defence measures with an existing level of protection of 100 years would be approximately reduced to protection from 30-year events.

The issue of the impact of climate change on flooding in Ireland has been considered in research carried out by the EPA [18]. NUI Maynooth prepared a report as part of the Environmental Research Technological Development and Innovation Programme 2000-2006. The report was carried out prior to the IPCC's Fourth Assessment Report (AR4) and therefore the Third Assessment Report (TAR) provided the basis of international research into the effects of climate change at the time. Within the international context there had been an observation that precipitation had increased over landmasses in temperate regions by 0.5% to 1% with the frequency of intense rainfall events in the northern hemisphere also appearing to be increasing. The TAR projections made indicated an increase in precipitation in mid to high latitudes particularly for the winter months [18]. These global projections for climate change have obvious effects for Ireland in a regional context. It is proposed that rainfall in Ireland will increase for the months of December to February. The significant portion of this increase is projected to occur in the northwest of the country [18]. With the east coast expected to experience little change it becomes obvious that the greatest impact on flooding from climate change will be experienced in western Ireland. While a reduction in the annual runoff is expected there is an expected increase in winter runoff and also in the magnitude and frequency of individual flood events in western Ireland [18].

To understand how results were attained during the study it is important to review the methods by which they were achieved. The assessment involved the downscaling of the Global Climate Models (GCM) that were discussed in section 5.1.1. The study involved the downscaling of the HadCM3 model, chosen due to practical considerations and the high degree of sophistication of the particular climate model. The technique of downscaling involves the translation of the relatively coarse grid of the GCM into a finer spatial scale. This enables the input of much more regionally significant information such as land type, catchment characteristics etc. There are a number of different methods to approach the task of downscaling GCM's. The EPA report involves a statistical downscaling technique. It incorporates meoscale predictor variables by establishing a correlation between the GCM output and surface observations. A key assumption on which the technique is based is that GCM's simulate meoscale aspects of climate more accurately than surface variables such as temperature. By establishing a link between upper atmosphere variables and local surface observations a link may be established which is assumed to be robust in a changing environment. These upper air variables are generated as an output of the GCM's. This provides a starting point from which to generate local surface variations in a changing climate by employing the relationship resulting from the analysis of observed data. The resolution of the regional climate provided satisfactory accuracy for Ireland's varied topographical features. Monthly climate data was used for the period 1961-1990 to build a baseline climate. This is the usual 30-year time period employed for such climate studies. This included data from 560 stations for precipitation. Certain problems existed in relation to availability of data to establish the climate baseline with a scarcity of weather stations measuring both incident solar radiation and potential evapotranspiration. The downscaling model was run for three separate time periods 1961-1990, 2041-2070 and 2061-2090. The differences between the 1961-1990 model and the other models could then be added on to the established baseline climate thus producing a projected climate for Ireland for 2041-2070 and 2061-2090. Accuracy of the model was undertaken for the period 1991-1997 comparing predicted with observed data. The verification of temperature was particularly good while predictions made relating to precipitation were less accurate as had been expected by the research team. Validation statistics for the different climatic aspects are shown below. The low level of certainty in relation to predictions regarding precipitation is apparent with a root mean square error in the region of 24 mm to 49 mm.

Table 5.1 – Validation Summary Using an Independent Dataset for the Period 1991-1997 [18]

Downscaled variables	Range of monthly values of Pearson's 'r'	Mean average error	Root mean square error
Maximum temperature	0.23–0.94	0.04°C	0.87°C
Minimum temperature	0.54–0.92	0.03°C	0.83°C
Precipitation	0.36–0.85	0.29–30.02 mm	24.24–48.72 mm
Radiation	−0.13–0.63	0.35 MJ day^{-1}	1.12 MJ day^{-1}

As a result of the study it was estimated that Ireland would experience increases in winter precipitation of 11% with the greatest increases coming in the northwest expected to be in the region of 20% by approximately 2050. An increase of 15% was projected in winter precipitation for the uplands of the southwest. However the report makes recurring reference to the unreliable nature of precipitation predictions made due to the inherent difficulties associated with GCM's.

There have been a number of studies carried out aiming to evaluate the correlation between climate variables and precipitation and subsequent runoff. Increases in annual precipitation and stream flow were observed by Kiely (1999) [18]. The increase in westerly winds was proposed as one of the driving factors behind the increase in flood events in Dublin in the second half of the 1900's by Sweeney (1997) [18]. Cunnane and Regan (1991) carried out a projection of future water resources, taking into account climate change, on the River Brosna for the year 2030 [18]. The study concluded that even though there would only be a relatively small increase in the magnitude of maximum and minimum flows that there would be a noticeable increase in the frequency of both flooding and drought. Further to the estimation of future precipitation the EPA report carried out a study on the potential runoff generated by such events [18]. This employed the hydrological simulation model HYSIM. Certain data was input in conjunction with the projected precipitation such as soil type, land use and channel characteristics. The model was set up on a grid basis. This gave rise to certain difficulties due to the non-catchment based approach. Validation of the model was carried out on a number of catchments of varying characteristics. The degree of accuracy of the model is shown in table 5.2 with the Shannon and the Bonet possessing the greatest degree of inaccuracy. It should be noted that this study focused on no catchments located in western Ireland. This is despite research suggesting that the increased precipitation associated with climate change will have a more significant effect in western areas.

Table 5.2 – Predicted and Observed Values of Annual Effective Runoff for Validation Catchments [18]

Effective runoff	Feale	Suir	Slaney	Shannon	Brosna	Bonet
Predicted (mm)	1058.93	617.27	566.55	645.86	475.88	950.12
Observed (mm)	1070.69	697.00	565.63	787.97	441.82	1232.20
% error	−1.10	−11.44	0.16	−18.03	7.71	−22.89

The results of the model indicated an increase in surface water runoff in the region of 10% due to climate change alone in the western half of the country for 2041-2070 during the wetter winter months. This would have significant consequences within catchments that already have a history of winter flooding. This predicted runoff increases to greater than 10% for the west and northwest for the period 2061-2090. While the report failed to make conclusive judgements in relation to flooding it did note that the increase in winter runoff especially in the period 2061-2090 was likely to have significant implications in relation to flood events. Most flooding occurs during the winter months when soil saturation levels are at their highest. Therefore an increase in runoff would contribute to increased flood risk during this time of year. These projected increases in both precipitation and corresponding runoff accompanied by expected increases in both the magnitude and frequency of intense precipitation events during the winter months indicate that an increase in the likelihood of flooding along with an increase in the extent of inundation should be expected.

The opinion that flood risk will increase as a result of changing climate is also contained within the report "Ireland in a Warmer World" [19]. Within the scope of the report an evaluation on the impact of climate change on the hydrology of Ireland was carried out. The study assessed nine catchments for a reference period of 1961-2000 and a future period of 2021-2060 considering the emission scenario SRES-A1B. The catchments studied are shown in Figure 5.1.

Figure 5.1 – Location of Study Catchments for Met Eireann & UCD Study [19]

The study is based on the downscaling of GCM's and does not address the unreliability associated with this data, which mainly pertains to precipitation. The study concluded that for the A1B scenario that there was a general increase in winter precipitation and decrease in summer precipitation. In all catchments the greatest increase in rainfall was expected in January ranging from an increase of 0.62 mm/day to 1.56 mm/day. The greatest increase was predicted for the catchments of the Bandon and the Feale situated in the southwest with the southeast projected as containing the driest catchments. Subsequent to validating the projected precipitation the impact of expected climate change on the hydrology of the nine catchments was analysed. This resulted in an expected decrease of 60% in stream flow from May to September and an increase in expected stream flow of 20% from October to April. There is a higher degree of certainty relating to the winter predictions as at this time of year soil is close to saturation and evaporation is low due to the lower temperatures. There is therefore a greater deal of confidence relating to the projected changes in winter flow. The Blackwater and Bandon catchments were deemed to be at greatest risks from the changing hydrology as what may previously have been considered a 40-year event would have a return period of approximately 10 years for the period 2021-2060. The Moy and the Suck, which are located closest to the Clare River System, produced mixed results with some events of given magnitudes predicted to possess longer return periods in the future. These catchments are characterised by damped, even hydrographs and would therefore respond to increases in precipitation over a longer time scale than would be associated with faster responding catchments. Overall it appears that the research suggests an increase in precipitation and subsequent runoff and stream flows. However these projections cannot be made with any great deal of certainty due to the uncertainty of predictions of future precipitation.

The OPW recognises the importance of factoring future changes into flood risk management plans and produced the guidelines 'Assessment of Potential Future Scenarios for Flood Risk Management' [20]. There are varying approaches outlined within the guidance document aimed at ensuring proper consideration of climate change in addressing flood risk. The assumptive approach assumes that there will be a certain degree of impacts as a result of climate change. The assumed degree of impact is incorporated into future flood related measures such as flood risk assessments and flood risk management strategies. The adaptive approach incorporates a capacity for adaptation in any flood strategy, plan or measure. This allows for these measures to be designed and implemented accounting for existing flood risk with the flexibility to change to account for increased flood risk due to climate change. This approach is deemed suitable for application to the design and implementation of strategies, plans and measures. The assumptive approach should be applied in the event of the adaptive approach not being appropriate, technically feasible or cost effective. The sensitivity-based approach considers the potential increase in flood risk due to influences of climate change in the future based on one or more scenarios. This approach is deemed most suitable for flood hazard/risk assessment and the development and assessment of flood strategies, plans and measures. No-physical provision is the final alternative. It does no make any provision for future climate change impacts. This measure is only deemed suitable for measures that serve to reduce current flood risk such as flood defence measures. The application of the assumptive, adaptive and sensitivity-based approach requires an estimation of the potential impacts associated with future climate change for varying scenarios. The OPW suggest that a minimum of two potential future scenarios should be considered [20]:

1. Mid-Range Future Scenario (MRFS)

   'This scenario is intended to represent a 'likely' future scenario, based on the wide range of predictions available and with the allowances for increased flow, sea level rise, etc. within the bounds of widely accepted projections' [20].

2. High-End Future Scenario (HEFS)

   'This scenario is intended to represent a more extreme potential future scenario, but one that is nonetheless not significantly outside the range of accepted predictions available, and with the allowances for increased flow, sea level rise, etc. at the upper the bounds of widely accepted projections' [20].

The allowances for both of these scenarios are shown in table 5.3. It shows that for the more extreme scenario (HEFS) there will be an increase in extreme rainfall depths and subsequent flood flows of 30%. Assuming the more probable scenario (MRFS) results in an increase in flood flows of 20%. This 20% increase corresponds to predictions based upon the 'Report of the Flood Policy Review Group' [17], the EPA study 'Climate Change: Scenarios and Impacts for Ireland' [18], and 'Ireland in a Warmer Climate' [19].

Table 5.3 – Allowances for Future Scenarios (100-year time horizon) [20]

	MRFS	HEFS
Extreme Rainfall Depths	+ 20%	+ 30%
Flood Flows	+ 20%	+ 30%
Mean Sea Level Rise	+ 500 mm	+ 1000 mm
Land Movement	- 0.5 mm / year[1]	- 0.5 mm / year[1]
Urbanisation	*No General Allowance – Review on Case-by-Case Basis*	*No General Allowance – Review on Case-by-Case Basis*
Forestation	- 1/6 Tp[2]	- 1/3 Tp[2] + 10% SPR[3]

Note 1: Applicable to the southern part of the country only (Dublin – Galway and south of this)

Note 2: Reduce the time to peak (Tp) by a third: This allows for potential accelerated runoff that may arise as a result of drainage of afforested land

Note 3: Add 10% to the Standard Percentage Runoff (SPR) rate: This allows for increased runoff rates that may arise following felling of forestry.

5.2 Key Precipitation Indicators

The EPA produced a report relating to key meteorological indicators of climate change in Ireland [21]. The methods outlined within the EPA report will be used to assess the data gathered for the Clare River Catchment. On a global status average annual precipitation over land areas has increased from 11 mm to 21mm from 1901 to 2004 [21]. However there are regional differences in both the spatial and temporal distribution of this increased precipitation. Evidence from Europe and the USA suggest that there is a disproportionate increase in heavy and extreme precipitation events relative to the total precipitation amount [21]. It is therefore necessary to strategically analyse data relating to the Clare River Catchment due to the regional variations in precipitation trends. The key precipitation indicators as outlined within the EPA report are outlined below [21]:

- A 10-year moving average provides a good indication of any dominant trend in the magnitude of rainfall.
- Evaluating the location of the wettest and driest years in time and comparison of annual precipitation to the mean rainfall also provides an indication of whether there has been an increase or decrease in the annual quantity of precipitation.
- The number of heavy and extreme precipitation events shows the frequency of heavier precipitation events. Extreme events produce the greatest damage and effect on the local population. For the purpose of this study precipitation thresholds will be defined as follows

  Wet Days – days with precipitation $\geq$ 1 mm
  Very Wet Days – days with precipitation $\geq$ 5 mm
  Heavy Precipitation Days – days with precipitation $\geq$ 10 mm
  Extreme Precipitation Days – days with precipitation $\geq$ 50 mm
- The maximum number of consecutive wet days provides an indication of the persistence of rainfall events.
- Greatest 3-day, 5-day and 10-day rainfall totals are important from the perspective of flooding and the impact on the local population and environment.

5.3 Precipitation Data from Clare River Catchment

Synoptic stations record meteorological elements on an hourly basis, such as air temperature, rainfall, humidity, wind speed, wind direction etc. Climatological stations record meteorological elements on a daily basis, such as rainfall and temperatures. Rainfall stations record daily rainfall amounts at 0900utc. There are neither synoptic nor climatological stations present within the Clare River catchment. There are a number of rainfall station records from within the catchment. A summary of these stations and extent of their records are shown in table 5.4.

Table 5.4 – Summary of Rainfall Stations Located in Clare River Catchment

Station Number	Station Name	River Catchment	Latitude	Longitude	Grid Reference	Height (m)	Year Opened	Year Closed
927	BALLYGLUNIN HSE.	ABBERT-CLARE	532530	84840	M461420	37	1946	1961
2127	BARNADERG G.S.	GRANGE-CLARE-L.CORRIB	532840	84320	M521478	61	1941	1988
4327	BELCLARE (AGR.RES.STN.)	CLARE-L.CORRIB	532800	85755	M359467	44	1977	1998
2927	CASTLE HACKET	CLARE-L.CORRIB	532950	85800	M359501	43	1943	1975
2027	COROFIN G.S.	CLARE-L.CORRIB	532610	85150	M426432	34	1941	1991
1327	DUNMORE G.S.	SINKING-CLARE	533710	84400	M515635	61	1941	1991
3127	GLENAMADDY (GORTNAGIER)	SINKING-CLARE	533610	83340	M629616	84	1944	
527	GURTEEN G.S.	ABBERT-CLARE	532150	83510	M610350	96	1941	1953
1827	KILCONLY G.S.	CLARE-L.CORRIB	533420	85900	M349585	46	1941	1998
2327	LAGHTGEORGE G.S.	CLARE-L.CORRIB	532120	85550	M380343	14	1941	1998
3027	MILLTOWN	CLARE-L.CORRIB	533643	85331	M410628	50	1944	
327	MONIVEA (FOR.STN.)	ABBERT-CLARE	532230	84145	M537363	82	1951	1952
1127	TUAM (AIRGLOONEY)	CLARE-L.CORRIB	533130	85230	M420531	34	1941	1981
4727	TUAM SUGAR FACTORY	CLARE-LOUGH CORRIB	533150	85230	M420538	37	1981	1985

As can be seen from the altitudes of each rainfall station the catchment is a relatively even landscape. For the purpose of this study rainfall station number 3127 at Glenamaddy (Gortnagier) and rainfall station number 3027 at Milltown were chosen to analyse their data. The reason for choosing these rainfall stations was due to the fact that both stations were currently in operation and would therefore allow for analysis of data incorporating the recent extreme events such as those that occurred in 2009. They also provided data as far back as 1944 thus providing a significant period of time. This would allow conclusions to be made with a reasonable level of confidence. They are also both situated relatively centrally within the catchment thus providing a good indication of rainfall throughout the catchment. It is thought by carrying out analysis on the data provided from these stations that a reasonable opinion relating to climatic rainfall and the potential impact of climate change on records up to 2009 can be obtained.

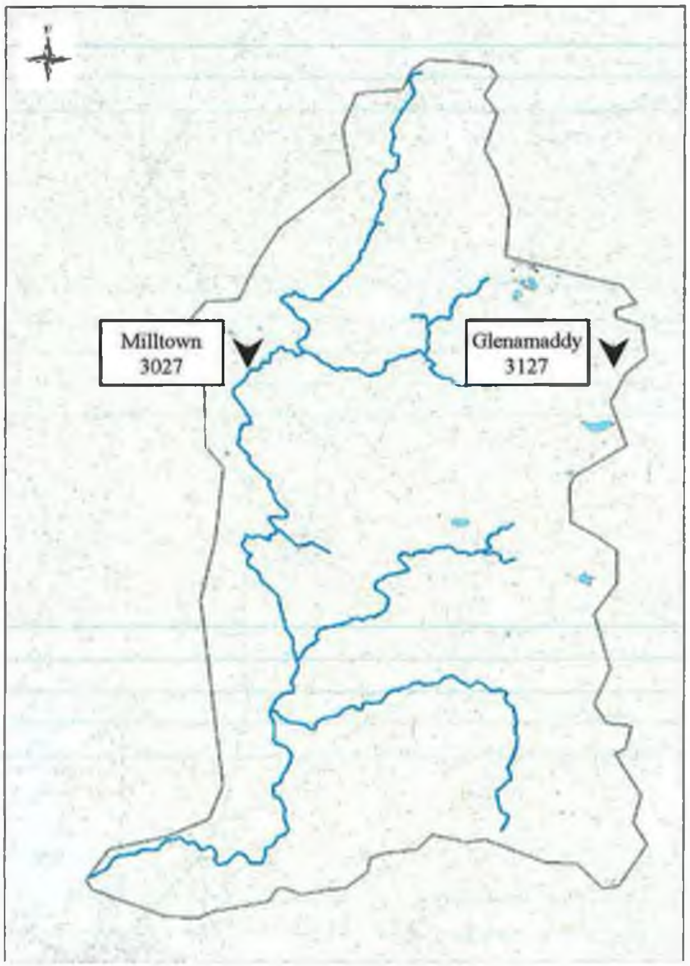

Figure 5.2 – Location of Rainfall Stations used in Study

5.4 Analysis of Precipitation Data

Prior to commencing the analysis of the rainfall data it was necessary to carry out a qualitative assessment of the available data. Rainfall data from both Glenamaddy and Milltown began in 1944 and was available up to the end of 2009. Due to the rainfall network being operated mainly by voluntary observers there are occasions where daily observations are missed. A cumulative value is entered for such situations. The observation is flagged as a cumulative total. The cumulative totals are redistributed across the preceding days of missed observations. Interpolating records from nearby stations provides an estimate of the ratio of redistribution. The cumulative rainfall is distributed according to ratio between the cumulative total and the interpolated total for the days in question. This process was carried out by interpolation between the available records from each station, as they are located relatively closely.

Due to gaps in rainfall records at Glenamaddy and Milltown it was necessary to ascertain which years within the records contained complete records and were therefore suitable to be included within the study. Only years providing a complete data set (i.e. complete daily rainfall amounts or cumulative totals that can be redistributed as explained above) were used within the scope of this study. Subsequent to identifying the years to be included in the study the analysis of the data was carried out guided by the methods outlined in section 5.2.

5.4.1 Milltown

Data Quality

The vast majority of absent records from the rainfall station at Milltown was absent from the central portion of the data set. Thus a certain amount of information was available for circa 1950 and also for recent records. The years that were deemed sufficiently complete to be included within the study were 1945-1950, 1953-1964, 1966-1967, 1987-1989, and 2001-2008. It should be noted that the time periods of 1966-1967 and 1987-1989 were relatively short duration and were therefore unsuitable for certain elements of the analysis. A total of 31 years of adequate data was provided within the 66 years of records.

Data Analysis

Table 5.5 shows the average annual precipitation for each of the periods of time analysed. The averages are similar and thus show no significant trend as to whether annual precipitation is increasing or decreasing.

Table 5.5 – Annual Rainfall Averages

Years	Rainfall (mm)
1945-1950	1196
1953-1964	1166
1966-1967	1066
1987-1989	1145
2001-2008	1154

Analysis of global precipitation has revealed that annual precipitation has increased by nearly 1% per decade between 1901 and 2004 [21]. This does not appear to be the case from rainfall records available at Milltown, which shows a reduction in annual precipitation of approximately 3% from 1945 to 2008. Global precipitation projections do acknowledge the regional differences that exist in these global trends. These regional differences have already been highlighted for Ireland in section 5.1.2 with precipitation gradients from the wetter northwest to the drier southeast expected to become more pronounced.

Table 5.6 shows the wettest and driest years of the available data set from Milltown. While two of the 5 wettest years occurred post 2000 so too did two of the five driest years. A lack of comprehensive records is thought to have effected this aspect of the analysis significantly as the 1990's was a decade which contained a considerable number of very wet years (1994, 1998, 1999) [21] which could potentially have skewed the occurrence of the wettest years comprehensively towards the more recent years in the data set. However available data suggests that there can be no significant conclusion drawn from annual rainfall regarding climatic trends. However, as noted within section 5.1.1 climate change is thought to differ in its effects both spatially and temporally. The western half of Ireland is expected to experience an increase in winter precipitation and in the frequency and severity of heavy precipitation events during winter months [21].

Table 5.6 – 5 Wettest and 5 Driest Years for Milltown

5 Wettest Years		5 Driest Years	
Year	Rainfall (mm)	Year	Rainfall (mm)
1954	1503	1987	911
2008	1427	2003	930
2006	1383	2001	999
1988	1293	1966	1010
1960	1278	1953	1011

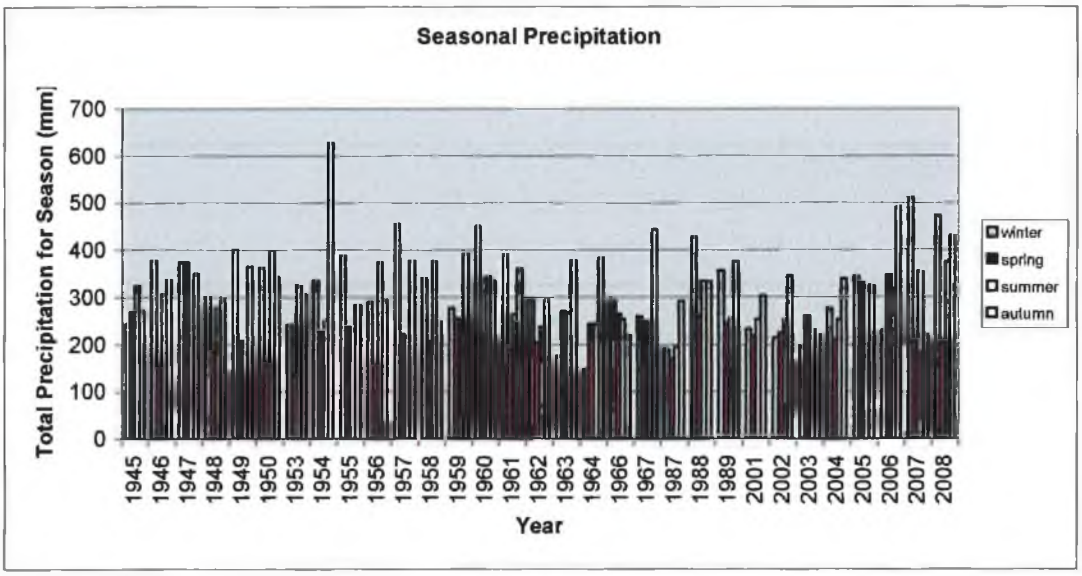

Figure 5.3 – Seasonal Precipitation at Milltown

Figure 5.3 provides a visual representation of seasonal precipitation. Individual graphs for seasonal precipitation are provided in Appendix E–1. Due to the fact that winter is not confined to one calendar year the season has been taken as January and February of the given year and December of the previous year for the purpose of this study, i.e. winter 2008 = December 2007 to February 2008. There are no obvious trends arising from analysis of these values. A considerable amount of precipitation fell during the autumn of 1954. Apart from this value the next three highest seasonal totals occurred in autumn 2007, winter 2007 and winter 2008. On closer analysis it does appear that winter precipitation has become slightly more erratic in recent years. This produces some winters with large total precipitation despite the observed decrease in the average winter precipitation as indicated by table 5.7.

Table 5.7 also provides standard deviation values for winter precipitation. The standard deviation of a data set is the square root of its variance. It is given by the formula:

$\{\Sigma [x_i-u]^2 / N\}^{1/2}$

where: x_i = data point i (i = 1,2 3…N)

u = mean of data set

N = number of points in data set

Standard deviation is a widely used measure of the variability of a data set. It shows how much variation there is from the average. A low standard deviation indicates that the data points tend to have a narrow range and are situated close to the mean. A high standard deviation indicates that the data is spread out over a wider range of values. From table 5.7 there is an obvious increase in the standard deviation and hence the variation of the data points from the mean in the more recent time periods. The period 1966-1967 does not fit the trend. This is expected from such a short period of time. The greater degree of dispersion from the mean is the reason why, even though a decrease has been observed in the average winter precipitation for more recent data sets, the most recent data set also contains relatively high winter rainfall totals with the two highest winter rainfall totals occurring in 2007 and 2008.

Table 5.7 – Winter Precipitation at Milltown

Years	Average Winter Precipitation (mm)	Standard Deviation
1945-1950	343.0	59.7
1953-1964	315.5	98.1
1966-1967	276.0	24.7
1987-1989	325.3	122.2
2001-2008	309.1	121.8

The number of wet days (≥ 1 mm precipitation) was calculated for each year. This was also done for very wet days (≥ 5 mm precipitation), heavy precipitation days (≥ 10 mm precipitation) and extreme precipitation days (≥ 50 mm precipitation). It was noted that extreme precipitation days only occurred at 4 times during the entire data sets with two of these occurring recently in December 2006 and December 2007. Table 5.8 shows the average values for each of the time periods.

Table 5.8 – Average Number of Wet, Very wet, Heavy Precipitation and Extreme Precipitation Days per Year for each Time Period

Years	Averages of no. of days per year for each time period			
	≥ 1mm (days)	≥ 5mm (days)	≥ 10mm (days)	≥ 50mm (days)
1945-1950	187.17	84.83	32.17	0.00
1953-1964	185.00	80.17	31.08	0.18
1966-1967	193.00	76.50	28.50	0.00
1987-1989	173.67	82.33	35.00	0.00
2001-2008	173.38	82.50	32.25	0.25

The number of wet days decreases over time. This reduction indicates that recent years have had fewer wet days than previous decades. This decline becomes less evident as the magnitude of the daily precipitation threshold increases. Analysis of the number of heavy precipitation days appears to show a reversal in the trend with a slight increase in the number of annual heavy precipitation days in more recent years. This reversal is further amplified in the analysis of extreme events with 2 of the 4 extreme precipitation days on record occurring between 2001-2008. These observations suggest that there has been a decrease in the number of days with low levels of precipitation and an increase in the number of days with higher levels of precipitation.

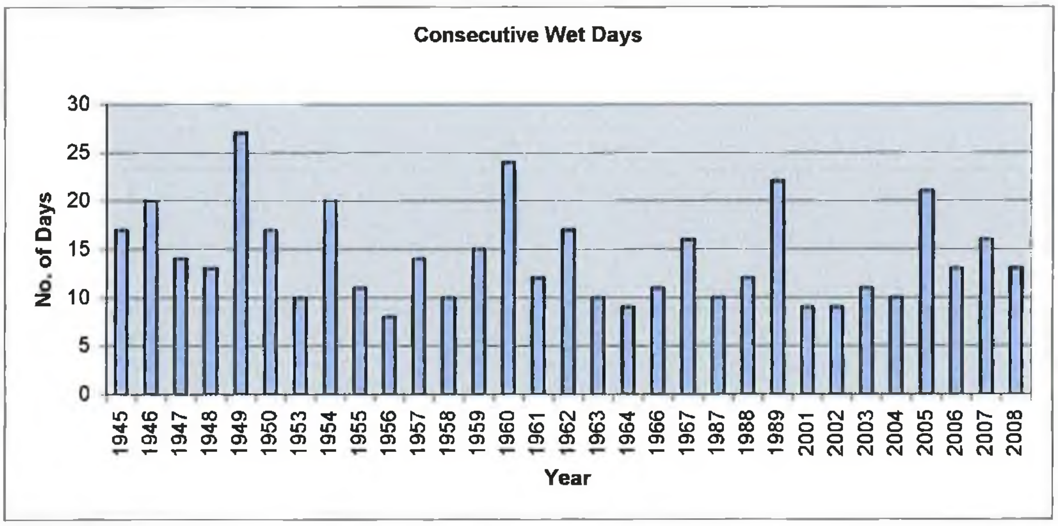

Figure 5.4 – Number of Consecutive Wet Days (≥ 1 mm)

The maximum number of consecutive wet days provides an indication of the persistence of precipitation events. Figure 5.4 shows these maximum values for each of the years studied.

These values appear relatively normal over time with occasional large values recorded. The large values in the early portion of the data set indicate rain of approximately 25 days duration. These large outliers tend to decrease to 20 days duration in the more recent portion of the data set. Despite this decrease in these occasional highs the mean tends to remain relatively constant for each complete time period at approximately 13 or 14 days apart from the period 1945-1950 which had an average maximum number of consecutive wet days of 18 days. If anything were to be inferred from this it would be that the duration of precipitation events has been slightly decreasing over time. The same process was carried out for very wet days ($\geq$ 5 mm rainfall) as shown in figure 5.5. Similar to the analysis of consecutive wet days no dominant trend was obvious from the graph of maximum annual consecutive very wet days.

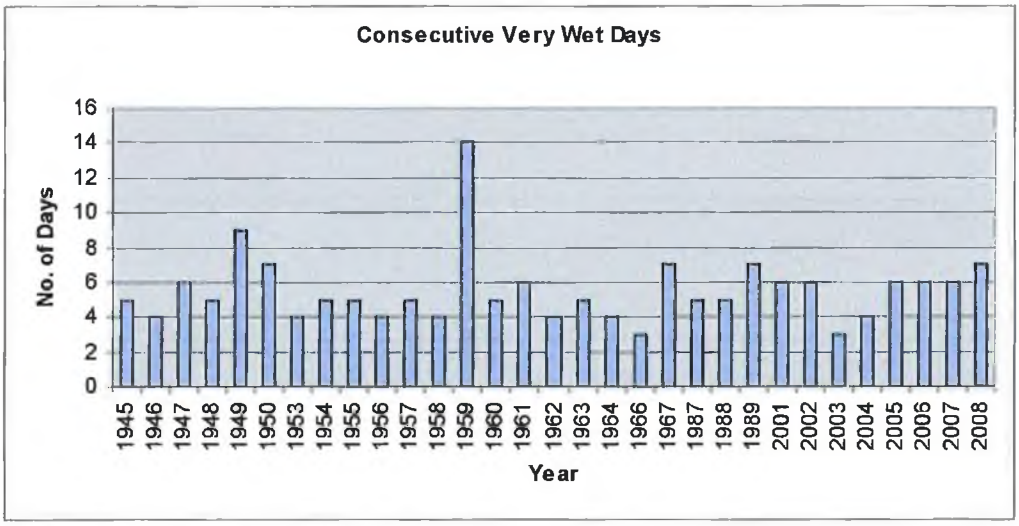

Figure 5.5 – Number of Consecutive Very Wet Days ($\geq$ 5 mm)

The greatest annual 3-day, 5-day and 10-day totals are deemed a key precipitation indicator in relation to flooding [21]. These annual maximums were calculated and an average was determined for each time period to evaluate whether there had been an increase in these values in the more recent years of the data set. The averages for each time period are shown in table 5.9. Both 1966-1967 and 1987-1989 have been omitted from this table due to their relatively short duration. It can be seen that there is an increase in the average of each of the 3-day, 5-day and 10-day totals from the initial time period 1945-1950 to the most recent time period 2001-2008. Percentage increases are also provided within table 5.9. The only stage at which there does not appear to be an increase in magnitude of these events over time is in the 3-day total from the period

1953-1964 to 2001-2008. The main reason for this is due to the considerably high rainfall in October 1954 that provided a 3-day total of 139 mm. This table suggests that there has been an increase in these values over the duration of the available records at Milltown. This is significant in that these totals are a particularly key indicator in the occurrence of flooding. An increase in their magnitude would infer an increase in the magnitude of flooding events. However it should be noted that rainfall records at Milltown were not continuous due to missing data as already highlighted. Therefore comprehensive conclusions cannot be made as to the likely impact of this on climatic trends.

Table 5.9 – Total Precipitation over 3, 5 and 10 days

Years	Average of Max. Rainfall Totals			% Increase from 1945-1950 Average		
	3-day (mm)	5-day (mm)	10-day (mm)	3-day %	5-day %	10-day %
1945-1950	52.0	64.5	102.6	0.0	0.0	0.0
1953-1964	64.7	76.9	108.8	24.5	19.2	6.1
2001-2008	64.3	79.8	113.2	23.7	23.6	10.4

5.4.2 Glenamaddy

Data Quality

The precipitation records at Glenamaddy were far more comprehensive than those at Milltown. There were 4 years in total between 1944 and 2009 that were deemed inadequate to be included in the study due to periods for which no data was available. These years are 1944, 1970, 1971 and 1979. Due to the more comprehensive data set available for Glenamaddy a 10-year running average was used for varying aspects of the data analysis. For 10-year averages that include years for which data was deemed inadequate as stated above the average was taken over the years of complete data located within that 10-year period. For analysis using a 10-year running average the results begin in 1954, 10 years after the first year of complete records (1945).

Data Analysis

Table 5.10 shows the average annual precipitation for decades of from 1950 up to 2009. It can be seen from the values that the average precipitation was greater during the earlier decades in the data set. This decrease in annual precipitation is displayed visually in Figure 5.6. The 10-year average can be seen to increase in the past 5 years, however the overall trend is one of decreasing annual precipitation.

Table 5.10 - Annual Rainfall Averages

Years	Rainfall (mm)
1950-1959	1112.20
1960-1969	1109.59
1970-1979	1006.69
1980-1989	1083.41
1990-1999	1068.70
2000-2009	1023.12

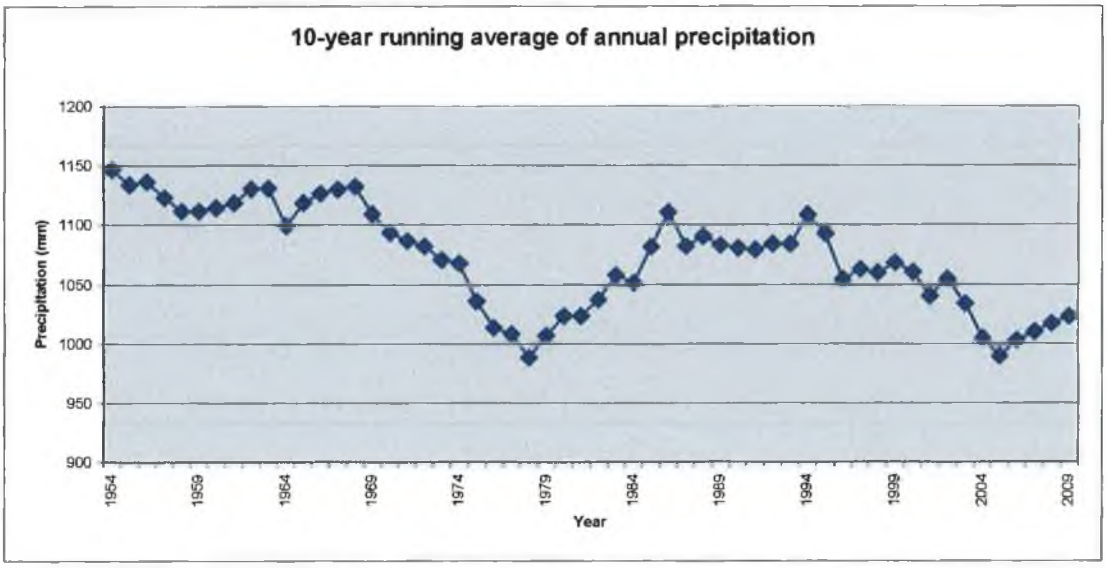

Figure 5.6 – 10-year Running Average of Annual Precipitation

The decreasing average for annual precipitation opposes the conclusions in relation to global trends that indicate an increase in annual precipitation by almost 1% per decade between 1901 and 2004 [21]. However the observed decrease in the average does not necessarily indicate a reduction in annual precipitation for individual years, as there may be considerably large annual precipitation values present within a data series with a relatively low average. Table 5.11 is a list of the 5 wettest and 5 driest years within the entire data set to provide an indication of when these recordings occurred.

Table 5.11 – 5 Wettest and 5 Driest Years at Glenamaddy from 1945 to 2009

5 Wettest Years		5 Driest Years	
Year	Rainfall (mm)	Year	Rainfall (mm)
1954	1416	2001	803
2002	1310	1987	861
1986	1273	1969	862
1994	1268	2003	874
1947	1256	1996	887

As was the case for data available from Milltown the wettest and driest years did not lean towards either end of the time scale with the location of the wettest and driest years dispersed relatively evenly throughout the data set. Upon evaluating the quantities of seasonal rainfall (Figure 5.7) the only season that displays a potential increase in rainfall quantities is winter. This increase in the 10-year running average occurs in the late 1990's and early 2000's. The years that are mainly responsible for this increase are 1994 and 1995 as shown in Figure 5.8. These winter seasons produced rainfall totals in the region of 500 mm over 70 mm greater than the third largest winter rainfall total. Analysis of annual and seasonal rainfall totals suggests that while annual precipitation is decreasing winter rainfall could potentially produce much larger values than climatic averages if winter rainfalls such as those experienced in 1994 and 1995 were to become more frequent.

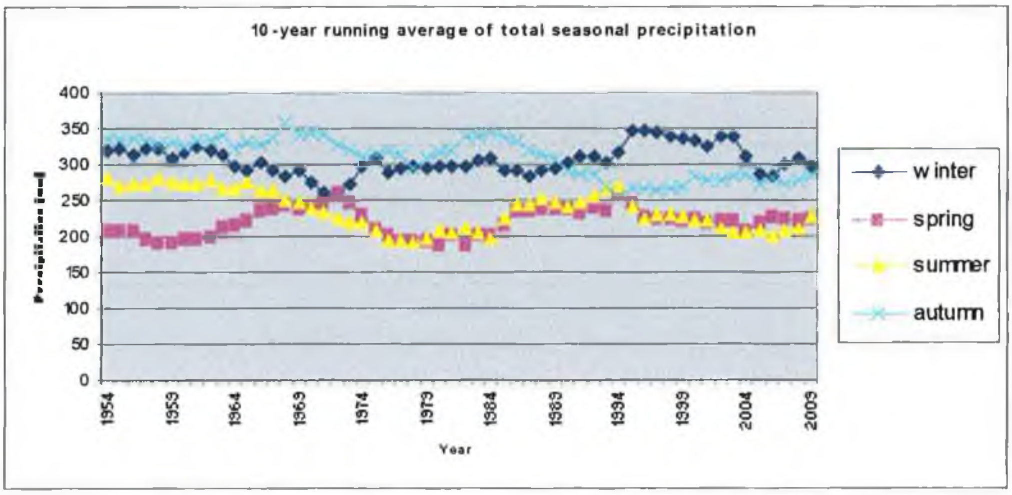

Figure 5.7 – 10-year Running Average of Total Seasonal Precipitation

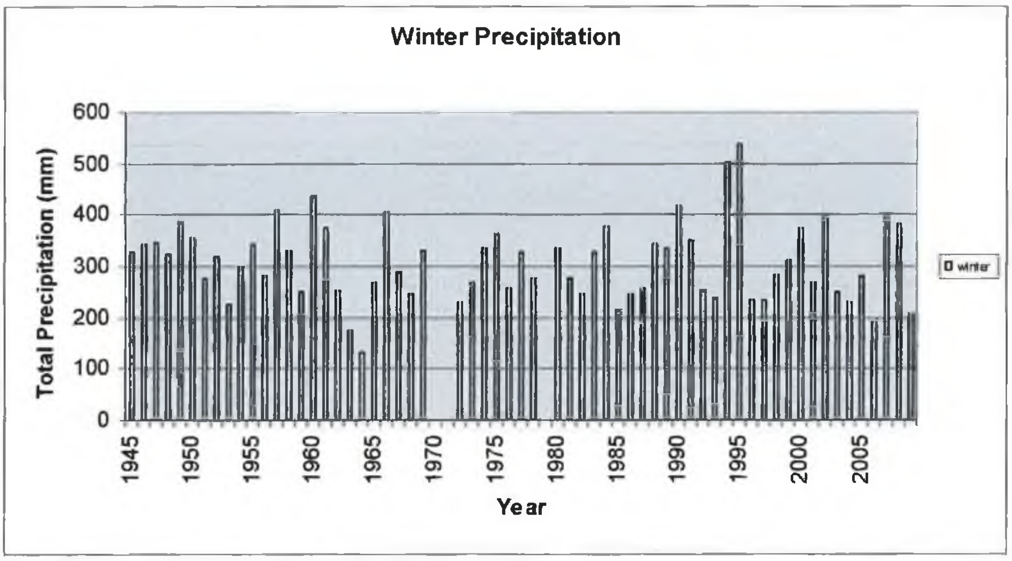

Figure 5.8 – Winter Precipitation Totals at Glenamaddy from 1945 to 2009

The number of wet days (≥ 1 mm precipitation), very wet days (≥ 5 mm precipitation), heavy precipitation days (≥ 10 mm precipitation) and extreme precipitation days (≥ 50 mm precipitation) was calculated for each year included in the study. A decrease was observed in the number of days associated with each threshold being surpassed over the duration of the data set. Table 5.12 shows the average annual number of days associated with each precipitation lower limit for decades from 1950 to 2009.

Table 5.12 – Average Number of Wet, Very wet, Heavy Precipitation and Extreme Precipitation Days per Year for each Decadal Period

Years	Averages of no. of days per year for each time period			
	≥ 1mm (days)	≥ 5mm (days)	≥ 10mm (days)	≥ 50mm (days)
1950-1959	178.80	76.80	30.40	0.10
1960-1969	177.20	78.50	29.30	0.20
1970-1979	176.00	72.29	22.57	0.00
1980-1989	179.10	79.00	28.00	0.10
1990-1999	177.70	74.80	29.20	0.00
2000-2009	178.00	72.50	27.00	0.20

The period 2000-2009 produces an average number of wet days similar to other decades within the study. There is no great variation across the decades for any of the precipitation thresholds. In the case of very wet and heavy precipitation days the decade 2000-2009 is among the lower values suggesting a possible decrease in the frequency of these events.

To evaluate the degree of persistence of rainfall throughout the data set the maximum number of consecutive wet days for each year being studied was calculated. The average for each decade is presented in table 5.13. The 10-year running average of consecutive wet days is shown in figure 5.9. The period from 1980-2000 produces higher values than at any other times throughout the recorded rainfall data with the largest number of consecutive wet days occurring in 1977 and 1995 (22 days each). The subsequent decline in observed number of consecutive wet days post-2000 results in the average value for 2000-2008 being equal to that observed in the period 1950-1959. The data set provides no clear pattern relating to this aspect of precipitation over the duration of the records.

Table 5.13 – Average of Annual Maximum Consecutive Wet days for each Decadal Period

Years	Average of annual max. consecutive wet days (days)
1950-1959	14.00
1960-1969	10.80
1970-1979	14.14
1980-1989	14.90
1990-1999	14.40
2000-2009	14.00

Figure 5.9 – 10-year Running Average for Annual Maximum Consecutive Wet Days

This process was repeated to analyse the occurrence of consecutive very wet days. The tabular and graphical data for this are provided in table 5.14 and figure 5.10. Once again the analysis does not produce any significant trend relating to the number of consecutive very wet days although the 10-year average may suggest a slight increase the maximum number of consecutive very wet days in more recent times. This increase would be in the region of 15% from 1954 to 2009.

Table 5.14 – Average of Annual Maximum Consecutive Very Wet Days for each Decadal Period

Years	Average of annual max. consecutive wet days
1950-1959	4.50
1960-1969	4.80
1970-1979	6.29
1980-1989	4.50
1990-1999	5.60
2000-2009	5.40

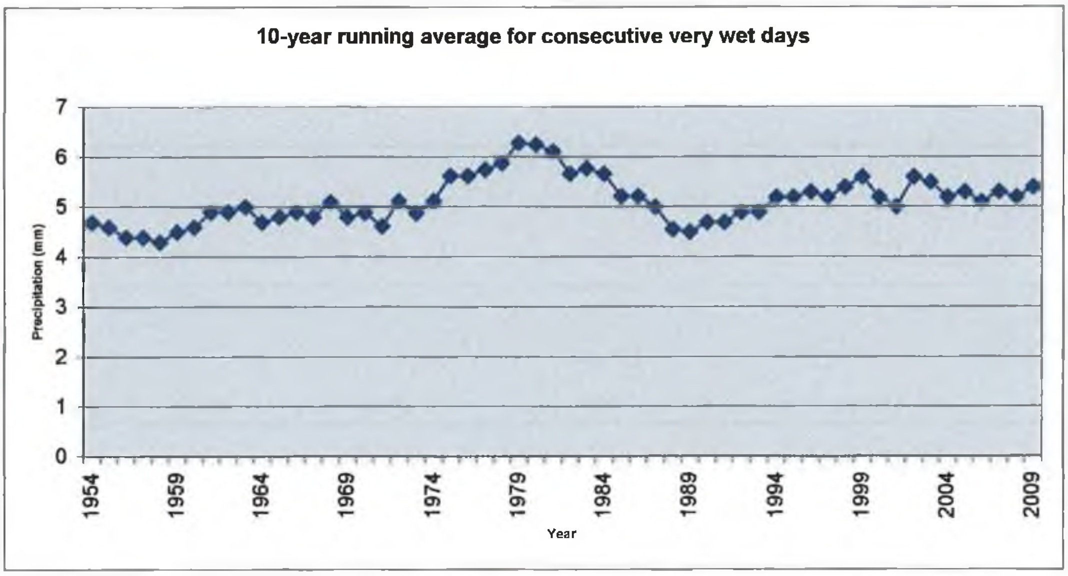

Figure 5.10 – 10-year Running Average for Annual Maximum Consecutive Very Wet Days

As indicated within section 5.2 a key indicator in relation to flooding are the maximum annual 3-day, 5-day and 10-day totals [21]. The magnitude of these is shown in table 5.15. Contrary to the observations made at Milltown these values at Glenamaddy have declined in recent times. This does not correspond with the opinion of the IPCC AR4 that suggests both an increase in the severity and frequency of extreme events as a result of climate change.

Table 5.15 – Total Precipitation over 3, 5 and 10-days

Years	Average of Max. Rainfall Totals			% Increase from 1949-1958 Average		
	3-day (mm)	5-day (mm)	10-day (mm)	3-day %	5-day %	10-day %
1950-1959	52.3	66.1	102.9	0.0	0.0	0.0
1960-1969	65.3	76.4	102.2	24.9	15.6	-0.7
1970-1979	50.7	66.1	99.9	-3.1	0.1	-2.9
1980-1989	53.5	66.8	94.6	2.3	1.1	-8.1
1990-1999	50.2	74.3	102.5	-4.1	12.5	-0.4
2000-2009	51.9	65.1	95.3	-0.9	-1.5	-7.3

5.5 Summary

The earth's climate is changing due to anthropogenic forces producing global warming. The main radiative forces behind global warming are greenhouse gases (GHG). These have resulted in a temperature increase of 0.76°C since pre-industrial time. The changing climate will have an effect on all climatic variables to varying degrees. The impact of climate change on precipitation may have a significant effect on flooding in Ireland. There is a great deal of uncertainty surrounding the prediction future precipitation trends and the influence of climate change on them. This reduces confidence in future projections of precipitation. Expert opinion proposes that climate change will result in an increase in the frequency and magnitude of precipitation events.

Analysis of rainfall within the Clare River catchment shows no significant indication of an increase in either frequency or magnitude of precipitation. Both rainfall stations recorded a decrease in annual precipitation over the duration of their respective data sets. This opposes the global trend that indicates a 1% increase per decade in annual precipitation between 1901 and 2004. The wettest and driest years in both data sets did not favour any particular period being relatively evenly distributed throughout the recorded period. Annual precipitation is proposed to have a different seasonal distribution due to climate change resulting in drier summers and wetter winters. Recent records at Milltown showed a decrease in the average winter rainfall. However they also showed a tendency to possess considerable rainfall totals due to having a wider range of values. Glenamaddy also produced some significant winter rainfall totals in the latter half of the available records. This suggests that there may be an increased probability of experiencing considerable winter rainfall totals in more recent times. There is no significant trend in the occurrence of wet days, very wet days, heavy precipitation days and extreme precipitation days for both rainfall stations. This is also the case for the analysis of annual maximum consecutive wet days and consecutive very wet day values. Therefore there is no evidence to suggest an increase in the persistence of rainfall. The 3, 5 and 10-day rainfall totals are expected to be the most significant precipitation indicator in relation to flooding. There was an increase in these values at Milltown suggesting an increase in the magnitude of precipitation events. Unfortunately the broken nature of the data at Milltown prevents an accurate assessment of this aspect of precipitation, as there is no similar trend evident at Glenamaddy.

The analysis of the available precipitation data for the Clare River catchment provides no clear indication of an increase in the frequency and magnitude of precipitation as a result of climate change. Analysis of rainfall throughout the catchment was not possible due to limited availability of data. Areas such as Claregalway situated southwest of available rainfall records may have experienced more rainfall during precipitation events due to the spatial variability in rainfall. The precautionary approach should be taken to ensure that potential future increases in precipitation are considered in decisions affected by flood risk. The OPW suggest that two scenarios should be considered to adequately account for potential increases in flood risk due to climate change. These are a most probable scenario (MRFS) and an extreme scenario (HEFS). These predict an increase in flood flows of 20% and 30% respectively. There appears to be a consensus of agreement among published documentation for the 20% estimate. Therefore it is suggested that factoring in a 20% allowance for future flow increases would be the most sensible option to ensure the potential impacts of climate change are adequately accounted for.

Chapter 6

Land Use

6.1 Influence of Land Use Changes on Flood Risk

The degree of flood risk in Ireland is expected to worsen due to predictions relating to the influence of climate change on precipitation. Flood risk is not determined by rainfall events alone. It is a combination of the likelihood of flooding occurring and the potential consequences arising from such a flood event. The rapid growth in property development during Ireland's economic growth over the past 20 years has contributed to the current level of flood risk. Engineered flood relief schemes are beneficial in addressing flood risk to existing development. However they are expensive and in many cases require the occurrence of a considerable flood event and subsequent flood damage to initiate their implementation e.g. Maynooth Flood Relief Scheme. They may also contribute to increased flood risk elsewhere if not properly designed i.e. through eliminating floodplain storage. Despite the economic downturn population and housing densities are still predicted to increase [30]. Therefore it is important to provide strategic policies that address the flooding issue in relation to planning and development.

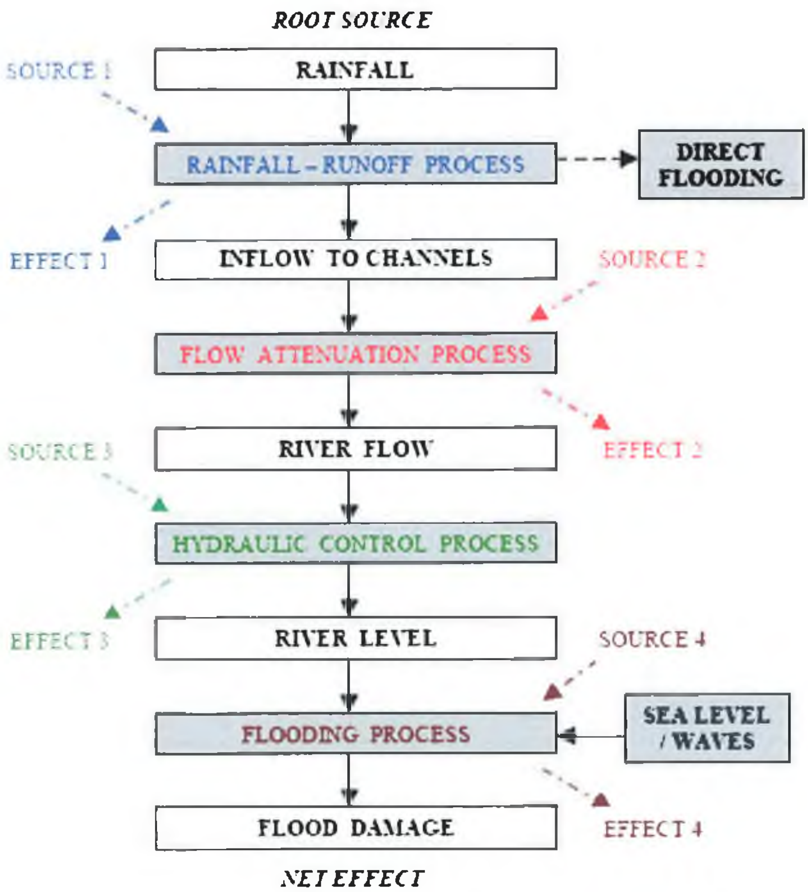

Figure 6.1 – Chain of Sources, Processes and Effects of Flooding [22]

There are a number of human factors that can influence the degree of flood risk. Precipitation is a natural phenomenon and outside of human control. Therefore it is important to ensure that controllable human influences are focused on considering flood risk. Every effort should be made to avoid increasing and when possible reduce flood risk. Figure 6.1 shows the chain of sources, processes and effects at stage 1, 2 and 3 that lead to flooding. This section is concerned with the potential increase in flooding as a result of land use changes and development. These factors can potentially influence source 1, 2 and 3 with a consequential impact on effect 1, 2 and 3. This increases the resulting overall net effect of flooding. Sea level is also mentioned in figure 6.1 but has no influence on the Clare River Catchment due to the river discharging into Lough Corrib. There is a potential influence from lake levels on flooding on the Clare River. This issue is discussed in section 7.3. It is important to identify the land use and development factors and their impact on flood risk and flood damage.

Rainfall is the primary source of flooding. The magnitude and frequency of rainfall events is uncontrollable despite the potential implications of climate change on increased rainfall. The most significant human influence on the rainfall-runoff process is factors influencing the rate of runoff. There are a number of processes that can remove water from the ground surface before it can potentially contribute to flooding as a result of surface water flooding or contributing to fluvial flooding. These processes are soil infiltration, evapotranspiration and interception.

The vegetation type and cover determines the degree of surface water interception that occurs. Surfaces with little vegetation have little capacity to intercept surface water. Evapotranspiration is the combination of evaporation and transpiration from vegetation. The rate of evapotranspiration is governed by energy supply, water supply and vapour transport [11]. The impact of changing vegetation cover on agricultural land is discussed in section 6.5. Removing vegetation has been identified as a key factor in increased surface water runoff most notably in tropical climates [22].

Changes in land use have a relatively insignificant impact on surface water runoff within a catchment unless they are carried out on a large scale or involve green field development. Development can have a major impact on soil infiltration and surface water runoff if not managed correctly due to the impermeable nature of construction materials. The rational method is one of the most widely used methods of calculating surface water runoff for the design of surface water sewers. It is the method preferred by the British Standards code of practice for building drainage [23]. The formula estimates the rate of peak surface water discharge in L/s (Q) from the rainfall intensity in mm/hr (i), contributing area in hectares (A) and a runoff coefficient (C). Q is given by:

$$Q = 2.78\ C\ i\ A$$

where: 2.78 is a measurement unit converter

The runoff coefficient (C) is the most difficult variable to predict. The percentage of the total rainfall that will reach the sewer network due to surface water runoff depends on factors such as permeability, slope and ponding character of the ground surface. The percentage will also depend on the severity and persistence of the rainfall event as this determines the wetness of the soil. Infiltration will decrease as rainfall persists thus increasing the quantity of runoff. Typical values of C are given in table 6.1. The topography of the Clare River catchment would provide a surface slope of 0-2% in most scenarios.

Table 6.1 – Runoff Coefficients (C) for use in the rational method [11]

	Return Period (years)						
Character of surface	**2**	**5**	**10**	**25**	**50**	**100**	**500**
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
Poor condition (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover on 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Undeveloped							
Cultivated Land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/Range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0–2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Comparing undeveloped pasture to the developed concrete/roof scenario there is a significant increase in the percentage of surface water runoff. A less severe 2-year return period event increases C from 0.25 to 0.75. A more extreme 100-year event results in an increase in C from 0.41 to 0.97. These represent increases of 50% and 56% respectively. It is also apparent that developed green areas may also produce differences in the runoff coefficient depending upon the condition and grass cover of the area. This is less significant than paved areas but still represents an 11% increase in runoff for grass areas in poor condition as opposed to those in good condition for a 100-year event. The rainfall runoff process contributes to fluvial flooding but can also produce isolated flooding as a result of collecting in depressions. As a result development located in low points may be at risk from flooding despite being a considerable distance from the nearest water body. Historically changing agricultural methods may have played a more significant role in the rainfall-runoff process due to deforestation and land cultivation. The most significant present day change in this process is as a result of development on green-field sites.

The flow attenuation process is the second process at which flooding can be affected. The surface water that is not removed through soil infiltration, evapotranspiration or interception makes its way to the drainage network i.e. river. The surface water combines with the flow in the river, which consists of the surface water and groundwater drained from higher up in the catchment. Depending upon the spatial and temporal distribution of rainfall in the catchment the flow in a channel at a given point will vary. The magnitude of flow at a point in the channel can be plotted on a hydrograph. This hydrograph can be used to show the magnitude of a flood event in relation to time. As the water moves downstream the timing of the peak flow is generally delayed. This is due to the time required for the peak flow to traverse between the two locations. This delay occurs in scenarios when there is insufficient inflow (e.g. rainfall–runoff, groundwater) into the river between the two locations to eliminate this lag time. The magnitude of the peak flow is also reduced as it moves downstream in scenarios where no inflow occurs between the two points due to attenuation. The channel provides temporary storage during times of in-bank flow. During periods of out-of-bank flow the floodplain also contributes to the temporary storage capacity of the system. The process of temporary storage of flows is known as attenuation. Figure 6.2 and 6.3 show how this increased temporary storage contributes to a delayed peak at the downstream location. Figure 6.2 shows in-bank flow with a fairly constant lag time of approximately 4 hours. The out-of-bank flow, shown in figure 6.3, shows that when the water level increases above bank level (approx. 2.5m in this example) that there is a noticeable reduction in the rate of increase of the flow at the downstream station. This results in an increased lag time for peak flow of 6 hours.

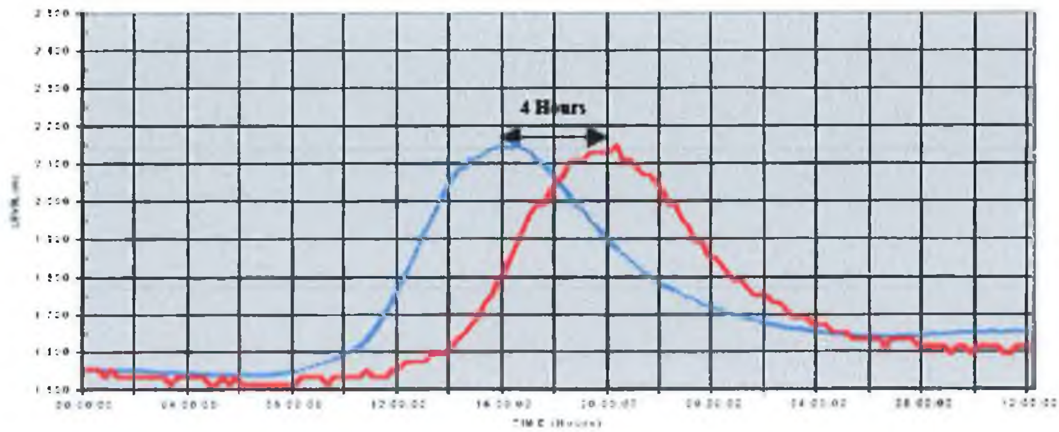

Figure 6.2 – In-Bank Hydrograph (Rathvilly – Tullow) [22]

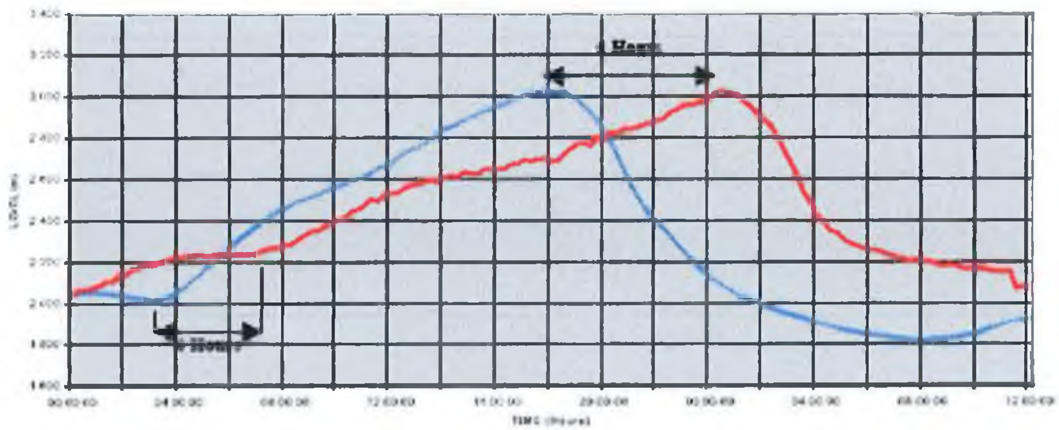

Figure 6.3 – Out-of-Bank Hydrograph (Rathvilly – Tullow) [22]

The attenuation of flow modifies the source (rainfall-runoff) to produce an effect. Therefore any adjustments to the attenuation capacity of a channel and its flood plain will influence the effect of flow events that utilise this storage capacity. Channel works and maintenance influence the attenuation capacity of a channel and its flood plain (see section 7.1). Development can also have a significant effect on flow attenuation occurring in the flood plain. Therefore development in a flood plain is not just increasing the flood risk of that site. Construction on a flood plain removes this storage volume through hard engineered flood defences, raising the ground level of the site or simply the volume of space occupied by the development. This reduces the attenuation capacity of the flood plain and increases the magnitude of flows. This would result in an increased flood risk at a location downstream of the development due to less temporary storage being available. This reduction of the floodplain can also have an impact on flooding upstream, acting as a hydraulic constraint as explained below. The most significant changes that could occur in relation to attenuation are if hard engineered flood defences were constructed to protect expanses of land that had previously provided temporary floodplain storage.

Hydraulic conditions dictate the rate at which water can be conveyed. The variables that can control the velocity of floodwater are the slope, condition, size and shape of a channel and its floodplain. These factors dictate the relationship between flow depth and discharge. There are a number of factors that can influence the hydraulic conditions both in the channel and on the floodplain. In-channel hydraulic constrictions would include construction of bridge piers in the channel, pipes (e.g. water mains) located along or across the channel, reduction of the channel width due to bank development (e.g. boat piers), running a channel through a culvert. Factors which can change the characteristics of a floodplain and hence the hydraulic conditions include raising the floodplain above flood level for development, development on floodplain (this includes low vulnerability development which may not be at risk of flood damage), embankments that effect flood flow such as landscaped embankments.

The relationship between water level and flow is known as the stage-discharge relationship. Figure 6.4 shows a stage-discharge relationship at a hydrometric station located a short distance upstream from a floodplain in which the construction of an embankment has restricted flow. The stage-discharge relationship is shown for pre-construction (red) and post-construction (blue). Both relationships are identical up until the water reaches the channels bank level. Above this the water spills out onto the floodplain. Flow on the floodplain is restricted in the post-construction scenario. As a result of this the water levels at the hydrometric station are approximately 300 mm higher for out-of-bank flow than the corresponding discharge water level pre-construction. Therefore altering the hydraulic control process will have implications on the net flooding effect. Hydraulic constraints can have a significant impact especially locally. As described in section 3.1.7 the water levels at Claregalway Bridge in November 2009 were 1 m higher on the upstream face of the bridge. This demonstrates how a bridge eye of insufficient cross-sectional area can act as a hydraulic constraint resulting in increased water levels upstream.

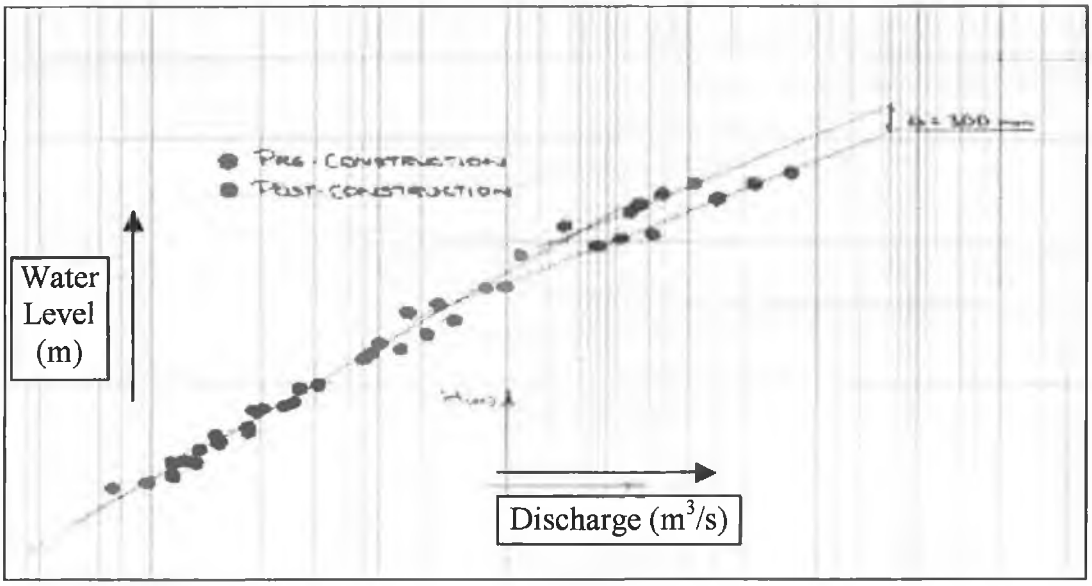

Figure 6.4 – Stage-Discharge Relationship Demonstrating the Effects of Floodplain Restriction [22]

The factors identified in figure 6.1 and subsequently discussed influence the magnitude of flooding. The flooding process is also affected by development. The flood damage is a function of both the flood magnitude and the extent and value of property within the area inundated by the floodwater. Development within an area at risk from flooding increases the potential flood damage resulting from a flood event. This development may include residential, commercial and key infrastructure. Potential damage arising may include physical damage to property, economic damage arising from failure of key infrastructure etc. Less vulnerable development within areas at risk from flooding reduces the potential flood damage arising from flood events. However it may have an impact on the other aspects of flood risk identified in figure 6.1. The flooding process is the key stage at which inappropriate development could contribute to a significant increase in flood risk.

6.2 Planning System and Flood Risk Management Guidelines

The Planning System and Flood Risk Management (PSFRM) Guidelines were developed by the Department of the Environment, Heritage and Local Government (DoEHLG) and the Office of Public Works (OPW) to ensure that flood risk is a key consideration in preparing development plans and local area plans and in assessing planning applications. They provide a systematic approach to flood risk management within a river catchment context. The main objectives of the guidelines are to avoid unnecessary development in areas at flood risk or that would increase flood risk elsewhere. They also aim to ensure effective mitigation measures are provided for development permitted in areas at flood risk and that planning procedures comply with EU and national law. There are a number of key principles that apply to planning and zoning. These are listed below in order of priority [24]:

1. Avoid development in flood risk areas
2. Substitute less vulnerable uses for flood risk areas where avoiding development is not achievable.
3. Provide mitigation and flood management measures in scenarios where option 1 and 2 are not achievable.

The application of flood risk management to different levels of the planning system is highlighted in table 6.2. The table shows that more comprehensive assessments are required as the scale of the policy instrument becomes more local.

Table 6.2 – Flood Risk Management and the Planning System [24]

Policy Documents / Instruments	Flood Risk Assessment Technique	Decision-making Tools
National Spatial Strategy, National Planning Guidelines	Flood Risk Management Guidelines	n/a
Regional planning guidelines	Regional Flood Risk Appraisal, Catchment Flood Risk Management Plans	Sequential approach, Strategic Environmental Assessment
City / county development plan	Strategic Flood Risk Assessment, Catchment Flood Risk Management Plans	Sequential approach, dev. plan Justification Test, SEA
Local area plan	Strategic Flood Risk Assessment	Sequential approach, dev. plan Justification Test, SEA
Master plan, non-statutory plan, site brief	Site-specific Flood Risk Assessment	Sequential approach, dev. plan Justification Test, SEA / Env. Impact Assessment
Planning application	Site-specific Flood Risk Assessment	Sequential approach, dev. management Justification Test, EIA

Flood zones provide a graphical indication of areas susceptible to flooding from events of varying return period (i.e. 100-year return period flood zone indicates area with 0.01% probability of flooding in any given year). The report defines three flood zones that should be incorporated into mechanisms involved in land zoning:

- Zone A – return period of 100 years or less (High probability of flooding)
- Zone B – return period of between 100 years and 1000 years (Medium probability of flooding)
- Zone C – return period of greater than 1000 years (Low probability of flooding)

Zone A is the region where most of the flooding will occur and should therefore be avoided for future development if possible. Zone B is less likely to be flooded but should consist of less vulnerable developments if being developed. Finished floor levels within zone A and B should take consideration of the water levels associated with extreme events. Vulnerable developments should be confined to Zone C where possible. The report states that flood defences should be ignored when determining these flood zones due to the risk of such defences being overtopped or breached. Table 6.3 shows the suitability of developments of varying vulnerability within each flood zone.

Table 6.3 – Suitability of Development of Varying Vulnerability within Flood Zones [24]

	Flood Zone A	Flood Zone B	Flood Zone C
Highly vulnerable development (including essential infrastructure)	Justification Test	Justification Test	Appropriate
Less vulnerable development	Justification Test	Appropriate	Appropriate
Water-compatible development	Appropriate	Appropriate	Appropriate

Due to uncertainties relating to impacts of climate change on flooding depth and extent a conservative approach should be taken to planning decisions. Factors of safety and the ability to adapt to climate change should be incorporated into future developments. This will help ensure that developments do not exacerbate or are not affected by increased flooding as a result of climate change in the future. The precautionary approach is a key priority of these guidelines in addressing flood risk. This includes measures such as setting finished floor levels (FFL) above 100-year flood levels.

Figure 6.5 shows the key principles to a risk based sequential approach to managing flood risk in the planning system. The primary objective is to avoid development in areas at food risk. In situations where development cannot be avoided less vulnerable development should be substituted to reduce the potential flood damage. Mitigation is a key element of development in flood risk areas. Inappropriate development that would result in increased flood risk should not be allowed. The justification test provides a method of justifying development in areas at flood risk due to planning need provided the appropriate mitigation measures are implemented. Only after this sequential approach has been applied successfully should development proceed. The level of detail required depends upon both the flood zone and vulnerability of the development as highlighted within figure 6.5. The guidelines prioritise that lands required for current and future flood management should be clearly identified in development plans and local area plan's (LAP). These lands should be protected from development to ensure that they are available to alleviate flooding.

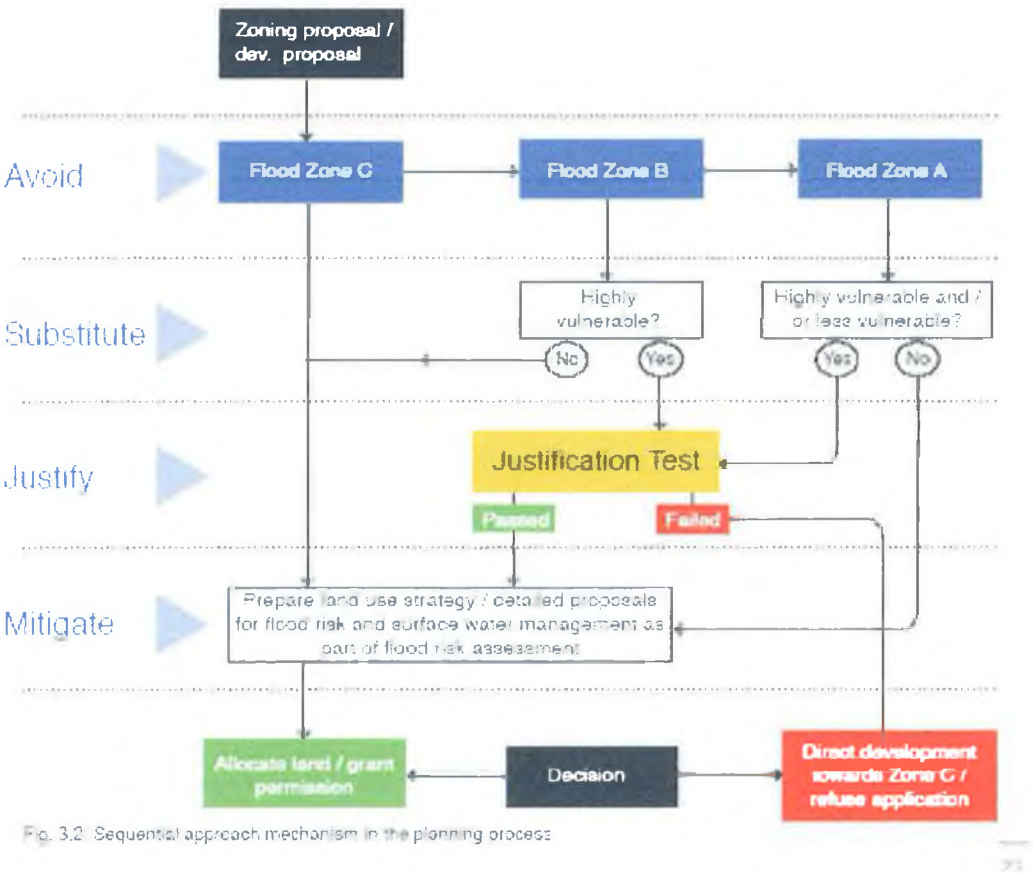

Figure 6.5 – Sequential Approach to Managing Flood Risk [24]

Flood risk management should be included in all aspects of spatial planning. The Strategic Environmental Assessment (SEA) is highlighted as a mechanism that can incorporate a flood risk assessment tool. The PSFRM guidelines should be incorporated into regional guidelines and the production of development plans. The guidelines highlight the importance of regional leadership in addressing flood risk. High-level flood risk appraisals are required in conjunction with Regional Planning Guidelines to ensure effective action is taken at local levels. The flood risk appraisal should identify high-level flood risk areas and spatial planning issues. It should also set out a high level policy framework for development plans and LAP's to address issues identified at regional level. Regional planning guidelines consideration of flood risk should be strategic in nature and regional in scope as development plans and LAP's will provide more detailed assessments. Regional flood risk appraisals will generally take the form of a desktop study. It should include a summary of the broad spatial distribution of flood risk and conflicts with growth areas. Supplementary information regarding areas where addressing flood risk is particularly important, e.g. notable urban settlements such as gateways and hubs, are another necessary element. Regional guidelines also provide a format to provide guidance on producing Strategic Flood Risk Assessments (SFRA). Integration of flood risk assessment into development plans is key. The statutory consultee for development plans is the OPW and as such should be consulted prior to the designation of any zoning strategy to ensure it does not impact on the objectives put forth by the regional flood risk appraisal and the planning system and flood risk management guidelines [24].

The PSFRM guidelines state that a less detailed approach will suffice at county level than is expected at local level except in cases where land is to be zoned or selecting locations for key infrastructure. A more detailed evaluation of the spatial distribution of flood risk is required at city and town level to identify zones A, B & C as described above. This would incorporate a SFRA of the area. This will provide improved understanding of flood risk in the development plan area. It will also act as a mechanism to evaluate existing flood defence infrastructure and the impact of failure of any flood defences. The natural flood plain should be identified and protected from development to preserve its hydrological function of accommodating and attenuating flood flows. Flood risk maps for key areas where there is interaction between development and flood risk will allow for zoning to be carried out in consideration of flood risk. The SFRA will also provide relevant information for the application of the Justification Test where necessary. Any proposed mitigation measures included in development plans should be evaluated to determine if they can reduce flood risk to an acceptable level without increasing flood risk elsewhere. Development plans should also include guidance on surface water management and information relevant to the application of site-specific flood risk assessments [24].

Major proposals for development must apply the sequential approach and justification test according to the PSFRM guidelines. Minor proposals such as extensions to houses will not have a significant effect on flooding and are not subject to the sequential approach or justification test as relocation would not be possible. However a commensurate assessment of the risks of flooding is required to accompany the application to ensure it does not exhibit adverse impacts on access to the watercourse, flood plain or flood defences for maintenance [24].

6.2.1 Flood Risk Assessment

The purpose of carrying out a Flood Risk Assessment (FRA) can be due to a number of reasons. It can be used to identify the extent to which flood risk is an issue, identify flood zones, inform decisions in relation to zoning and planning applications or to develop appropriate flood risk mitigation and management measures for development sited in flood risk areas. There are a number of key scales at which an FRA can be carried out which are regional, strategic (county/city development plans and LAP's) and site-specific.

FRA's should be proportionate to risk, scale and location of development. A competent person such as a hydrologist or engineer should carry out the FRA as soon as possible in the planning process. This will ensure that decisions made are informed as to the implications of flood risk. The FRA should include all relevant information, i.e. extent of previous flood events, and also focus on prediction of more extreme events and potential impacts of climate change. Their main purpose is to identify flood risks and how they will be managed with consideration for flood risk elsewhere. They also provide a format to consider the impact of modifying flood defences and the potential impact of their failure.

FRA's consider the source, pathway, and receptor model. This model is based on the principle that these three factors must be present for a flood risk to occur. The source of flooding is the primary contributor e.g. rainfall. The pathway links the source and receptor e.g. water over spilling a riverbank and entering the floodplain due to the increased quantity of surface water runoff. The receptor is the recipient of the damage arising from the flooding e.g. house located in floodplain. The source is predominantly uncontrollable except in such cases where flooding involves failure of infrastructure such as dams. Therefore consideration of the remaining two factors is required to ensure proper consideration of flood risk. Table 6.4 shows a breakdown of the main objectives of FRA's at regional, county/local and site-specific level.

Table 6.4 – Hierarchy of Flood Risk Assessment [25]

FRA	Code	Purpose	Responsibility
Regional Flood Risk Appraisal	RFRA	RFRAs provide a broad overview of the source and significance of all types of flood risk across a region and also highlighting areas where further more detailed study will be required. At this level, they are an appraisal and not an assessment.	Regional authorities in consultation with the OPW, river basin management bodies and LAs. CFRAM Study outputs, when available, will be an important and prime input to the appraisal
Strategic Flood Risk Assessment for development plan and LAP	SFRA	To provide a broad (area-wide) assessment of all types of flood risk to inform strategic land-use planning decisions. SFRAs enable the LA to undertake the sequential approach, including the Justification Test, allocate appropriate sites for development and identify how flood risk can be reduced as part of the development plan process. The level of detail required will differ for county and city development plans	LAs in consultation with the OPW and emergency services. The Flood risk management plan arising from the CFRAM programme will heavily inform the SFRA. In its absence local authorities may need to commission extensive flood risk assessments, albeit at a strategic level. OPW will provide advice on the specifications that should be applied
Site-specific Flood Risk Assessment	Site FRA	To assess all types of flood risk for a new development. FRAs identify the sources of flood risk, the effects of climate change on this, the impact of the development, the effectiveness of flood mitigation and management measures and the residual risks that remain after those measures are put in place. Must be carried out in all areas where flood risk have been identified but level of detail will differ if SFRA at development plan level has been carried out	Those proposing the development in consultation with the LA and emergency planners

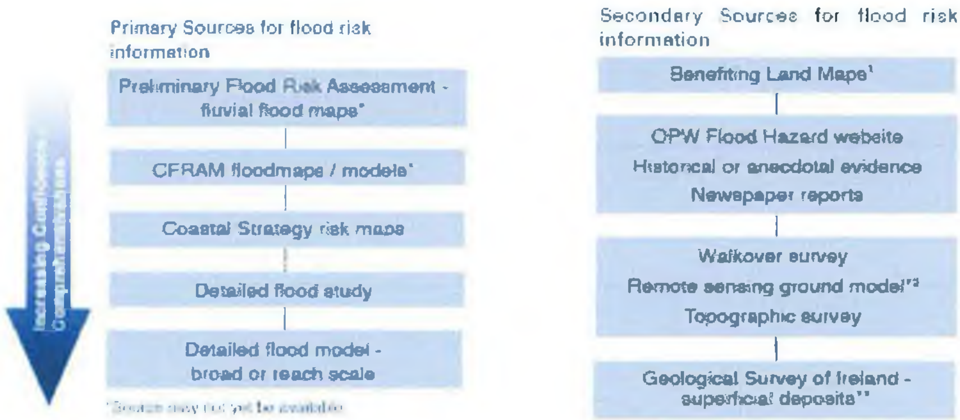

Figure 6.6 – Sources of Flood Risk Information [25]

There are a number of indicators that are typically used in the assessment of flood risk including flood probability, flood depth, flood velocity and rate of onset of flooding. These indicators are suitable for application at both strategic and site-specific level. There are a number of key stages in carrying out a FRA [25].

Stage 1 – Flood Risk Identification

The purpose of this stage is to identify whether there may be any issues with flooding or surface water management in relation to the site/area to be zoned that warrant further analysis. Information that may be availed of at this stage is outlined in figure 6.6. It should be noted that not all of these sources are available for every location.

Stage 2 – Initial Flood Risk Assessment

This second stage requires confirmation of the sources of flooding outlined in stage 1. The quantity and quality of available flood risk information is evaluated. The extent of analysis required to provide the required level of spatial resolution of flood risk should be determined. This process may require producing indicative flood risk maps. The key elements of the initial assessment are described in table 6.5.

Stage 3 – Detailed Risk Assessment

This final stage provides a comprehensive quantitative appraisal of potential flood risk to the area and elsewhere and also includes the expected impact of mitigation measures. This will usually involve using or constructing a hydraulic model of a wide enough area to appreciate the catchment scale impacts of the development. It should take account of actual and residual flood risks.

Table 6.5 – Key Elements of Initial Assessment [25]

Elements of initial assessment	Type of flood risk assessment		
	SFRA for county plan	SFRA for city development plan or LAP	FRA for site
An examination of all sources of flooding that may affect a plan area	C Z	✓ C	✓
An appraisal of the availability and adequacy of existing information	✓	✓ C	✓
Produce flood zone map where not available	U	✓ (But not all areas)	S
Determine what technical studies are appropriate	✓ Z	✓ C	✓
Describe what residual risks will be assessed	✓ Z	✓ C	✓
Potential impact of development on flooding elsewhere	✓ Z	✓ C	✓
Scope of possible mitigation measures and what compensation works may be required and what land may be needed	U	✓ C	✓
Set out requirements for subsequent stages of FRA	✓	✓ C	n/a

✓ = Expected activity

U = Unlikely initial assessment will undertake this element

Z = detail will differ in County Plan where zoning is being considered

C = Confirmation of details provided in county wide SFRA or RFRA

S = FRA's main purpose is not to challenge the flood zone map, but concentrate on the flood risk issues. Where no SFRA has been produced flood zones should be produced in accordance with OPW specifications.

n/a = Not applicable

If stage 1 finds there is no flood risk from assessing available information then it will end here. If not the FRA will progress on to stage 2. This avoids costly evaluation work being carried out unnecessarily. At site-specific level indicative flood plains should be estimated to be subject to a detailed FRA. Decisions can be made on limited data so long as conservative estimations are taken. Table 6.6 shows which of the stages outlined above are required when applying flood risk assessment at different spatial scales. It shows that site-specific FRA's require the most detailed analysis.

Table 6.6 – Flood Risk Assessment Stages Required per Scale of Study Undertaken [25]

	Flood risk identification	Initial flood risk assessment	Detailed flood risk assessment
Regional Flood Risk Appraisal	✓	U	U
Strategic Flood Risk assessment – County-wide	✓	P	U
Strategic Flood Risk Assessment – City or town within a county plan	✓	✓	P
Site-specific flood risk assessment	✓	✓	✓

P = Probably needed to meet the requirements of the Justification Test

U = Unlikely to be needed

✓ = Required to be undertaken

Figure 6.7 shows a graphical representation of the different elements required within the scope of a FRA. The outputs of a FRA depend upon the spatial scale at which it was applied. A FRA carried out at a regional level should show the broad spatial distribution of flood risk and any conflicts arising from growth objectives such as those outlined in the National Spatial Strategy. It is also expected to highlight areas of particular importance due to significant flood risk or growth objectives i.e. Tuam which is a hub town. At a regional scale it provides a mechanism to suggest policies for sustainable flood risk management and guidance for producing city and county development plans. A FRA carried out at strategic level (i.e. city or county) should identify key rivers/areas at flood risk and impacts of flood risk on key growth areas. Allowances for climate change should also be incorporated into flood zoning. It should also identify locations and areas protected by flood risk management infrastructure and flood warning systems and assess the performance and consequence of failure of such systems [25]. Floodplains should be identified at county and local level so that they can be maintained to protect their natural accommodation and attenuation function. Areas where site-specific FRA's should be required should also be defined. Land that is likely to be affected by current or future flood risk should be identified as well as land where development would increase flood risk elsewhere. Mitigation measures to deal with the flood risk should be evaluated to determine whether they would comply with justification test or whether development should not be permitted [25].

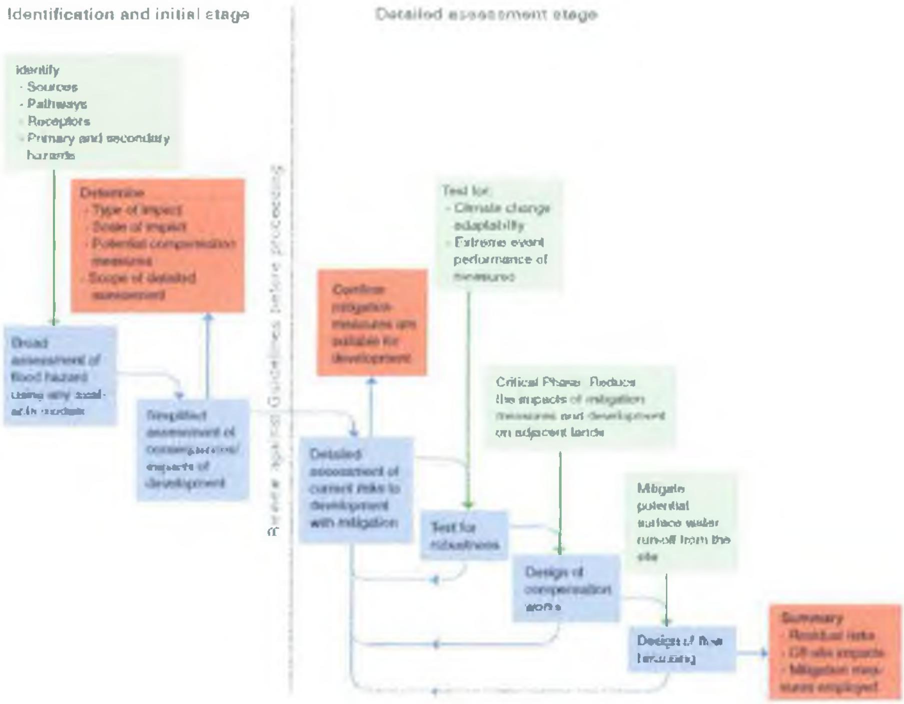

Figure 6.7 – Stages of Flood Risk Management [25]

Site-specific flood risk assessments require the most detailed approach. They should be undertaken at an appropriate spatial scale so as to determine if the development poses any influence on flood risk elsewhere. The information included in a typical site-specific FRA includes [25]:

Plans;

- Location plan including watercourses
- Plan showing existing site and development proposals
- Identify any structures that may influence river hydraulics

Surveys;

- Existing and proposed site levels (mAod)
- Cross section of site showing FFL and road levels relative to watercourse levels.
- Anticipated water levels and associated probabilities

Assessment;

- Consideration of flood zone and that development is suitable given vulnerability
- Existing flood alleviation measures
- Information on all potential sources of flooding
- The impact of flooding including:
  - The likely rate at which flooding might occur (i.e. rapid onset or slow rise of flood water)
  - The speed of flow of flood water
  - The order in which various parts of the location or site might flood
  - The likely duration of flood events
  - The economic, social and environmental consequences of flooding on occupancy of the site
  - Information on extent and depth of previous flooding
  - Access and egress from site under routine and emergency conditions (frequent and extreme flood conditions)
  - Proposals for surface water management

Any information relevant to on site drainage should also be included such as soil porosity, existing and proposed drainage, impact on runoff and proposed surface water management methods such as SUDS.

FRA's provide a strategic assessment of the flood risk associated with development. There already exists the Strategic Environmental Assessment mechanism within which it can be incorporated. A key element of a FRA is the mathematical modeling of the relationship between a development and flooding. This enables informed decision making in relation to planning. The application of FRA would be most beneficial on a strategic scale that incorporates entire catchments. This scale of evaluation would consider all probable influences on flooding throughout the catchment and is the basis on which current projects such as CFRAMS are being carried out in line with requirements of the EU Floods Directive.

6.2.2 Justification Test

The justification test is a key requirement for development in Zone A and B. It is outlined as a key tool to ensure that development in flood risk areas is carried out in consideration of the flood risk. The provision of mitigation of flood risk using measures such as hard engineered flood defences is a necessary requirement to alleviate flood risk pressures imposed by developing in flood risk areas. However it is not an acceptable justification of development in these areas. The justification test has two processes. These are the plan making justification test and the development management justification test [24].

The plan making justification test ensures all necessary steps are taken to avoid increasing flood risk due to zoning of land. The following criteria must be satisfied for developments to be carried out in Zone A or B [24]:

1. The urban settlement is targeted for growth under the National Spatial Strategy and regional planning guidelines.
2. Zoning of the land is required to achieve proper planning and sustainable development i.e.
    a. Essential to facilitate regeneration or expansion of centre of urban settlement
    b. Land includes considerable previously developed or under-utilised land.
    c. Within/adjacent to core of urban settlement
    d. Essential to facilitate compact and sustainable urban growth
    e. No alternative options of lower flood risk within/adjacent to core of urban settlement.

3. Flood risk assessment carried out in conjunction with SEA demonstrates that mitigation measures will reduce flood risk to acceptable level without increasing flood risk elsewhere.

There are a number of other considerations when evaluating planning strategies. Riparian strips should be maintained to allow for river maintenance. The local authority should also develop management standards and checklists to provide a structured and effective method of evaluating planning applications where flood risk may be an issue and consider impacts to other sources of flooding such as overloading of artificial drainage networks [24]. Standards should also be provided for managing flood risk i.e. hard-engineered defences, SUDS.

Consideration of flood risk in development management addresses flood risk for individual planning applications. The same basic methods apply as required for land zoning. The sequential approach should be implemented to avoid development on land at flood risk. FRA's should accompany planning applications where necessary. The justification test should be implemented in situations in which development in areas at flood risk is unavoidable There are a number of stages in development management. The consideration of flood risk as early as possible in the planning process ensures informed decisions are made. Pre-application is the first stage. It is the responsibility of the applicant to gather relevant information to identify any flood risks relating to their site. It should be identified if a site specific FRA is required. The application is subsequently lodged along with the FRA if necessary. The detail of FRA depends on scale and sensitivity of the development and if a SFRA has been carried out for the area, as this would already provide information on the flood risk associated with the site. FRA's should include plans and the relationship between the site and waterbodies. Information should be provided on any structures that may act as hydraulic controls. Topographical surveys relating site levels to potential flood levels should also be included. The FRA should also provide an assessment of any potential causes of flooding, existing flood mitigation measures, potential impact of site on flooding on the site and elsewhere, how the site layout can address any impact on flood risk, surface water management methods and a description and expected performance of mitigation measures to be implemented.

The next stage of planning is the assessment stage. It is at this stage that the development management justification test should be applied if the development is located within zone A or B. The key parameters that should be adhered to as set out in this justification test are [24]:

1. The site is located in a zone that has been designated for the particular use proposed in an operative development plan that has taken account of the PSFRM guidelines.
2. The planning application has included an appropriate FRA that demonstrates the development will not increase flood risk elsewhere and will include measures to manage any residual flood risk effectively. These risks should be addressed in a method that is in keeping with the objectives of the planning strategy.

Details of mitigation measures for development in zone A or B justified by the test should also be provided to the major emergency management committee (MEMC). This is an essential aspect in ensuring comprehensive emergency plans can be produced for dealing with extreme events.

By following the above procedures it is felt that informed decisions can be made in consideration of flood risk. Planning applications can be rejected on flood risk grounds without compensation under planning legislation, Schedules of Planning and Development Act, 2000 as amended. Every attempt should be made to avoid development that would be affected by or could contribute to flood risk.

6.3 Area Planning Guidelines on Flood Risk

The draft Regional Planning Guidelines (RPG), 2010-2022, was produced due to the requirement to review the Regional Planning Guidelines 2004-2016 under the Planning and Developments Act 2000-2007 and 2009 regulations. Its aim is to provide a framework for long-term strategic development of the west in line with the National Spatial Strategy (NSS), 2002-2020. There are a number of scenarios outlined in the report relating to the distribution of growth throughout the region. The preferred scenario involves a dispersion of development among the major urban centres (hubs and gateways) thus encouraging growth in adjacent urban settlements. This is economically beneficial due to the stimulation of growth and opportunities being distributed evenly across the region. This scenario will require strategic flood risk assessments for urban settlements to ensure that existing residential areas, drinking water supplies and the surrounding environment are protected from the potential adverse effects of increased flood risk. Many towns have grown on or near watercourses. Increased development has put a greater strain on existing drainage networks. Flooding can occur at any time and can have a significant effect on the economy and society of a region depending on the magnitude and location of the flood event. Flood risk is identified as having a need for cross border co-operation due to flooding and water movement crossing regional boundaries. The PSFRM guidelines outline a transparent flood risk assessment system incorporated into all stages of the planning process and that a regional floods risk appraisal and management system is a requirement for clear and informed decisions to be made at a local scale. The draft Regional Flood Risk Appraisal (RFRA) was published in association with the draft RPG. It is intended to influence decisions made in preparing development plans and local area plans (LAP) in relation to flood risk. The purpose of considering flood risk is not to limit development but to enable sustainable growth while managing flood risk in an appropriate manner. The guidelines state that county development plans and local area plans should include a strategic flood risk assessment. Urban centres such as Tuam, that require continued growth due to its status as a hub town in the National Spatial Strategy (NSS), should be developed in such a way that considers flood risk and implements suitable land uses in areas at risk, i.e. flood plain protection.

The draft Regional Flood Risk Appraisal (RFRA) for the west region outlines a number of different causes of flooding within the region such as fluvial flooding, groundwater flooding and flooding due to artificial drainage systems. The report makes reference to increased rainfall intensity resulting from climate change being particularly problematic in western Ireland due to its already wet climate. However it also acknowledges that the exact impacts of any potential change are unknown due to the uncertainties surrounding climatic rainfall predictions.

The draft RFRA outlines that there is a potential risk of overloading existing artificial drainage networks through increased development and impermeable surfaces leading to increased runoff. It suggests that the main impact in relation to natural surface water drainage networks such as the Clare River would be as a result of developing on flood plains. This would put these new developments at risk from flooding and would also potentially exacerbate the extent of fluvial flooding as it reduces the floodplains capacity to accommodate and attenuate flood flows as outlined in section 6.1. The RFRA favours an approach to flood risk assessment that avoids potential flood risk rather than attempting to justify development by including attenuation or hard-engineered flood defences in line with the principles of the PSFRM guidelines. It states that development should not occur in areas at risk from flooding unless it is necessary, justifiable and there is a capacity to manage the flood risk without increasing flood risk elsewhere. The order in which flood risk management is to be approached is avoidance, reduction and mitigation of flood risk [26]. It is essential that flood risk assessment of vulnerable areas becomes a key element of planning applications and appeals.

The RFRA outlines the sources of flood risk assessment information. The amount and quality of the information varies depending on the source. The OPW are the primary body that deals with flooding as a result of natural causes such as fluvial flooding. The OPW provides information in relation to past flood events at www.floodmaps.ie. This along with other sources, e.g. local authority & GSI, should be considered when reviewing planning applications. A key element of flood risk assessment is that it cannot be effectively carried out on one site in isolation from its surroundings. FRA's must be carried out on a catchment scale in order to account for all possible eventualities arising from altering the hydrology of a catchment. The OPW are currently carrying out Preliminary Flood Risk Assessments (PFRA) under the EU Floods Directive with a view to producing more detailed Catchment-based Flood Risk Assessments in regions identified as Areas with Potentially Significant Risk (APSR). The sequence of the implementation flood risk assessment measures is shown in figure 6.8.

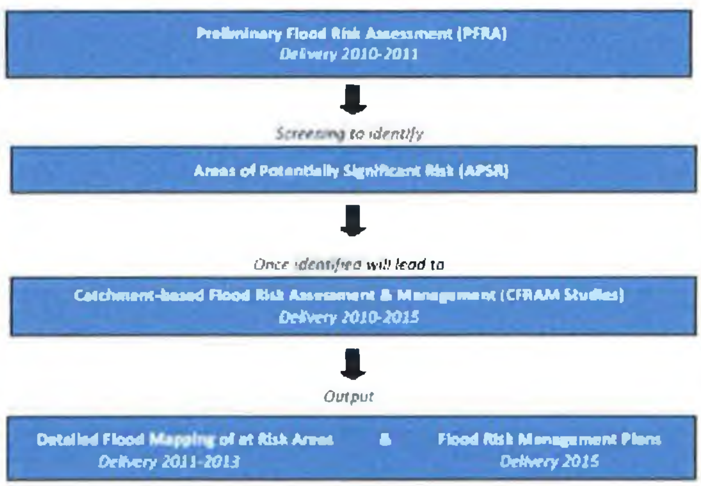

Figure 6.8 – Hierarchy of Flood Risk Assessment Plans [26]

The draft RFRA is predominantly qualitative and only focuses on key urban settlements (gateways, hub and linked hubs as identified by the NSS). The responsibility for carrying out more detailed Strategic Flood Risk Assessments (SFRA) of flood risk areas lies with the production of county development plans and local area plans carried out by local authorities. Information included in the RFRA only relates to the town of Tuam within the Clare River catchment due to its 'hub' status. Fluvial flooding is identified as having a low to high risk in Tuam. The town is susceptible to flash flooding from heavy rain with lands adjacent to the Nanny and Clare River susceptible to fluvial flooding. It proposes that Flood Risk Assessment's should be mandatory for proposed developments. Setting back developments from watercourses and floodplains, and zoning flood plains for amenity purposes should be carried out to preserve the hydrologic function of these areas [26]. SUDS are proposed as a method of reducing the impact of increased runoff due to new developments. It is also important that areas benefiting from flood defences should be zoned only after consideration of the level of protection provided by such defences and the potential increase in flood defence failure as a result of climate change.

A key element of the RFRA is that it provides a list of best practices in dealing with flood risk. Some of these recommendations that would pertain to the Clare River catchment are [26]:

- Protect natural flood plains that have not yet been developed on and include appropriate flood defences and mitigation measures when redeveloping brown field sites on flood plains.
- No development should be allowed on land required for flood management purposes.
- Strategic Flood Risk Assessments (SFRA) are a necessary part of land zoning so that future development occurs in areas of low flood risk.
- Key infrastructure (existing and future) should be evaluated to ensure that no unnecessary disruption occurs due to decisions made without consideration of flood risk.

The general format of the RFRA is what would be expected from requirements of the PSFRM guidelines. It is generally qualitative in nature identifying situations where flood risk compromises locations identified for growth. These areas should then be addressed in a more detailed manner within city development plans and local area plans.

The Galway County Development Plan was produced for the period 2009-2015. It requires the council to seek to prepare flood zone maps for all zoned lands within the county. All local area plans are required to prepare flood risk zone areas. It does not produce any significant flood extent maps or strategic flood risk assessments. Figure 6.9 is provided within the plan as an indication of flood risk areas within the county. The flood events map shows a band densely populated with flood events situated along a central north-south corridor. This part of the county possesses a significant flood risk relative to the rest of Galway. This area includes the Clare River Catchment. The majority of flood events are located in the Gort area.

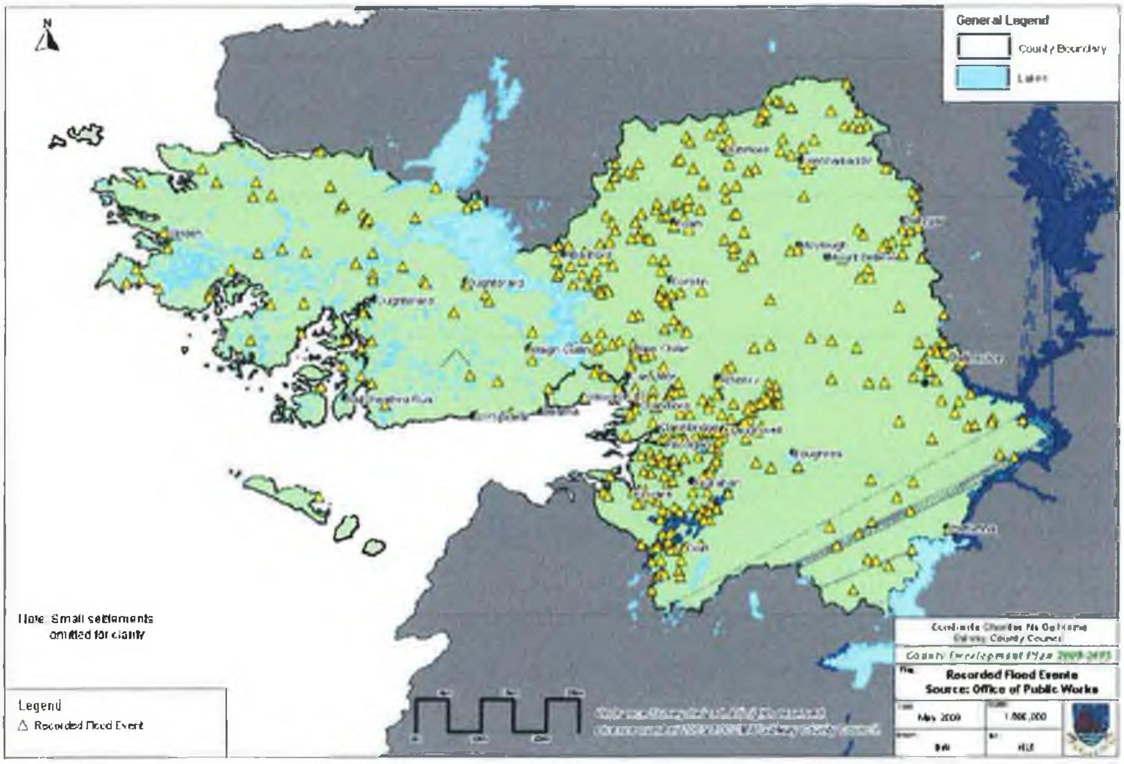

Figure 6.9 – Flood Event Map for Galway [27]

A number of flood risk management and assessment policies are set out within the scope of the county development plan. It states that inappropriate development in areas at risk from flooding should be restricted. SUDS are to be incorporated into all significant developments. Development in flood risk areas will only be permitted when it can be verified that mitigation measures will reduces flood risk to the development to an acceptable level without increasing flood risk elsewhere. Flood studies are required with all planning applications proposed in flood risk areas to ensure that the development does not increase the flood risk in the relevant catchment. Generally a flood impact assessment will be required with all significant developments and a certificate from a competent person stating that the development will not contribute to flooding within the relevant catchment will be required with all small developments of areas of 1 hectare or less [27].

The Galway county development plan aims to carry out flood risk management in line with the PSFRM guidelines. It is intended to ensure appropriate zoning of land and to restrict land use in consideration of flood risk and flood extent. The development plan also states that development in areas at flood risk will only be considered along with mitigation measures in line with the PSFRM guidelines justification test. The development plan highlights the importance of consideration of flood risk in relation to key infrastructure. Such infrastructure should not increase the runoff characteristics of the catchment and should not be located in areas at risk from flooding unless justified and appropriate mitigation measures are put in place to reduce flood risk without increasing flood risk elsewhere [27]. Flood risk assessments are to be carried out in conjunction with planning decisions where necessary. Flood design standards proposed by the plan indicate the consideration of a 100-year event for urban and built up areas and a 25-year event for less vulnerable rural areas.

The Claregalway Local Area Plan (LAP) does acknowledge the importance of addressing flooding within its development strategy. Surface water attenuation proposals are required for all developments over 0.5 ha to ensure that there is no increased flood risk due to an increase in the rate and quantity of surface water runoff. Developments over 1 ha require a flood risk assessment and hydrological report. A certificate from a competent person that the development is not liable to flooding, and will not contribute to flooding within the catchment of River Clare and associated watercourses, must accompany applications for planning permission for development of areas of 1 ha or less, within and directly adjacent to the indicative floodplain area. These measures meet the requirements of the Galway County Development Plan.

Figure 6.10 accompanies the Claregalway LAP and shows an indicative flood plain area that determines an area of potentially high flood risk. This indicative flood plain corresponds reasonably well with the spatial extent of the November 2009 floods. The

2009 flooding was most probably a 100-year event, as suggested by the more extensive records from Ballygaddy and Corofin even though analysis of the relatively short records at Claregalway suggests that it may be a more frequent event. Therefore the indicative flood plain would be in line with the county development plan, which states that development within urban settlements should consider a design flood of 100 years. It does state that should the OPW produce flood plain maps during the lifetime of the LAP that they will supersede the LAP's indicative floodplain. However the spatial extent of the November 2009 events determined by this study suggests that the indicative map provides a good match for actual flooding. The zoning of land takes due consideration of the indicated floodplain. The floodplain land is zoned for amenity and agricultural purposes as shown in figure 6.11. The LAP does not alter land zoning in consideration of the turlough that forms during the winter months when the river is in flood. In fact the formation of this turlough is only mentioned briefly in the introduction of the LAP. The turlough and fluvial flooding associated with the November 2009 events is shown in figure 6.12. It should also be noted that the proposed N17 bypass shown to the east of Claregalway in figure 6.10 possesses a significant potential to affect river discharge if not designed correctly. Water infiltration areas should be provided along the new road to ensure that there is no increase in surface water runoff. However the most significant impact it could have on flows in the Clare River would be due to the new bridge that would be required upstream from the existing Claregalway Bridge. It has been identified that in times of high flow the existing Claregalway Bridge produces increased floodwater levels in the section of river immediately upstream of its location due to acting as a hydraulic constraint. A new bridge should be designed to provide adequate capacity to accommodate significant flood flows (i.e. 100-year event plus 20% allowance for climate change). Bridge piers should be located outside of the channel and preferably outside of the floodplain also.

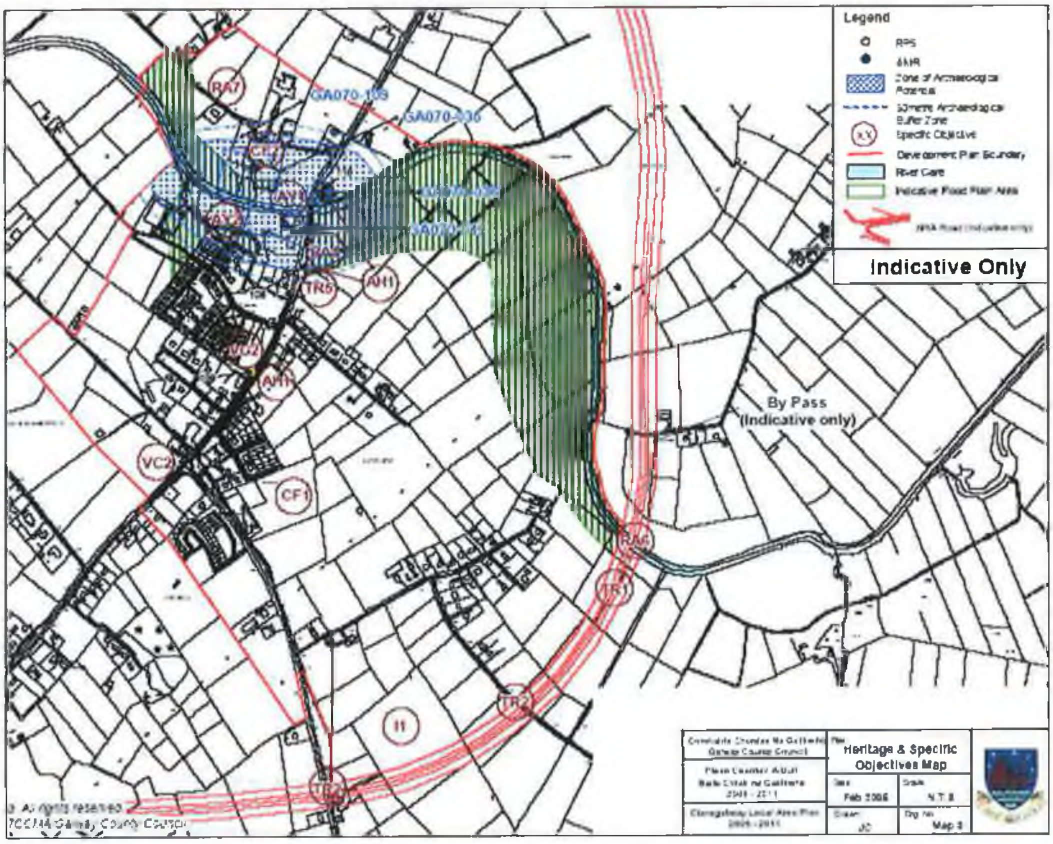

Figure 6.10 – Indicative Floodplain for Claregalway LAP [28]

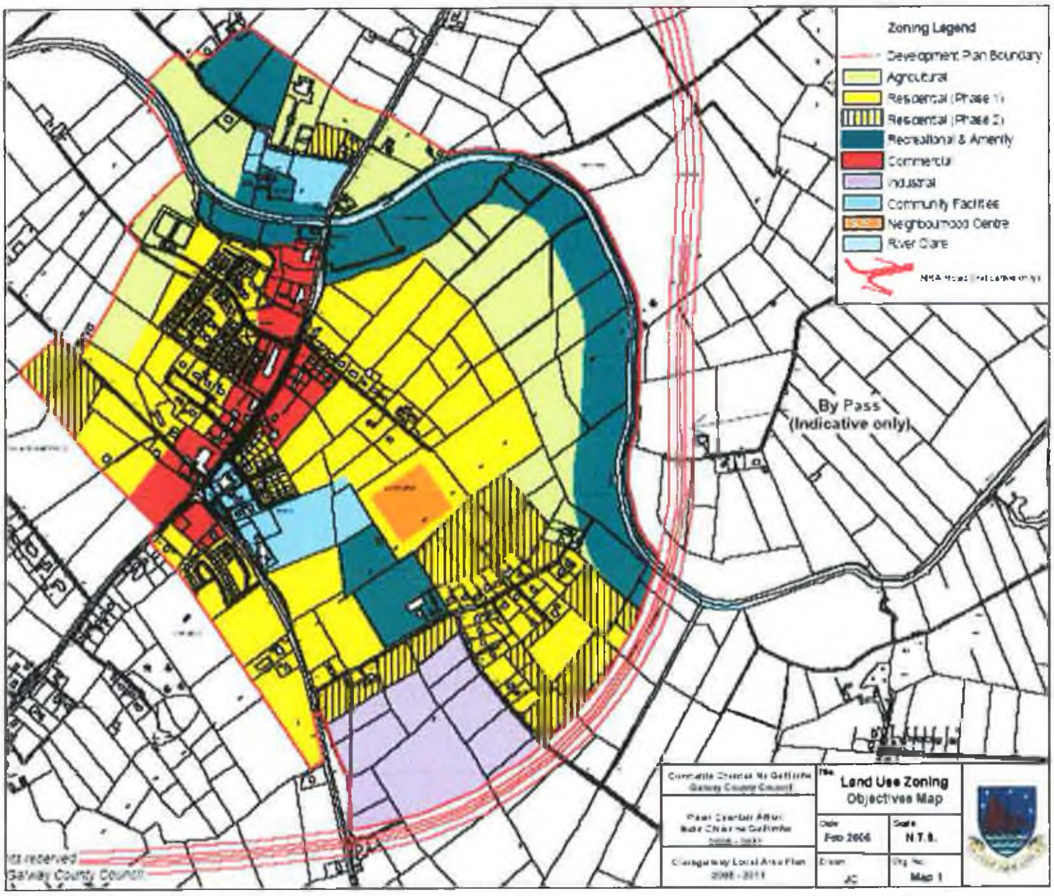

Figure 6.11 – Land Zoning for Claregalway LAP [28]

Figure 6.12 – Flood Extent Map for November 2009 Flood at Claregalway

The LAP states that developments within Claregalway must make reasonable attempts to reduce surface water runoff by employing measures such as SUDS and surface water attenuation. All new developments are required to be designed to meet the 200-year design flood standards. Any developments within the floodplain should consider the impact on flows in the river and floodplain e.g. wide bridge piers, embankments etc. The Claregalway LAP appears to satisfactorily deal with the issue of flood risk in relation to planning and development. This will help to ensure that development within Claregalway is carried out in a manner that will avoid increasing flood risk in the area. However the lack of an indicative floodplain for the wider Claregalway area may result in poorly informed decisions being made for more rural one-off development in areas such as Montiagh and Caherlea, which were both significantly affected by the flooding of November 2009. These areas should therefore make use of flood extent maps associated with the 2009 floods to indicate areas where development requires a detailed consideration of flood risk.

Tuam has taken a far less proactive approach than Claregalway. Tuam was identified in the draft Regional Planning Guidelines flood risk appraisal as a town that had flooding issues. The draft RFRA stated that mandatory flood risk assessments should be required. It is also a hub town identified for future growth. Therefore it should address flood risk within the parameters of its LAP to ensure that future development adheres to the objectives of the PSFRM guidelines. Certain areas close to the confluence of the Nanny-Clare are zoned for industrial and residential by the LAP despite flooding in the area in November 2009 as highlighted in figure 6.13 and 6.14. Tuam LAP provides no indicative floodplains to support the land zoning decisions. The sequential approach followed by a justification test if necessary would be an essential mechanism in ensuring that development is carried out in a sustainable manner that does not increase flood risk locally or elsewhere. The impact of development on the hydrology of the Clare River should be considered more carefully as planning decisions made in Tuam will have an effect locally and also in the wider catchment. Poorly informed decisions may result in increasing flood risk elsewhere where the effect of flood risk may be far more significant than in the Tuam locality.

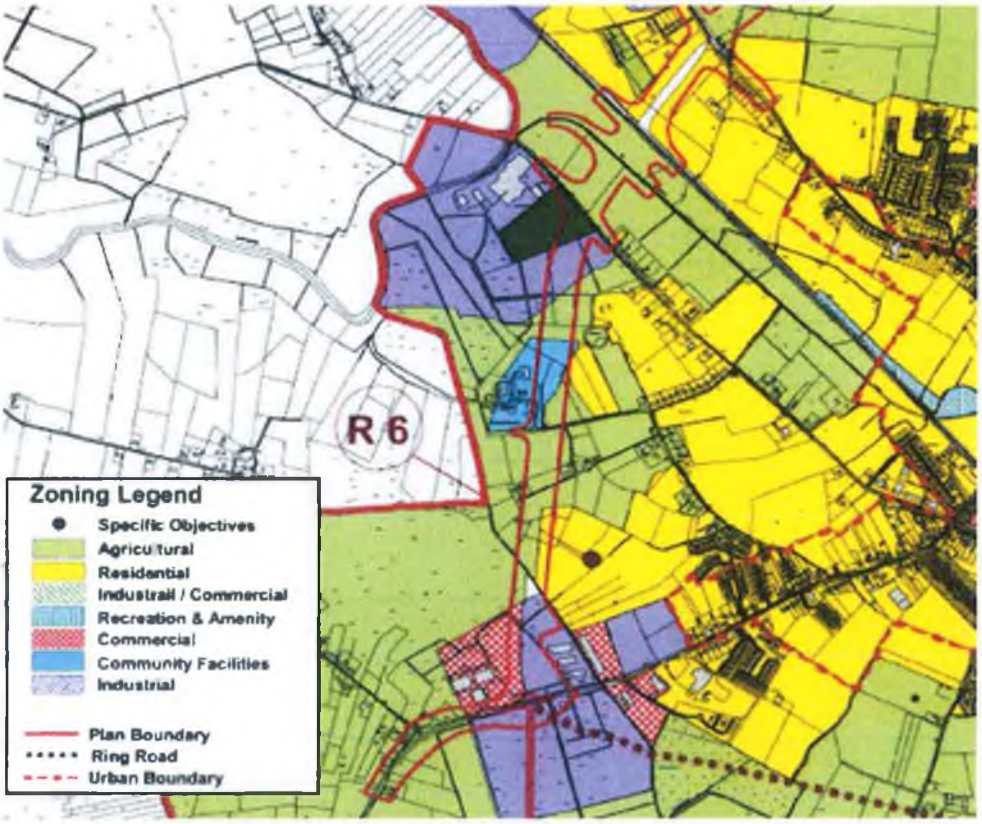

Figure 6.13 – Land Zoning For Tuam Local Area Plan [29]

Figure 6.14 – Flood Extent Map for November 2009 Flood at Tuam

6.4 Land Use in Clare River Catchment

The Clare River catchment is primarily rural. Figure 6.15 shows the corine land cover for the region with an accompanying legend. Each land cover is described by the legend descriptors provided in Appendix B–5. The vast majority of the catchment is described by the corine code 2.3.1 indicating pastureland. The second most notable land cover is peat bogs indicated by code 4.1.2. There are also some areas of complex cultivation patterns, signified by 2.4.2, dispersed throughout the southern extent of the catchment. Urban and developed areas are indicated by corine codes beginning with 1. There is only a very small portion of the catchment described using these codes. Only 3.98 km^2 is described as urban fabric, 0.17 km^2 is described as industrial, commercial and transport units and 0.39 km^2 is described as mine, dump and construction sites. The majority of the urban fabric is identified at Tuam. The total developed land cover equates to 4.5 km^2, less than 0.5% of the entire catchment area. This is thought to underestimate the urban land area within the catchment, as it does not describe areas such as Claregalway as urban. The true urban land cover is conservatively estimated at about 3 times this figure as shown in section 6.6.

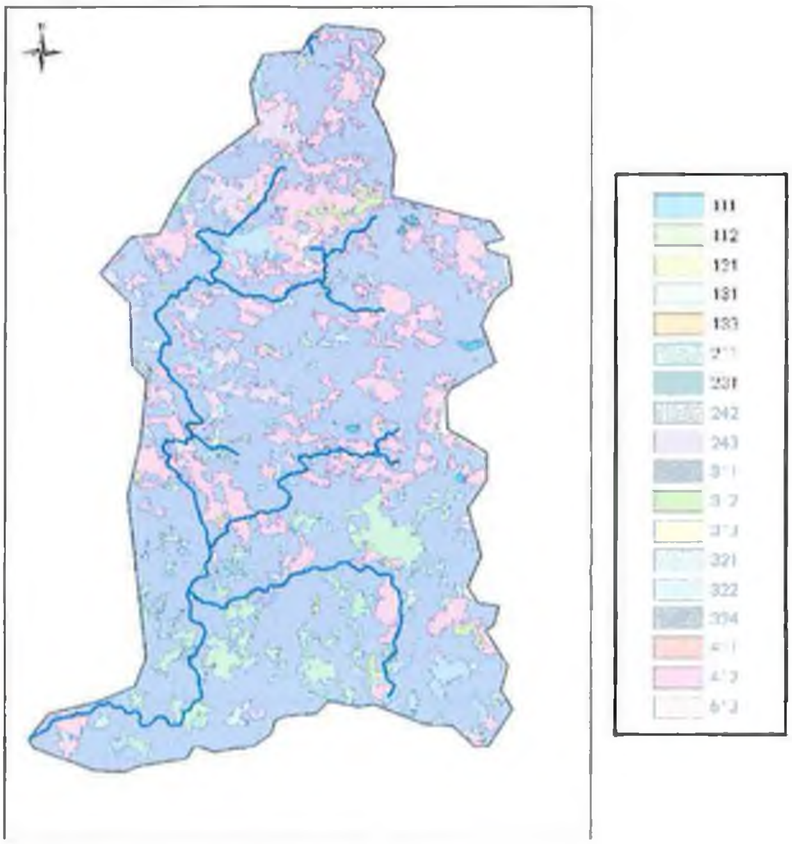

Figure 6.15 – Corine Land Cover

The majority of the Clare River catchment is rural. Figure 6.16 shows the spatial distribution of housing throughout the Galway portion of the catchment in 1996. It shows that the highest housing density is located in the west and southwest of the catchment. The majority of the catchment exhibits a housing density of less than 10 houses per km^2.

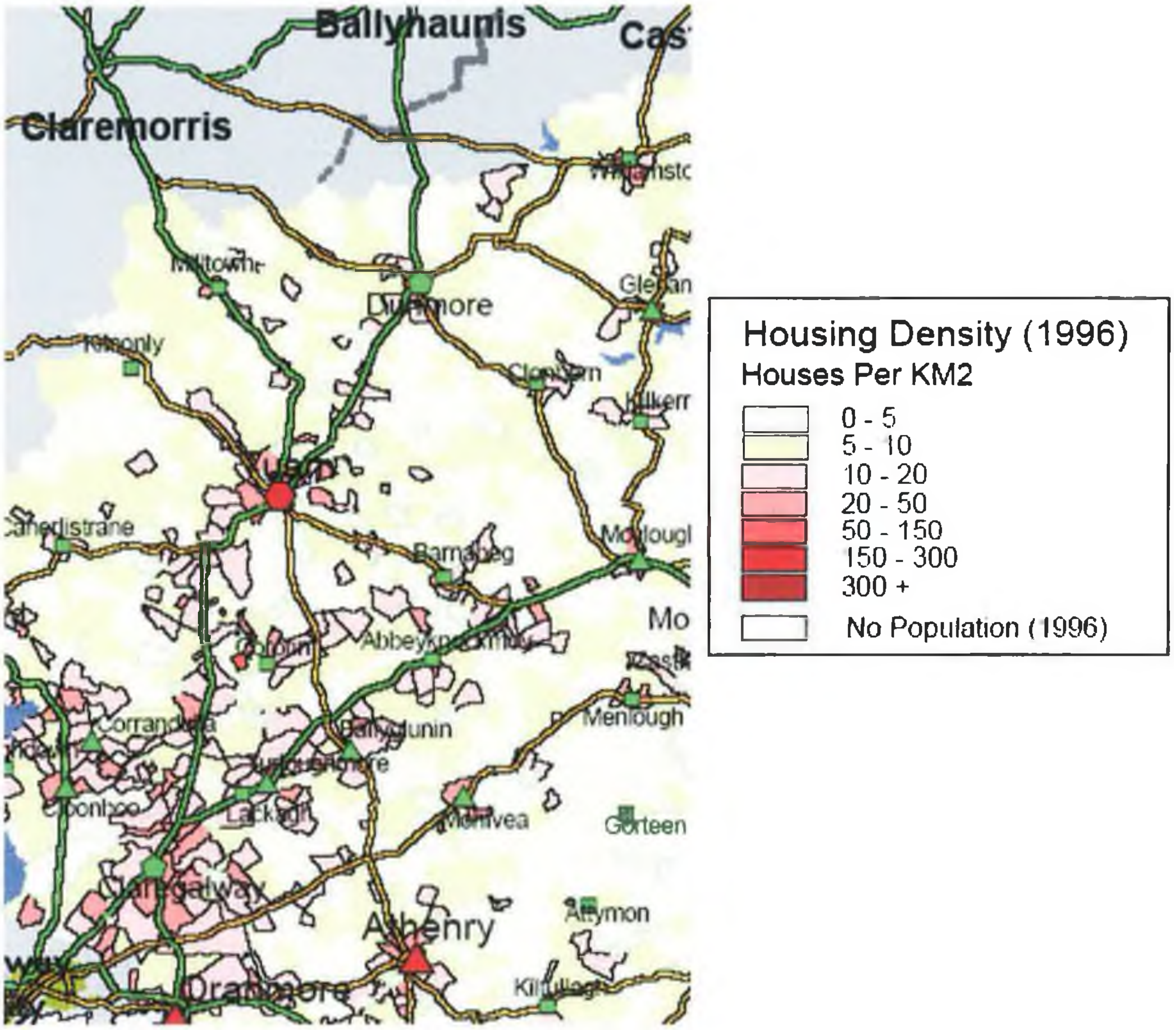

Figure 6.16 – Housing Density [27]

The distribution of the population throughout the urban settlements in the catchment is shown in figure 6.17. The most populated town located within the catchment is Tuam. The population of Tuam was estimated at 4,622 in 2006. There were 2,104 residences located within the Tuam area in 1996 according to the demographic report accompanying the Galway County Development Plan 2009-2015 [30]. The second most notable urban settlement is Ballyhaunis with a population of 2,649 in 2006. Other notable urban centres include Claregalway, Dunmore, Corofin, Monivea, Glenamaddy and Turloughmore.

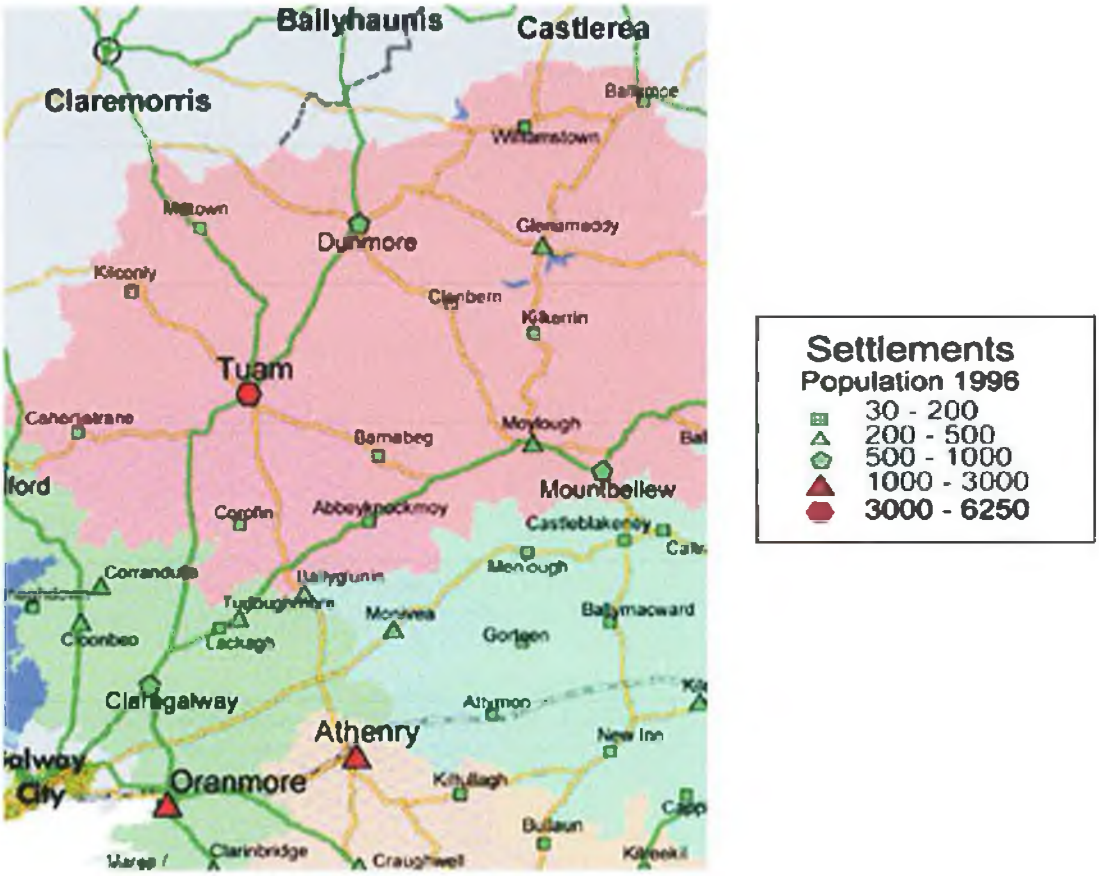

Figure 6.17 – Population of Urban Settlements [27]

The vast majority of land within the catchment is rural. The contribution of one-off housing to surface water runoff in the catchment is expected to be insignificant due to the low housing density of rural areas. Tuam, Ballyhaunis and Claregalway are the most significant of the population centres. Section 6.6 analyses the potential impact that the urban areas may have on flows in the Clare River.

6.5 Influence of Vegetation Cover

The vegetation type and cover determines the degree of surface water that is intercepted. Surfaces with little vegetation have little capacity to intercept surface water. Evapotranspiration is the combination of evaporation and transpiration from vegetation. The rate of evapotranspiration is governed by energy supply, water supply and vapour transport. Energy supply comes from heat gains from the sun. Vapour transport is concerned with the rate at which the water vapour is removed from near the evaporation surface and replaced with less humid air that has the capacity to absorb more water vapour. In particularly dry conditions it is the water supply that becomes the limiting factor for evapotranspiration as water evaporates quicker than it can be supplied to the vegetation cover. The basic rate is known as the reference crop evapotranspiration. This is "the rate of evapotranspiration from an extensive surface of 8cm to 15cm tall green grass cover of uniform height, actively growing, completely shading the ground and not short of water" [31]. The combination approach is proposed as the most accurate approach of estimating evapotranspiration rate [31]. This approach combines the aerodynamic and energy balance method. The aerodynamic method assumes that energy supply is not limiting while the energy balance method assumes that vapour transport is not limiting. In most cases both energy supply and vapour transport are limiting and therefore the combined approach is used [11]. The formulae for calculating the aerodynamic and energy balance method and subsequent combination method are shown below [11]:

Energy Balance Method:

The latent heat of vaporisation is given by:

$$l_v = 2.501 \times 10^6 - 2370 \times T \qquad (6.5.1)$$

where: l_v = latent heat of vaporisation (J/kg)

T = air temperature (°C)

The evapotranspiration assuming vapour transport is not limiting is given by:

$$E_r = (R_n / l_v \rho_w)(8.64 \times 10^7) \qquad (6.5.2)$$

where: E_r = rate of evapotranspiration (mm/day)

R_n = radiation intensity (W/m^2)

ρ_w = density of water (kg/m^3)

l_v = latent heat of vaporisation

Aerodynamic Method:

The vapour transport coefficient is given by:

$B = 0.0027\ (1 + u / 100)$ (6.5.3)

where: u = 24hr wind run at height 2m in km/day

(Distance that an air particle would travel in the air stream at 2m)

The vapour pressure at the ground surface is calculated using:

$e_{as} = 611 \exp [(17.27\ T) / (237.3 + T)]$ (6.5.4)

where: e_{as} = vapour pressure at the ground surface (Pa)

T = air temperature (°C)

The ambient vapour pressure in air is given by:

$e_a = R_h\ e_{as}$ (6.5.5)

where: e_a = ambient vapour pressure in air (Pa)

R_h = relative humidity ($0 \geq R_h \leq 1$)

The results of equations 6.5.3 to 6.5.5 are then combined in equation 6.5.6 to give the evapotranspiration assuming energy supply is not limiting:

$E_a = B\ (e_{as} - e_a)$ (6.5.6)

where: E_a = evapotranspiration rate (mm/day)

B = vapour transport coefficient

e_{as} = vapour pressure at surface (Pa)

e_a = ambient vapour pressure in air (Pa)

Combination Method:

The gradient of the saturated water vapour pressure curve is given by:

$\Delta = 4098\ e_{as} / (237.3 + T)^2$ (6.5.7)

where: Δ = gradient of the saturated water vapour pressure curve (Pa / °C)

e_{as} = vapour pressure at surface (Pa)

T = air temperature (°C)

The psychometric constant is given by:

$\gamma = C_p\ K_h\ p / 0.622\ l_v\ K_w$ (6.5.8)

where: C_p = specific heat at constant pressure p = atmospheric pressure

l_v =latent heat of vapourisation

K_h / K_w = heat diffusivity / water vapour diffusivity

Note: K_h / K_w is commonly taken as 1

The reference crop evapotranspiration rate is obtained by combining the results from equations 6.5.2, 6.5.6, 6.5.7 and 6.5.8 in equation 6.5.9:

$$E_{tr} = (\Delta E_r) / (\Delta + \gamma) + (\gamma E_a) / (\Delta + \gamma) \qquad (6.5.9)$$

where: E_{tr} = reference crop evapotranspiration (mm/day)

E_r = evapotranspiration rate from energy balance method (mm/day)

E_a = evapotranspiration rate from aerodynamic method (mm/day)

Δ = gradient of the saturated water vapour pressure curve (Pa / °C)

γ = psychometric constant (Pa / °C)

The potential evapotranspiration is then calculated by multiplying E_{tr} by a crop coefficient (k_c):

$$E_t = k_c E_{tr} \qquad (6.5.10)$$

where: E_t = Potential evapotranspiration (mm/day)

k_c = crop coefficient

E_{tr} = reference crop evapotranspiration (mm/day)

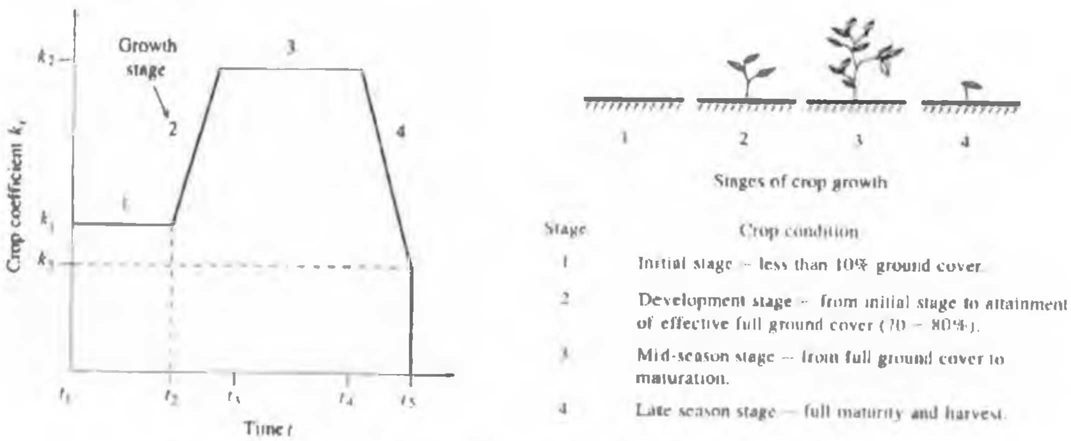

Figure 6.18 – The Relationship Between the Crop Coefficient (k_c) and the Stage of Crop Growth [11]

The value of k_c varies depending upon the vegetation cover. The typical range varies from 0.2 to 1.3. Figure 6.18 shows the relationship between the crop coefficient and the stage of crop growth. The actual evapotranspiration is found by multiplying by a stress coefficient (k_s). This takes into account factors such as water supply and soil structure. The following example is carried out to demonstrate the difference in the potential evapotranspiration rate depending upon different vegetation types. This will indicate the effect of crop cover on surface water runoff.

6.5.1 Estimation of Potential Evapotranspiration for Varying Vegetation Cover

In order to provide an example similar to conditions in the Clare River catchment the example is carried out from data recorded as close to the catchment as was available. This was obtained from Met Eireann monthly reports and was taken from August and September values of 2009. This will give an indication of potential evapotranspiration rates leading into the wetter winter months.

R_n = net radiation = 130 W/m^2

T = air temperature = 13.6 °C

R_h = relative humidity = 82% = 0.82

Wind speed @ height of 10m = 5 m/s

P = Atmospheric Pressure = 101.3 kPa

C_p = specific heat at constant pressure = 1005 J / kg K for air

ρ_w = density of water = 999.3 kg/m^3 @ 13.6 °C

$K_h / K_w = 1$

Energy Balance Method:

Using equation 6.5.1:

$l_v = 2.501 \times 10^6 - 2370 \times T$

$= 2.501 \times 10^6 - 2370 \times (13.6) = 2{,}468{,}768$ J/kg

Using equation 6.5.2:

$E_r = (R_n / l_v \rho_w)(8.64 \times 10^7)$

$= (200 / (2468768 \times 999.3))(8.64 \times 10^7) = 4.55$ mm/day

Aerodynamic Method:

Using equation 6.5.3:

$B = 0.0027\,(1 + u / 100)$

u is the wind run at height 2m in km/day. The wind speed records indicate a speed of 5m/s @ height 10m. The height at 2m can be interpolated using a wind profile. This mathematical method of interpolating wind speed at different heights is given by:

$u @ z = u @ h\,(z / h)^{1/7}$

where: u = wind speed (u @ h = 5 m/s)

z = height for wind speed to be calculated = 2m

h = height of initial measurement = 10m

Therefore:

$u @ 2m = 5\,(2 / 10)^{1/7} = 3.97$ m/s

Using equation 6.5.3:

$B = 0.0027\,(1 + u / 100) = 0.0027\,(1 + 3.97 / 100) = 0.01196$

Using equation 6.5.4:

$e_{as} = 611 \exp[(17.27\,T) / (237.3 + T)]$

$= 611 \exp[(17.27\,(13.6)) / (237.3 + 13.6)] = 1558.1$ Pa

Using equation 6.5.5:

$e_a = R_h\, e_{as} = 0.82\,(1558.1) = 1277.6$ Pa

Using equation 6.5.6:

$E_a = B\,(e_{as} - e_a) = 0.01196\,(1558.1 - 1277.6) = 3.35$ mm/day

Combination Method:

Using equation 6.5.7:

$\Delta = 4098\, e_{as} / (237.3 + T)^2 = 4098\,(1558.1) / (237.3 + 13.6)^2 = 101.4$ Pa/°C

Using equation 6.5.8:

$\gamma = C_p\, K_h\, p / 0.622\, l_v\, K_w = 1005\,(1)\,101300 / (0.622\,(2468768)) = 66.3$ Pa/°C

Using equation 6.5.9:

$E_{tr} = (\Delta E_r) / (\Delta + \gamma) + (\gamma E_a) / (\Delta + \gamma)$

$= (101.4 (4.55)) / (101.4 + 66.3) + (66.3 (3.35)) / (101.4 + 66.3)$

$= 4.08$ mm/day

This reference crop evapotranspiration (E_{tr}) can then be converted to potential evapotranspiration (E_t) by multiplying by the crop coefficient (k_c). The value of k_c varies depending upon the land use. Some values of k_c and the corresponding value of E_t are given in table 6.7. The stage indicated refers to the stage of crop development as defined in figure 6.18.

Table 6.7 – Crop Coefficients and Potential Evapotranspiration Rate for Example 6.5.1

Type of Crop	Development Stage	Crop Coefficient k_c [32]	Potential Evapotranspiration $E_t = k_c (E_{tr})$ (mm/day)
Grassland (Rotated Grazing)	k_3	0.85 - 1.05	3.47 - 4.28
Grassland (Extensive Grazing)	k_3	0.75	3.06
Hay	k_3	0.9014	3.68
Hay	k_4	0.85	3.47
Cereals (e.g. Barley, Oats)	k_3	1.15	4.69
Cereals (e.g. Barley, Oats)	k_4	0.4	1.63
Conifer Trees (@ height 10m)	k_1 - k_4	1	4.08
Wetlands	k_3	1.2	4.90

Table 6.7 shows that overgrazing of grassland pastures can lead to a difference of 1.2 mm/day in E_{tr} for conditions assumed by this example. There is a negligible difference between the potential evapotranspiration rate of hay before and after harvest. Cereals produce a high value for E_{tr} in a fully developed stage pre-harvest (k_3). However the value of E_{tr} for cereals drops significantly after harvesting to 1.63 mm/day. Most pastures would be harvested during the summer. Therefore grassland would provide more beneficial evapotranspiration than cereals leading into winter months. Conifer trees provide a year round crop coefficient of 1.0. Wetlands provide the best condition from the above table with E_{tr} equating to 4.90 mm/day. Therefore in scenarios where wetlands are reclaimed for grazing purposes and subsequently extensively grazed there would be a drop in E_{tr} of 1.84 mm/day.

It should be noted that these values are only indicative of a specific set of conditions and are only provided to give an idea of the effect of different agricultural land usage on the potential evapotranspiration rate. This rate is also not a measure of the actual evapotranspiration that may occur. Actual evapotranspiration rate is also a function of other factors such as soil conditions and water supply and must therefore be multiplied by a further stress coefficient (k_s) as mentioned above.

6.6 Impact of Urban Development on the Synthetic Unit Hydrograph

The Flood Studies Report (FSR) method was used to assess the potential impact that the urban portion of the Clare River catchment has on flood flows. A synthetic unit hydrograph was produced for two scenarios. Scenario A estimated that urban development accounted for 1.3% of the total catchment area as described in the calculations. Scenario B assumed no urban development in the catchment. The FSR provides a method for synthesising a 1-hr unit hydrograph for an ungauged catchment. As the Clare River catchment is only gauged for current conditions an ungauged method is required to generate the 1-hr unit hydrograph assuming there is 0% urban development. The method must then also be applied to an urban portion of 1.3% to provide a comparison scenario as figures generated by the FSR method would not be directly comparable to those generated using statistical analysis techniques such as the EV1 distribution (Gumbel) method.

Scenario A

1) The catchment area (AREA) was determined from the catchment layout map generated on GIS software.

   AREA = 1078 km^2

   The mainstream length (MSL) was determined from catchment layout map

   MSL = 93 km

2) The channel slope (S1085) is the average of the slope in m per km between two points at 10% and 85% of the mainstream length from the outlet.

   Elevation at 85% MSL = 64.5 mAod

   Elevation at 10% MSL = 6 mAod

   Distance between points = 69.75 km

   S1085 = 0.84

3) The average annual rainfall (SAAR) was taken as the average of 1971-2000 for the catchment to Claregalway.

   SAAR = 1201.7 mm

Figure 6.19 – RSMD (mm) for Ireland [33]

4) The RSMD can be estimated both graphically and mathematically. The RSMD is the 1-day M5 rainfall less effective mean soil moisture deficit. Figure 6.19 provides a graphical estimation of RSMD in the area of the catchment (approximately 42.5). The RSMD was calculated mathematically as shown below for more accurate results.

   2-day M5 rainfall (average for catchment) = 60 mm (Appendix D–2)

   r = 30% (Appendix D–3)

   r is the ratio of 60-minute M5 to 2-day M5

   Therefore using table 6.8 the M5 rainfall amount as percentage of 2-day M5 rainfall was estimated as 0.85. This is multiplied by the 2-day M5 rainfall to give the 24-h M5 rainfall: 0.85 x 60 = 51mm

   Note: M5 rainfall is the rainfall depth with a return period of 5-years. It is adopted as the reference frequency [33].

Table 6.8 – Model for M5 Rainfall for Durations up to 48-hours [33]

r (per cent)	M5 rainfall (amounts as percentages of 2-day M5)												
	1 min	*2 min*	*5 min*	*10 min*	*15 min*	*30 min*	*60 min*	*2 h*	*4 h*	*6 h*	*12 h*	*24 h*	*48 h*
12	0.8	1.4	2.7	4.2	5.4	8.1	12	18	26	33	49	72	106
15	1.2	2.1	3.8	5.8	7.2	10.5	15	21	30	37	53	75	106
18	1.6	2.8	5.0	7.4	9.2	12.9	18	25	34	41	56	77	106
21	2.1	3.5	6.3	9.2	11.2	15.5	21	28	38	45	60	80	106
24	2.5	4.3	7.6	11.0	13.3	18.1	24	31	41	48	63	81	106
27	3.0	5.0	9.0	12.9	15.5	20.7	27	35	44	51	65	83	106
30	3.3	5.7	10.3	14.8	17.7	23.3	30	38	48	55	68	85	106
33	3.8	6.5	11.7	16.7	19.9	26.0	33	41	51	57	71	87	106
36	4.1	7.2	13.0	18.6	22.2	28.7	36	44	54	60	73	88	106
39	4.6	8.0	14.5	20.6	24.5	31.5	39	47	57	63	75	89	106
42	5.0	8.7	16.0	22.7	26.9	34.2	42	50	60	66	77	91	106
45	5.4	9.5	17.4	24.7	29.2	37.0	45	53	63	68	79	92	106

This must then be divided by the multiplying factor, table 6.9. This converts it into the 1-day M5 value:

51 / 1.11 = 45.95 mm

Table 6.9 – Factors to Relate M5 Values for Rainfall Hours and Rainfall Days [33]

Rainfall days	1	2	4	8
Multiplying factor	1.11	1.06	1.03	1.015
Rainfall hours	24	48	96	192

This is multiplied by the areal reduction factor (ARF) estimated as 0.89 from table 6.10 to give the 1-day M5 catchment rainfall = 0.89 x 45.95 = 40.9 mm

Table 6.10 – Areal Reduction Factor (ARF) [33]

Duration D	*Area A (km^2)*									
	1	*5*	*10*	*30*	*100*	*300*	*1000*	*3000*	*10 000*	*30 000*
1 min	0.76	0.61	0.52	0.40	0.27	–	–	–	–	–
2 min	0.84	0.72	0.65	0.53	0.39	–	–	–	–	–
5 min	0.90	0.82	0.76	0.65	0.51	0.38	–	–	–	–
10 min	0.93	0.87	0.83	0.73	0.59	0.47	0.32	–	–	–
15 min	0.94	0.89	0.85	0.77	0.64	0.53	0.39	0.29	–	–
30 min	0.95	0.91	0.89	0.82	0.72	0.62	0.51	0.41	0.31	–
60 min	0.96	0.93	0.91	0.86	0.79	0.71	0.62	0.53	0.44	0.35
2 h	0.97	0.95	0.93	0.90	0.84	0.79	0.73	0.65	0.55	0.47
3 h	0.97	0.96	0.94	0.91	0.87	0.83	0.78	0.71	0.62	0.54
6 h	0.98	0.97	0.96	0.93	0.90	0.87	0.83	0.79	0.73	0.67
24 h	0.99	0.98	0.97	0.96	0.94	0.92	0.89	0.86	0.83	0.80
48 h	–	0.99	0.98	0.97	0.96	0.94	0.91	0.88	0.86	0.82
96 h	–	–	0.99	0.98	0.97	0.96	0.93	0.91	0.88	0.85
192 h	–	–	–	0.99	0.98	0.97	0.95	0.92	0.90	0.87
25 days	–	–	–	–	0.99	0.98	0.97	0.95	0.93	0.91

Soil moisture deficit (smd) is estimated from Appendix D-4 as 2.3mm. This is subtracted from the 1-day M5 catchment rainfall to give:
RSMD = 40.9 – 2.3 = 38.6 mm

5) The fraction of the catchment under urban development (URBAN) was estimated from evaluation of the corine land cover map and population distribution from the Galway County Development Plan 2009-2015. The corine land cover estimated that 4.5 km^2 in the catchment was urban. This was thought to be a low estimate, as the corine land cover did not define some areas such as Claregalway as urban. Using population figures for the main urban towns [34] and upper estimates of population for the remaining urban settlement [27] the population living in urban settlements was estimated as 14,006 people. 2 persons per residence was assumed as estimated from population density and housing density figures for 1996 [27]. An average area of 2,000 m^2 (0.5 acres) was assigned per residence to see if this provided an approximation of total urban area associated with housing densities. This was then used to calculate the area of towns such as Tuam of known population and housing density to see if calculated area was a reasonable match for mapped information. It was felt that assigning 2000 m^2 to housing density estimates provided a reasonably good estimate of entire urban area, including commercial and industrial, associated with housing density figures. It also provided a degree of overestimation providing a factor of safety. The housing density within urban settlements was estimated as 7,003 residences. This was estimated to indicate a total urban land area of 14 km^2 in the Clare River catchment. This equated to 1.3% of the entire catchment.
URBAN = 0.013

6) The time to peak (T_p) of the 1-hr unit hydrograph measured from the start of response runoff is calculated using:

$$T_p = 46.6\,(MSL)^{0.14}\,(S1085)^{-0.38}\,(1+URBAN)^{-1.99}\,(RSMD)^{-0.4}$$

$$= 46.6\,(93)^{0.14}\,(0.84)^{-0.38}\,(1+0.013)^{-1.99}\,(38.6)^{-0.4}$$

$$T_p = 21.23 = 21 \text{ hrs}$$

The peak of the unit hydrograph in m^3/s per $100km^2$ is given by:

$$Q_p = 220 / T_p = 10.5 \ m^3/s \text{ per } 100km^2$$

This equates to 112.9 m^3/s for the entire 1078 km^2 catchment.

The time base (TB) is the width of the base of the runoff hydrograph and is given by:
TB = 2.52 Tp = 2.52 (21.23) = 53.5 = 54 hrs

7) The basic data interval (T) is approximately T_p / 5. Therefore T was taken as 3 hours.

8) The design storm duration (D) can now be calculated using the following equation:
$D = (1 + SAAR/1000)\ T_p$
$= (1 + 1201.7 / 1000)\ 21 = 46$
D is taken as 45 hours, an odd integer multiple of T, for calculation purposes.

9) The next step is to estimate the storm return period associated with the flow return period being analysed. The return period of a storm associated with a 100-year peak flow is 140-years (figure 6.20)

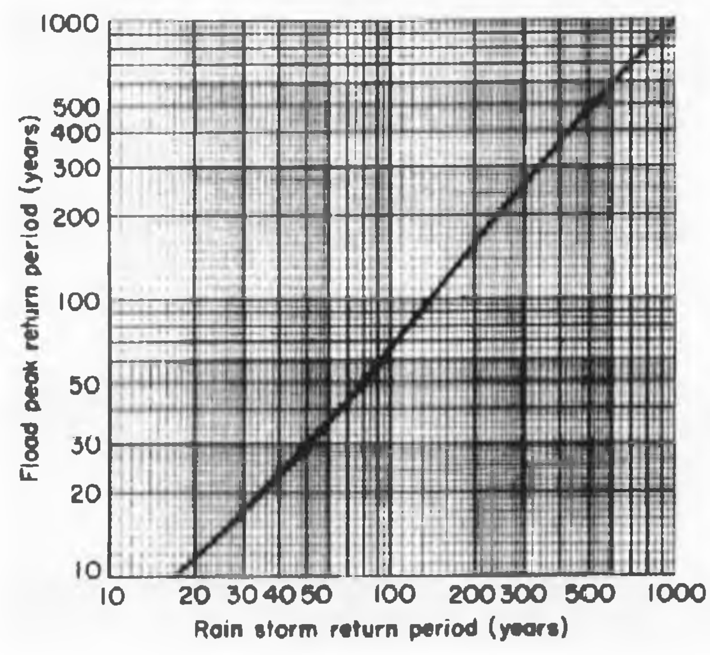

Figure 6.20 – Recommended Storm Return Period to Yield a Flood Peak of Required Return Period by the Design Method [33]

10) The rainstorm is 45 hours duration and has a return period of 140-years. r = 30% from part 4. Therefore the M5 rainfall amount as percentage of 2-day M5 rainfall was estimated as 103.4% from table 6.8. This is used to factor the 2-day M5 rainfall of 60 mm: 45hr M5 value = 1.034 x 60 = 62.04 = 62mm

11) The 45-hr M5 value is now converted to 45 hr M140 value by estimating the growth factor from table 6.11.

Growth factor = 1.7

45-hr M140 = 1.7 x 62 = 105.4 mm

Table 6.11 – Growth Factors MT / M5 [33]

M5 (mm)	*Partial duration series* 2M	1M	*Annual maximum series* M2	M10	M20	M50	M100	M1000	M10 000
0.5	0.55	0.68	0.76	1.14	1.30	1.51	1.71	2.54	3.78
2	0.55	0.68	0.76	1.15	1.31	1.54	1.75	2.65	4.01
5	0.54	0.67	0.76	1.16	1.34	1.62	1.86	2.94	4.66
10	0.55	0.68	0.75	1.18	1.38	1.69	1.97	3.25	5.36
15	0.55	0.69	0.75	1.18	1.38	1.70	1.98	3.28	5.44
20	0.56	0.70	0.76	1.18	1.37	1.66	1.93	3.14	5.12
25	0.57	0.71	0.77	1.17	1.36	1.64	1.89	3.03	4.85
30	0.58	0.72	0.78	1.17	1.35	1.61	1.85	2.92	4.60
40	0.59	0.74	0.79	1.16	1.33	1.56	1.77	2.72	4.16
50	0.60	0.75	0.80	1.15	1.30	1.52	1.72	2.57	3.85
75	0.62	0.77	0.82	1.13	1.26	1.45	1.62	2.31	3.30
100	0.63	0.78	0.83	1.12	1.24	1.40	1.54	2.12	2.92
150	0.64	0.79	0.84	1.10	1.20	1.33	1.45	1.90	2.50
200	0.65	0.80	0.85	1.09	1.18	1.30	1.40	1.79	2.30
500	0.66	0.80	0.86	1.08	1.14	1.20	1.27	1.52	–
1000	0.66	0.80	0.86	1.07	1.12	1.18	1.23	1.42	–

12) This point rainfall value must be reduced to a catchment average using the areal reduction factor (ARF) estimated from table 6.10:

ARF (for 45hr and 1078 km^2) = 0.906

Rainfall P = 0.906 x 105.4 = 95.5 mm

13) The catchment wetness index (CWI) is estimated from figure 6.21 using SAAR = 1201.7 mm. Therefore: CWI = 125

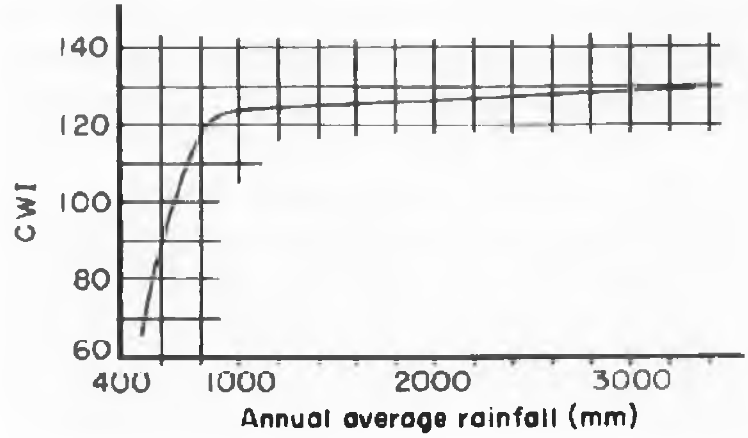

Figure 6.21 – Recommended Design Values for Catchment Wetness Index (CWI) [33]

14) The percentage runoff figure is dependent on the soil type in the catchment and the urban fraction. Soils are classified from 1 to 5 in order of decreasing permeability. Due to the even topography of the catchment and the high percentage of till the soil in the Clare River catchment is 50% S_1 and 50% S_2 as shown in Appendix D–5. The soil index (SOIL) is calculated from the formula:

$$SOIL = (0.15S_1 + 0.30S_2 + 0.40S_3 + 0.45S_4 + 0.50S_5) / (S_1 + S_2 + S_3 + S_4 + S_5)$$

$$= (0.15(0.5) + 0.3(0.5)) / (0.5 + 0.5)$$

$$= 0.225$$

The standard percentage runoff (SPR) is derived using:

$$SPR = 95.5\ SOIL + 0.12\ URBAN$$

$$= 95.5\ (0.225) + 0.12\ (0.013)$$

$$= 21.49\ \%$$

The appropriate percentage runoff for the design event is given by:

$$PR = SPR + 0.22\ (CWI - 125) + 0.1\ (P\text{-}10)$$

$$= 21.49 + 0.22\ (125 - 125) + 0.1\ (95.5 - 10)$$

$$= 30\%$$

The net rainfall to be applied to the synthetic unit hydrograph is given by:

Net rainfall = P (PR) / 100 = 95.5 x 0.30 = 28.68 mm

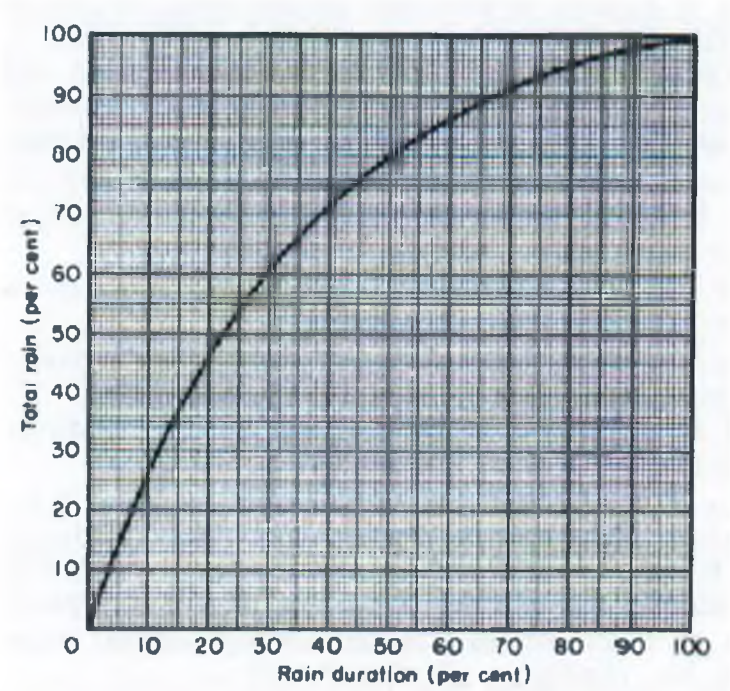

Figure 6.22 – The 'Winter 75%' Storm Profile [33]

15) The net rain is now applied to the unit hydrograph using the 75% winter storm profile shown in figure 6.22. A stepped distribution graph of fifteen 3-hr periods provides the 45-hr storm period. Each 3-hour interval accounts for approximately 6.7% of the overall duration. The rain percentage estimated from figure 6.22 is a cumulative percentage of rain for time intervals centred on the centre of the storm duration. It is then divided into incremental rainfall depths for time intervals located equal temporal distances from the centre of the storm duration, i.e. incremental rainfall for time increment 6 hours to 39 hours will consist of rainfall during the time periods 6 hours to 9 hours and 36 hours to 39 hours. The calculations are shown in table 6.12.

Table 6.12 – Net Rain Distributed for Scenario A Using the 'Winter 75%' Storm Profile

Time increment of duration (hrs)	Duration (time incr./D)*100 (%)	Rain percentage (%)	Increment Rain percentage (%)	Increment Rain Depth (mm)	Increment Rain Depth (cm)
21 to 24	6.7	17.5	17.5	5.02	0.50
18 to 17	20	46	28.5	8.18	0.82
15 to 30	33.3	64.5	18.5	5.31	0.53
12 to 33	46.7	77	12.5	3.59	0.36
9 to 36	60	86	9	2.58	0.26
6 to 39	73.3	92.5	6.5	1.86	0.19
3 to 42	86.7	97.5	5	1.43	0.14
0 to 45	100	100	2.5	0.72	0.07

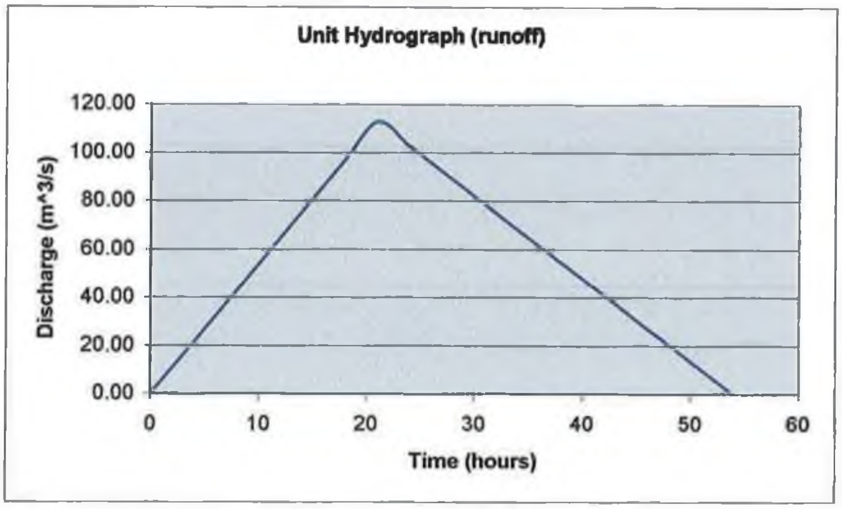

Figure 6.23 – Synthetic Unitgraph for Scenario A

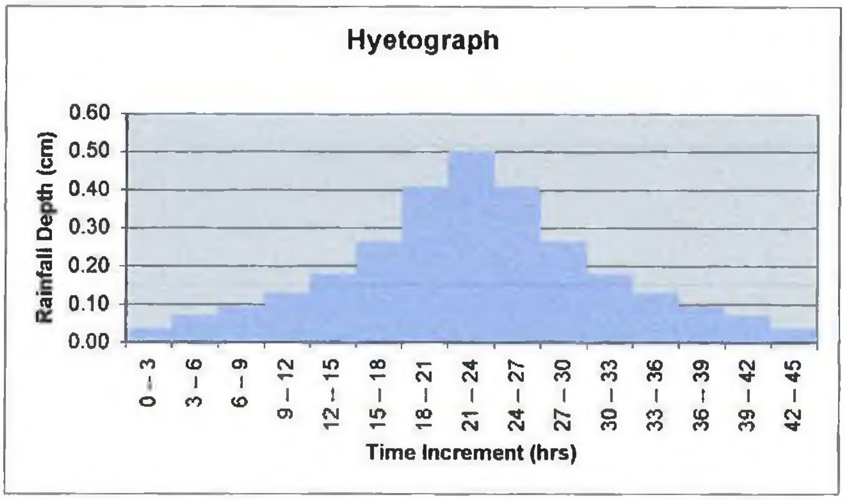

Figure 6.24 – Hyetograph for Scenario A

The unit hydrograph for the catchment and associated flows is shown in figure 6.23. The rainfall from table 6.12 is shown in the stepped distribution graph (figure 6.24) arranged symmetrically about the centre line. It is then applied to the unit hydrograph (net rain column of Appendix F–1).

Each 3-hour increment of rainfall is multiplied in turn by each 3-hour ordinate of the unit hydrograph, successive products being moved 3 hours (1 interval) to the right. The flood flows are calculated by summing the columns for each 3 increment. The following equation is used to estimate the average non-separated flow (ANSF). It has been derived through regression analysis of CWI and catchment characteristics in the British Isles [33]: $ANSF = (3.26 \times 10^{-4})(CWI - 125) + (7.4 \times 10^{-4}) RSMD + (3 \times 10^{-3})$

Therefore:

$$ANSF = (3.26 \times 10^{-4})\,(125 - 125) + (7.4 \times 10^{-4})\,38.6 + (3 \times 10^{-3})$$
$$= 0.0316 \text{ m}^3\text{/s per km}^2$$

ANSF is multiplied by the catchment area to provide the base flow:

$$0.0316 \times 1078 = 34 \text{ m}^3\text{/s}$$

This can then be added to the flood flows to find the total flow at each 3-hr time increment (Appendix F–1). The hydrograph is shown graphically in Figure 6.25. The largest of these values is taken as the peak flow. For the Clare River catchment the peak flow was estimated as 276.83 m^3/s, which occurred 45 hours after the start of response runoff. Flows returned to normal base flow after 81 hours. The FSR method estimates the 100-year event at a greater magnitude than the 180 m^3/s at Claregalway predicted by EV1 distribution (Gumbel) method. This is most probably due to the considerable groundwater leakage from the catchment to the west, which would be included in EV1 distribution as it is a statistical analysis of actual recorded data. This section is assessing the impact of urban development on storm flows and is therefore merely a comparison study. Therefore the impact of groundwater leakage from the Clare River to the west on the flood flow will not be considered.

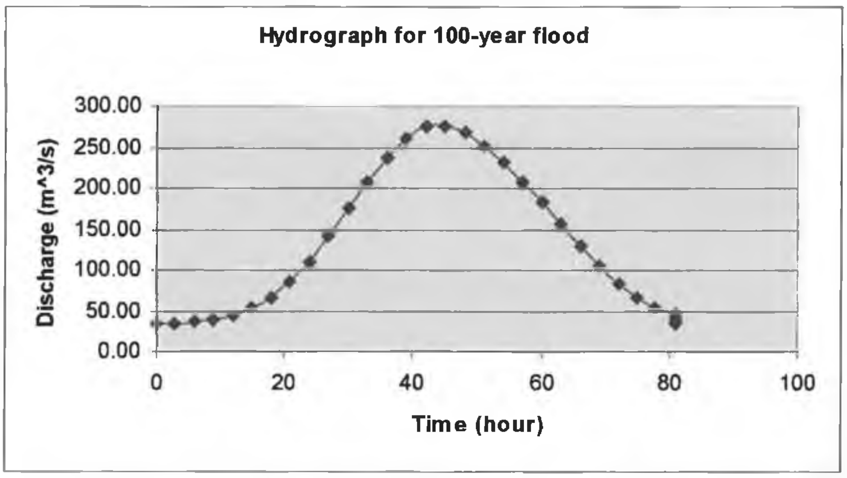

Figure 6.25 – Hydrograph for 100-year Flood for Scenario A

Scenario B

This scenario follows the same method as that used for scenario A. The only difference in the primary data is that the urban fraction of the catchment is zero. Therefore only the calculations that are a function of the URBAN factor will be shown in this section.

1) AREA = 1078 km^2

   MSL = 93 km

2) S1085 = 0.84

3) SAAR = 1201.7

4) RSMD = 40.9 – 2.3 = 38.6 mm

5) This scenario assumes that there is no urban development in the catchment.

   URBAN = 0.0

6) The time to peak (T_p) of the 1-h unit hydrograph measured from the start of response runoff is calculated using:

   $T_p = 46.6\ (MSL)^{0.14}\ (S1085)^{-0.38}\ (1+URBAN)^{-1.99}\ (RSMD)^{-0.4}$

   $= 46.6\ (93)^{0.14}\ (0.84)^{-0.38}\ (1+0)^{-1.99}\ (38.6)^{-0.4}$

   $T_p = 21.78 = 22$ hrs

   The peak of the unit hydrograph in m^3/s per $100km^2$ is given by:

   $Q_p = 220 / T_p = 10\ m^3/s$ per $100km^2$

   This equates to 107.8 m^3/s for the entire catchment.

   The time base (TB) is the width of the base of the runoff hydrograph and is given by:

   TB = 2.52 Tp = 2.52 (22) = 55.5 = 56 hrs

7) The basic data interval (T) is approximately T_p / 5. Therefore T was taken as 2 hours.

8) The design storm duration (D) can now be calculated using the following equation:

   $D = (1 + SAAR/1000)\ T_p$

   $= (1 + 1201.7 / 1000)\ 22 = 48$

9) The return period of a storm associated with a 100-year peak flow is 140-years (Figure 6.20)

10) The rainstorm is 48 hours duration and has a return period of 140-years. r = 30% from part 4. Therefore the M5 rainfall amount as percentage of 2-day M5 rainfall was estimated as 106 % from table 6.8. This is used to factor the 2-day M% rainfall of 60mm:

    48hr M5 value = 1.06 x 60 = 63.6 = 64 mm

11) The 48-hr M5 value is now converted to 48 hr M140 value by estimating the growth factor from table 6.11.

    growth factor = 1.7

    48-hr M140 = 1.7 x 64 = 108.8 mm

12) This point rainfall value must be reduced to a catchment average using the areal reduction factor (ARF) estimated from table 6.10:

    ARF (for 48hr and 1078 km^2) = 0.909

    Rainfall P = 0.909 x 108.8 = 98.9 mm

13) The catchment wetness index (CWI) is the same as scenario A:

    CWI = 125

14) The soil index (SOIL) is the same as scenario A:

    $$SOIL = (0.15S_1 + 0.30S_2 + 0.40S_3 + 0.45S_4 + 0.50S_5) / (S_1 + S_2 + S_3 + S_4 + S_5)$$
    $$= (0.15(0.5) + 0.3(0.5)) / (0.5 + 0.5)$$
    $$= 0.225$$

    The standard percentage runoff (SPR) experienced no significant change as the change in URBAN value was relatively small:

    $$SPR = 95.5\ SOIL + 0.12\ URBAN$$
    $$= 95.5\ (0.225) + 0.12\ (0)$$
    $$= 21.49\ \%$$

The appropriate percentage runoff for the design event is given by:

PR = SPR + 0.22 (CWI – 125) + 0.1 (P-10)

= 21.49 + 0.22 (125 – 125) + 0.1 (95.5 – 10)

= 30.38 %

The net rainfall to be applied to the synthetic unit hydrograph is given by:

Net rainfall = P (PR) / 100 = 98.9 x 0.3038 = 30.04 mm

15) The net rain is now applied to the unit hydrograph using the 75% winter storm profile shown in figure 6.22. A stepped distribution graph of 24 2-hr periods provides the 48-hr storm period. Each 2-hour interval accounts for approximately 4.2 % of the overall duration. The rain percentage estimated from figure 6.22 is a cumulative percentage of rain for time intervals centred on the centre of the storm duration. It is then divided into incremental rainfall depths for time intervals located equal temporal distances from the centre of the storm duration, i.e. incremental rainfall for time increment 10 hours to 38 hours will consist of rainfall during the time periods 10 hours to 12 hours and 36 hours to 38 hours. The calculations are shown in table 6.13.

Table 6.13 – Net Rain Distributed for Scenario B Using the 'Winter 75%' Storm Profile

Time increment of duration (hrs)	Duration (time incr./D)*100 (%)	Rain percentage (%)	Increment Rain percentage (%)	Increment Rain Depth (mm)	Increment Rain Depth (cm)
22 to 26	8.3	21.5	21.5	6.46	0.65
20 to 28	16.7	40	18.5	5.56	0.56
18 to 30	25.0	53.6	13.6	4.09	0.41
16 to 32	33.3	64.5	10.9	3.27	0.33
14 to 34	41.7	72.6	8.1	2.43	0.24
12 to 36	50.0	80	7.4	2.22	0.22
10 to 38	58.3	85	5	1.50	0.15
8 to 40	66.7	89.8	4.8	1.44	0.14
6 to 42	75.0	93.5	3.7	1.11	0.11
4 to 44	83.3	96.2	2.7	0.81	0.08
2 to 46	91.7	98.3	2.1	0.63	0.06
0 to 48	100.0	100	1.7	0.51	0.05

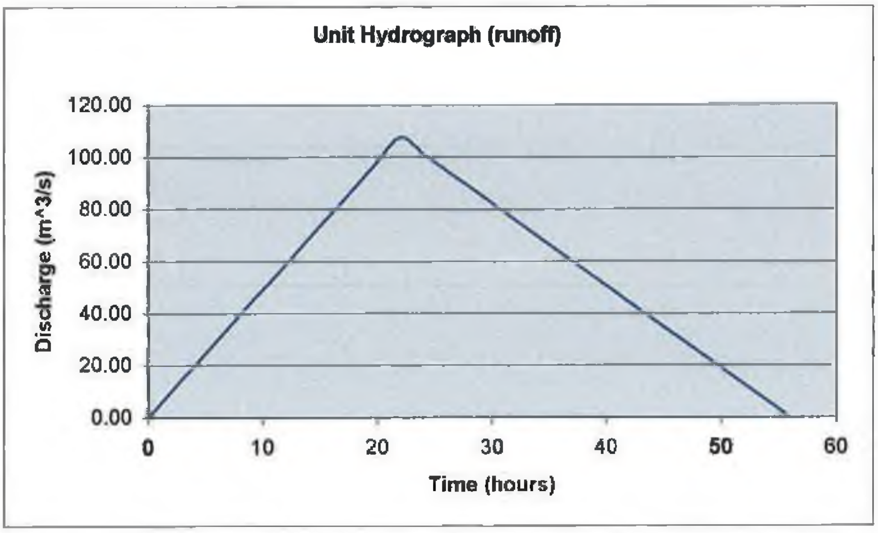

Figure 6.26 – Synthetic Unitgraph for Scenario B

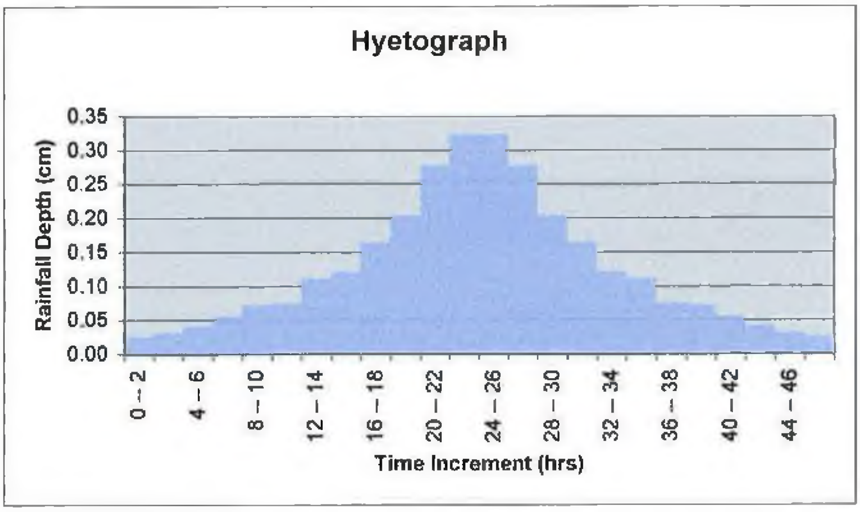

Figure 6.27 – Hyetograph for Scenario B

The unit hydrograph for the catchment is shown in figure 6.26. The rainfall from table 6.13 is shown in the stepped distribution graph of rain is shown in figure 6.27 arranged symmetrically about the centre line. It is then applied to the unit hydrograph (net rain column of Appendix F–2).

Each 2-hour increment of rainfall is multiplied in turn by each 2-hour ordinate of the unit hydrograph, successive products being moved 2 hours (1 interval) to the right. The flood flows are calculated by summing the columns for each 2-hour increment. The following equation is used to estimate the average non-separated flow (ANSF) in the same method as scenario A [33]:

ANSF = (3.26×10^{-4}) (CWI – 125) + (7.4×10^{-4}) RSMD + (3×10^{-3})

Therefore:

ANSF = (3.26×10^{-4}) (125 – 125) + (7.4×10^{-4}) 38.6 + (3×10^{-3})

= 0.0316 m^3/s per km^2

ANSF is multiplied by the catchment area to provide the base flow:

0.0316 x 1078 = 34 m^3/s

This can then be added to the flood flows to find the total flow at each 2-hr time increment (Appendix F–2). The hydrograph is shown graphically in Figure 6.28. The largest of these values is taken as the peak flow. For the Clare River catchment with no urban development the peak flow was estimated as 274.41 m^3/s, which occurred 46 hours after the start of response runoff. The river flows return to base flow after 102 hours. This is an increase in time of 21 hours and is due to the release of attenuated waters from the increased attenuation capacity over a longer period. The peak flow for scenario B is only 2.42 m^3/s less than scenario A and occurs 1 hour later. This reduction in peak flow and increase in the time to peak is due to the increased attenuation capability of the catchment due to there being no development. The absence of developed impermeable areas increases the percentage of permeable land available for infiltration. However the decrease in peak flow is relatively small, in the order of 1 %. Therefore the urban development in the Clare River catchment is expected to exhibit an insignificant influence on either the Clare Rivers peak flow or the time to peak. There is also an increase in the duration of increased river flows.

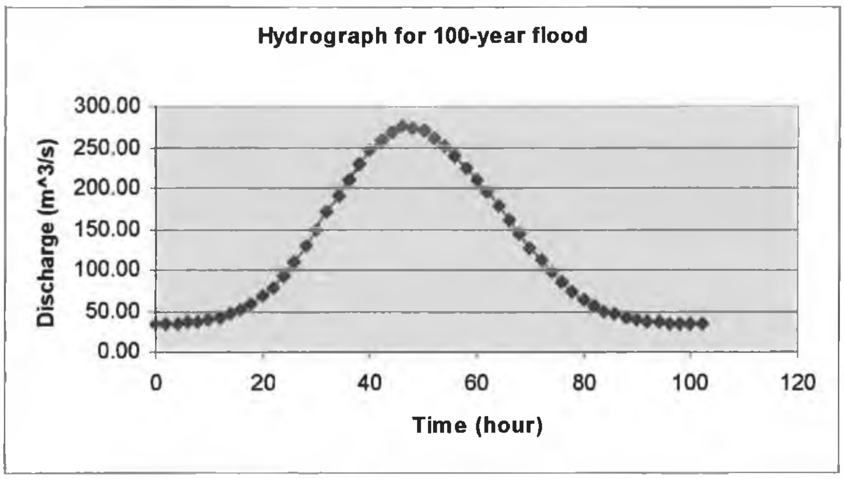

Figure 6.28 – Hydrograph for 100-year Flood for Scenario B

The relationship between increase in flood flow and increase in water level was graphed for Claregalway (figure 6.29). The relationship between the two was used to estimate the increase in water level that would be associated with a 2.42 m^3/s increase in flood flow at Claregalway. It was estimated that such an increase in flood flow would produce an increase in peak water level of approximately 35 mm at Claregalway. This is a reasonably small increase considering that the floodplain at Claregalway is not extensive. However it does highlight the effect that extensive development could potentially have on flood flows due to decreased attenuation and increased runoff rate. Therefore implementation of surface water management techniques such as SUDS will alleviate pressures resulting from surface water runoff from new development.

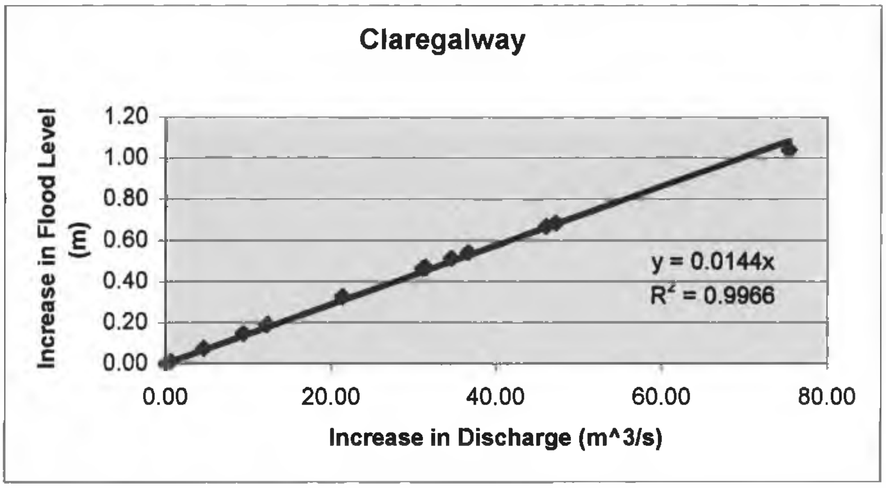

Figure 6.29 – Height Vs Discharge for Increase in Flood Flow

6.7 Summary

Land use changes have been identified as having the potential to make a significant contribution to flood risk. A change in agricultural land use can have an affect on the interception of surface water runoff. This effect is more significant in tropical climates. Development can have a significant effect on flood risk. The rainfall-runoff process, flow attenuation process, hydraulic control process and flood damage process are all affected by development. Development can significantly increase the percentage runoff due to reducing the permeability of the landscape. This can increase flooding downstream of the development. Development that affects the hydraulics of the channel or floodplain can lead to increased flood risk upstream and downstream of the development location. Downstream pressures are as a result of decreased attenuation capacity due to the reduction of available temporary storage volume in the channel/floodplain. Increased flood levels upstream are experienced as result of constriction of the channel/floodplain due to the development. These effects are usually experienced immediately upstream/downstream of the development location.

There is recognition of the importance of addressing flood risk in planning at every level of planning regulations. The Planning System and Flood Risk Management (PSFRM) Guidelines released in 2009 provides a sequential approach for addressing flood risk in relation to planning. The main aim is to avoid development in flood risk areas. The most significant increase in flood risk from development arises due to the increase in potential flood damage from developing in flood risk areas. The PSFRM guidelines aim to substitute less vulnerable development into such areas in instances where development is unavoidable. The Justification test is provided as a method of justifying such required development. A key element of developing in flood risk areas is that mitigation measures should be provided to reduce flood risk at the location without increasing flood risk elsewhere. However mitigation measures should not be used as a means of justifying development. Avoiding development in flood risk zones is a much better option than investing in costly mitigation measures that may have unforeseen consequences on peak flood levels elsewhere along the channel.

Flood risk assessments are a necessary element of evaluating the potential flood risks associated with planning proposals. The application of flood risk assessment varies depending upon the scale at which it is implemented and the required outcomes. The

Regional Flood Risk Appraisal provides a broad outlook on flooding throughout the catchment. The Claregalway LAP provides a greater level of detail in producing an indicative floodplain to inform land zoning. Tuam has not addressed flood risk sufficiently within its LAP. Tuam is a hub town identified for future growth. It is also susceptible to fluvial flooding near the confluence of the Nanny and the Clare River. The Regional Flood Risk Appraisal stated that Tuam should require mandatory flood risk assessments with planning proposals due to flood risk in the area. Land zoning in Tuam has been carried out without reference to indicative floodplain maps, which is identified as a key requirement within the Galway County Development Plan. It is suggested that flood extent maps from the November 2009 event would provide a good indication of the extent of a 100-year event and should therefore be used as indicative floodplain maps until such time as a more detailed hydraulic model of flood extent can be carried out. This would comply with regulations and guidelines that identify the 100-year event as the design flood that should be considered in urban areas. Such maps would also provide an indicative floodplain for other areas to identify areas that would require flood risk assessments in conjunction with planning proposals. The wider Claregalway area including Caherlea, Lisheenavala and Montiagh are an example of areas where flood risk is particularly prevalent that would benefit from an informed approach.

It is thought that land use changes involving a change in agricultural practice will have very little effect on runoff in the Clare River catchment. There is little potential for significant change in the catchment with the vast majority of land used as pastureland or peat bogs. However urban development could produce a significant change in runoff. Evaluation of the current level of urban development showed that it had little effect on peak flood flows than if there were no urban settlements within the catchment, with a difference between both scenarios of 2.42 m^3/s. Large-scale development in the catchment could potentially have a significant effect on flood levels. However it is unlikely that there will be such development especially since the recent decline in the construction industry. It is felt that the most significant impact on flood risk could arise due to development in flood risk zones leading to an increase in the potential flood damage that can be caused. Therefore the sequential approach of the PSFRM should be followed wherever possible. Making use of flood extent maps will also provide an effective method of delineating areas that should be exempt from development and identifying areas where flood risk assessments should be required with all planning proposals. The production of catchment scale flood risk assessments such as the CFRAMS will be of significant benefit in addressing flood risk and the results of such studies should be incorporated into regional and local planning guidelines.

Chapter 7

Channel Conditions

7.1 Effect of Channel Conditions on Flooding

Figure 7.1 shows the chain of sources, processes and effects of flooding that were discussed in section 6.1. The rainfall runoff process is not influenced by channel conditions. However subsequent processes can be significantly influenced by the condition in which the river channel is maintained.

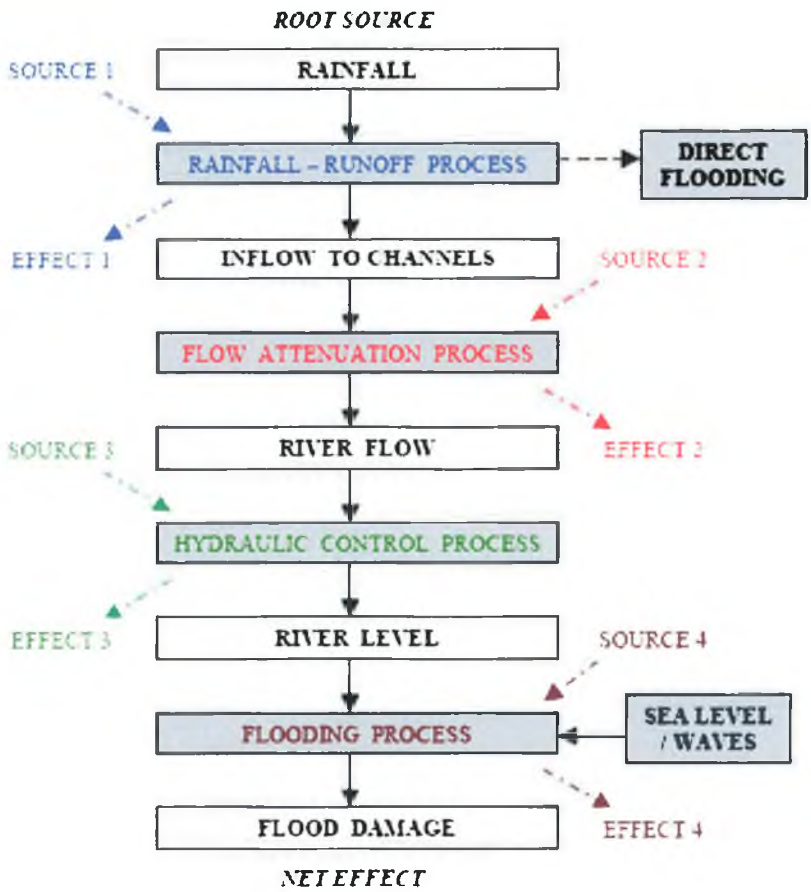

Figure 7.1 – Causes of Flooding: Chains, Sources and Effects [22]

Localised drainage can alleviate floodwaters locally by increasing channel capacity. However channel works can also reduce the attenuation capacity [22]. This produces increased flows and can lead to increased flood risk downstream. This increased flood risk is usually confined to immediately downstream of the works carried out unless the works are carried out on a large scale. Increased flood risk due to major channel works, such as channel excavation, is eliminated in cases where arterial drainage works are carried out along the entire length of a channel. Such extensive work would provide benefits along the entire length of the channel thus negating any increased flood risk downstream of localised works. Seasonal maintenance is sometimes provided to remove vegetation from a channel. This results in similar impacts as those experienced from channel excavation due to an increase in discharge and a corresponding reduction in attenuation capacity. These changes in the flow attenuation process can lead to increased flood risk if not properly considered when carrying out channel maintenance. Arterial drainage works are best implemented beginning at the river outlet and working upstream to ensure that increased flows do not increase flood risk downstream.

The hydraulic control process is also influenced by the condition of the channel. A channel or floodplain can be further restricted by blockages as a result of natural or human debris. Siltation can reduce the available cross-sectional area of a channel. Discharge is a function of both velocity and cross-sectional area. Therefore this would result in a reduction in discharge. Lack of channel maintenance can result in increased vegetation growth within the channel and floodplain. This also leads to a reduction in the channel carrying capacity. The reduction in discharge capacity can lead to flooding upstream of the blockage if flow is significantly constrained. The effects that alterations to the hydraulic control process have on flooding are explained more fully in section 6.1.

Manning's equation shows how the stage discharge relationship can be significantly affected by the condition of the river channel. The Manning equation can be derived from the Darcy-Weisbach equation for head losses due to wall friction, and is used to calculate open channel flow for a given set of hydraulic conditions. Manning's equation is given by [11]:

$$Q = A\,V = A\,R^{2/3}\,S_f^{1/2} / n$$

where: Q = discharge (m^3/s)

A = cross-sectional area of flow (m^2)

V = fluid (water) velocity (m/s)

R = hydraulic radius (m) = A / P

P = wetted perimeter of channel (m)

S_f = friction slope ($S_f = S_0$ for uniform flow where S_0 = channel slope)

n = Manning roughness coefficient

Manning roughness coefficient provides a method of factoring in channel conditions when determining the channel flows. Table 7.1 gives an example of some Manning roughness coefficients. The table shows that the same rule applies to flow in the floodplain. Therefore a floodplain constituting of dense brush will reduce discharge and provide greater attenuation than pastureland. The average value of n for streams is 0.035. Since the discharge (Q) is inversely proportional to Manning's roughness coefficient (n) an increase of 0.001 in n represents a decrease of approximately 3% in discharge.

Table 7.1 – Manning Roughness Coefficients for Various Open Channel Surfaces [11]

Material	Typical Manning roughness coefficient
Concrete	0.012
Gravel bottom with sides — concrete	0.020
— mortared stone	0.023
— riprap	0.033
Natural stream channels	
Clean, straight stream	0.030
Clean, winding stream	0.040
Winding with weeds and pools	0.050
With heavy brush and timber	0.100
Flood Plains	
Pasture	0.035
Field crops	0.040
Light brush and weeds	0.050
Dense brush	0.070
Dense trees	0.100

7.2 Condition of Clare River Channel

The OPW is the lead agency for flood risk management in Ireland. It was established in 1831. The main concern of the OPW is to manage and reduce flood risk. They are responsible for the collection of hydrometric data for a wide network of stations throughout Ireland. They also provide assistance to local authorities in response to flooding. The OPW provide information on historical flooding and flood risk to inform decision-making that may impact or be impacted on by flood risk. They are the principle authority for coordinating the assessment of flood risk in line with national and EU legislation and are responsible for carrying out any remedial works to the countries natural drainage network. Any works carried out on the Clare River System are the responsibility of the OPW.

The Clare River has undergone significant arterial drainage schemes resulting in a considerably altered drainage network. Figure 7.2 shows the drainage network that existed in east Galway in the 1800's, prior to arterial drainage schemes. The drainage pattern for the upper portion of the Clare River catchment was similar to the current drainage network. The mid to lower section of the catchment was considerably different from present day conditions. The Abbert River terminated in a turlough at Ballyglunin prior in the early nineteenth century. This water escaped as groundwater flows resurfacing elsewhere. Water sinking at Ballyglunin has been found to remerge at Auclogeen Spring near the source of the Cregg River, 10 km to the west [1]. This is potentially the path that the majority of water from the Abbert River took prior to being connected by a surface water channel to the Clare River. There also existed a permanent lake at Corofin, which experienced considerable groundwater losses due to the karst nature of the area [35]. This lake corresponds to the modern Cloonkeen turlough. This turlough forms during intense precipitation events such as those experienced in November 2009 and is shown in Appendix A–2.4. There was also a considerable turlough located between Corofin and Turloughmore at the end of the upper portion of the surface water system. This turlough was almost 9 km in length. There was no surface water channel flowing from this turlough. Water discharged through swallow holes and underground conduits re-emerging at springs elsewhere and flowing to Lough Corrib [35]. This historical drainage layout highlights the high level of karstification that exists within the catchment that accommodates groundwater flows.

Figure 7.2 – River Network in East Galway Prior to Arterial Drainage Schemes (19th Century) [36]

Initial arterial drainage works during the nineteenth century served to connect these isolated drainage networks with Lough Corrib via surface water channels as shown in figure 7.3. The construction of a surface water channel from Turloughmore to Claregalway connected the upper and lower portions of the Clare River. A channel from Ballyglunin was provided to connect the Abbert River to the Clare River. These works provided a complete surface water drainage system throughout the Clare River catchment. Prior to these works extensive flooding would have occurred at Ballyglunin, north of Corofin at Cloonkeen and from Corofin to Turloughmore. This arterial drainage scheme would have resulted in significantly diminished flooding at all of these locations [35]. However groundwater flows still play an important role in the catchment as explained in section 2.2.

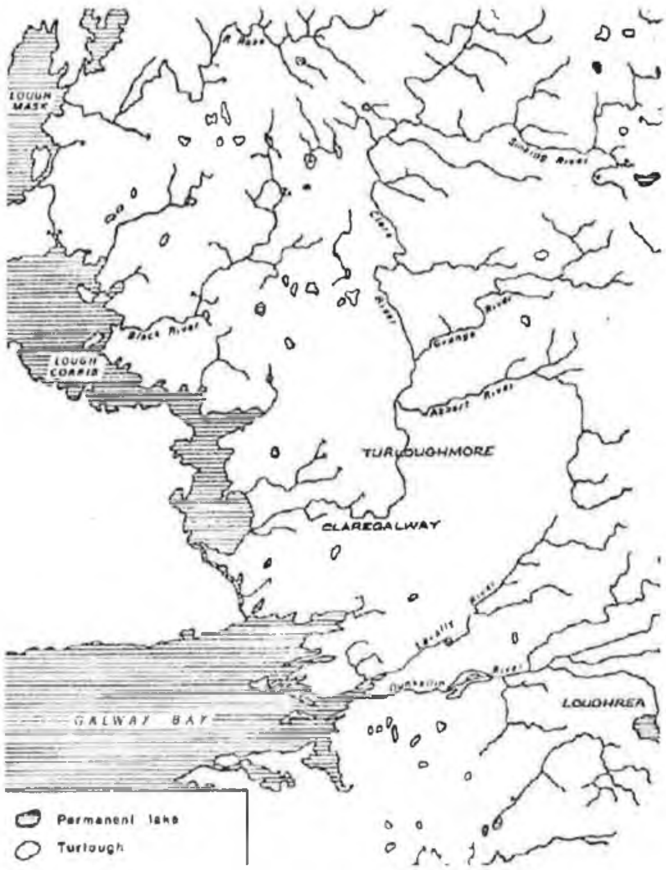

Figure 7.3 – River Network in East Galway Post Arterial Drainage Schemes [36]

The next significant arterial drainage scheme on the Clare River commenced around 1950. The works were carried out by the OPW as a result of the Arterial Drainage Act, 1945. The OPW are the responsible authority for carrying out arterial drainage schemes and flood relief schemes. The Arterial Drainage Act 1945 was introduced as a result of the findings of the Browne Commission (1938), which concluded that drainage practice was of a poor standard [37]. The 1945 act provides the principle legislation that enables the OPW to carry out catchment wide arterial drainage schemes to reduce flooding. The act was primarily focused on improving the drainage of agricultural land. A design flood with a return period of 3-years was used for designing channel improvements to address the flooding of such agricultural lands. The Arterial Drainage Amendment Act 1995 introduced the protection of urban areas as a key priority in flood risk management. This came as a result of increased flooding of urban settlements in the 1980's and 1990's.

The works carried out on the Clare River system in the 1950's were to improve agricultural land as set out in the 1945 act. An initial survey was carried out to establish existing channel conditions for the entire length of the Clare River and its tributaries.

The survey informed the design process to alter channel characteristics to enable the channel to convey water more effectively. The proposed alterations were designed to cater for a 3-year flood event. This is a relatively small flood event in comparison with the extreme events that are responsible for more serious flooding. Ireland experiences most of its flooding from October to March. The crop-growing season is located in the drier months between March and October. Therefore the 3-year design flood will actually reduce the likelihood of flooding by a much greater factor of approximately 15 years during the growing season [37]. The works involved excavation carried out over the entire channel length. This included the excavation of 1.35 x 10^6 m^3 of earth and 350,000 m^3 of rock. There was a significant amount of new cuts carried out on the river during these works to improve conveyance. This served to straighten the rivers meandering flow, which occurred due to the low lying, undulating topography of the catchment. The significant amount of work carried out on the Clare River system is evident along its course as shown in figure 7.4 and 7.5. According to the OPW these works have been successful with post drainage flood levels observed to be lower than those pre-drainage [35].

Figure 7.4 – Canalised Channel, Looking d/s from N17 Bridge South of Tuam

Figure 7.5 – Channel Cut into Rock, Looking d/s from Bridge at Lackagh

Subsequent to the major works carried out in the 1950's there has been continuous maintenance of the river system to maintain the condition of the channel as designed at the time. This maintenance originally took a less strategic approach with maintenance being carried as and when required with no particular structured pattern. This changed in around 1990 to a more structured 5-year cycle [38]. The maintenance is carried out by machine, or hand labour for smaller channels. It involves works such as removing any build up of sediment/debris, removing vegetation, etc. These cyclical maintenance works do not include small tributaries such as field drains. They also do not apply to sections of the channel with significantly large flows. The lower portion of the Clare River has only undergone significant maintenance once in the past 25 years. The lower section of the Clare River does experience slow flows resulting from shallow gradient and water level influence from Lough Corrib. Annual maintenance by boat is required in this section of the river to manage vegetation growth [38]. While the lower sections of the Clare River do appear to be in good condition there is an obvious lack of maintenance in the upper reaches resulting in considerable growth of vegetation at some locations, as shown in figure 7.6 and 7.7. This is not thought to significantly effect flooding in the catchment as the significant flooding locations are situated below these reaches. The presence of this vegetation would, if anything, reduce flooding in the lower areas of the catchment due to providing increased attenuation as described in section 7.1.

Figure 7.6 – Sinking River above Confluence with Dalgan

Figure 7.7 – Dalgan River above Confluence with Sinking

Table 7.2 – Comparison of Arterial Drainage Design Cross-Sections to Current Cross-Sections [10]

	Cross Section		Lowest Bed Level (m OD Malin)		Δ Elevation (m)*
	OPW Section No.	RH Section No.	Original Design (1950s)	Current (2010)	
c23 Curragmore Br	19/0	c15	3.87	2.94	**-0.93**
c73 Claregalway Br	55/0	c42	4.14	2.45	**-1.69**
	96/0	c74	5.89	5.18	**-0.71**
	123/0	c86	6.8	5.91	**-0.89**
c103 Cregmore Br	143/0	c95	6.48	8.00	**1.52**
c123 Lackaghmore Br	174/0	c109	14.23	13.65	**-0.58**
	210/0	c127	17.21	16.09	**-1.12**
	245/0	c145	18.92	18.00	**-0.92**
c158 Clare – Abbert Confluence	269/0	c156	20.59	18.88	**-1.71**
	298/0	c170	23.5	21.77	**-1.73**

Note: (-) indicates a drop in bed levels since design stage; (+) represents a rise.

A bathymetric survey was carried out in conjunction with the Clare River Flood Study [10]. These results were compared to design cross-sections for the arterial drainage scheme carried out in the 1950's. The results of the comparison are shown in table 7.2. The results show that the only area that experienced siltation in the lower reaches of the Clare River was at cross section reference C95. This is located halfway between Cregmore Bridge and Crusheen Bridge (figure 7.8). The River changes direction at this point from the southerly direction that it has maintained over the majority of its length to a westerly direction to flow to Lough Corrib. The bed level data from the original design in the 1950's shows a significant drop in bed level from the bed level upstream of C95 down to C95. The channel gradient levels out again at and below C95. It is therefore probable that sediment picked up upstream of this point, due to the steeper slope, falls out of suspension at this location due to a decrease in the velocity of flow. This section should be returned to its original design depth and monitored to ensure siltation does not increase the bed level datum in the future.

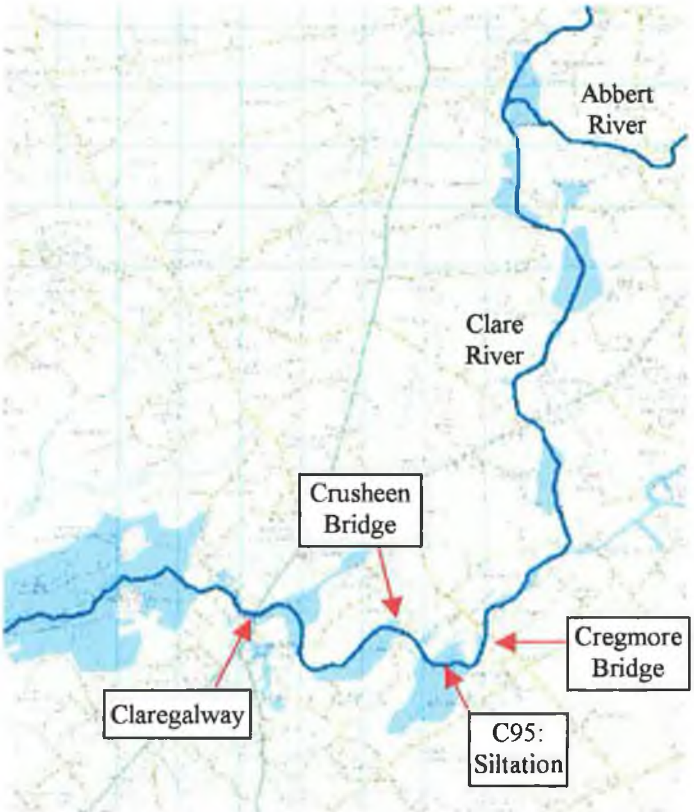

Figure 7.8 – November 2009 Flood Extent Map Including Location of Siltation at C95

The other comparisons in table 7.2 show that the bed level is lower by as much as 1.73 m from design levels. There may be a number of reasons for this. The original excavation works in the 1950's would have been carried out by machine. It is probable that over excavation occurred to ensure that the design dimensions were provided. Also machine maintenance occurred along the lower reaches of the Clare River once since the arterial drainage scheme was completed. This could have also resulted in over excavation of the channel. The river flows may also have produced scouring of the riverbed. Scouring is the erosion of waterway soils and sediments [39]. The increased discharge capacity is not expected to provide an increased flood risk downstream of Crusheen Bridge, as the channel is deepened throughout the lower reaches. The sediment can fall out of suspension downstream at changes in the river morphology (e.g. change in direction, gradient becomes less steep) such as that observed downstream of Cregmore Bridge. The impact of the raised bed level at this location due to siltation accompanied by the fact that Crusheen Bridge could potentially act as a hydraulic constraint in times of high flow may have potentially exacerbated flood levels in this region. Figure 7.9 and 7.10 show aerial photos of this section of river during the November 2009 events. The area south of this point (Caherlea, Lisheenavala and Islandmore) experienced extensive flooding during November 2009 as shown in figure 7.8, 7.9 and 7.10.

Figure 7.9 – November 2009 Flood Event Upstream of Crusheen Bridge, Looking South

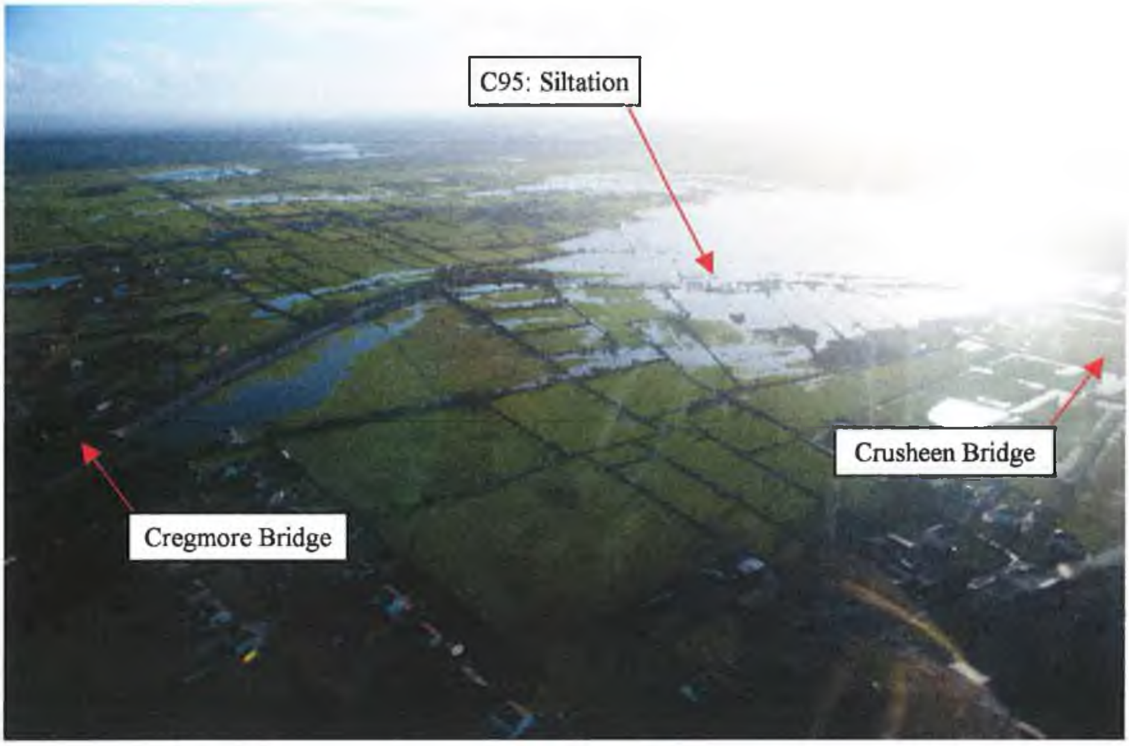

Figure 7.10 – November 2009 Flood Event Upstream of Crusheen Bridge, Looking South

7.3 Lough Corrib Water Level

The Corrib River conveys water from Lough Corrib to the sea at Galway Bay. The discharge of this water is controlled by sluice gates at the Salmon Weir Bridge. The construction of this sluice barrage was completed in 1959 and allows for water discharge to be controlled by the manipulation of 14 steel gates and 2 wooden gates. The manipulation of these gates influences water levels in Lough Corrib which in turn influences water levels not just along the lake margins but also in the lower reaches of inflowing rivers such as the Clare River due to water backing up along the channel. The backwater effect occurs due to water level or flow rate being changed at a particular point in a channel carrying subcritical flow [11]. The effects of these changes propagate back upstream. The main purpose of constructing the sluice barrage was to maintain a sufficiently high minimum lake water level to service uses such as boating, fishing and water abstraction while minimising peak lake levels. These gates are opened in times of high inflow to alleviate high lake levels. During the design of the sluice barrage the desired minimum lake level was set at 5.83 mAod. A high lake level of 6.44 mAod for discharging the high design flow of 311.5 m^3/s was set as the maximum water level target. Studies carried out by the OPW in the late 1970's suggested that achieving a peak lake level of 6.44 mAod for discharging this high flow was over optimistic target of what could be economically achieved [35].

The OPW carried out an assessment of the impact the sluice gates had on Lough Corrib water levels in 1987 [40]. The main purpose of the study was to evaluate whether the operation of the sluice gates maintained lake levels within the set parameters (5.83 mAod to 6.44 mAod) and what effect changing the gate manipulation strategy would exhibit on water levels in Lough Corrib. The report found that since the initial installation of the sluice barrage in 1959 up until the time at which this report commenced that the water levels in Lough Corrib were maintained at or above the minimum design target for all years. The maximum design target of 6.44 mAod was exceeded on all but 4 years for the period 1960 to 1986. The 1987 report generated a series of rating curves for water flows at the sluice barrage. A rating curve is a graph that shows the relationship between water level and discharge at a certain cross-section in a river [33]. These rating curves are shown in figure 7.11 with water level above Poolbeg and Malin on the y-axis. They were produced for all sluice gate opening combinations. Figure 7.11 shows that water level at the sluice barrage is 6.02 mAod when discharging 311.5 m^3/s with all sluice gates open.

This equated to a water level of 6.88 mAod in lower Lough Corrib [40]. This is 0.44 m above the maximum lake level target as set out in the design process. The report concluded that the reason for lower Lough Corrib water levels being in excess of the design level was due to design calculations underestimating actual energy losses due to friction along the River Corrib. Therefore the report concluded that the River Corrib channel from Lough Corrib to the sluice barrage is the controlling factor in not maintaining lake levels below 6.44 mAod for the design flow. In order to discharge the design flow at 6.44 mAod excavation of approximately 700,000 m^3 of material would be required along the River Corrib channel length [35]. Further analysis of data from 1960 to 1986 resulted in the report concluding that the design flow of 311.5 m^3/s was in fact a 20-year event. The Arterial Drainage Scheme under which the sluice barrage was constructed had proposed this magnitude as a 3-year event. A 3-year event at Galway Sluice Barrage would result in a flow of 265 m^3/s. According to the rating curves this occurs at 5.91 mAod if all gates are open and 6.02 mAod if just the 14 steel gates were open. The corresponding water level in lower Lough Corrib for this discharge is 6.63 mAod (still 0.19 m above the maximum design target set for the discharge of 311.5m^3/s) [40]. Excavation of 300,000 m^3 of material along the 8km length of the River Corrib channel from the lake to the sluice barrage would be required to achieve this discharge at a lake level of 6.44 mAod [35].

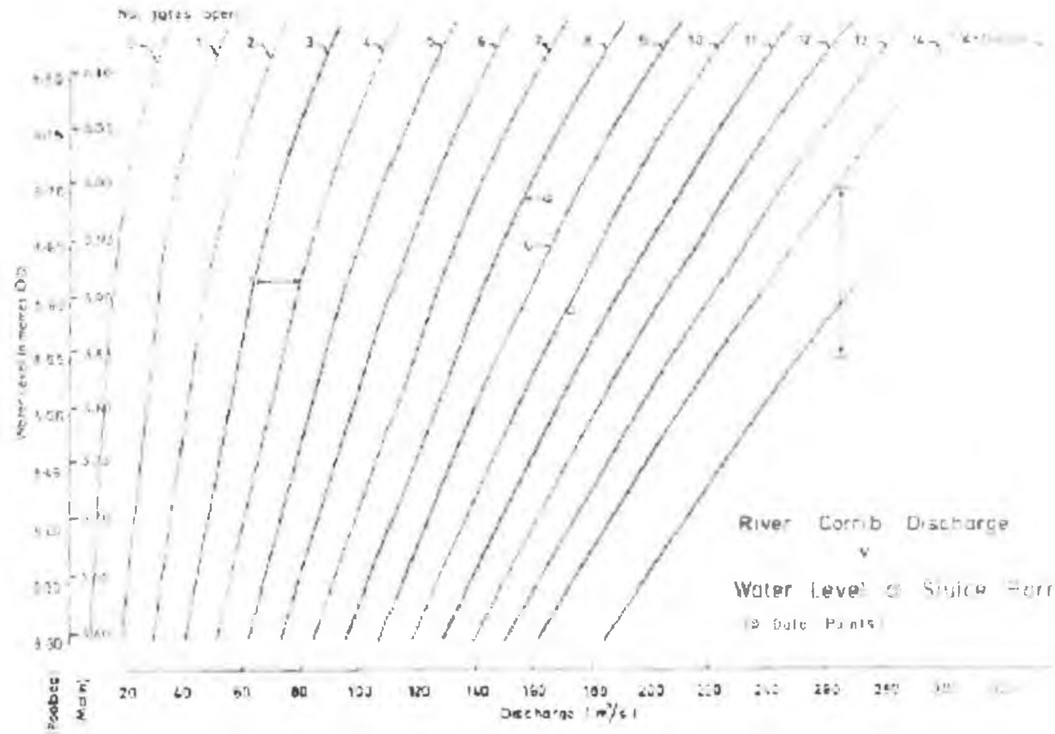

Figure 7.11 – Rating Curves for Different Gate Opening Combinations at Galway Sluice Barrage [10]

The report analysed 4 gate manipulation policies. A computer-based model was used along with recorded hydrometric data and rating curves to generate water levels for the period 1981 to 1986. The results of the different gate manipulation policies were then compared to the actual recorded lake levels for the period. The 4 gate manipulation policies analysed were [35]:

1. All gates open all year – This analysed the impact of the gates remaining open all year round on low and high water levels in Lough Corrib.
2. Narrow band gate control – This scenario opens all gates when water levels reach an upper limit and closes all gates when water levels reach a lower limit. The limits chosen for the 1987 study were 5.88 mAod and 5.98 mAod. [10]
3. Lower limit gate control – The gates are opened to give the maximum possible discharge while maintaining lake levels at or above 5.81 mAod [10].
4. All gates closed all year round – The gates remain closed for the entire year in this scenario. The implications of this were found to increase high lake levels by about 1 m.

Scenario 1 (all gates open all year) would obviously produce the lowest possible lake levels. Therefore if this scenario cannot lower winter floods there is no possibility of any gate manipulation strategy diminishing flooding during extreme events. The 1987 report concluded that while opening all 16 gates for the entire year would reduce lake levels during milder winters it would not have any effect on peak lake levels during particularly wet winters in which flooding occurs. Opening all gates would have lowered winter lake levels by 0.4 m during the relatively mild winter and resultant low lake levels of 1981. However it would not have had any effect on the higher lake levels that resulted from the wet winter of 1986 [10]. The performance of each of the gate manipulation policies and actual lake level for 1986 is shown in figure 7.12. The graph shows a significant decrease in lake level during spring, summer and early autumn when lake levels were recorded at approximately 6 mAod. However there was no reduction in the peak lake water level of approximately 7 mAod, which occurred in December. This peak lake water level of about 7 mAod is similar to the 6.928 mAod lake water level that was recorded during the November 2009 events as shown in figure 7.12. (Figure 7.12 shows lake water level above Poolbeg and Malin on the y-axis as in figure 7.11). This suggests that opening all gates all year round would have had no significant effect on lake levels during the 2009 floods.

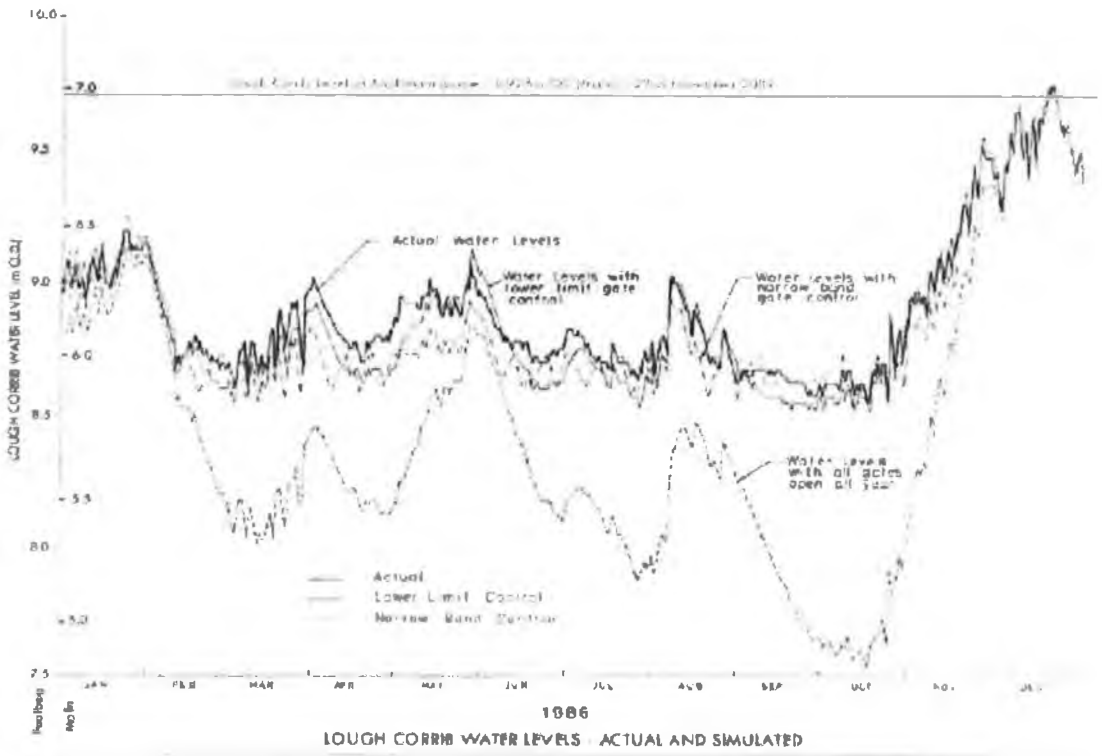

Figure 7.12 – Actual Lough Corrib Water Levels for 1986 and Modelled Lough Corrib Water Levels for Different Gate Manipulation Strategies for 1986 [10]

It can be concluded that the gate manipulation policy implemented by the OPW at Galway sluice barrage was as effective at managing high water levels for the period 1981 to 1986 as leaving all the gates open all year round. The gate manipulation policy operated by the OPW has remained unchanged. Therefore there is no benefit from changing the existing gate manipulation policy in alleviating high lake levels. The 1987 report concluded that the main reason for lake levels not being maintained below 6.44 mAod was due to the constraints of the River Corrib channel.

GSI Report

The impact of lake levels on flooding along the Clare River was also reviewed in a report carried out by the Geological Survey of Ireland (GSI) [35]. The report analysed flooding in the Claregalway area in relation to the flooding experience in 1990 and 1991. The report considered 3 possible causes of the flooding:

1. Exceptionally heavy precipitation
2. High water levels in Lough Corrib during the heavy precipitation
3. Channel restrictions along the River Clare.

The rainfall was determined to be the primary cause of flooding with 1.6 times the normal rainfall in January 1990 and 3.2 times the normal in February 1990 at UCG. It was recorded as 3.6 times the normal at Craughwell with a return period of 70-years. Rainfall was twice the normal for the period from 18th December 1990 to 11th January 1991. The GSI report concluded that rainfall was the major reason for the flooding of March 1990 and January 1991.

The impact of lake levels was also considered. Heavy precipitation causes water levels in Lough Corrib to rise resulting in flooding around Lough Corrib and also causing water to back up along the River Clare. The GSI report identified three factors that could potentially contribute to the high water levels in Lough Corrib:

- The River Corrib channel upstream of the sluice gates
- The sluice gates at the salmon weir bridge
- Wind set up

It's review of the OPW report produced in 1987 concluded that the carrying capacity of the Corrib channel was the controlling factor that produced lake levels in excess of 6.44 mAod, which consequently produces flooding around Lough Corrib and water backing up along the lower reaches of the Clare River. The report looked at the gate manipulation policy of leaving all gates open all year round. It concluded that the gate manipulation policy had no affect on the flooding in 1990 and 1991. The report suggested that wind set-up due to the high winds recorded in February 1990 and January 1991 could have exacerbated lake levels at the mouth of the Clare River during these two flood events. Wind blowing across a lake can increase water levels on the leeward shore and reduce them on he windward shore. The rise above the still-water level is known as the wind set-up. Differences in water levels on Lough Corrib have been recorded by the OPW in the region of 0.4 m and are thought to be due to wind set-up [35]. Winds over Lough Corrib are predominantly westerly and southwesterly. These would lead to increased water levels on the eastern shore of Lough Corrib where the Clare River discharges to the lake. The wind could also reduce the water level at the lake outfall thus reducing the discharge via the River Corrib channel. This would maintain high lake levels for longer periods. Lake levels at Annaghdown on the eastern shore of Lough Corrib were in the region of 50 mm lower than lake water level recorded at Barrusheen near Oughterard on the western shore during November 2009. This is what would be expected as Barrusheen is also situated slightly farther away from the outlet of the lake and should therefore exhibit slightly higher water levels. Therefore it is not expected that wind set-up played a part in dictating Lough Corrib water levels during the November 2009.

The GSI report suggested that due to the shallow gradient from Claregalway to Lough Corrib that improvement of the Clare River channel in this region would have had a minimal effect on flooding in 1990 and 1991 as lake levels would have influenced the water levels as far as Claregalway. It recommends that the most effective way of alleviating flooding from Claregalway to Lough Corrib would be to improve the River Corrib channel thus lowering lake levels. It suggests that improvements in the Clare River upstream of Claregalway could alleviate the flooding from Corofin to Turloughmore.

The principal recommendations proposed by the report for further evaluation were:

- Improvement (excavation) along River Corrib channel to reduce peak lake level for discharge for a 3-year event.
- Practicality and benefits of a flood forecasting system. The report recommends that a flood forecasting system to inform sluice gate operation and warning of major floods for farmers and others should be looked at. It suggests that better sluice gate operation may reduce flooding in the early part of the wet season.
- Improvement of Clare River between Claregalway and Turloughmore.

Impact of Lake Level on November 2009 Flood Event

The impact of Lough Corrib water level along the lower reaches of the Clare River was evaluated in the Clare River Flood Study [10]. The hydraulic simulation was carried out using the computer based model HEC-RAS. Extensive information was gathered relating to hydrometric data and physical characteristics of the channel. The model simulated the November 2009 flood event under 2 scenarios. The first scenario replicated existing conditions at the time of the events with the recorded lake level of 7.1 mAod. The second scenario set the lake level at 6 mAod. The water levels observed along the lower reaches of the River Clare are shown in table 7.3. The results show that decreasing the lake level has no significant effect on flooding at and above Claregalway with peak water level dropping by only 30mm at Claregalway Bridge when lake level is dropped by 1.1 m. Floodwater level decreases more significantly in regions closer to Lough Corrib. Montiagh flood levels decrease by between 100 mm and 210 mm. This would be a significant drop in peak flood level at a location, which experienced flooding and was isolated during the 2009 flood event. There is an even more significant 460 mm drop in flood levels at Curraghmore Bridge, at the N84. This area has a low flood risk due to its comparatively low population and building density. Areas upstream of Claregalway would experience negligible difference in peak flood levels. Crusheen Bridge (near Caherlea, Lisheenavala and Islandmore) experienced no decrease in high water level from dropping the lake level by 1.1 m. These results correspond with the views of the GSI report that lower lake levels would only have a significant effect on flood levels between Claregalway and Lough Corrib.

Table 7.3 – HEC-RAS Model Results for Different Water Levels at Lough Corrib for November 2009 Flood Event [10]

Location	Distance from outlet (m)	November 2009 Flood Actual lake level (mAod)	November 2009 Flood Lake level set at 6 mAod (mAod)	Difference (m)
Lough Corrib	0	7.10	6.00	1.10
Curraghmore Bridge (N84)	2628	7.64	7.18	0.46
Montiagh South	6276	8.27	8.06	0.21
Montiagh (North)	7628	8.70	8.60	0.10
Claregalway Bridge (d/s face)	8506	9.49	9.46	0.03
Claregalway Bridge (u/s face)	8557	10.37	10.34	0.03
Kinishka	8936	10.58	10.56	0.02
Lakeview, Cuirt na hAbhainn	9785	11.02	11.01	0.01
Gortaleva	10785	11.53	11.52	0.01
Crusheen Bridge (d/s face)	12153	11.79	11.79	0.00
Crusheen Bridge (u/s face)	12163	12.41	12.41	0.00
Islandmore	12856	12.92	12.92	0.00

7.4 Summary

Maintenance of a channel can have an effect on both the flow attenuation process and the hydraulic control process in determining the water level in the channel. Influences such as vegetation growth in the channel/floodplain can reduce the discharge capacity of a river. Carrying out localised drainage works to increase the channel carrying capacity can result in increased flood levels downstream due to an increase in discharge and corresponding decrease in attenuation capacity at the location of such works. Lack of channel maintenance can result in blockages and siltation. This reduces the cross-sectional area available for flow. Therefore the reduced cross-sectional area can act as a hydraulic constraint producing increased flood levels upstream. Increased water levels due to changes in the flow attenuation process or hydraulic control process are usually confined to immediately downstream or upstream of the location where the maintenance works are carried out unless the works are carried out on a large scale. These increases in flood risk can be avoided if the works are carried out on such a large scale so as to include the entire river channel such as in arterial drainage schemes carried out by the OPW.

The Clare River system has undergone major arterial drainage schemes resulting in a significant change in the drainage network. Prior to these schemes the upper and middle sections of the river network were only linked to Lough Corrib by underground flows. The works carried out in the 19th century linked these sections of the drainage network with the lower section of the Clare River thus providing a complete surface water network to drain surface water from the catchment. These works reduced flooding in the regions around Turloughmore and Corofin where a significant amount of land was underwater due to turloughs and a permanent lake. These works were also beneficial as underground flows can be very unpredictable with collapse of underground conduits resulting in a significant increase in flood risk for areas reliant on them carrying away floodwater. Further works carried out in the 1950's were aimed at benefiting agricultural land through increasing the Clare Rivers capacity to cope with a 3-year flood event. Arterial drainage schemes have reduced flood levels along the Clare River. A bathymetric survey carried out in the lower section of the Clare River identified only one point where siltation had resulted in raised riverbed levels. This raised riverbed level could have potentially exacerbated flooding in the region of Caherlea, south of its location and should be returned to its original design conditions. The remainder of the lower reaches of the Clare River were surveyed as being significantly lower than the 1950's OPW design levels demonstrating that siltation has had little impact on the lower reaches of the Clare River. It is not expected that these reduced bed levels contributed to flooding downstream as they were surveyed to be lower throughout the lower reach of the Clare River except upstream from Crusheen Bridge where siltation may have exacerbated flooding. Considerable vegetation growth is evident in the upper reaches of the Clare River system. However it is felt that this has little significant impact on flooding, as it is located above the areas that suffer from the most significant flood risk. Therefore they may help to reduce flood risk by providing increased attenuation capacity in the upper reaches.

The water levels on Lough Corrib influence water levels in the lower reaches of the Clare River. The operation of the sluice gates at the Salmon Weir Bridge was found to have no effect on peak water levels in lower Lough Corrib during significant flood events. The carrying capacity of the River Corrib channel was found to be the main cause of peak lake levels not being maintained below the upper limit of 6.44 mAod for the design flood. The lower reaches of the Clare River up to and including Montiagh could potentially benefit from reducing the lake levels during flood events. Claregalway and regions above this would have experienced little reduction in peak flood levels for the November 2009 event for a significant (1.1 m) drop in lake level. The cost of excavating large volumes of material from the River Corrib channel to reduce flood levels below Claregalway may not be viable. This region is not as densely populated as areas such as Claregalway, which may benefit more significantly from a similar allocation of funds. The section of river upstream of Crusheen Bridge should be returned to its original design conditions as it may exacerbate flooding in a region that experienced considerable flood damage in November 2009.

Chapter 8

Flood Risk Management

8.1 Flood Risk Management Measures

Inappropriate development has been identified as a significant contributor to flood risk. Ireland experiences a relatively low level of flood risk due to its low population density, especially in western Ireland. However the increase in the density of urban settlements has produced an associated increase in flood vulnerability in instances where they are located in flood risk areas. National legislation has recognised this with more informed zoning and planning processes that consider the associated flood risk.

Flood risk management is a method that applies to a broad spectrum of factors that influence flood risk. It involves identifying, managing and reducing existing and future flood risk. The mitigation measures can include strategies, plans, hard-engineered defences and early warning systems. A comprehensive flood risk management strategy requires that all aspects of flood risk be considered including implications on development plans, society and the environment. Historically flood risk management has been reactive. Flood mitigation measures have generally been implemented following the occurrence of a flood event. Considering future flood risk allows for a more comprehensive and cost effective implementation of flood relief measures in the long-term. This proactive approach will ensure that increased surface water discharge associated with the predicted impacts of climate change will not result in an increase in flood risk. It will also ensure that decision-making in flood risk areas is well informed and that increasing flood risk due to poorly informed decision-making is avoided. It has already been identified that decisions made at one point in a river network can exhibit unfavourable implications on flood risk elsewhere. Comprehensive flood risk management techniques require that all these possible implications be considered. Therefore it is essential that flood risk management be carried out on a catchment scale that incorporates all flood risks associated with a drainage network. A comprehensive evaluation of flood risk could only be examined on a spatial scale that considers the entire catchment. However there is significant cost associated with such studies and therefore such an approach may not always be feasible.

Information is a key requirement for effective flood risk management. Cooperation with the public can provide an invaluable source of information to assist the plan making process and can also increase the rate of uptake of flood risk management schemes. Flood extent maps that show the spatial distribution of flooding associated with a flood event of a given magnitude greatly increase the effectiveness of flood alleviation measures. It has been recognised as a key element of understanding flood risk by many organisations such as the UN who stated 'Identification and mapping of flood hazards and high-risk areas should be integrated into land-use planning policies' [41]. Flood zoning can take the form of historical flood extent maps such as those provided in Appendix A–2 for November 2009. Geographical Information Systems (GIS) can be very beneficial for utilising flood extent information in planning processes. Their ability to solve complex spatial problems and to integrate multiple objectives is extremely beneficial in visualising the spatial variability of flooding. It enables data from various sources to be collated for visual analysis to assist in decision-making that requires the consideration of spatial information. Advanced computer based hydraulic models have enabled flood risk zones to be predicted accurately by inputting boundary conditions. This approach should be used to its full effect to ensure flood risk management measures are implemented in consideration of future events. The 2009 flood was estimated as most probably a 100-year event after taking into consideration the more comprehensive hydrometric data at Ballygaddy and Corofin. Therefore the flood extent maps provided in Appendix A–2 would provide a reasonably accurate match for a 100-year flood extent map produced by a hydraulic model. The benefit of a hydraulic model is that it allows for the input of potential changes (e.g. development, climate change) that could potentially affect flood zones. The implication of such changes can then be addressed in a proactive manner without having to wait for flood events to occur for flood relief measures to be implemented.

Catchment flood risk management plans should deal with all aspects of flood risk. The spatial scale allows for the feasibility of varying flood risk management measures to be assessed in a comprehensive manner. Catchment flood risk management plans should include evaluation of hard-engineered flood defences, flood warning options, emergency response, future flood risk predictions due to changes in flood processes (i.e. floodplain constriction), flood risk zones (where development should be distributed according to the sequential approach as outlined in the Planning System and Flood Risk Management Guidelines), potential environmental impacts and best practice in managing surface water runoff (e.g. SUDS, agricultural practice). Figure 8.1 shows the information requirements and the role that catchment flood risk management plans can play in various aspects of flood risk management. The layout of the chart is based on the river basin flood risk management plan published in the report of the Flood Policy Review Group [17].

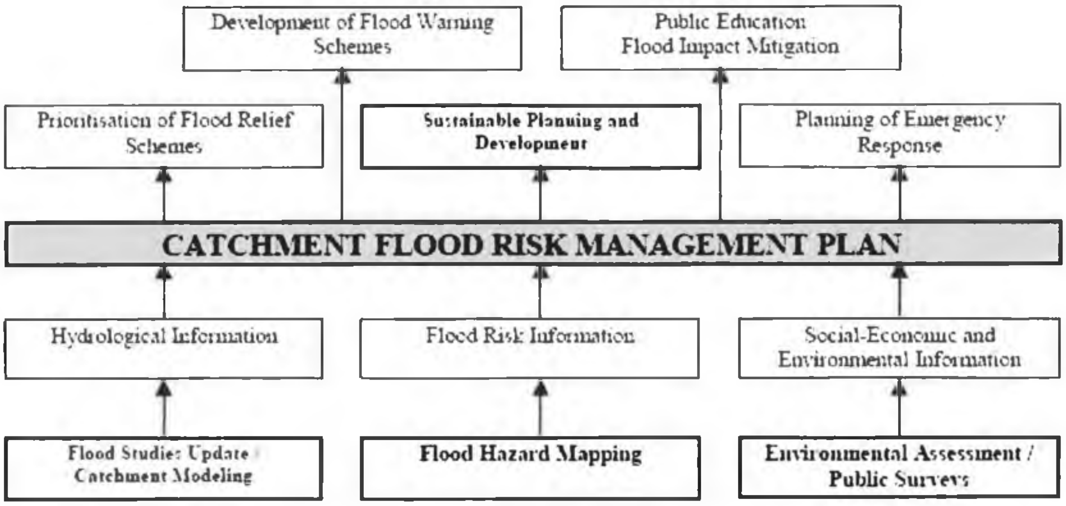

Figure 8.1 – Role of Catchment Flood Risk Management Plans [22]

Flood relief includes any mitigation measures that are taken to reduce flood risk. It may include hard-engineered flood defences, flood warning systems, arterial drainage schemes etc. A government statement outlined that 'the OPW should be the lead agency in devising and implementing measures to deal with flooding' [23]. The Arterial Drainage Act of 1945 enables the OPW to undertake flood relief works on a catchment scale. It is under this scheme that the Clare River Arterial Drainage Scheme was undertaken as discussed in section 7.2. The Amendment Act 1995 was introduced to address more localised flooding in relation to particularly vulnerable areas (e.g. urban settlements). Figure 8.2 outlines the method of implementation of a flood relief scheme under the structure used by the OPW. The process is discussed further below.

The preliminary assessment is a qualitative assessment of the requirement and viability of implementing a flood relief scheme. Should the location be found to be sufficiently vulnerable to flooding the process moves to the pre-feasibility stage. This stage allows for a more detailed study of readily available information without the commitment of considerable funds. This prevents wasting of funding carrying out a detailed feasibility study for a project that is determined to be unfeasible by this secondary stage due to financial, technical, environmental or societal reasons. The detail of this stage is limited by cost as it is only intended to justify the implementation of the more detailed and expensive feasibility study and outline design stage. Outcomes of the pre-feasibility study include an analysis of historical data, indicative flood risk from historical and design flow information, an approximate estimation of potential flood damage and benefit of scheme, potential obstructions in implementing the scheme and preliminary evaluation of outline proposals for flood relief measures. It also provides helpful information for the following stage.

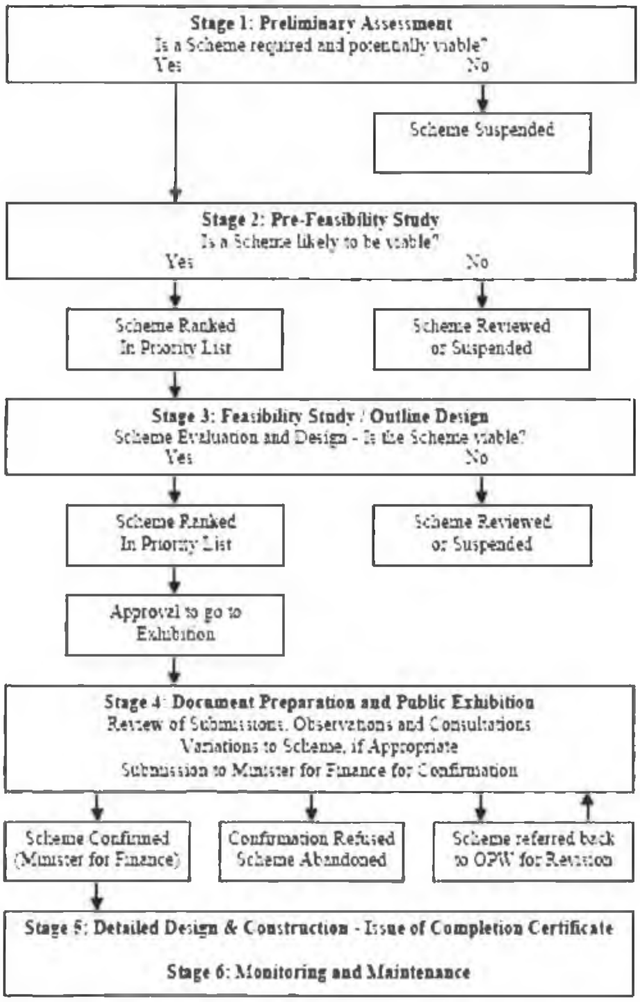

Figure 8.2 – OPW Flood Relief Scheme Implementation Process [22]

The feasibility and outline stage involves a detailed assessment of potential flood relief measures from a technical, economical, environmental and societal perspective. It requires extensive surveys and analysis of data. It is based on the same principles as the pre-feasibility stage 2 but is a far more in-depth evaluation. The gathering of data is a crucial part in planning an effective flood relief scheme. Historical data provides a very good indication of the spatial extent of flooding, flood levels and potential flood damage arising from flood events. Historical flood data, hydrometric data and meteorological data help to increase the accuracy of the outcomes of the study. Accurate site information is also a key requirement for the prediction of flood risk. The topography of the floodplain, soil characteristics, hydraulic constraints (e.g. bridges) and the channel size, shape and condition are all required elements in producing an accurate hydraulic model to predict flood flows. Subsequent to the collection of all relevant data a comprehensive analysis of flood risk can be carried out. Statistical analysis (as demonstrated in chapter 4) is required to provide input data for hydraulic models along with the site-specific information gathered from area surveys (e.g. bathymetric, geotechnical, topographical). Calibration of computer based hydraulic models against historical behaviour of the watercourse and floodplain is necessary to ensure the accuracy of future predictions of flood extent and flood risk. The model can then generate hydraulic models of the area concerned for flood events of varying return periods and in consideration of different pressures/boundary conditions. This enables potentially harmful decisions that could increase flood risk to be avoided and also enables unavoidable changes (e.g. climate change and unavoidable development) to be factored into flood alleviation measures.

The estimation of potential damages arising from flood events is required to evaluate the benefit of providing a flood relief scheme. This is the value of expected Average Annual Damage assuming that no flood relief works are carried out and equates to the sum of the products of event damages and annual probabilities of occurrence. The method relates the predicted flood levels to property levels and applies a suitable methodology of estimating potential damage such as that outlined in the FLAIR report (1990). Average Annual Damage includes all aspects of damage such as direct economic damage (property), indirect economic damage (disruption of travel and information transfer network) and intangible damage (no investment in business growth in the area due to perceived flood risk) [22]. The benefit of the scheme is calculated as the Net Present Value of the reduction in flood damage that would be achieved were the flood relief scheme to be implemented.

Constraints to the provision of flood relief measures vary greatly depending upon the location and characteristics of the catchment. Environmental impacts of flood relief schemes require adequate consideration. The public and business sector may be significantly impacted by proposed flood alleviation measures. For instance considerable changes in the hydraulic characteristics of a river may result in significant changes to the habitat and species. The construction of the sluice barrage at the Salmon Weir Bridge would have prevented the migratory salmon from returning to breed. This would be of particular concern to anglers and authorities such as the Western Regional Fisheries Board (WRFB). Provision of a fish pass at the sluice barrage offered a suitable solution to the problem. Environmental Impact Assessments (EIA) are carried out on proposed works to identify and address such implications in advance of implementation of works. Aesthetic and archaeological constraints also exist depending upon the location. Technical constraints may also produce limitations. These may be due to reasons such as spatial constraints or ground bearing capacity. Provision of flood relief measures should not only aim to address all these constraints but should also aim to improve upon existing conditions through enhancement of the area.

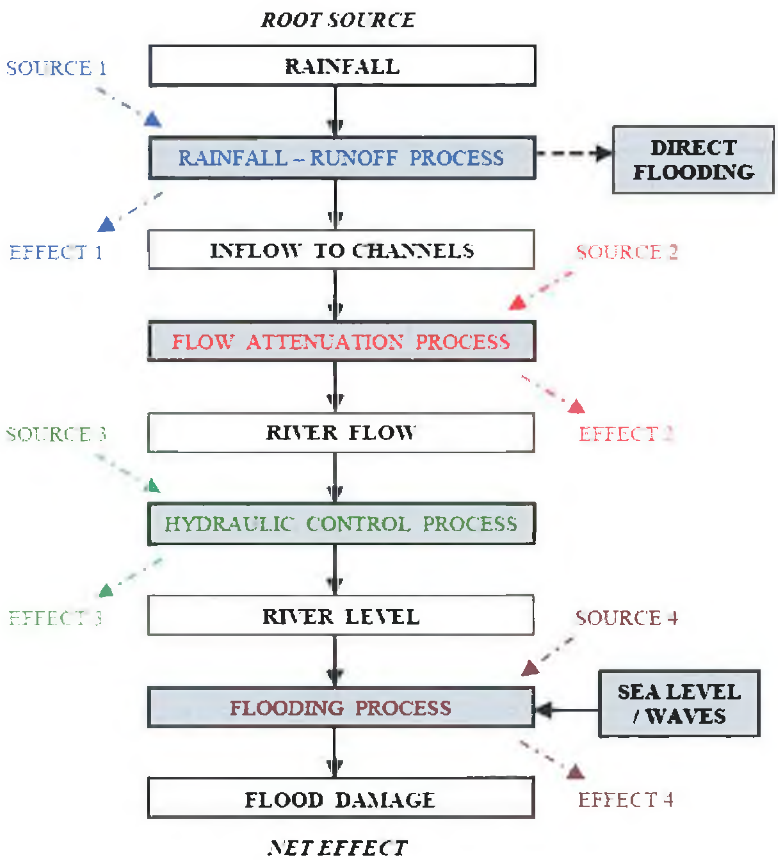

Figure 8.3 – Causes of Flooding: Chain of Sources, Processes and Effects [22]

Following identification of the flood risk, flooding mechanisms and constraints the flood relief measures can be considered and evaluated. There are a number of potential measures that can be employed in reducing flood risk. The method by which they reduce flood risk is by intervening in one of the processes shown in figure 8.3, which have been discussed previously in chapter 6 and 7. The 'Do Nothing' approach involves no action being taken and is the benchmark against which all other options may be evaluated to determine their benefit as flood relief measures.

Flood Containment provides physical defences to prevent floodwater from entering areas that could result in flood damage. It has an effect on the flooding process (figure 8.3). Flood containment requires a considerable investment of resources and should be used only after careful consideration due to the significant effect it can have on flood levels elsewhere. This is due to its influence on the flow attenuation process and hydraulic control process by confining flow to a conveyance route of reduced cross sectional area. However sometimes it may provide the only option due to site constraints e.g. spatial limitations in urban area. Flood containment measures can involve construction of permanent hard-engineered flood defences. There are also semi-permanent options that provide protection as required and enable the site to be returned to normal conditions in time of steady flow. Demountable rigid defences are one such option. They are raised in response to rising waters and removed when water levels return to normal. Floating defences provide an option that removes the requirement for human intervention for semi-permanent flood defence installations. They are raised and lowered due to being acted upon by the lifting force of the water. This is particularly beneficial in an area that is characterised by a flashier flood hydrograph. Such catchments receive minimal warning prior to peak flood levels. Self-deployed defences such as this should be protected from potential obstructions that would reduce their effectiveness by preventing them from deploying effectively. There are significant risks associated with the failure/breach of such flood defences, which should be carefully considered before their implementation.

Increasing Flow Capacity is a method of flood alleviation that can reduce flood levels at and upstream of the location where it is carried out. This method of flood alleviation includes channel excavation (widening/deepening), removal of vegetation/obstructions and increasing dimensions of floodplain. It influences the hydraulic control process to reduce the overall net effect of flooding. It can also influence the flow attenuation process, which can have negative impacts on flood risk downstream. Increasing the channel carrying capacity increases the rate at which water is supplied to downstream locations as explained in chapters 6 and 7. Therefore it is important to consider such implications carefully prior to the implementation of such measures.

Retention and storage of floodwaters involves any means that increases the attenuation capacity of the channel and floodplain. This includes structures, embankments, excavation and any such works that provides a storage volume where floodwater can be stored safely during times of high inflow allowing it to be released more gradually over time. It can also involve allowing the condition of the drainage network to degrade and become overgrown. This increases the attenuation capacity due to alteration of the stage discharge relationship as described in section 7.1. Sluice gates or weirs may be used to retain water. This technique delays the time to peak and reduces the magnitude of flood peaks downstream resulting in a more damped and even hydrograph. This is particularly effective for flashier catchments with limited natural attenuation capacity. The quantity of storage required may be problematic for rivers with significant flood flow peaks due to spatial requirements. There will also be consequences arising from the temporary flooding of land that should be given proper consideration (e.g. impact on landowners).

Channels can also be diverted. This involves re-routing of flows from the existing channel. It can involve diverting the entire flow of water through a less vulnerable location. Usually it involves diverting a portion of channel discharge to accommodate flood flows. Provision of an overflow channel can alleviate flood flows without having to divert a complete river channel. These works may exhibit negative environmental implications if not designed correctly (e.g. effect on habitat of aquatic species).

Pumping provides a method of removing floodwaters by mechanical means. It usually requires that it be used in conjunction with other alleviation measures that sufficiently reduce the rate of inflow of floodwaters to enable pumping to manage and remove the floodwater effectively.

A flood warning process can be a very important part in effective reduction of flood risk. This process does not reduce the magnitude of flooding but it can reduce the flood damage by providing sufficient warning for action to be taken. It requires reliable forecasting of potential flooding through an effective and timely warning system. Its benefit is increased when the population and response services are well informed and prepared to take effective action to reduce the extent of flood damage. The effectiveness of this process increases exponentially when used in conjunction with temporary flood defence systems that can be erected to provide protection from floodwaters. Local authorities are currently the primary authority responsible for emergency services and the development and activation of major emergency plans in response to flooding [17]. Timely flood warning will greatly increase the effectiveness of such services.

Individual property protection may be utilised in instances where the isolated nature of the property results in it not being cost-effective to provide more significant and expensive flood relief measures. Underground seepage can be a significant obstacle to this option being effective. The cost of providing sufficient protection for isolated property may be too significant to justify. In such cases a relocation package should be considered as a potentially viable solution. This would allow for the natural functions of the floodplain to be maintained thus avoiding any potential increase in flood risk elsewhere due to the implementation of hard-engineered flood defences adversely effecting the flow attenuation and hydraulic control process.

Surface water runoff management is a key element in ensuring that future growth does not adversely impact flood risk. It mainly applies to new developments but can also be applied to existing development in an effort to reduce the rate of runoff within the catchment. This method is particularly effective in catchments with a high urban fraction. It has been identified in chapter 6 that development can significantly increase the surface water runoff by reducing the ability for soil infiltration. This reduces the natural attenuation capacity of a catchment resulting in an increase in peak flood levels. There are numerous surface water attenuation techniques available. Infiltration tanks provide storage that enables ground infiltration. Sustainable Urban Drainage Systems (SUDS) provide sustainable surface water management techniques that can reduce surface water emitted from a site and also increase the biodiversity of the area. Examples of SUDS options include swales, retention ponds, land drains etc. Flow control methods can be used to limit outflows from sites into surface water sewers thus preventing overloading of the public sewer network. This is particularly beneficial in urban areas. Urban areas that provide a combined sewer network (foul and surface water) should employ such measures to avoid contamination of property and environment due to foul water overflow.

The chosen flood relief measures should be evaluated to ensure that they consider and comply with all relevant aspects of flooding. The measures should suitably consider environmental, societal, technical and economic impacts and constraints of their implementation. There may also be less obvious implications such as structural issues relating to construction in the channel and floodplain (e.g. bridges), effect on local drainage networks (e.g. raising water level for storage above the invert of a sewer outfall may result in flooding the artificial drainage network and area which it services). Future increases in flood flows should be considered. A 20% increase in flood flows would result in defences that provide protection from 100-year events being reduced to protecting from 30-year events as discussed in section 5.1.2. Therefore the flood alleviation measures would be best advised to incorporate allowances for such eventualities as described in section 5.1.2.

Once the design has been finalised the relevant documentation, drawings and feasibility report are produced for consultation. The feasibility report may also include further information and suggestions such as future development implications or requested improvements in the data collection network (e.g. hydrometric data) to provide a greater deal of accuracy in generating hydraulic models.

The inclusion of landowners, community and stakeholders in the decision making process is important to ensure that there is complete cooperation from the concerned population. The document preparation and public exhibition stage enables the public to become aware of proposed flood relief schemes and the effect it may have on them. It also provides a forum for people to voice any concerns they may have. The final stages of the process involve the design and construction phase. This should be followed up by a monitoring phase that ensures the measures are performing to minimum requirements as set out by the design process. Quality control of the flood relief scheme is essential to ensure that the considerable planning process realises its full potential.

8.2 Flood Relief Measures for Clare River Catchment

This study has identified a number of different factors that contribute to flood risk in the Clare River catchment. This section aims to identify potential measures that should be undertaken to alleviate the flood risk. These measures will be identified from the most downstream point moving upstream in a sequential manner. This is also the manner in which such works should be carried out to avoid increasing flood risk downstream due to interfering in the flow attenuation process.

The benefit and cost effectiveness of increasing the discharge capacity of the River Corrib channel from Lough Corrib to the sluice barrage at the Salmon Weir Bridge should be evaluated. It has been identified that decreasing lake levels will have an effect on river levels below Claregalway. Excavation of approximately 300,000 m^3 of material would be required to discharge a 3-year event at 6.44 mAod. It would require excavation of 700,000 m^3 of material to discharge a 20-year event at this lake level. These levels would equate to a drop in lake level of 0.19 m and 0.44 m respectively. It was identified that a drop of 1.1 m in lake level would have reduced flood levels by 100 mm to 210 mm in the region of Montiagh during November 2009. Therefore the quantities of excavation of the Corrib River channel mentioned above may provide no significant change in flood levels at this location. The low density of property in the area below Claregalway may also make such investment of funds difficult to justify. However the benefit of lowering lake levels will not just benefit the lower reaches of the Clare River. It will also benefit land along the lake margins and the lower reaches of other inflowing rivers and streams. Therefore the complete benefit of such works should be evaluated to determine if the option is feasible.

Statistical analysis of the flood flows along the Clare River has estimated the magnitude of discharge associated with events of varying return periods. Analysis of hydrometric data and meteorological data for the catchment has not identified any significant increase in the frequency or magnitude of flooding. A review of research studies carried out in relation to climate change has concluded that there could be a significant increase in flood flows in the future. Two possible scenarios have been identified. The more extreme High-End Future Scenario (HEFS) proposes an increase in extreme rainfall depths and subsequent flood flows of 30%. The more probable Mid-Range Future Scenario (MRFS) predicts an increase in flood flows of 20%. This 20% increase corresponds to predictions based upon the 'Report of the Flood Policy Review Group' [17], the EPA study 'Climate Change: Scenarios and Impacts for Ireland' [18], and 'Ireland in a Warmer Climate' [19]. Therefore it is suggested that the MRFS be used to factor in the potential impact of climate change on future flood flows. Planning Regulation proposes the 100-year event as that which should be used in relation to flood risk i.e. flood relief, land zoning, planning etc. The 20% increase of the MRFS should be applied to the 100-year event estimate to provide the design flood flow. This estimated future scenario flow is shown in table 8.1. It can be seen that its magnitude is significantly greater than that observed at all hydrometric stations for the November 2009 flood event.

Table 8.1 – Estimated Future Scenario Flow That Should be used as Design Flood Flow

	Ballyhaunis 30020 (m^3/s)	Ballgaddy 30007 (m^3/s)	Corofin 30004 (m^3/s)	Claregalway 30012 (m^3/s)
Estimated 100-year Return Period Flow	6.44	109.28	185.21	181.07
Allowance for Mid-Range Future Scenario (MRFS)	1.29	21.86	37.04	36.21
Estimated Future Scenario Flow including MRFS	7.73	131.14	222.26	217.29
Peak Flow of November-2009 Flood Event	5.91	108.90	193.00	163.19

It is not expected that increasing the discharge capacity of the Clare River below Claregalway would provide any significant change in flood levels due to the shallow gradient in this region. Every effort should be made to maintain the natural floodplain in this lower reach of the river. This area has been identified as having an expansive floodplain as shown in Appendix A–2.1. Flood relief works to reduce flood risk arising from new development in this region would be extremely costly and technically difficult to achieve due to the spatial distribution of floodwater and the poor soil conditions present.

It is proposed that the hydrometric gauge at Claregalway be move approximately 300 m downstream from the Claregalway Bridge. This point in the river is less affected by floodplain flows bypassing the channel such as those that occur around Claregalway Bridge during significant flood flows. Water levels this distance below Claregalway Bridge were also observed to be 200 mm higher than those recorded on the downstream face of the bridge in 2009 due to turbulence in the vicinity of the bridge. Movement of the hydrometric gauge will enable more accurate recording of water level and discharge data for more accurate statistical analysis of flows at Claregalway. This step should only be required if other works outlined for the Claregalway bridge to increase capacity at this location to accommodate flood flows are not undertaken.

The flood flows of November 2009 demonstrated that the Claregalway Bridge acted as a hydraulic constraint to flood flows. Water levels on the upstream face of the bridge were 1.056 m higher than those recorded on the downstream face and were 1.251 m above the bridge soffit level. The discharge capacity at the bridge should be increased to at least accommodate the 2009 flow, as it has been determined that it is most likely a 100-year flow. The more conservative approach of using the 100-year design flow of 181 m^3/s should be used to provide a factor of safety. Inclusion of a 20% allowance in accordance with climate change will correspond to a discharge of 217.29 m^3/s as shown in table 8.1. Analysis of the stage-discharge relationship of flood flows at Claregalway determined that this flow corresponds to a peak water level of 10.07 mAod. This is 790 mm above the peak water level of 9.28 mAod recorded at the downstream face of the bridge during the November 2009 flood event. Increasing the soffit height above this peak flow may be considered. Suitable structural design and minimisation of the thickness of the bridge deck will help to minimise the implications on road surface level due to raising the soffit level the required 985 mm. A hydraulic model will be required to assess potential implications of various bridge layouts. Provision of a stepped channel and flood eye should be considered as a potential solution. This would allow for alleviation of floodwaters without affecting low summer flows. The invert of this stepped channel should be kept above 5.86 mAod. This is the minimum water level recorded at Claregalway and provides a water depth in the main channel of 500 mm. Consultation with the Western Regional Fisheries Board (WRFB) suggests that an overflow channel invert level of 6.06 mAod would provide more favourable conditions for aquatic species. This would result in low flows remaining unaffected up to a river depth of 700 mm at which point water would enter the upper tier of the stepped channel. A multi-tiered option was also proposed by the WRFB to provide a more gradual dissipation of flood flows.

Other bridges along the Clare River should also be considered to increase their capacity due to potential flooding and also structural implications to the bridge due to build up of head on the upstream face. The most notable of these are Cregmore Bridge and Crusheen Bridge just upstream of Claregalway. Flood-bypass channels such as those proposed for Claregalway Bridge should be considered. Significant flooding in Caherlea, Lisheenavalla and Islandmore occurred just upstream of Crusheen Bridge and Downstream of Cregmore Bridge. This area is also located adjacent to the section of river channel where siltation was observed as described in section 7.2. Channel deepening of 1.52 m should be carried out to return this section of river to its original conditions as designed within the scope of the arterial drainage works carried out by the OPW in the 1950's. Continual monitoring and maintenance of this section of river should be carried out due to its vulnerability to incur siltation as described in section 7.2.

Development in Claregalway town takes consideration of flood risk associated with fluvial flooding. It is suggested that the location and impact of the turlough described in section 6.3 and shown in figure 6.12 also be included in the Claregalway LAP, as it impacts on current land zoning areas. Efforts should also be made to extend the benefit of the Claregalway LAP to encompass the surrounding areas that are at particular flood risk, e.g. Montiagh, Caherlea, Lisheenavalla and Islandmore. The flood extent maps provided in Appendix A–2 should be used as an indicative flood plain of the spatial extent of a 100-year event. Analysis of the significant quantity of recorded data at Corofin and Ballygaddy suggests that the November 2009 flooding was of a magnitude close to a 100-year event. This indicative 100-year flood zone corresponds to the high-risk flood zone A as outlined in the Planning System and Flood Risk Management Guidelines (2009). A more detailed hydraulic model may be constructed to provide predictions of flood zones of various return periods. Such flood zone maps provide an indication of area that should be exempt from development. Flood risk assessments should be compulsory for development in adjacent lands and for unavoidable development in such flood risk zones, which has been validated by the justification test as outlined in section 6.2.2.

The potential to retain floodwater in the Cloonkeen turlough in times of high flow by installation of a flow control device (e.g. weir) should be assessed. The Cloonkeen turlough consists of agricultural land. Prior to arterial drainage schemes the area was a permanent lake. It provides a natural retention basin that could potentially be used for attenuation of floodwater to relieve flood risk downstream. Such a significant change in the hydraulics of the channel should be evaluated comprehensively using suitable modelling software to evaluate all potential implications as it may result in unsuitable levels of flood risk in the locality of the turlough and also upstream in areas such as Tuam, which could be significantly affected by increased flood levels. The potentially catastrophic consequences of a breach or failure resulting in an instant release of the volume of stored water should also be given adequate consideration. This measure would require the acquisition of land or consent from landowners for using the land for this purpose.

A complete revision of the Tuam LAP is required to ensure all aspects of the development plan adequately consider the implications of flood risk. Tuam is a hub town. It has therefore been identified as a location for focused future growth in accordance with the objectives of the National Spatial Strategy (NSS). The area is at risk from fluvial flooding, as it is located at the confluence of the Nanny and the Clare River. It is also potentially at risk from urban flooding due to surface water runoff of extreme events exceeding the capacity of artificial drainage networks. Land zoning should be revised to consider indicative floodplains. The flood extent map for November 2009 provided in Appendix A–2 provides a good indication of a 100-year event in the Tuam area as the flood was estimated as having a return period of 97-years at Ballygaddy just upstream from Tuam.

There has been considerable degradation of channel conditions due to vegetation growth identified in the more remote reaches of the Clare River system. The implication of improving channel conditions should be considered fully before clearing any channel. Channel improvement works would increase the discharge capacity of the channel thus reducing its flow attenuation capacity. This may have negative impacts on flood risk downstream if not considered fully prior to commencing works.

There are a number of other suggestions that apply to the entire catchment. Indicative floodplain maps, such as those provided in Appendix A–2, should be included in land zoning and planning throughout the entire catchment. The application of the sequential approach and Justification test is an important mechanism in reaching well-informed decisions. These methods combined with a spatial representation of flood risk will serve to inform decision-making. Every effort should also be made to ensure that new development provides sufficient surface water management plans and techniques to mitigate their impact on flood flows. Green areas should be kept in good condition to promote soil infiltration. Surface water attenuation and SUDS measures should be implemented to ensure surface water is managed efficiently on-site. These surface water management techniques may include storm water attenuation tanks, infiltration tanks, swales and retention ponds. SUDS techniques also provide the opportunity to enhance the biodiversity of an area. These opportunities should be taken if available.

In instances where development unavoidably leads to a reduction in floodplain storage compensatory floodplain storage should be provided. The objective of maintaining floodplain storage is to ensure that water stored in the floodplain at any point along a watercourse is the same following a development as it was prior to the changes. The compensatory storage volume should be located at the same elevation as the original storage volume and is known as 'level for level' storage provision [42]. It involves excavation of adjacent lands to provide this compensatory storage volume. It should be provided at or as near as possible to the development site. The feasibility of including this method as a flood mitigation measure for new construction relies upon the extent of compensatory storage required, topography of site, spatial constraints, land use and environmental issues. The works should ensure that there is no net loss of floodplain storage subsequent to development being carried out. Figure 8.4 provides a graphical representation of the provision of 'level for level' compensatory flood plain storage.

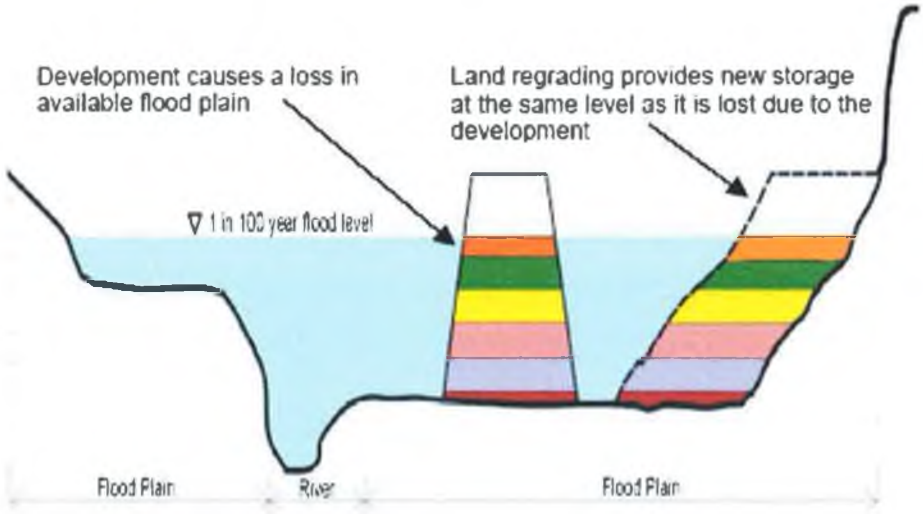

Figure 8.4 – Level for Level Compensatory Flood Plain Storage [43]

The inclusion of these outlined proposals is expected to serve to alleviate flood risk. The inclusion of a flood warning system should also be considered particularly for urban settlement such as Claregalway. There are also constraints that may present difficulties. The entire length of the Clare River is designated a Special Area of Conservation (SAC). This may result in environmental opposition to significant flood relief works. The proximity of the castle at Claregalway may provide an archaeological constraint to development due to its historical significance. Also the nature of the catchment may give rise to unforeseen technical constraints such as poor ground bearing capacity or the karstified nature of the catchment. This would require increased structural performance of flood defences and foundations. It is felt that all of these constraints are not insurmountable and provided the design process is carried out in an informed and comprehensive manner with sufficient interaction with the public and interested parties that a successful implementation of flood relief works can be achieved.

Chapter 9

Conclusions and Recommendations

The Clare River catchment is located in east Galway. It is approximately 1,078 km^2 and is situated within the Lough Corrib catchment area. Its topography is relatively even, sloping gradually in a southwesterly direction. The catchment is drained by a large network of streams and rivers that flow to the Clare River and onto Lough Corrib. There is a great deal of groundwater and surface water interaction due to the karstified nature of the limestone bedrock. Groundwater moves in a predominantly westerly direction. Groundwater losses from the river and greater catchment area are not unusual with groundwater sinking in the Clare River catchment identified as resurfacing in separate river catchments to the west such as the Cregg River. Soil is primarily till with some areas of peat present throughout the catchment. The Clare River flows approximately 93 km in a southerly and then westerly direction to its outfall at Lough Corrib.

Flooding has been recorded along the Clare River as far back as 1968. There have been a number of flood events throughout the 1990's and 2000's. The most significant flood event at most hydrometric stations was the flood event of November 2009. The only flood event of greater magnitude was recorded at Corofin during November 1968. The statistical analysis of flooding along the Clare River during November 2009 estimated that the flooding was most probably a 100-year event. The return periods estimated at Ballyhaunis and Claregalway were considerably less than those estimated at Ballygaddy and Corofin mainly due to the fact that the annual maximum distribution series were considerably shorter at these locations. The statistical analysis of hydrometric data produced estimated flow for varying return periods as shown in section 4.3. The 100-year flow was chosen as the design flood flow for the area in line with planning recommendations. The 100-year flow was subsequently modified by adding 20% in consideration of climate change impacts to provide an estimated future flood flow scenario as demonstrated in section 8.2. The design flows provided in table 8.1 should be used in flood risk management and informing decision-making such as the sizing of bridges and culverts. Statistical analysis of historical flood events shows that while the November 2009 floods were significant that this does not follow a trend of more frequent and severe floods in recent times. Analysis of the first and second half of the annual maximum series at Ballygaddy and Corofin showed a slight increase in the frequency and magnitude of flooding in the latter half of each series. However this was not significantly comprehensive to suggest that it is part of a climatic trend.

A review of climate change resulted in an estimated 20% increase in flood flows being factored into future flow predictions to make allowance for climate change in accordance with the Mid-Range Future Scenario (MRFS). Rainfall played a major role in flooding throughout the catchment. Analysis of meteorological data did not suggest any trend that would indicate an increase in the intensity or severity of rainfall that would correspond to climate change theory. 3-day, 5-day and 10-day totals which are significant precipitation indicators from a flooding perspective did not display any significant climatic trend. However the precautionary approach of incorporating the MRFS into methodologies, strategies and plans should be adopted.

The Clare River catchment is predominantly agricultural with peat bogs dispersed throughout. Although changes in agricultural processes can contribute to increased flood risk it is not expected that they will have a significant impact on the rainfall-runoff process in the Clare River catchment. Urban development can have a significant effect on flooding due to its potential effect on the rainfall-runoff process, flow attenuation process, hydraulic control process and flood damage process. The impact of urban development on the rainfall-runoff process within the Clare River catchment is evaluated to be minimal accounting for only 2.42 m^3/s, approximately 1% of peak 100-year flood flow. The most significant effect that future development could have in flood risk is expected to be due to the implications to the flood damage process. It is important that development plans and planning processes fully consider the implications associated with construction. Assessments and decisions should be carried out in line with the sequential approach and justification test if necessary. Flood risk assessments should become a mandatory element of planning proposals and land zoning decisions for areas in and adjacent to flood zones. The flood extent maps provided in Appendix A–2 should be used in conjunction with such processes to identify areas that require these measures to be carried out. The planning process at Claregalway is determined to be in line with requirements set out at regional and national level. However Tuam LAP does not adequately address flood risk. A full review of the Tuam LAP should be carried out to adequately address flood risk, most notably flood risk zones should influence spatial changes in the existing land zoning policy.

The condition of a watercourse can have a significant effect on flooding within a catchment due to its effect on the flow attenuation process and the hydraulic control process. It is felt that the condition of the Clare River channel is only sufficiently degraded from vegetation growth in the upper reaches to significantly reduce discharge capacity. It is determined that this could only reduce flood risk in the catchment by increasing the attenuation capacity of the channel in these reaches and therefore diminishing the peak flood flow in downstream areas of more significant flood risk. Considerable arterial drainage schemes were carried out along the entire Clare River drainage system. These have linked the upper and lower sections of the drainage network and increased the channel carrying capacity. This has greatly diminished flooding and reduced flood levels. Siltation has been observed upstream of Crusheen Bridge. The channel should be returned to its 3-year design capacity at this point. Water levels in Lough Corrib have not been maintained below the maximum design level as set out in the arterial drainage scheme that resulted in the construction of the Galway sluice barrage. The reason for this is expected to be due to the limited carrying capacity of the River Corrib channel and not due to the manipulation policy of the sluice gates. Water levels in Lough Corrib would have to be significantly reduced to achieve significant benefits in the lower reaches of the Clare River with no noticeable effect being experienced at Claregalway for a simulated reduction in lake level of 1.1 m for the November 2009 flooding.

There are a number of flood risk management measures that can be implemented to reduce flood risk. A list and description of proposed measures for the Clare River catchment is provided in section 8.2. Below is a list of outcomes of the study along with key recommendations derived from section 8.2 that it is felt should be given priority during implementation of a flood relief scheme:

- Estimated 100-year flows including allowance factor for climate, as provided in table 8.1, should be applied to the design of flood management measures and considered in relation to processes or decisions which may affect flood risk.
- The flood extent map should be used as an indicative floodplain map of a 100-year event for areas that do not possess detailed hydraulic models of the spatial extent of flood risk. Lands within or adjacent to this indicative floodplain should be subjected to appropriate levels of scrutiny which should aim to avoid development in the floodplain and ensure that adjacent development does not adversely effect flood risk. This should be done by implementing the sequential

approach, justification test and flood risk assessment as necessary in accordance with the Planning System and Flood Risk Management Guidelines of 2009.

- New developments should provide sufficient surface water management techniques on site to reduce surface water load from the site on drainage networks to an acceptable level.
- Carry out a full review of the Tuam LAP to adequately consider flood risk
- Include the location and impact of turlough in Claregalway town within the scope of the Claregalway LAP.
- Increase the discharge capacity at Claregalway Bridge, Crusheen Bridge and Cregmore Bridge.
- Carry out channel excavation upstream of Crusheen Bridge to return the channel to its original 3-year design dimensions as set out in the arterial drainage scheme carried out in the 1950's. Continuous monitoring should be carried out to ensure siltation is avoided at this location in the future.
- Carry out a review of the potential advantages and disadvantages of utilising the natural storage capacity of Cloonkeen turlough during flood events to alleviate flooding in areas downstream of Corofin with consideration of potential implications to flood risk elsewhere.

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Appendices

Appendix A:	Clare River Catchment Maps
Appendix B:	Legends for Catchment Characteristic Maps
Appendix C:	Hydrometric Data
Appendix D:	Tables and Graphs
Appendix E:	Rainfall Data
Appendix F:	Synthetic Unit Hydrograph (FSR Method)

Appendix A

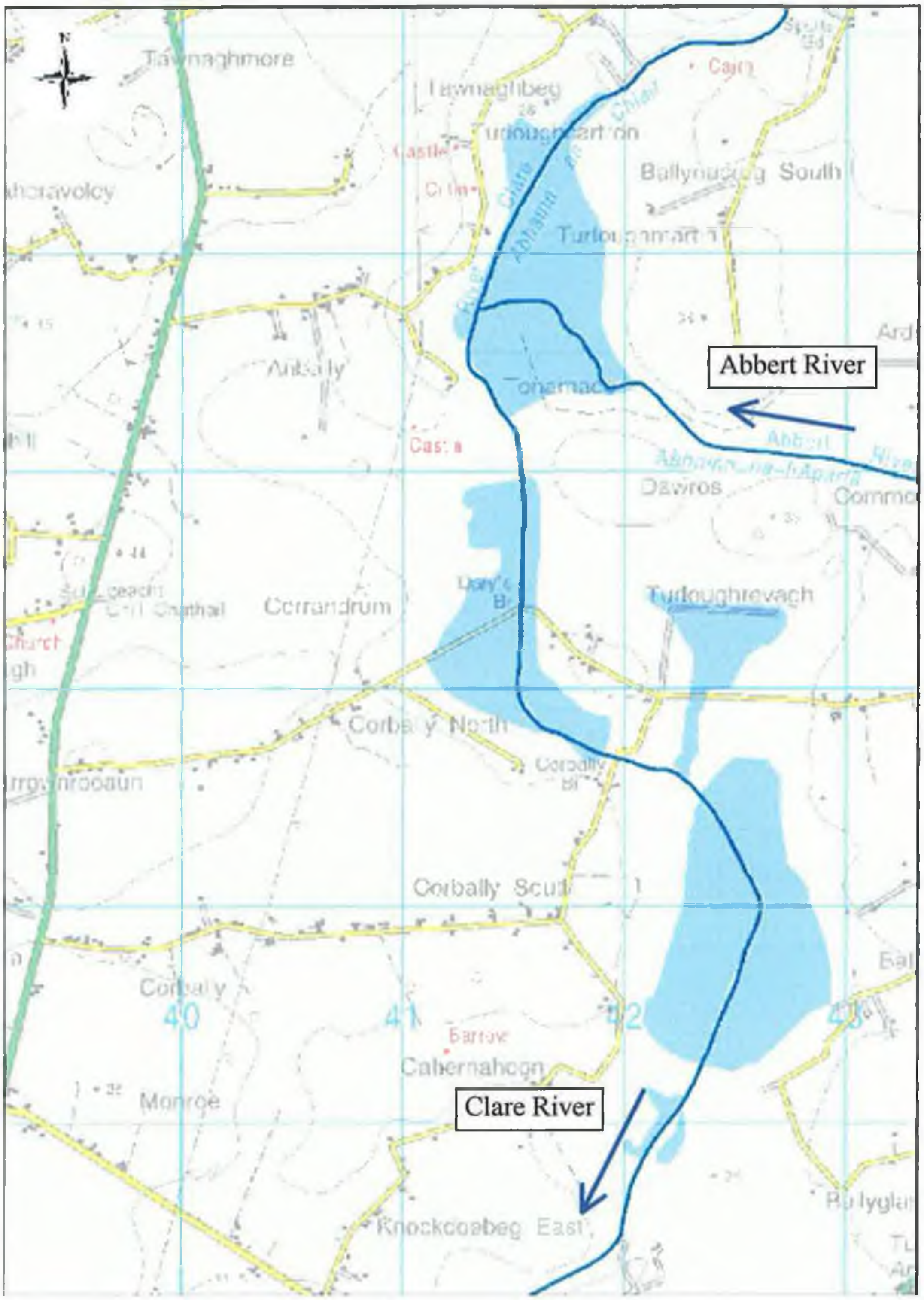
Clare River Catchment Maps:

1. River and Stream Network for Clare River Catchment
2. Flood Extent Maps for November 2009 Flood Event
3. Map of Significant Locations

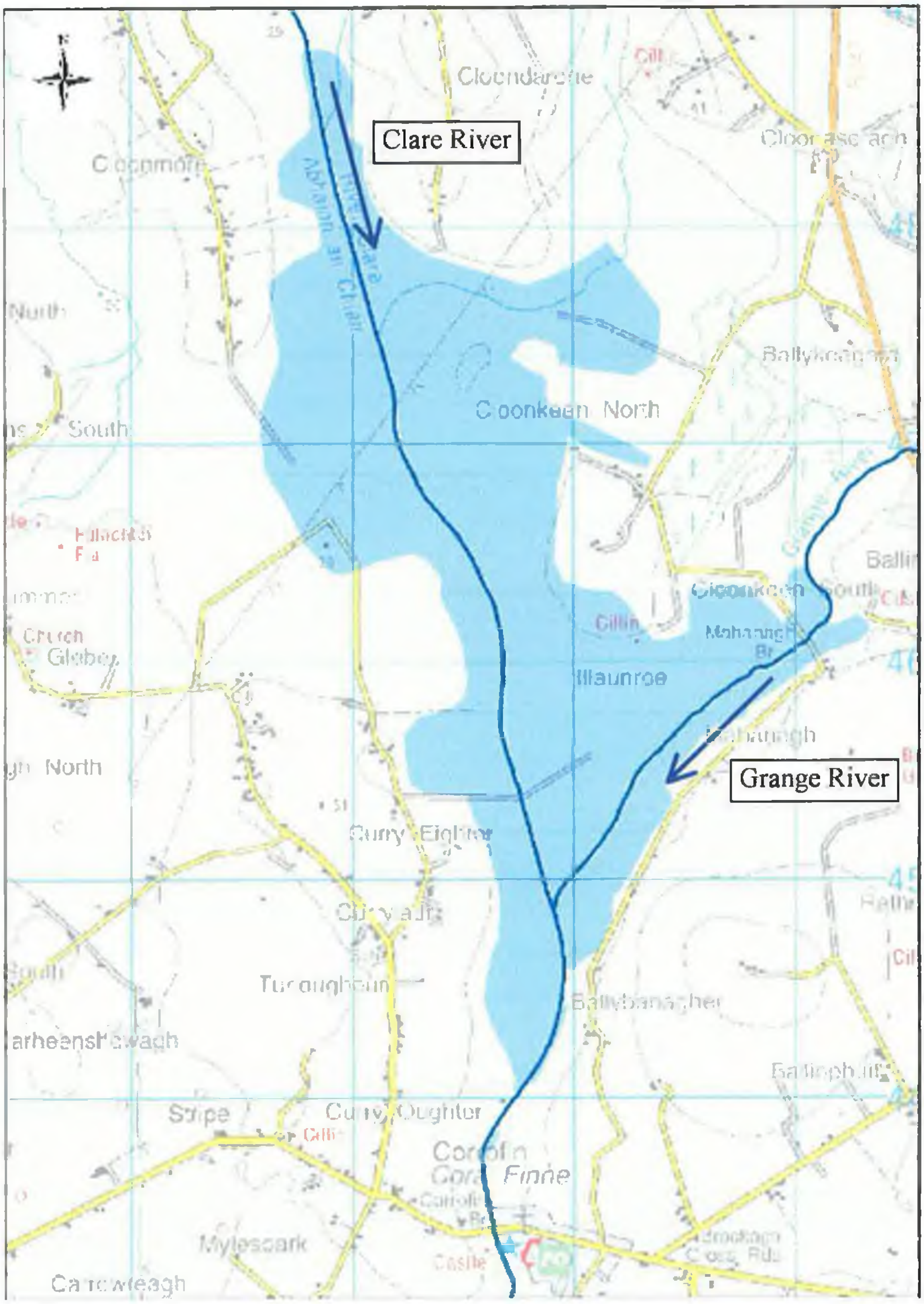
A–1 – River and Stream Network for Clare River Catchment

A–2.1 – Flood Extent Map for Clare River from Tuam to Lough Corrib for November 2009 Flood Event

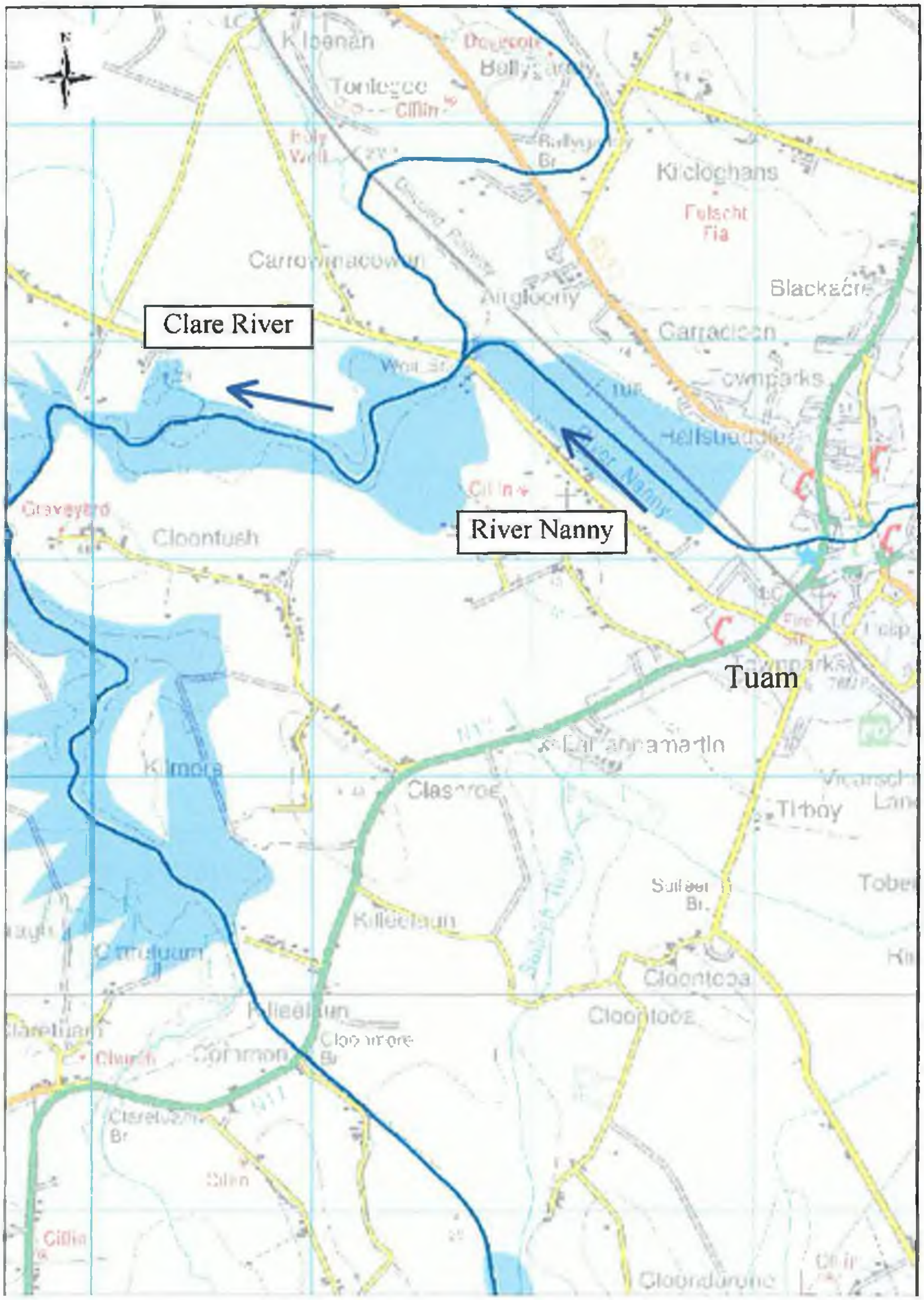
A–2.2 – Flood Extent Map for Clare River at Claregalway
for November 2009 Flood Event



A–2.3 – Flood Extent Map for Clare River at Confluence of Abbert River for November 2009 Flood Event

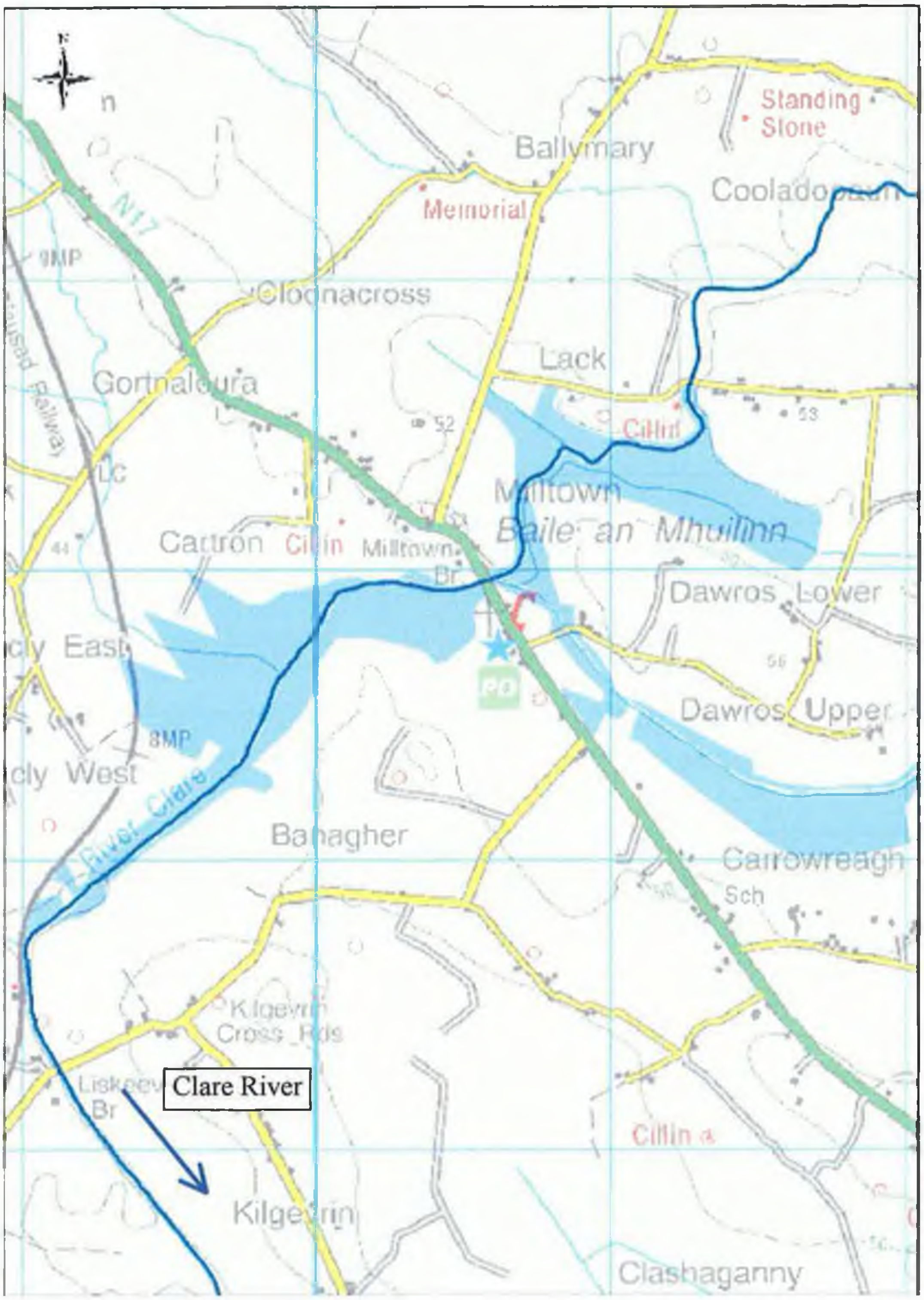


A–2.4 – Flood Extent Map for Clare River at Cloonkeen Turlough upstream from Corofin for November 2009 Flood Event



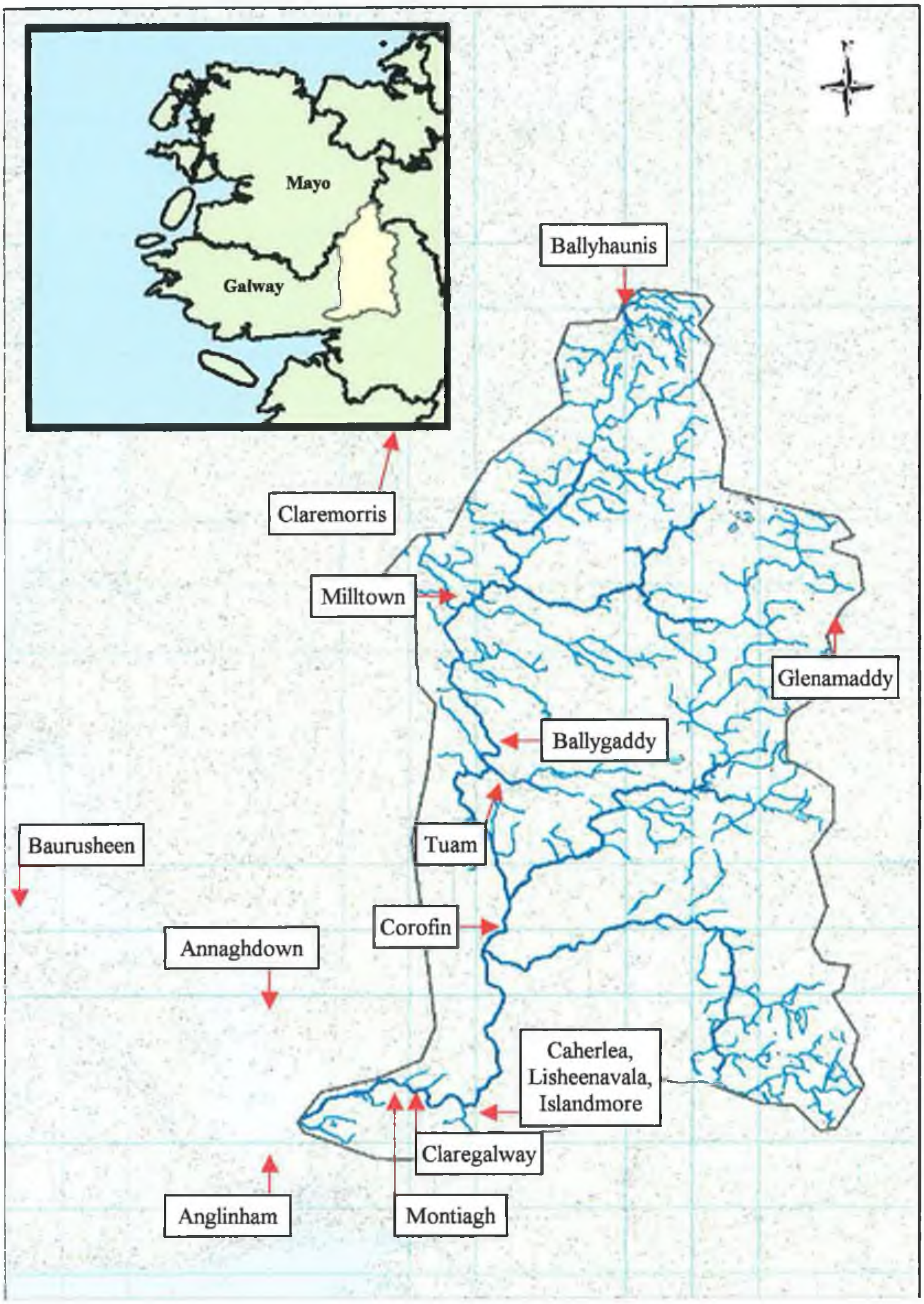
A–2.5 – Flood Extent Map for Clare River at Tuam
for November 2009 Flood Event

Note: Jagged lines to west indicate that floodplain extends farther but there was insufficient information to identify the flood extent accurately for this location



A–2.6 – Flood Extent Map for Clare River at Milltown
for November 2009 Flood Event

Note: Jagged lines to west indicate that floodplain extends farther but there was insufficient information to identify the flood extent accurately for this location



A –3 – Map of Significant Locations

Appendix B

Legends for Catchment Characteristic Maps:

1. Bedrock
2. Aquifer
3. Subsoil (Soil Parent Material)
4. Soil
5. Corine Land Cover

Code	Rock Unit Name	Description
BA	Ballysteen Formation	Dark Muddy Limestone, Shale
BO	Boyle Sandstone Formation	Sandstone, Siltstone, Black Mudstone
CO	Cong Limestone Formation	Thick Bedded Pure Limestone
CT	Coranellistrum Formation	Medium to Thick-Bedded Pure Limestone
CfFe	Caledonian Cloonfad Felsite	Felsite
KA	Knockmaa Formation	Thick Bedded Pure Limestone
Katm	Two Mile Ditch Member	Thick-Bedded Limestone Clay Wayboards
KL	Kilbryan Limestone Formation	Dark Nodular Calcarenite & Shale
LU	Lucan Formation	Dark Limeston & Shale ('calp)
NL	Cong Canal Formation	Medium to Thick-Bedded Pure Limestone
OK	Oakport Limestone Formation	Pale Grey Massive Limestone
VIS	Visean Limestones (undifferentiated)	Undiffeerentiated Limestone
WA	Waulsortian Limestones	Massive Unbedded Lime-Mudstone

B–1 – Bedrock Legend

Reference	Aquifer Type	Comments
LI	Locally Important	Bedrock which is Moderately Productive only in Local Zones
PI	Poor	Bedrock which is Generally Unproductive except for Local Zones
Rkc	Regionally Important karstified	karstified (conduit)
Unclassified	Unclassified	

B–2 – Aquifer Legend

B–3 – Subsoil (Soil Parent Material) Legend

Tills:

Till type	Texture	Text on map	Layer Code
Sandstone till (Cambrian/Precambrian)	Sandy	TCSs	Pet-TO-TCSs-T
Shale till (Cambrian/Precambrian)	Clayey	TCS	Pet-TO-TCS-T
Sandstone and shale till (Cambrian/Precambrian)	Clayey	TCSsS	Pet-TO-TCSsS-T
Greywacke till (Cambrian/Precambrian)	Stony	TCGw	Pet-TO-TCGw-T
Sandstone till (Lower Palaeozoic)	Sandy/ silty	TLPSs	Pet-TO-TLPSs-T
Shale till (Lower Palaeozoic)	Clayey	TLPS	Pet-TO-TLPS-T
Sandstone and shale till (Lower Palaeozoic)	Clayey	TLPSsS	Pet-TO-TLPSsS-T
Greywacke till (Lower Palaeozoic)	Stony	TLPGw	Pet-TO-TLPGw-T
Sandstone till (Lower Palaeozoic/Devonian)	Sandy	TLPDSs	Pet-TO-TLPDSs-T
Sandstone till (Devonian)	Sandy	TDSs	Pet-TO-TDSs-T
Sandstone till (Devonian/Carboniferous)	Sandy	TDCSs	Pet-TO-TDCSs-T
Sandstone and shales till (Devonian/Carboniferous)	Sandy	TDCSsS	Pet-TO-TDCSsS-T
Limestone till (Carboniferous)	Variable	TLs	Pet-TO-TLs-T
Sandstone till	Sandy	TSs	Pet-TO-TSs-T
Shales and sandstones till (Namurian)	Clayey	TNSSs	Pet-TO-TNSSs-T
Sandstone till (Triassic)	Sandy	TTrSs	Pet-TO-TTrSs-T
Chert till	Stony	TCh	Pet-TO-TCh-T
Quartzite till	Stony	TQz	Pet-TO-TQz-T
Acid volcanic till	Variable	TAv	Pet-TO-TAv-T
Granite till	Sandy	TGr	Pet-TO-TGr-T
Basic igneous till	Clayey	TBi	Pet-TO-TBi-T
Metamorphic till	Variable	TMp	Pet-TO-TMp-T
Sandstone till (Cambrian/Precambrian) with matrix of Irish Sea Basin origin	Clayey	IrSTCSs	Pet-TI-IrSTCSs-T
Shale till (Cambrian/Precambrian) with matrix of Irish Sea Basin origin	Clayey	IrSTCS	Pet-TI-IrSTCS-T
Sandstone and shale till (Cambrian/Precambrian) with matrix of Irish Sea Basin origin	Clayey	IrSTCSsS	Pet-TI-IrSTCSsS-T
Greywacke till	Clayey	IrSTCGw	Pet-TI-IrSTCGw-T

(Cambrian/Precambrian) with matrix of Irish Sea Basin origin			
Sandstone till (Lower Palaeozoic) with matrix of Irish Sea Basin origin	Clayey	IrSTLPSs	Pet-TI-IrSTLPSs-T
Shale till (Lower Palaeozoic) with matrix of Irish Sea Basin origin	Clayey	IrSTLPS	Pet-TI-IrSTLPS-T
Sandstone and shale till (Lower Palaeozoic) with matrix of Irish Sea Basin origin	Clayey	IrSTLPSsS	Pet-TI-IrSTLPSsS-T
Greywacke till (Lower Palaeozoic) with matrix of Irish Sea Basin origin	Clayey	IrSTLPGw	Pet-TI-IrSTLPGw-T
Sandstone till (Lower Palaeozoic/Devonian) with matrix of Irish Sea Basin origin	Clayey	IrSTLPDSs	Pet-TI-IrSTLPDSs-T
Sandstone till (Devonian/Carboniferous) with matrix of Irish Sea Basin origin	Clayey	IrSTDCSs	Pet-TI-IrSTDCSs-T
Limestone till (Carboniferous) with matrix of Irish Sea Basin origin	Clayey	IrSTLs	Pet-TI-IrSTLs-T
Sandstone till with matrix of Irish Sea Basin origin	Clayey	IrSTSs	Pet-TI-IrSTSs-T
Chert till with matrix of Irish Sea Basin origin	Clayey	IrSTCh	Pet-TI-IrSTCh-T
Quartzite till with matrix of Irish Sea Basin origin	Clayey	IrSTQz	Pet-TI-IrSTQz-T
Acid volcanic till with matrix of Irish Sea Basin origin	Clayey	IrSTAv	Pet-TI-IrSTAv-T
Granite till with matrix of Irish Sea Basin origin	Clayey	IrSTGr	Pet-TI-IrSTGr-T
Basic igneous till with matrix of Irish Sea Basin origin	Clayey	IrSTBi	Pet-TI-IrSTBi-T
Metamorphic till with matrix of Irish Sea Basin origin	Clayey	IrSTMp	Pet-TI-IrSTMp-T

Glaciofluvial sands and gravels:

Sands and gravels type	**Texture**	**Text on map**	**Layer Code**
Sands and gravels (undifferentiated)	Gravelly	G	Pet-SG-G-T
Esker sands and gravels	Gravelly	Esk	Pet-SG-Esk-T
Sandstone sands and gravels (Cambrian/Precambrian)	Gravelly	GCSs	Pet-SG-GCSs-T
Shale sands and gravels	Gravelly	GCS	Pet-SG-GCS-T

(Cambrian/Precambrian)			
Sandstone and shale sands and gravels (Cambrian/Precambrian)	Gravelly	GCSsS	Pet-SG-GCSsS-T
Greywacke sands and gravels (Cambrian/Precambrian)	Gravelly	GCGw	Pet-SG-GCGw-T
Sandstone sands and gravels (Lower Palaeozoic)	Gravelly	GLPSs	Pet-SG-GLPSs-T
Shale sands and gravels (Lower Palaeozoic)	Gravelly	GLPS	Pet-SG-GLPS-T
Sandstone and shale sands and gravels (Lower Palaeozoic)	Gravelly	GLPSsS	Pet-SG-GLPSsS-T
Greywacke sands and gravels (Lower Palaeozoic)	Gravelly	GLPGw	Pet-SG-GLPGw-T
Sandstone sands and gravels (Lower Palaeozoic/Devonian)	Gravelly	GLPDSs	Pet-SG-GLPDSs-T
Sandstone sands and gravels (Devonian)	Gravelly	GDSs	Pet-SG-GDSs-T
Sandstone sands and gravels (Devonian/Carboniferous)	Gravelly	GDCSs	Pet-SG-GDCSs-T
Limestone sands and gravels (Carboniferous)	Gravelly	GLs	Pet-SG-GLs-T
Sandstone sands and gravels	Gravelly	GSs	Pet-SG-GSs-T
Shales and sandstones sands and gravels (Namurian)	Gravelly	GNSSs	Pet-SG-GNSS-T
Sandstone sands and gravels (Triassic)	Gravelly	GTrSs	Pet-SG-GTrSs-T
Chert sands and gravels	Gravelly	GCh	Pet-SG-GCh-T
Quartzite sands and gravels	Gravelly	GQz	Pet-SG-GQz-T
Acid volcanic sands and gravels	Gravelly	GAv	Pet-SG-GAv-T
Granite sands and gravels	Gravelly	GGr	Pet-SG-GGr-T
Basic igneous sands and gravels	Gravelly	GBi	Pet-SG-GBi-T
Metamorphic sands and gravels	Gravelly	GMp	Pet-SG-GMp-T

Glaciolacustrine deposits:

Textural labels for sorted sediments	**Texture**	**Text on map**	**Layer Code**
Lacustrine Sediments			
Lake sediments undifferentiated	Variable	L	Pet-L-T
Gravelly	Gravelly	Lg	Pet-Lg-T
Sandy	Sandy	Ls	Pet-Ls-T
Silty	Silty	Lsi	Pet-Lsi-T
Clayey	Clayey	Lc	Pet-Lc-T

Alluvium:

Textural labels for sorted sediments	**Texture**	**Text on map**	**Layer Code**

Alluvial Sediments			
Alluvium undifferentiated	Variable	A	Pet-A-T
Gravelly	Gravelly	Ag	Pet-Ag-T
Sandy	Sandy	As	Pet-As-T
Silty	Silty	Asi	Pet-Asi-T
Clayey	Clayey	Ac	Pet-Ac-T

Marine deposits:

Textural labels for sorted sediments	Texture	Text on map	Layer Code
Marine Deposits			
Marine sands and gravels	Gravelly	MGs	Pet-MGs-T
Beach/raised beach sand	Sandy	Mbs	Pet-Mbs-T
Beach/raised beach gravel	Gravelly	Mbg	Pet-Mbg-T
Beach/raised beach sands and gravels	Gravelly	Mbs	Pet-Mbsg-T
Marine silts	Silty	Msi	Pet-Msi-T
Marine clays	Clayey	Mc	Pet-Mc-T
Estuarine sediments (silts/clays)	Clayey	MEsc	Pet-MEsc-T

Peat:

Peat type	Texture	Text on map	Layer code
Blanket peat	Peaty	BktPt	Pet-OT-BktPt-T
Raised peat	Peaty	RsPt	Pet-OT-RsPt-T
Fen peat	Peaty	FenPt	Pet-OT-FenPt-T
Cutover peat	Peaty	Cut	Pet-OT-CutPt-T

Other deposits:

Aeolian.

Aeolian sediment type	Texture	Text on map	Layer code
Aeolian Sediments undifferentiated	Sandy/silty	Aeo	Pet-W-T
Blown sand	Sandy	Ws	Pet-Ws-T
Blown sand in dunes	Sandy	Wsd	Pet-Wsd-T

Deposit type	Texture	Text on map	Layer code
Colluvium (slope deposits, including head)	Variable	Clv	Pet-OT-Clv-T
Marl (Shell)	Clayey/ silty	Mrl	Pet-OT-Mrl-T
Residuals (weathered *in situ* bedrock)	Variable	Resid	Pet-Ot-Resid-T
Scree	Blocky	Scree	Pet-Ot-Scree-T
Made ground	Variable	Made	Pet-Ot-Mde-T

Marsh	Marshy	Marsh	Pet-Ot-Mrsh-T
Tidal marsh	Marshy	TdlMr	Pet-Ot-TdlMr-T
Bedrock at surface	n/a	Rck	Pet-OT-Rck-T
Bedrock close to surface (within 1m with till veneer)	n/a	Subrck	Pet-OT-Subrck-T
Karstified limestone bedrock at surface	n/a	KaRck	Pet-OT-KaRck-T

B–4 – Soil Legend

IFS soil categories

Friday, 17 September 2010

IFS Soil	IFS Attribute	IFS Code
Deep well drained mineral		**1**
Derived from mainly acidic parent materials	AminDW	11
Derived from mainly basic parent materials	BminDW	12
Shallow well drained mineral		**2**
Derived from mainly acidic parent materials	AminSW	21
Derived from mainly basic parent materials	BminSW	22
Deep poorly drained mineral		**3**
Derived from mainly acidic parent materials	AminPD	31
Derived from mainly basic parent materials	BminPD	32
Poorly drained mineral soils with peaty topsoil		**4**
Derived from mainly acidic parent materials	AminPDPT	41
Derived from mainly basic parent materials	BminPDPT	42
Podsolised soils with/without peaty topsoil		
Mineral podsolised soils and peaty topsoil with occasional iron pan layer	PodPDPT	43

Alluviums		**5**
Mineral alluvium	AlluvMIN	51
Peaty alluvium	AlluvPT	52
Marl type soils	AlluvMRL	53
Alluvium undifferentiated	AlluvUND	55
Lacustrine alluviums	AlluvLk	56
Peats		
(Raised)		**6**
Raised bog	RsPT	61
Raised bog (cutaway)	Cut	62
(Blanket)		
Mountain	BktPt	63
Lowland	BktPt	64
Cutaway	Cut	65
Miscellaneous		**7**
Scree	Scree	70
Aeolian undifferentiated	AeoUND	71
Aeolian sands	AeoSands	71
Beach sand and gravels	MarSands	72
Marine/ Estuarine sediments	MarSed	73
Reed Swamp/Marsh	Swamp	75
Made	Made	74
Lake	Water	76
Reservoir	Water	76
Unclassified	Unclass	77
No data	Unclass	77

B–5 – Corine Land Cover Legend

Code	Group Description	Description
1.1.1	Urban Fabric	Continuous Urban Fabric
1.1.2		Discontinuous Urban Fabric
1.2.1	Industrial ,Commercial & Transport Units	Industrial or Commercial Units
1.2.2		Road & Rail Networks & Associated Land
1.2.3		Port Areas
1.2.4		Airports
1.3.1	Mine, Dump & Construction Sites	Mineral Extraction Sites
1.3.2		Dump Sites
1.3.3		Construction Sites
1.4.1	Artificial, Non-Agricultural Vegetated Areas	Green Urban Areas
1.4.2		Sport & Leisure Facilities
2.1.1	Arable Land	Non Irrigated Arable land
2.1.2		Permanently Irrigated Land
2.1.3		Rice Fields
2.2.1	Permanent Crops	Vineyards
2.2.2		Fruit Trees & Berry Plantations
2.2.3		Olive Groves
2.3.1	Pasture	Pastures
2.4.1	Heterogeneous Agricultural Areas	Annual Crops Associated with Permanent Crops
2.4.2		Complex Cultivation Patterns
2.4.3		Land Principally Ocupied by Agriculture, with Significant Areas of Natural Vegetation
2.4.4		Agro Forestry Areas
3.1.1	Forest	Broad Leaved Forest
3.1.2		Coniferous Forest
3.1.3		Mixed Forest
3.2.1	Scrub &/or Herbaceous Vegetation Associations	Natural Grasslands
3.2.2		Moors & Heathland
3.2.3		Sclerophyllous Vegetation
3.2.4		Traditional Woodland Scrub
3.3.1	Open Spaces with Little or no Vegetation	Beaches, Dunes, Sands
3.3.2		Bare Rocks
3.3.3		Sparsley Vegetated Areas
3.3.4		Burnt Areas
3.3.5		Glaciers & Perpetual Snow
4.1.1	Inland Wetlands	Inland Marshes

4.1.2		Peat Bogs
4.2.1	Maritime Wetlands	Salt Marshes
4.2.2		Salines
4.2.3		Intertidal Flats
5.1.1	Inland Waters	Water Courses
5.1.2		Water Bodies
5.2.1	Marine Waters	Coastal Lagoon
5.2.2		Estuaries
5.2.3		Sea & Ocean

Appendix C

Hydrometric Data:

1. Annual Maxima Distribution Series
2. EV1 Distribution (Gumbel), Method of Moments
3. EV1 Distribution (Frequency Factor)
4. EV1 Distribution (Gringorten), Probability Plotting
5. Comparison of Plotted Data with Lognormal Distribution Fitted to them by Frequency Factor
6. Standard Error and Confidence Limits
7. Statistical Analysis of Historical Flood Events
8. Statistical Analysis of Return Period Flows for 1st and 2nd Half of Annual Maxima Data Series

C–1.1 – Annual Maxima Distribution Series for Ballyhaunis

Station No.:	30020
Station Name:	Ballyhaunis
Water body:	Dalgan River
Catchment Area (km^2):	21.4
Partial Distribution Series:	Annual Maxima

Hydrmoetric Year	Water Level (mAod - Malin)	S.G. Reading (m)	Estimated Flow (m^3/s)	Date	
1991	72.32	0.94	4.2	08/01/1992	1
1992	72.23	0.85	3.4	01/12/1992	2
1993	72.13	0.76	2.6	01/02/1994	
1994	72.19	0.81	3.1	16/01/1995	
1995	72.33	0.96	4.4	26/10/1995	3
1996	72.10	0.72	2.4	25/02/1997	
1997	72.10	0.72	2.4	10/01/1998	
1998	72.20	0.83	3.2	02/01/1999	
1999	72.35	0.98	4.7	28/11/1999	4
2000	72.09	0.72	2.4	04/12/2000	5
2001	72.20	0.82	3.2	10/03/2002	
2002	72.09	0.71	2.3	27/10/2002	
2003	72.17	0.79	3.0	01/02/2004	6
2004	72.17	0.80	3.0	08/01/2005	
2005	72.03	0.66	1.8	24/10/2005	
2006	72.31	0.94	4.2	03/12/2006	
2007	72.22	0.84	3.3	09/12/2007	7
2008	72.21	0.84	3.3	10/10/2008	8
2009	72.48	1.11	5.9	19/11/2009	9

Hydrometric year X begins on September 1st of year X and ends on August 31st of year X +1

Staff Gauge Zero History: 1991-Present 71.375 mAod Malin

1 - Incomplete Hydrometric year, missing 9th - 10th June

2 - Incomplete Hydrometric year, missing 13th - 28th April

3 - Incomplete Hydrometric year, missing 18th February - 4th April

4 - Incomplete Hydrometric year, missing 18th April

5 - Incomplete Hydrometric year, missing 9th - 22nd February

6 - Incomplete Hydrometric year, missing 11th March - 22nd April and 25th June - 9th September

7 - Incomplete Hydrometric year, missing 22nd - 31st May

8 - Incomplete Hydrometric year, missing 10th April - 3rd June

9 - Incomplete Hydrometric year, ended June 20th

The absent data is primarily located in the summer months with no absent winter data
Therefore it is not expected to have an impact on the annual maxima series

However results should be treated with caution and are merely provided to demonstrate
the magnitude of events in the upper reaches of the Clare River System

Weir installed 15/12/1988 to maintain summer flows; no significant effect on annual maximum flows

C–1.2 – Annual Maxima Distribution Series for Ballygaddy

Station No.:	30007
Station Name:	Ballygaddy
Water body:	Clare River
Catchment Area (km^2):	469.9
Partial Distribution Series:	Annual Maxima

Hydrometric Year	Water Level (mAod - Malin)	S.G. Reading (m)	Estimated Flow (m^3/s)	Date	
1974	34.25	1.79	51.4	16/01/1975	
1975	34.36	1.90	57.6	09/01/1976	
1976	34.07	1.61	42.0	20/01/1977	
1977	34.48	2.02	64.8	08/11/1977	
1978	34.30	1.84	54.2	28/12/1978	
1979	34.54	2.08	68.5	27/11/1979	
1980	34.50	2.04	66.0	03/11/1980	
1981	34.26	1.80	52.0	10/03/1982	
1982	34.51	2.05	66.7	20/12/1982	
1983	34.45	1.99	63.0	17/01/1984	
1984	34.50	2.04	66.0	27/05/1985	
1985	34.58	2.12	71.1	07/08/1986	
1986	34.66	2.20	76.3	05/12/1986	
1987	34.48	2.02	64.8	19/01/1988	
1988	34.46	2.00	63.6	10/03/1989	
1989	34.94	2.48	96.0	07/02/1990	
1990	34.53	2.07	67.9	19/03/1991	
1991	34.56	2.10	69.8	09/01/1992	
1992	34.50	2.04	66.0	03/12/1992	
1993	34.24	1.78	50.9	01/02/1994	
1994	34.38	1.92	58.8	22/01/1995	
1995	34.49	2.03	65.4	27/10/1995	
1996	34.12	1.66	44.5	18/02/1997	
1997	34.19	1.73	48.2	09/01/1998	
1998	34.28	1.82	53.1	03/01/1999	
1999	34.92	2.46	94.5	29/11/1999	
2000	34.18	1.72	47.6	06/11/2000	
2001	34.42	1.96	61.0	11/03/2002	
2002	34.21	1.75	49.2	11/03/2003	
2003	34.35	1.89	57.0	03/02/2004	
2004	-	-	58.9	09/01/2005	1
2005	34.21	1.63	43.0	22/05/2006	
2006	34.35	2.32	84.5	05/12/2006	
2007	34.35	1.89	57.1	04/02/2008	
2008	34.39	1.93	59.4	12/10/2008	
2009	-	-	108.9	20/11/2009	2

Reliable Limit = $70m^3/s$; Discharges above this magnitude are extrapolated and should be treated with caution

Hydrometric year X begins on September 1st of year X and ends on August 31st of year X +1

Staff Gauge Zero History:	1974-Present	32.46	mAod Malin

1 - Estimated Level

2 - Incomplete Hydrometric year, ended December 31st

C–1.3 – Annual Maxima Distribution Series for Corofin

Station No.:	30004
Station Name:	Corofin
Water body:	Clare River
Catchment Area (km^2):	699.9
Partial Distribution Series:	Annual Maxima

Hydrometric Year	Water Level (mAod - Malin)	S.G. Reading (m)	Estimated Flow (m^3/s)	Date	
1964	25.53	3.02	93.4	07/10/1964	
1965	25.47	2.96	90.3	25/11/1965	
1966	24.98	2.47	66.9	02/12/1966	
1967	25.89	3.38	113.0	10/10/1967	
1968	27.30	4.79	207.0	02/11/1968	
1969	25.62	3.11	98.2	22/12/1969	
1970	25.10	2.59	72.4	03/11/1970	
1971	25.17	2.66	75.6	02/04/1972	
1972	25.90	3.39	83.5	12/12/1972	
1973	25.83	3.32	80.1	11/11/1973	
1974	26.31	3.80	105.0	22/01/1975	
1975	25.94	3.43	85.5	09/01/1976	
1976	25.51	3.00	65.3	07/02/1977	
1977	26.07	3.56	92.1	07/11/1977	
1978	25.79	3.28	78.2	28/12/1978	
1979	-	-	88.7	26/11/1979	1
1980	26.33	3.82	106.0	03/11/1980	
1981	25.57	3.06	68.0	15/12/1981	
1982	26.07	3.56	92.1	20/12/1982	
1983	25.95	3.44	86.0	17/01/1984	
1984	25.96	3.45	86.5	29/11/1984	
1985	26.13	3.62	95.3	07/08/1986	
1986	-	-	95.0	05/12/1986	1
1987	25.91	3.40	84.0	04/02/1988	
1988	25.95	3.44	86.0	10/03/1989	
1989	26.63	4.12	123.0	08/02/1990	
1990	26.19	3.68	98.5	29/12/1990	
1991	26.34	3.83	107.0	09/01/1992	
1992	26.21	3.70	99.5	03/12/1992	
1993	25.94	3.43	85.5	09/12/1993	
1994	26.71	4.20	128.0	14/12/1994	
1995	26.25	3.74	102.0	27/10/1995	
1996	25.92	3.41	84.5	17/02/1997	
1997	26.09	3.58	93.2	26/12/1997	
1998	25.88	3.37	82.6	03/01/1999	
1999	26.76	4.25	131.0	30/11/1999	
2000	25.94	3.43	85.5	06/11/2000	
2001	26.21	3.70	99.5	05/02/2002	
2002	25.71	3.20	74.4	11/11/2002	
2003	26.13	3.62	95.3	03/02/2004	
2004	26.40	3.89	110.0	10/01/2005	
2005	25.71	3.20	74.4	22/09/2006	

2006	27.04	4.53	148.0	06/12/2006	
2007	26.25	3.74	102.0	06/02/2008	
2008	26.35	3.84	107.0	12/10/2008	
2009	27.14	4.63	193.0	21/11/2009	2

Reliable Limit = $100m^3/s$; Discharges above this magnitude are extrapolated and should be treated with caution

Hydrometric year X begins on September 1^{st} of year X and ends on August 31^{st} of year X +1

Staff Gauge Zero History: 1964-Present 22.51 mAod Malin

1 - Estimated Level

2 - Incomplete Hydrometric year, ended December 31^{st}

C–1.4 – Annual Maxima Distribution Series for Claregalway

Station No.:	30012
Station Name:	Claregalway
Water body:	Clare River
Catchment Area (km^2):	1072.9
Partial Distribution Series:	Annual Maxima

Hydrometric Year	Water Level (mAod - Malin)	S.G. Reading (m)	Estimated Flow (m^3/s)	Date	
1996	8.25	2.53	88.5	26/02/1997	
1997	8.43	2.71	100.2	06/01/1998	
1998	8.39	2.66	97.3	17/01/1999	
1999	8.91	3.18	134.0	25/12/1999	
2000	8.57	2.84	109.3	07/11/2000	
2001	8.70	2.98	118.9	05/02/2002	
2002	8.31	2.59	92.5	11/11/2002	
2003	8.43	2.71	100.1	04/02/2004	
2004	8.75	3.03	122.5	10/01/2005	
2005	8.24	2.52	87.9	25/10/2005	
2006	8.92	3.20	135.1	07/12/2006	
2007	8.71	2.99	119.4	10/12/2007	
2008	8.78	3.06	124.5	12/10/2008	
2009	9.28	3.55	163.2	22/11/2009	2

Hydrometric year X begins on October 1st of year X and ends on September 30st of year X +1

Staff Gauge Zero History: 1996-Present 5.724 mAod Malin

1 - Estimated Level

2 - Incomplete Hydrometric year, ended June 20th

C–2.1 – Ballyhaunis: EV1 Distribution (Gumbel MOM)

Max. Value	5.91	
s	1.00	standard deviation of Q
x_{av}	3.30	average Q
α	0.78	scale parameter (eqn. 4.2.1.3)
u	2.86	location parameter (eqn. 4.2.1.4)

Return Period (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.5	0.37	3.14
5	0.2	1.50	4.02
10	0.1	2.25	4.61
25	0.04	3.20	5.35
50	0.02	3.90	5.89
100	0.01	4.60	6.44
500	0.002	6.21	7.69
	1/T	(eqn. 4.2.1.15)	(eqn. 4.2.1.16)

C–2.2 – Ballygaddy: EV1 Distribution (Gumbel MOM)

Max. Value	96.00	
s	14.74	standard deviation of Q
x_{av}	63.05	average Q
α	11.49	scale parameter (eqn. 4.2.1.3)
u	56.41	location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	60.63
5	0.20	1.50	73.65
10	0.10	2.25	82.28
25	0.04	3.20	93.18
50	0.02	3.90	101.26
100	0.01	4.60	109.28
500	0.002	6.21	127.83
	1/T	(eqn. 4.2.1.15)	(eqn. 4.2.1.16)

C–2.3 – Corofin: EV1 Distribution (Gumbel MOM)

Max. Value	207.00	
s	27.74	← standard deviation of Q
x_{av}	98.22	← average Q
α	21.63	← scale parameter (eqn. 4.2.1.3)
u	85.74	← location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	93.66
5	0.20	1.50	118.17
10	0.10	2.25	134.40
25	0.04	3.20	154.90
50	0.02	3.90	170.12
100	0.01	4.60	185.21
500	0.002	6.21	220.11
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

C–2.4 – Claregalway: EV1 Distribution (Gumbel MOM)

Max. Value	163.19	
s	21.44	← standard deviation of Q
x_{av}	113.81	← average Q
α	16.72	← scale parameter (eqn. 4.2.1.3)
u	104.16	← location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.5	0.37	110.29
5	0.2	1.50	129.24
10	0.1	2.25	141.78
25	0.04	3.20	157.64
50	0.02	3.90	169.40
100	0.01	4.60	181.07
500	0.002	6.21	208.05
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

C–3.1 – Ballyhaunis: EV1 Distribution (Frequency Factor)

x_{av}	3.30	← average Q
s	1.00	← standard deviation of Q

Return Period T	Exceedance Probability p	Frequency Factor Kt	Discharge Q (m^3/s)
2	0.5	-0.164	3.141
5	0.2	0.719	4.023
10	0.1	1.305	4.608
25	0.04	2.044	5.346
50	0.02	2.592	5.894
100	0.01	3.137	6.438
500	0.002	4.395	7.694
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.2.2)

C–3.2 – Ballygaddy: EV1 Distribution (Frequency Factor)

x_{av}	63.05	← average Q
s	14.74	← standard deviation of Q

Return Period T (years)	Exceedance Probability p	Frequency Factor Kt	Discharge Q (m^3/s)
2	0.5	-0.164	60.626
5	0.2	0.719	73.653
10	0.1	1.305	82.278
25	0.04	2.044	93.175
50	0.02	2.592	101.260
100	0.01	3.137	109.284
500	0.002	4.395	127.829
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.2.2)

C–3.3 – Corofin: EV1 Distribution (Frequency Factor)

x_{av}	98.22	← average Q
s	27.74	← standard deviation of Q

Return Period T (years)	Exceedance Probability p	Frequency Factor Kt	Discharge Q (m^3/s)
2	0.5	-0.164	93.661
5	0.2	0.719	118.172
10	0.1	1.305	134.400
25	0.04	2.044	154.904
50	0.02	2.592	170.116
100	0.01	3.137	185.215
500	0.002	4.395	220.106
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.2.2)

C–3.4 – Claregalway: EV1 Distribution (Frequency Factor)

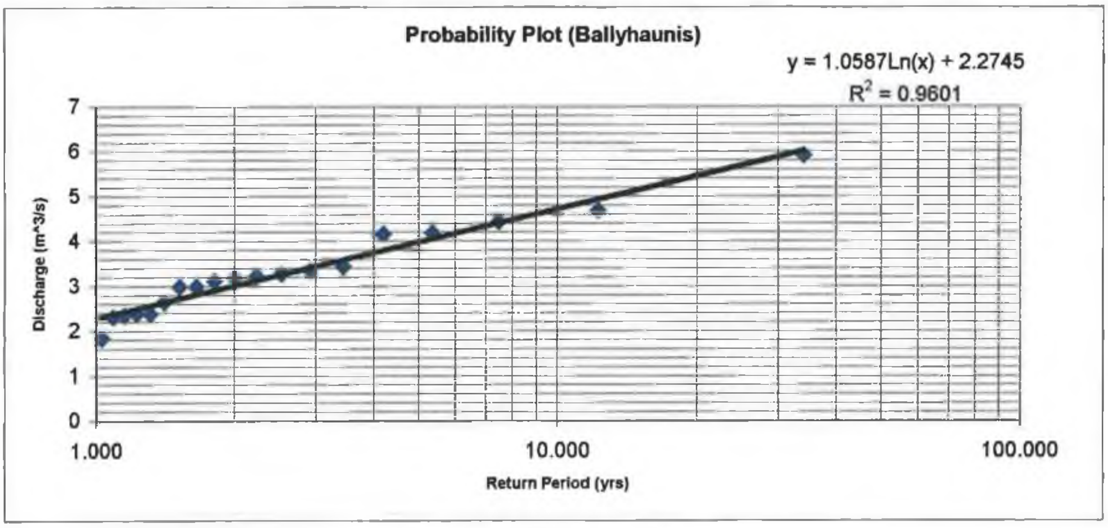
x_{av}	113.81	← average Q
s	21.44	← standard deviation of Q

Return Period T (years)	Exceedance Probability p	Frequency Factor Kt	Discharge Q (m^3/s)
2	0.5	-0.164	110.287
5	0.2	0.719	129.237
10	0.1	1.305	141.784
25	0.04	2.044	157.637
50	0.02	2.592	169.398
100	0.01	3.137	181.072
500	0.002	4.395	208.049
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.2.2)

C–4.1 – Ballyhaunis: EV1 Distribution (Gringorten) Probability Plotting

Discharge Q (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Log Q from recorded data
5.91	1	0.029	34.143	0.772
4.68	2	0.082	12.256	0.670
4.43	3	0.134	7.469	0.646
4.18	4	0.186	5.371	0.621
4.16	5	0.238	4.193	0.619
3.44	6	0.291	3.439	0.537
3.34	7	0.343	2.915	0.524
3.27	8	0.395	2.529	0.515
3.22	9	0.448	2.234	0.508
3.19	10	0.500	2.000	0.504
3.12	11	0.552	1.811	0.494
2.98	12	0.605	1.654	0.474
2.98	13	0.657	1.522	0.474
2.64	14	0.709	1.410	0.422
2.38	15	0.762	1.313	0.377
2.38	16	0.814	1.229	0.377
2.35	17	0.866	1.155	0.371
2.31	18	0.918	1.089	0.364
1.83	19	0.971	1.030	0.262
		(eqn. 4.2.3.1)	(eqn. 4.2.3.2)	Log (Q)

b	0.44	Gringorten Value
n	19	No. of observation/years in series



Return Period T	Discharge from lognormal distribution (m^3/s)
2	3.0
5	4.0
10	4.7
25	5.7
50	6.4
100	7.1
500	8.9

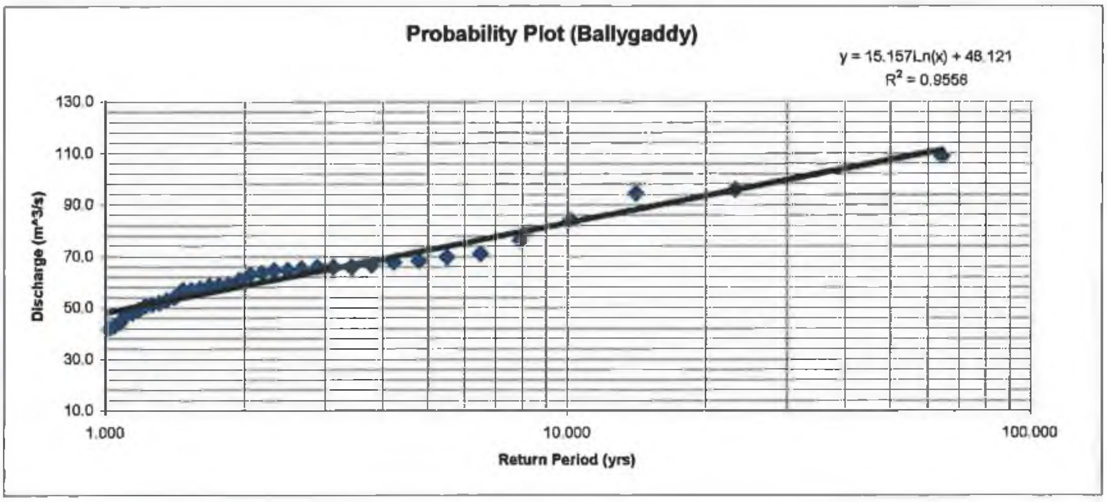
$1.0587\ \mathrm{Ln}(T) + 2.2745$

Eqn. of line from graph

C–4.2 – Ballygaddy: EV1 Distribution (Gringorten) Probability Plotting

Discharge Q (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Log Q from recorded data
108.9	1	0.016	64.500	2.037
96.0	2	0.043	23.154	1.982
94.5	3	0.071	14.109	1.975
84.5	4	0.099	10.146	1.927
76.3	5	0.126	7.921	1.883
71.1	6	0.154	6.496	1.852
69.8	7	0.182	5.506	1.844
68.5	8	0.209	4.778	1.836
67.9	9	0.237	4.220	1.832
66.7	10	0.265	3.778	1.824
66.0	11	0.292	3.420	1.820
66.0	12	0.320	3.125	1.820
66.0	13	0.348	2.876	1.820
65.4	14	0.375	2.664	1.816
64.8	15	0.403	2.481	1.812
64.8	16	0.431	2.321	1.812
63.6	17	0.458	2.181	1.803
63.0	18	0.486	2.057	1.799
61.0	19	0.514	1.946	1.785
59.4	20	0.542	1.847	1.774
58.9	21	0.569	1.757	1.770
58.8	22	0.597	1.675	1.769
57.6	23	0.625	1.601	1.760
57.1	24	0.652	1.533	1.757
57.0	25	0.680	1.471	1.756
54.2	26	0.708	1.413	1.734
53.1	27	0.735	1.360	1.725
52.0	28	0.763	1.311	1.716
51.4	29	0.791	1.265	1.711
50.9	30	0.818	1.222	1.707
49.2	31	0.846	1.182	1.692
48.2	32	0.874	1.144	1.683
47.6	33	0.901	1.109	1.678
44.5	34	0.929	1.076	1.648
43.0	35	0.957	1.045	1.633
42.0	36	0.984	1.016	1.623
		↑ (eqn. 4.2.3.1)	↑ (eqn. 4.2.3.2)	↑ Log (Q)

b	0.44	← Gringorten Value
n	36	← No. of observation/years in series



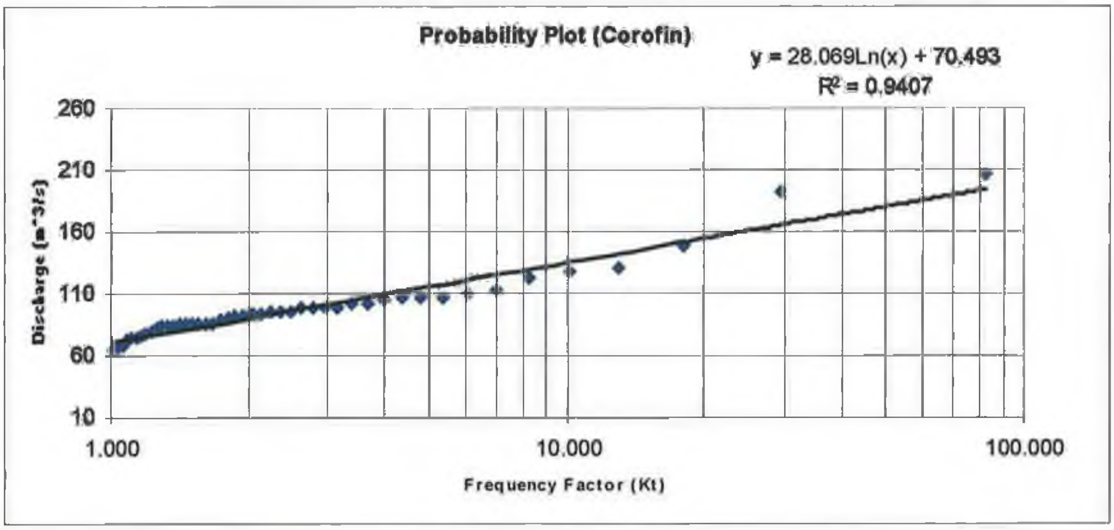
Return Period T (years)	Discharge Q (m^3/s)
2	58.6
5	72.5
10	83.0
25	96.9
50	107.4
100	117.9
500	142.3

15.157 Ln(T) + 48.121
Eqn. of line from graph

C–4.3 – Corofin: EV1 Distribution (Gringorten) Probability Plotting

Discharge Q (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Log Q from recorded data
207	1	0.012	82.357	2.316
193	2	0.034	29.564	2.286
148	3	0.056	18.016	2.170
131	4	0.077	12.955	2.117
128	5	0.099	10.114	2.107
123	6	0.121	8.295	2.090
113	7	0.142	7.030	2.053
110	8	0.164	6.101	2.041
107	9	0.186	5.388	2.029
107	10	0.207	4.824	2.029
106	11	0.229	4.367	2.025
105	12	0.251	3.990	2.021
102	13	0.272	3.672	2.009
102	14	0.294	3.401	2.009
99.5	15	0.316	3.168	1.998
99.5	16	0.337	2.964	1.998
98.5	17	0.359	2.785	1.993
98.2	18	0.381	2.626	1.992
95.3	19	0.402	2.485	1.979
95.3	20	0.424	2.358	1.979
95	21	0.446	2.243	1.978
93.4	22	0.467	2.139	1.970
93.2	23	0.489	2.044	1.969
92.1	24	0.511	1.958	1.964
92.1	25	0.533	1.878	1.964
90.3	26	0.554	1.804	1.956
88.7	27	0.576	1.736	1.948
86.5	28	0.598	1.673	1.937
86	29	0.619	1.615	1.934
86	30	0.641	1.560	1.934
85.5	31	0.663	1.509	1.932
85.5	32	0.684	1.461	1.932
85.5	33	0.706	1.416	1.932
84.5	34	0.728	1.374	1.927
84	35	0.749	1.334	1.924
83.5	36	0.771	1.297	1.922
82.6	37	0.793	1.261	1.917
80.1	38	0.814	1.228	1.904
78.2	39	0.836	1.196	1.893
75.6	40	0.858	1.166	1.879
74.4	41	0.879	1.137	1.872
74.4	42	0.901	1.110	1.872
72.4	43	0.923	1.084	1.860
68	44	0.944	1.059	1.833
66.9	45	0.966	1.035	1.825
65.3	46	0.988	1.012	1.815
		↑ (eqn. 4.2.3.1)	↑ (eqn. 4.2.3.2)	↑ Log (Q)

b	0.44	← Gringorten Value
n	46	← No. of observation/years in series



Return Period T (years)	Discharge Q (m^3/s)
2	89.9
5	115.7
10	135.1
25	160.8
50	180.3
100	199.8
500	244.9

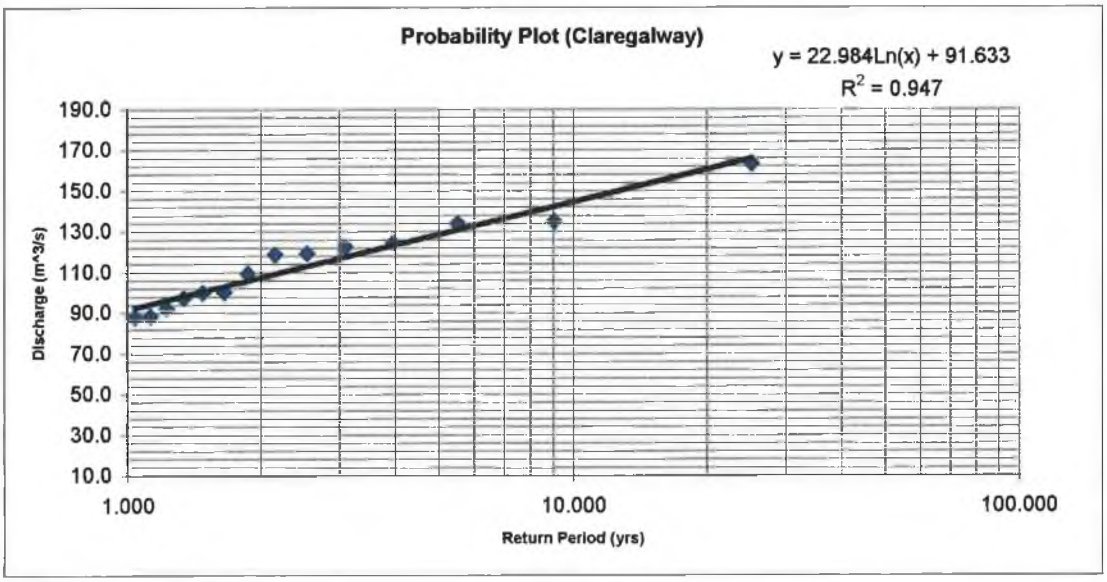
28.069 Ln(T) + 70.493
Eqn. of line from graph

C–4.4 – Claregalway: EV1 Distribution (Gringorten) Probability Plotting

Discharge Q (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Log Q from recorded data
163.2	1	0.040	25.214	2.213
135.1	2	0.110	9.051	2.131
134.0	3	0.181	5.516	2.127
124.5	4	0.252	3.966	2.095
122.5	5	0.323	3.096	2.088
119.4	6	0.394	2.540	2.077
118.9	7	0.465	2.152	2.075
109.3	8	0.535	1.868	2.039
100.2	9	0.606	1.650	2.001
100.1	10	0.677	1.477	2.000
97.3	11	0.748	1.337	1.988
92.5	12	0.819	1.221	1.966
88.5	13	0.890	1.124	1.947
87.9	14	0.960	1.041	1.944

(eqn. 4.2.3.1) (eqn. 4.2.3.2) Log (Q)

b	0.44	← Gringorten Value
n	14	← No. of observation/years in series



Return Period T (years)	Discharge Q (m^3/s)
2	107.6
5	128.6
10	144.6
25	165.6
50	181.5
100	197.5
500	234.5

$22.984\ Ln(T) + 91.633$
Eqn. of line from graph

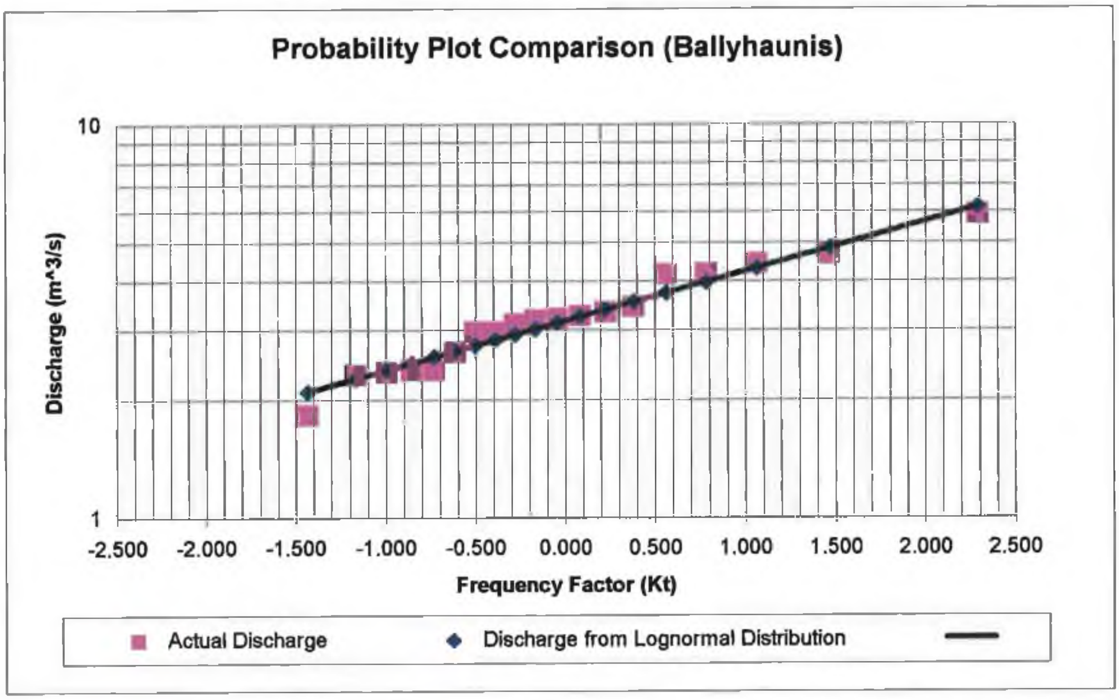
C–5.1 – Ballyhaunis: Comparison of Plotted Data with Lognormal Distribution

Fitted to them by the Frequency Factor Method

Actual Discharge (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Frequency Factor Kt	Log Q from lognormal distribution	Log Q from recorded data	Discharge from lognormal distribution (m^3/s)
5.91	1	0.029	34.143	2.291	0.790	0.772	6.16
4.68	2	0.082	12.256	1.471	0.687	0.670	4.86
4.43	3	0.134	7.469	1.062	0.635	0.646	4.32
4.18	4	0.186	5.371	0.782	0.600	0.621	3.98
4.16	5	0.238	4.193	0.564	0.572	0.619	3.74
3.44	6	0.291	3.439	0.383	0.550	0.537	3.55
3.34	7	0.343	2.915	0.226	0.530	0.524	3.39
3.27	8	0.395	2.529	0.085	0.512	0.515	3.25
3.22	9	0.448	2.234	-0.043	0.496	0.508	3.13
3.19	10	0.500	2.000	-0.164	0.481	0.504	3.03
3.12	11	0.552	1.811	-0.280	0.466	0.494	2.93
2.98	12	0.605	1.654	-0.392	0.452	0.474	2.83
2.98	13	0.657	1.522	-0.503	0.438	0.474	2.74
2.64	14	0.709	1.410	-0.615	0.424	0.422	2.66
2.38	15	0.762	1.313	-0.731	0.410	0.377	2.57
2.38	16	0.814	1.229	-0.855	0.394	0.377	2.48
2.35	17	0.866	1.155	-0.995	0.376	0.371	2.38
2.31	18	0.918	1.089	-1.166	0.355	0.364	2.26
1.83	19	0.971	1.030	-1.434	0.321	0.262	2.10

		(eqn. 4.2.3.1)	(eqn. 4.2.3.2)	(eqn. 4.2.2.1)	(eqn. 4.2.4.1)	Log (Q)	(eqn. 4.2.4.2)

b	0.44	← Gringorten Value
n	19	← No. of observation/years in series
y_{av}	0.50	← average of log Q from recorded data
s_y	0.13	← standard deviation of log Q from recorded data


Probability Plot Comparison (Ballyhaunis)
Discharge (m^3/s)
10
1
-2.500
-2.000
-1.500
-1.000
-0.500
0.000
0.500
1.000
1.500
2.000
2.500
Frequency Factor (Kt)
Actual Discharge
Discharge from Lognormal Distribution

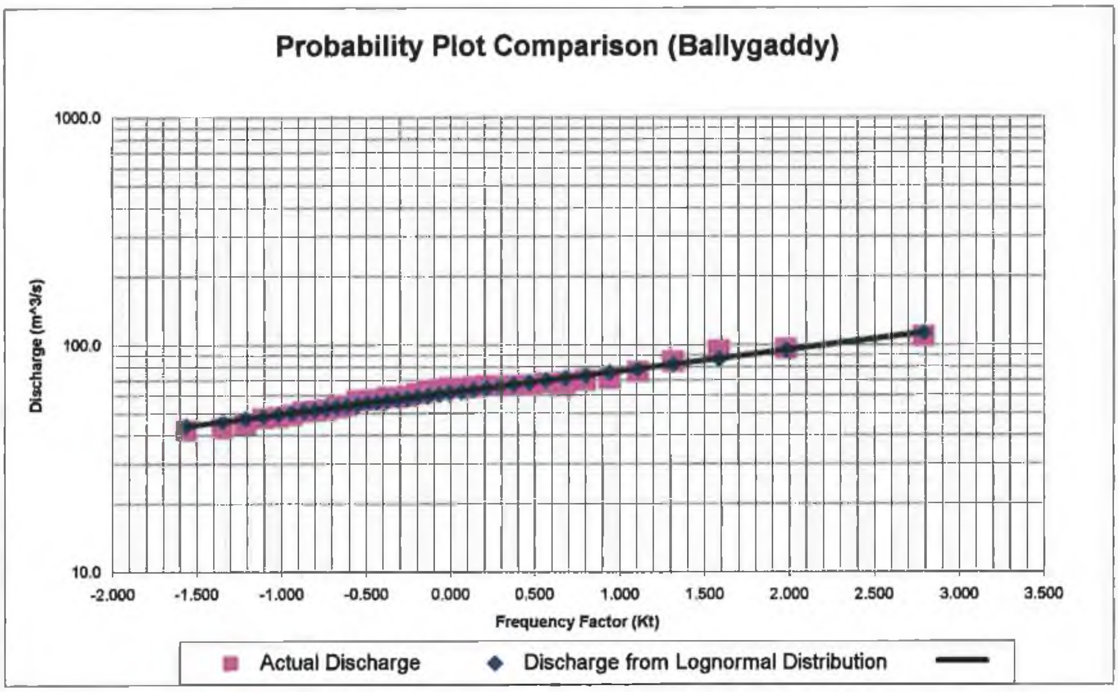
C–5.2 – Ballygaddy: Comparison of Plotted Data with Lognormal Distribution

Fitted to them by the Frequency Factor Method

Actual Discharge (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Frequency Factor Kt	Log Q from lognormal distribution	Log Q from recorded data	Discharge from lognormal distribution (m^3/s)
108.9	1	0.016	64.500	2.793	2.053	2.037	112.98
96.0	2	0.043	23.154	1.983	1.977	1.982	94.74
94.5	3	0.071	14.109	1.585	1.939	1.975	86.89
84.5	4	0.099	10.146	1.316	1.914	1.927	81.96
76.3	5	0.126	7.921	1.112	1.894	1.883	78.39
71.1	6	0.154	6.496	0.945	1.878	1.852	75.59
69.8	7	0.182	5.506	0.803	1.865	1.844	73.30
68.5	8	0.209	4.778	0.680	1.853	1.836	71.36
67.9	9	0.237	4.220	0.569	1.843	1.832	69.67
66.7	10	0.265	3.778	0.470	1.834	1.824	68.17
66.0	11	0.292	3.420	0.378	1.825	1.820	66.83
66.0	12	0.320	3.125	0.293	1.817	1.820	65.60
66.0	13	0.348	2.876	0.213	1.809	1.820	64.47
65.4	14	0.375	2.664	0.138	1.802	1.816	63.42
64.8	15	0.403	2.481	0.066	1.795	1.812	62.44
64.8	16	0.431	2.321	-0.003	1.789	1.812	61.52
63.6	17	0.458	2.181	-0.069	1.783	1.803	60.64
63.0	18	0.486	2.057	-0.133	1.777	1.799	59.80
61.0	19	0.514	1.946	-0.195	1.771	1.785	59.00
59.4	20	0.542	1.847	-0.256	1.765	1.774	58.22
58.9	21	0.569	1.757	-0.316	1.759	1.770	57.47
58.8	22	0.597	1.675	-0.375	1.754	1.769	56.73
57.6	23	0.625	1.601	-0.434	1.748	1.760	56.01
57.1	24	0.652	1.533	-0.493	1.743	1.757	55.30
57.0	25	0.680	1.471	-0.552	1.737	1.756	54.60
54.2	26	0.708	1.413	-0.611	1.732	1.734	53.89
53.1	27	0.735	1.360	-0.672	1.726	1.725	53.19
52.0	28	0.763	1.311	-0.734	1.720	1.716	52.47
51.4	29	0.791	1.265	-0.799	1.714	1.711	51.74
50.9	30	0.818	1.222	-0.866	1.707	1.707	50.98
49.2	31	0.846	1.182	-0.939	1.701	1.692	50.19
48.2	32	0.874	1.144	-1.017	1.693	1.683	49.34
47.6	33	0.901	1.109	-1.105	1.685	1.678	48.41
44.5	34	0.929	1.076	-1.209	1.675	1.648	47.33
43.0	35	0.957	1.045	-1.343	1.662	1.633	45.97
42.0	36	0.984	1.016	-1.563	1.642	1.623	43.82

	↑ (eqn. 4.2.3.1)	↑ (eqn. 4.2.3.2)	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.4.1)	↑ Log (Q)	↑ (eqn. 4.2.4.2)

b	0.44	←	Gringorten Value
n	36	←	No. of observation/years in series
y_{av}	1.79	←	average of log Q from recorded data
s_v	0.09	←	standard deviation of log Q from recorded data



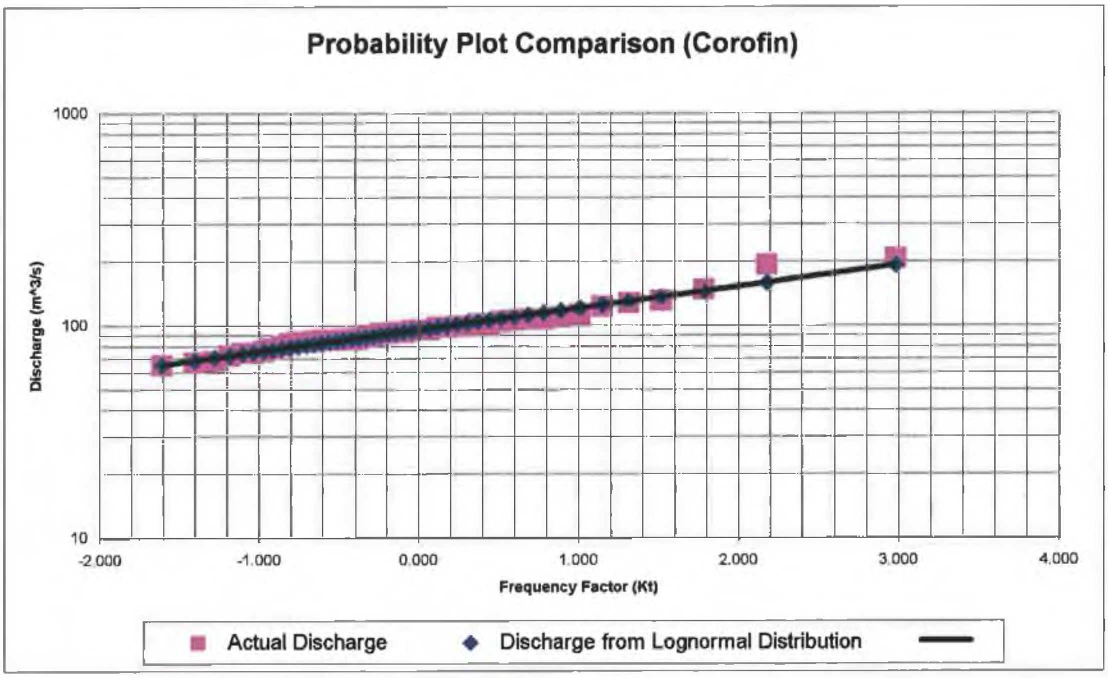
Probability Plot Comparison (Ballygaddy)
1000.0
100.0
10.0
Discharge (m^3/s)
-2.000
-1.500
-1.000
-0.500
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
Frequency Factor (Kt)
Actual Discharge
Discharge from Lognormal Distribution

C–5.3 – Corofin: Comparison of Plotted Data with Lognormal Distribution Fitted to them by Frequency Factor

b	0.44	←	Gringorten Value
n	46	←	No. of observation/years in series
y_{av}	1.98	←	average of log Q from recorded data
s_y	0.10	←	standard deviation of log Q from recorded data

Actual Discharge (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Frequency Factor Kt	Log Q from lognormal distribution	Log Q from recorded data	Discharge from lognormal distribution (m^3/s)
207	1	0.012	82.357	2.984	2.285	2.316	192.74
193	2	0.034	29.564	2.177	2.202	2.286	159.29
148	3	0.056	18.016	1.782	2.162	2.170	145.11
131	4	0.077	12.955	1.516	2.134	2.117	136.28
128	5	0.099	10.114	1.314	2.114	2.107	129.93
123	6	0.121	8.295	1.150	2.097	2.090	125.00
113	7	0.142	7.030	1.012	2.083	2.053	120.98
110	8	0.164	6.101	0.891	2.070	2.041	117.59
107	9	0.186	5.388	0.784	2.059	2.029	114.67
107	10	0.207	4.824	0.688	2.050	2.029	112.09
106	11	0.229	4.367	0.600	2.041	2.025	109.79
105	12	0.251	3.990	0.519	2.032	2.021	107.70
102	13	0.272	3.672	0.443	2.024	2.009	105.80
102	14	0.294	3.401	0.373	2.017	2.009	104.04
99.5	15	0.316	3.168	0.306	2.010	1.998	102.41
99.5	16	0.337	2.964	0.242	2.004	1.998	100.89
98.5	17	0.359	2.785	0.182	1.998	1.993	99.46
98.2	18	0.381	2.626	0.123	1.992	1.992	98.10
95.3	19	0.402	2.485	0.068	1.986	1.979	96.82
95.3	20	0.424	2.358	0.013	1.980	1.979	95.59
95	21	0.446	2.243	-0.039	1.975	1.978	94.41
93.4	22	0.467	2.139	-0.090	1.970	1.970	93.28
93.2	23	0.489	2.044	-0.140	1.965	1.969	92.19
92.1	24	0.511	1.958	-0.189	1.960	1.964	91.14
92.1	25	0.533	1.878	-0.236	1.955	1.964	90.11
90.3	26	0.554	1.804	-0.284	1.950	1.956	89.11
88.7	27	0.576	1.736	-0.330	1.945	1.948	88.13
86.5	28	0.598	1.673	-0.377	1.940	1.937	87.18
86	29	0.619	1.615	-0.423	1.936	1.934	86.23
86	30	0.641	1.560	-0.469	1.931	1.934	85.30
85.5	31	0.663	1.509	-0.515	1.926	1.932	84.38
85.5	32	0.684	1.461	-0.561	1.922	1.932	83.47
85.5	33	0.706	1.416	-0.608	1.917	1.932	82.55
84.5	34	0.728	1.374	-0.655	1.912	1.927	81.63
84	35	0.749	1.334	-0.703	1.907	1.924	80.71
83.5	36	0.771	1.297	-0.753	1.902	1.922	79.77

82.6	37	0.793	1.261	-0.804	1.897	1.917	78.82
80.1	38	0.814	1.228	-0.856	1.891	1.904	77.84
78.2	39	0.836	1.196	-0.912	1.886	1.893	76.83
75.6	40	0.858	1.166	-0.971	1.879	1.879	75.77
74.4	41	0.879	1.137	-1.034	1.873	1.872	74.64
74.4	42	0.901	1.110	-1.104	1.866	1.872	73.42
72.4	43	0.923	1.084	-1.183	1.858	1.860	72.06
68	44	0.944	1.059	-1.278	1.848	1.833	70.47
66.9	45	0.966	1.035	-1.401	1.835	1.825	68.45
65.3	46	0.988	1.012	-1.607	1.814	1.815	65.20
		↑ (eqn. 4.2.3.1)	↑ (eqn. 4.2.3.2)	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.4.1)	↑ Log (Q)	↑ (eqn. 4.2.4.2)

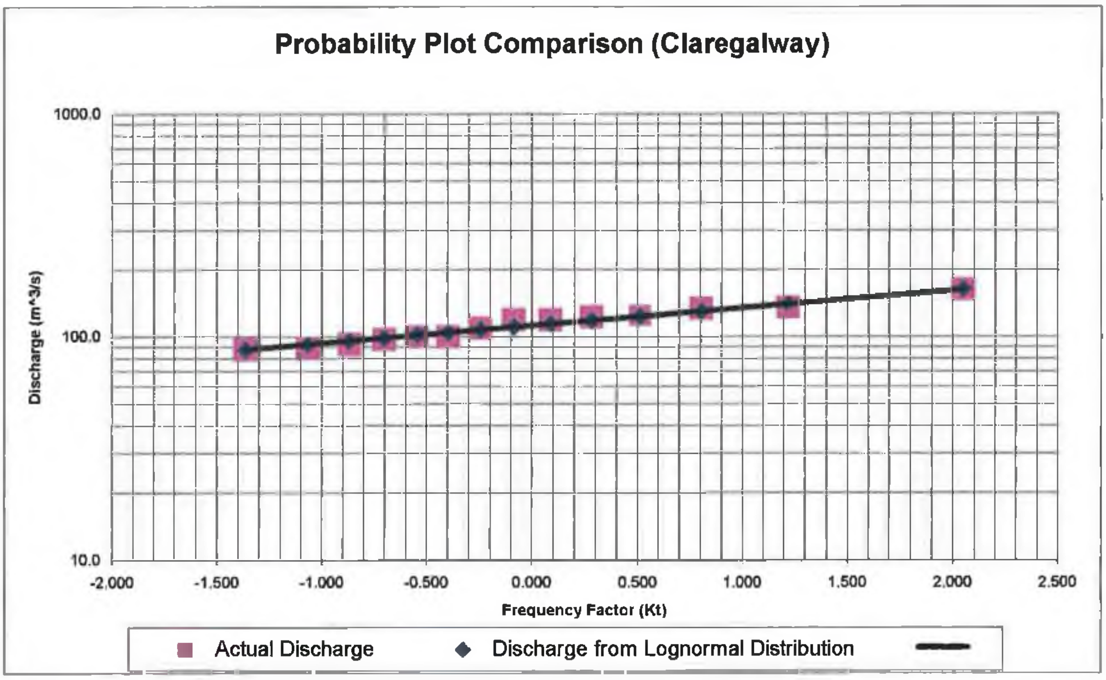


C–5.4 – Claregalway: Comparison of Plotted Data with Lognormal Distribution

Fitted to them by the Frequency Factor Method

Actual Discharge (m^3/s)	Rank m	Exceedance Probability p	Return Period T	Frequency Factor Kt	Log Q from lognormal distribution	Log Q from recorded data	Discharge from lognormal distribution (m^3/s)
163.2	1	0.040	25.214	2.051	2.212	2.213	162.82
135.1	2	0.110	9.051	1.222	2.146	2.131	140.00
134.0	3	0.181	5.516	0.805	2.113	2.127	129.73
124.5	4	0.252	3.966	0.514	2.090	2.095	123.03
122.5	5	0.323	3.096	0.284	2.072	2.088	117.99
119.4	6	0.394	2.540	0.090	2.056	2.077	113.88
118.9	7	0.465	2.152	-0.083	2.043	2.075	110.35
109.3	8	0.535	1.868	-0.243	2.030	2.039	107.18
100.2	9	0.606	1.650	-0.395	2.018	2.001	104.25
100.1	10	0.677	1.477	-0.546	2.006	2.000	101.43
97.3	11	0.748	1.337	-0.700	1.994	1.988	98.61
92.5	12	0.819	1.221	-0.867	1.981	1.966	95.65
88.5	13	0.890	1.124	-1.066	1.965	1.947	92.25
87.9	14	0.960	1.041	-1.364	1.941	1.944	87.37
		(eqn. 4.2.3.1)	(eqn. 4.2.3.2)	(eqn. 4.2.2.1)	(eqn. 4.2.4.1)	Log (Q)	(eqn. 4.2.4.2)

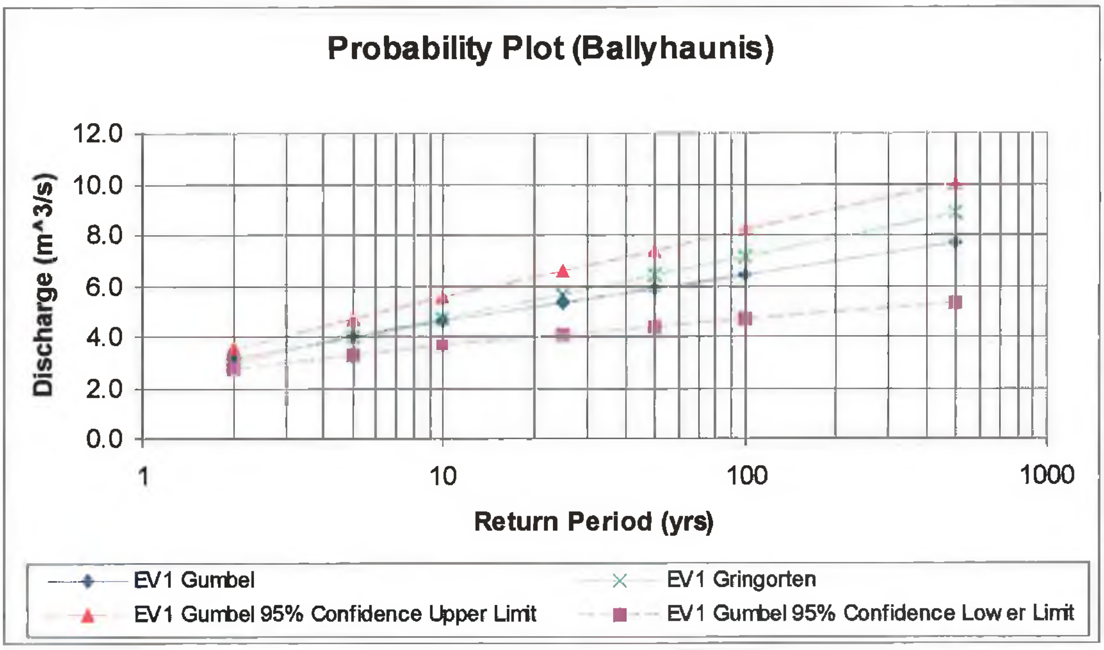
b	0.44	← Gringorten Value
n	14	← No. of observation/years in series
y_{av}	2.05	← average of log Q from recorded data
s_y	0.08	← standard deviation of log Q from recorded data



C–6.1 – Ballyhaunis: Standard Error and Confidence Limits

x_{av}	3.30	average Q
s	1.00	standard deviation Q
n	19	No. of observation/years in series
β	0.95	confidence level
α	0.025	significance level (eqn. 4.2.5.1)
z_α	1.96	standard normal variable with exceedance probability α

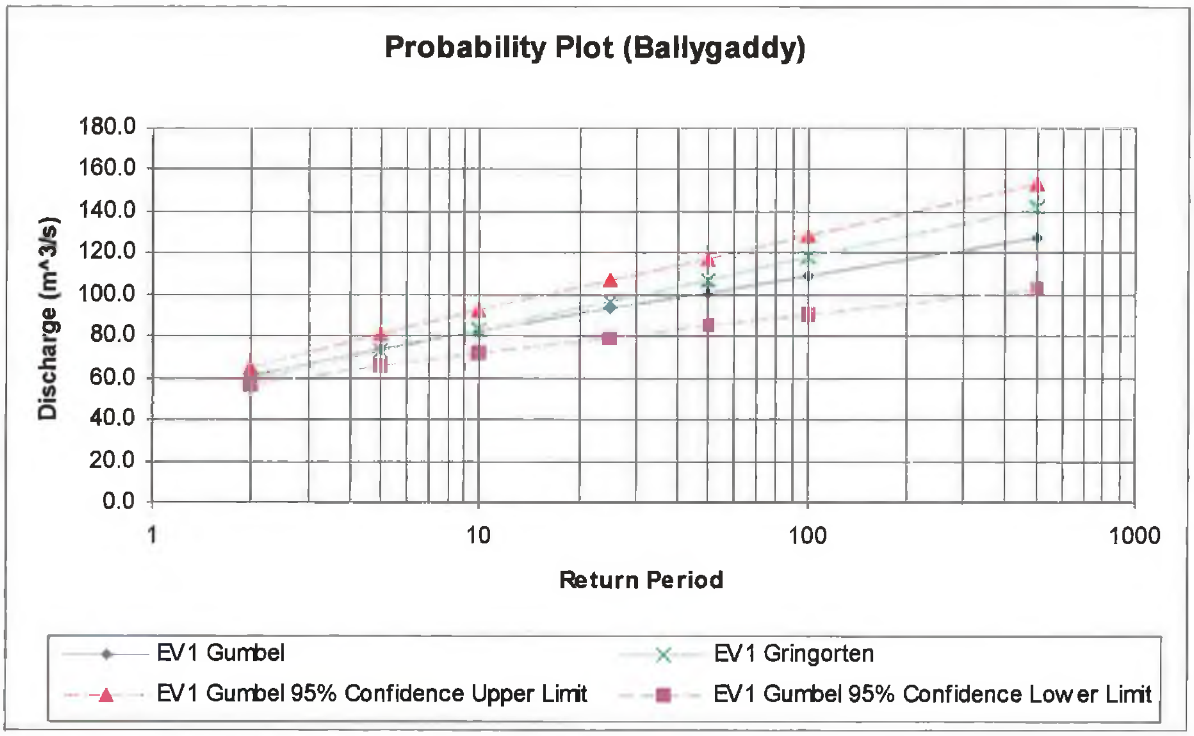
Return Period T	Exceedance Probability p	Frequency Factor Kt	Standard Error (m^3/s)	Confidence Limits $s_e z_\alpha$ (m^3/s)	Discharge (Q) from EV1 distribution (m^3/s)	Confidence Interval (m^3/s)
2	0.5	-0.164	0.2	0.4	3.1	3.1 +/- 0.4
5	0.2	0.719	0.4	0.7	4.0	4.0 +/- 0.7
10	0.1	1.305	0.5	0.9	4.6	4.6 +/- 0.9
25	0.04	2.044	0.6	1.3	5.3	5.3 +/- 1.3
50	0.02	2.592	0.8	1.5	5.9	5.9 +/- 1.5
100	0.01	3.137	0.9	1.8	6.4	6.4 +/- 1.8
500	0.002	4.395	1.2	2.3	7.7	7.7 +/- 2.3
	1/T	(eqn. 4.2.2.1)	(eqn. 4.2.5.2)	(eqn. 4.2.5.3)	x_T	(eqn. 4.2.5.3)



C–6.2 – Ballygaddy: Standard Error and Confidence Limits

x_{av}	63.05	← average Q
s	14.74	← standard deviation Q
n	36	← No. of observation/years in series
β	0.95	← confidence level
α	0.025	← significance level (eqn. 4.2.5.1)
z_α	1.96	← standard normal variable with exceedance probability α

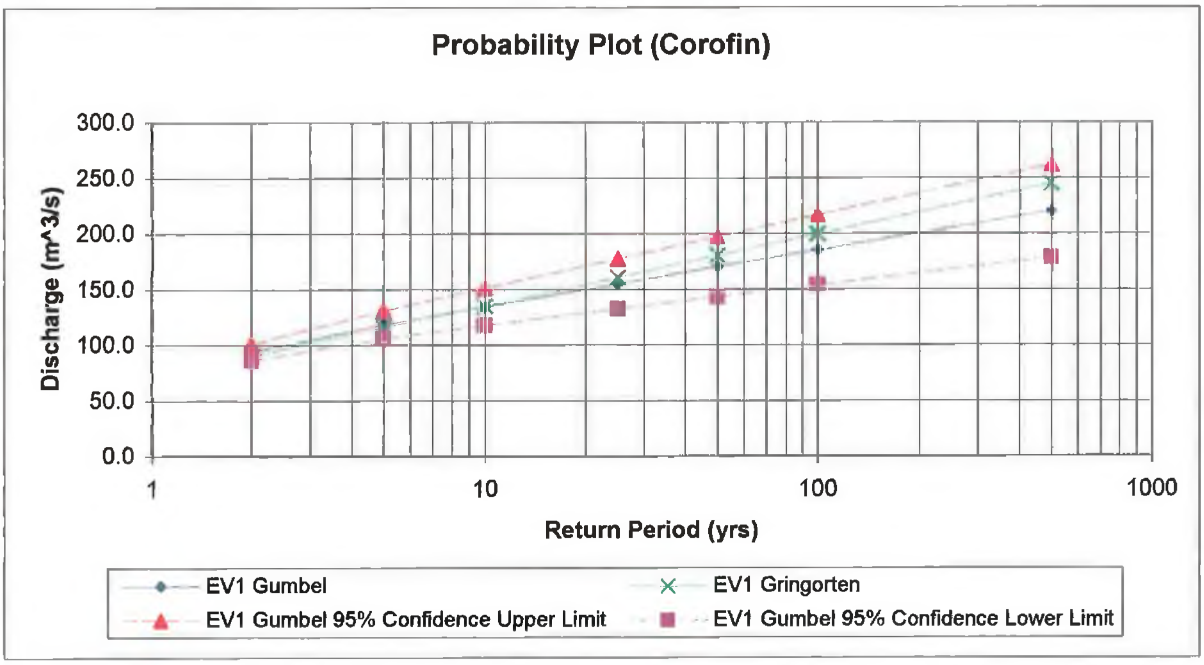
Return Period T	Exceedance Probability p	Frequency Factor Kt	Standard Error (m^3/s)	Confidence Limits $s_e z_\alpha$ (m^3/s)	Discharge (Q) from EV1 distribution (m^3/s)	Confidence Interval (m^3/s)
2	0.5	-0.164	2.3	4.4	60.6	60.6 +/- 4.4
5	0.2	0.719	3.8	7.4	73.7	73.7 +/- 7.4
10	0.1	1.305	5.1	10.1	82.3	82.3 +/- 10.1
25	0.04	2.044	6.9	13.6	93.2	93.2 +/- 13.6
50	0.02	2.592	8.3	16.2	101.3	101.3 +/- 16.2
100	0.01	3.137	9.6	18.9	109.3	109.3 +/- 18.9
500	0.002	4.395	12.8	25.1	127.8	127.8 +/- 25.1
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.5.2)	↑ (eqn. 4.2.5.3)	↑ x_T	↑ (eqn. 4.2.5.3)



C–6.3 – Corofin: Standard Error and Confidence Limits

x_{av}	98.22	average Q
s	27.74	standard deviation Q
n	46	No. of observation/years in series
β	0.95	confidence level
α	0.025	significance level (eqn. 4.2.5.1)
z_α	1.96	standard normal variable with exceedance probability α

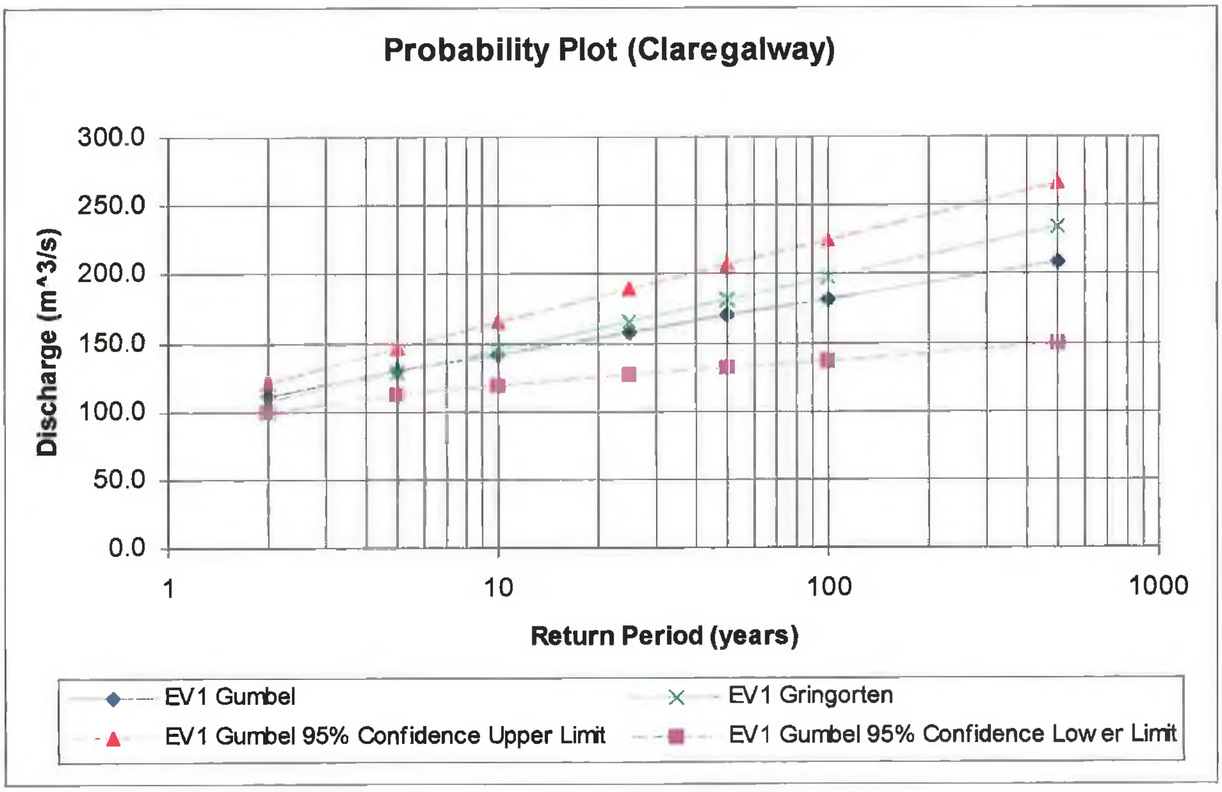
Return Period T	Exceedance Probability p	Frequency Factor Kt	Standard Error (m^3/s)	Confidence Limits $s_e z_\alpha$ (m^3/s)	Discharge (Q) from EV1 distribution (m^3/s)	Confidence Interval (m^3/s)
2	0.5	-0.164	3.8	7.4	93.7	93.7 +/- 7.4
5	0.2	0.719	6.3	12.4	118.2	118.2 +/- 12.4
10	0.1	1.305	8.5	16.7	134.4	134.4 +/- 16.7
25	0.04	2.044	11.5	22.6	154.9	154.9 +/- 22.6
50	0.02	2.592	13.8	27.0	170.1	170.1 +/- 27.0
100	0.01	3.137	16.0	31.5	185.2	185.2 +/- 31.5
500	0.002	4.395	21.3	41.8	220.1	220.1 +/- 41.8
	1/T	(eqn. 4.2.2.1)	(eqn. 4.2.5.2)	(eqn. 4.2.5.3)	x_T	(eqn. 4.2.5.3)



C–6.4 – Claregalway: Standard Error and Confidence Limits

x_{av}	113.81	← average Q
s	21.44	← standard deviation Q
n	14	← No. of observation/years in series
β	0.95	← confidence level
α	0.025	← significance level (eqn. 4.2.5.1)
z_α	1.96	← standard normal variable with exceedance probability α

Return Period T	Exceedance Probability p	Frequency Factor Kt	Standard Error (m^3/s)	Confidence Limits $s_e z_\alpha$ (m^3/s)	Discharge (Q) from EV1 distribution (m^3/s)	Confidence Interval (m^3/s)
2	0.5	-0.164	5.3	10.3	110.3	110.3 +/- 10.3
5	0.2	0.719	8.9	17.4	129.2	129.2 +/- 17.4
10	0.1	1.305	12.0	23.5	141.8	141.8 +/- 23.5
25	0.04	2.044	16.1	31.6	157.6	157.6 +/- 31.6
50	0.02	2.592	19.3	37.8	169.4	169.4 +/- 37.8
100	0.01	3.137	22.5	44.1	181.1	181.1 +/- 44.1
500	0.002	4.395	29.9	58.6	208.0	208.0 +/- 58.6
	↑ 1/T	↑ (eqn. 4.2.2.1)	↑ (eqn. 4.2.5.2)	↑ (eqn. 4.2.5.3)	↑ x_T	↑ (eqn. 4.2.5.3)



C–7.1 – Ballyhaunis: Statistical Analysis of Historical Flood Events using Frequency Factor

x_{av}	3.30
s	1.00

← average Q
← standard deviation of Q

Event	Discharge Q (m^3/s)	Frequency Factor Kt	Return Period T (years)
Nov.-1968	-	-	-
Feb.-1990	-	-	-
Winter 1990-91	-	-	-
Dec.-1999	4.7	1.377	11
Jan.-2005	3.0	-0.325	2
Dec.-2006	4.2	0.856	6
Nov.-2009	5.9	2.608	51

(eqn. 4.2.2.2) (eqn. 4.2.2.3)

C–7.2 – Ballygaddy: Statistical Analysis of Historical Flood Events using Frequency Factor

x_{av}	63.05
s	14.74

← average Q
← standard deviation of Q

Event	Discharge Q (m^3/s)	Frequency Factor Kt	Return Period T (years)
Nov.-1968	-	-	-
Feb.-1990	96.0	2.235	32
Winter 1990-91	64.6	0.105	3
Dec.-1999	94.5	2.134	28
Jan.-2005	58.9	-0.281	2
Dec.-2006	84.5	1.455	12
Nov.-2009	108.9	3.111	97

(eqn. 4.2.2.2) (eqn. 4.2.2.3)

C–7.3 – Corofin: Statistical Analysis of Historical Flood Events using Frequency Factor

x_{av}	98.22
s	27.74

← average Q

← standard deviation of Q

Event	Discharge Q (m^3/s)	Frequency Factor Kt	Return Period T (years)
Nov.-1968	207.0	3.922	273
Feb.-1990	123.0	0.894	6
Winter 1990-91	98.5	0.010	2
Dec.-1999	131.0	1.182	9
Jan.-2005	110.0	0.425	4
Dec.-2006	148.0	1.795	18
Nov.-2009	193.0	3.417	143

(eqn. 4.2.2.2) (eqn. 4.2.2.3)

C–7.4 – Claregalway: Statistical Analysis of Historical Flood Events using Frequency Factor

x_{av}	113.81
s	21.44

← average Q

← standard deviation of Q

Event	Discharge Q (m^3/s)	Frequency Factor Kt	Return Period T (years)
Nov.-1968	-	-	-
Feb.-1990	-	-	-
Winter 1990-91	-	-	-
Dec.-1999	134.0	0.940	6
Jan.-2005	122.5	0.404	4
Dec.-2006	135.1	0.993	7
Nov.-2009	163.2	2.303	35

(eqn. 4.2.2.2) (eqn. 4.2.2.3)

C–8.1 – Ballygaddy: Statistical Analysis of Return Period Flows for 1st and 2nd Half of Annual Maxima Data Series

1974-1991

s	11.46	←	standard deviation of Q
x_{av}	64.54	←	average Q
α	8.93	←	scale parameter (eqn. 4.2.1.3)
u	59.38	←	location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	62.66
5	0.20	1.50	72.78
10	0.10	2.25	79.48
25	0.04	3.20	87.95
50	0.02	3.90	94.24
100	0.01	4.60	100.47
500	0.002	6.21	114.88
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

1992-2009

s	17.65	←	standard deviation of Q
x_{av}	61.56	←	average Q
α	13.76	←	scale parameter (eqn. 4.2.1.3)
u	53.61	←	location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	58.66
5	0.20	1.50	74.25
10	0.10	2.25	84.58
25	0.04	3.20	97.62
50	0.02	3.90	107.30
100	0.01	4.60	116.91
500	0.002	6.21	139.11
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

C–8.2 – Corofin: Statistical Analysis of Return Period Flows for 1st and 2nd Half of Annual Maxima Data Series

1964-1986

s	27.95	←	standard deviation of Q
x_{av}	92.35	←	average Q
α	21.79	←	scale parameter (eqn. 4.2.1.3)
u	79.77	←	location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	87.76
5	0.20	1.50	112.46
10	0.10	2.25	128.82
25	0.04	3.20	149.48
50	0.02	3.90	164.81
100	0.01	4.60	180.02
500	0.002	6.21	215.19
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

1987-2009

s	26.84	←	standard deviation of Q
x_{av}	104.08	←	average Q
α	20.93	←	scale parameter (eqn. 4.2.1.3)
u	92.00	←	location parameter (eqn. 4.2.1.4)

Return Period (T) (years)	Exceedence Probability	Reduced Variate y	Variate x (m^3/s)
2	0.50	0.37	99.67
5	0.20	1.50	123.39
10	0.10	2.25	139.10
25	0.04	3.20	158.94
50	0.02	3.90	173.66
100	0.01	4.60	188.27
500	0.002	6.21	222.03
	↑ 1/T	↑ (eqn. 4.2.1.15)	↑ (eqn. 4.2.1.16)

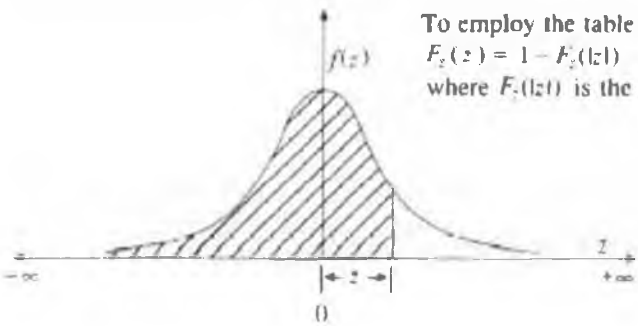
Appendix D

Tables and Graphs:

1. Cumulative Probability of the Standard Normal Distribution
2. 2-day M5 Rainfall
3. Ratio r = 60-minute M5 / 2-day M5 (%)
4. Effective Mean Soil Moisture Deficit
5. Soil Classification for Runoff Potential

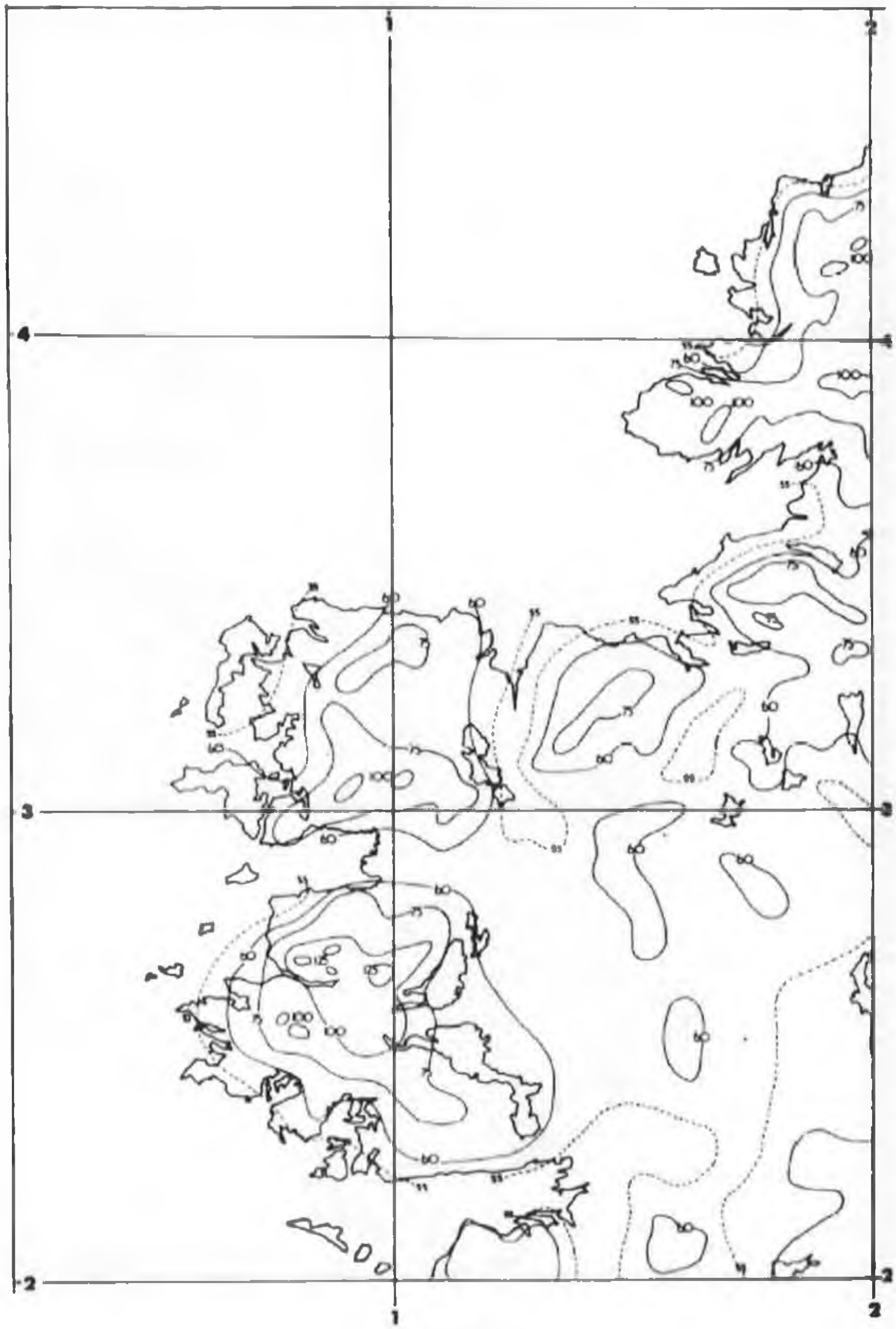
z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998

Source: Grant, E. L., and R. S. Leavenworth, *Statistical Quality and Control*, Table A, p.643, McGraw-Hill, New York, 1972. Used with permission.



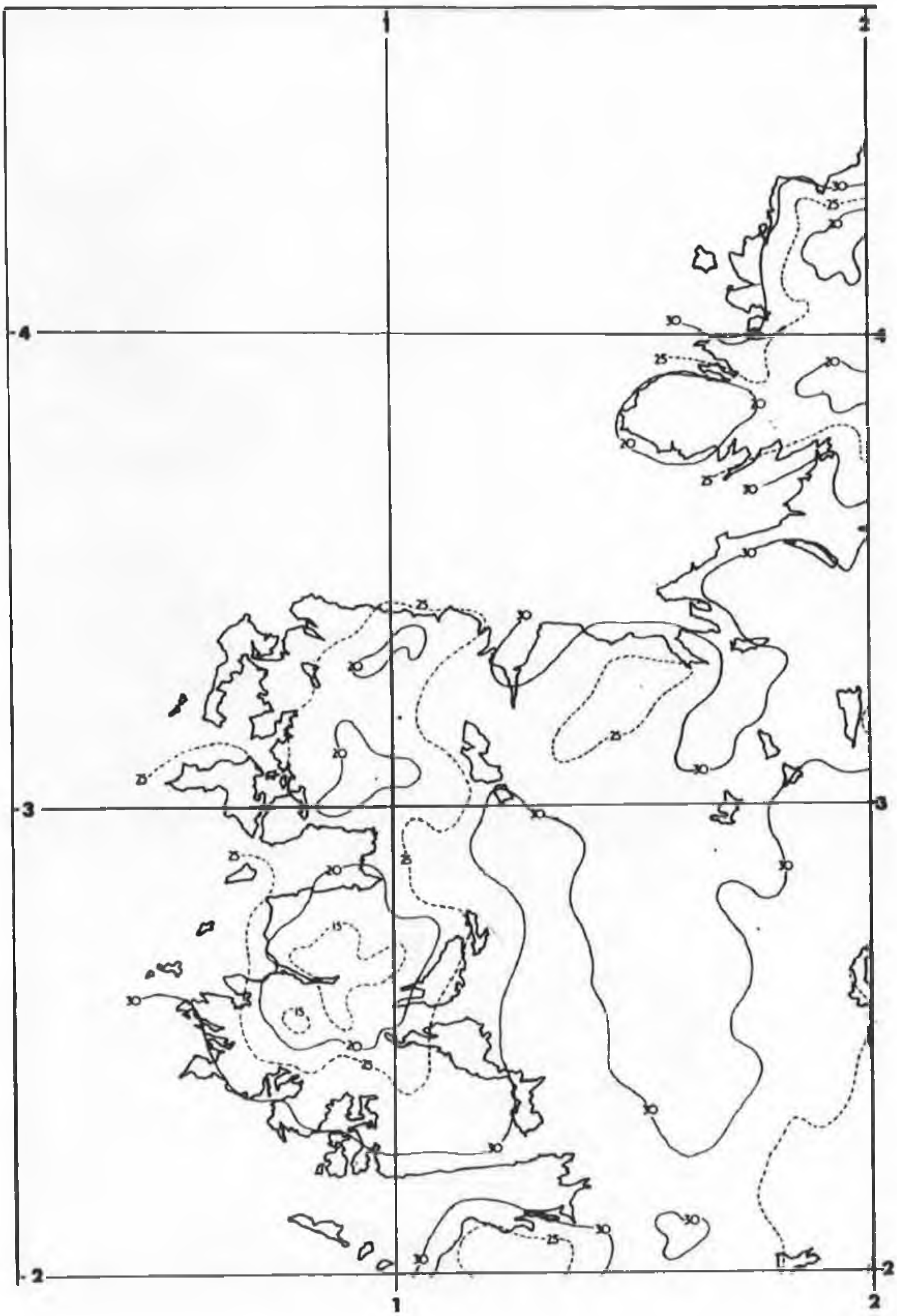
To employ the table for $z < 0$, use
$F_z(z) = 1 - F_z(|z|)$
where $F_z(|z|)$ is the tabulated value.

D–1 – Cumulative Probability of the Standard Normal Distribution



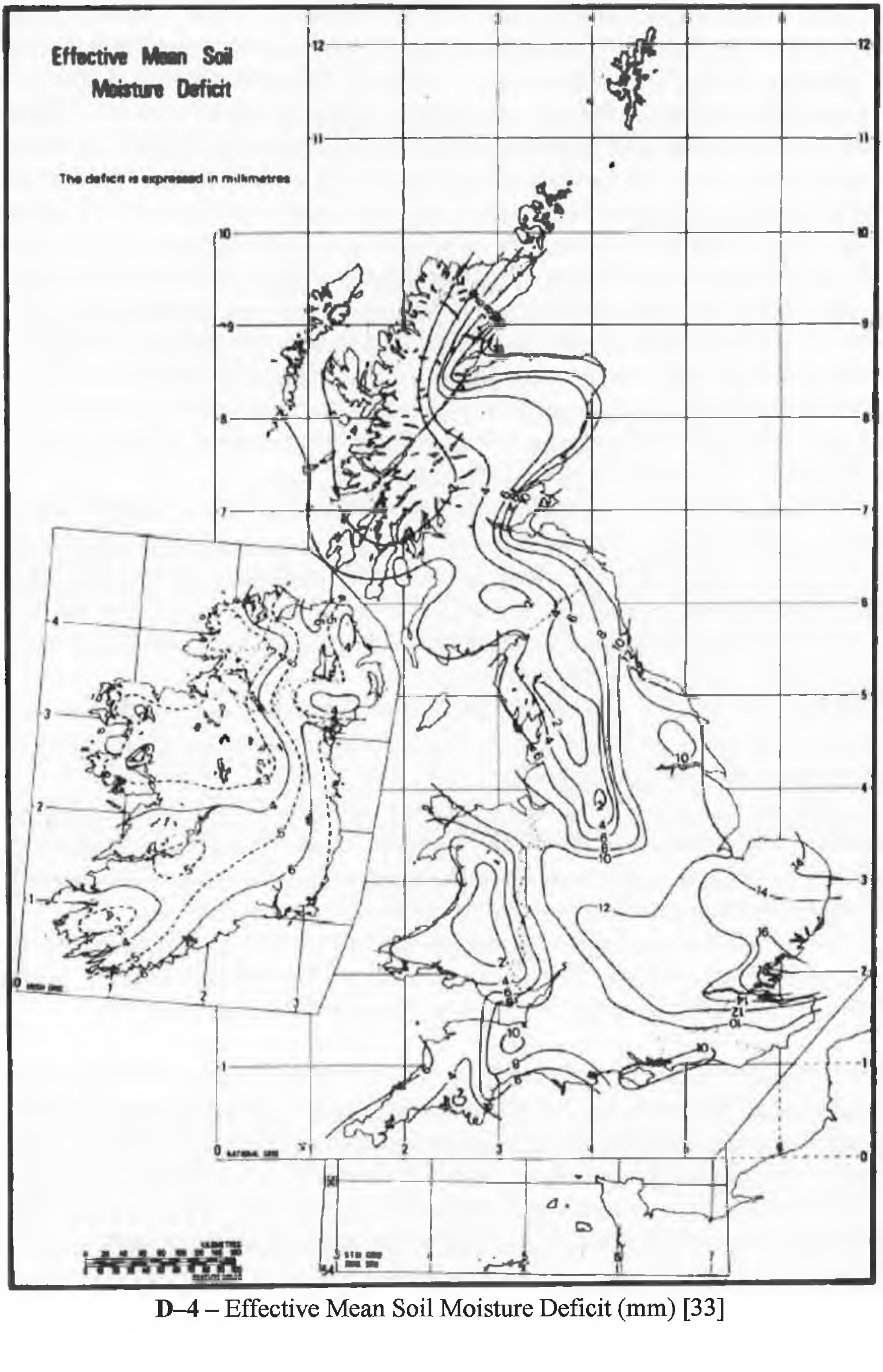
I 2DM5.3

D–2 – 2-day M5 Rainfall [33]

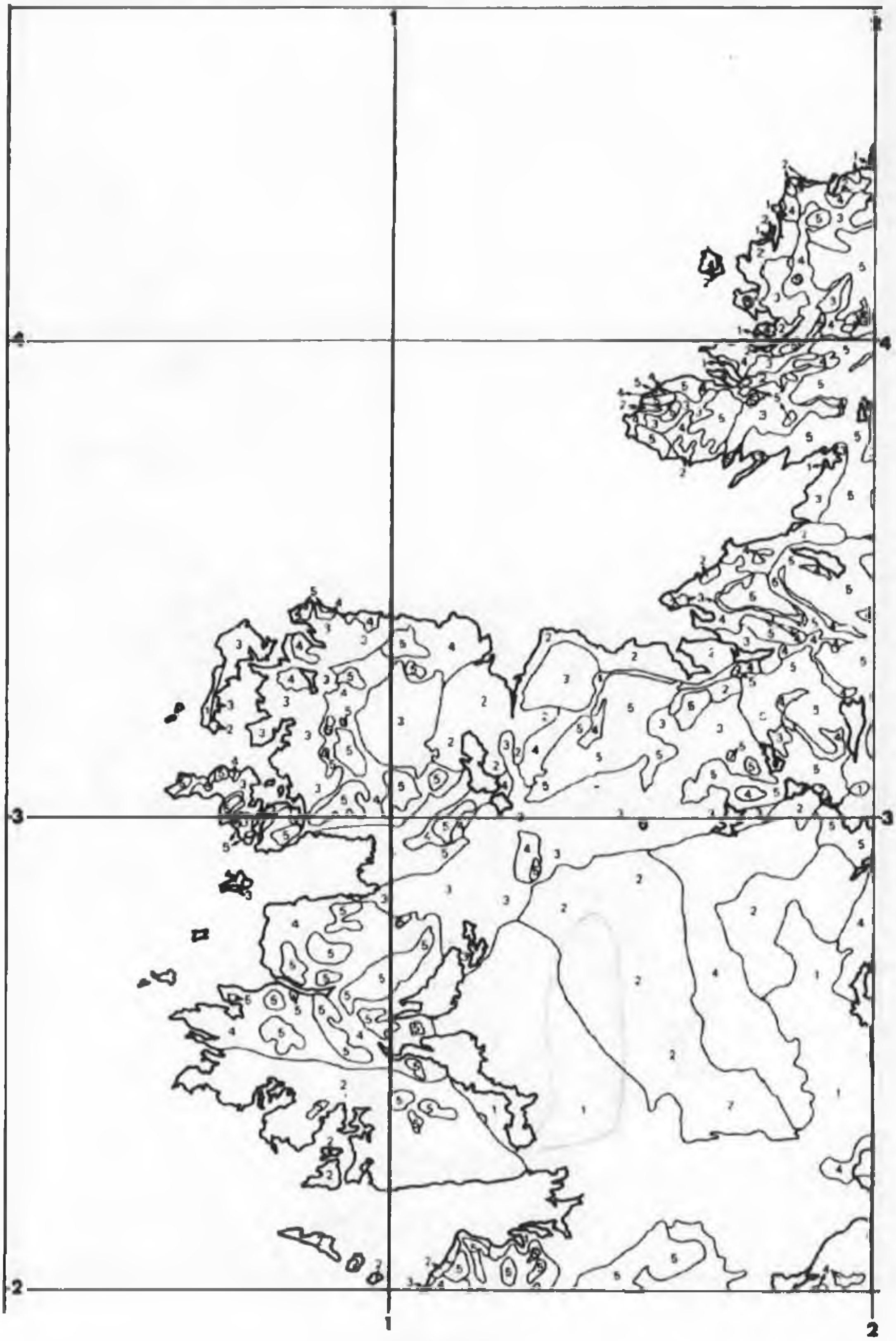


I r.3

D–3 – ratio r = 60-minute M5 / 2-day M5 (%) [33]



D–4 – Effective Mean Soil Moisture Deficit (mm) [33]



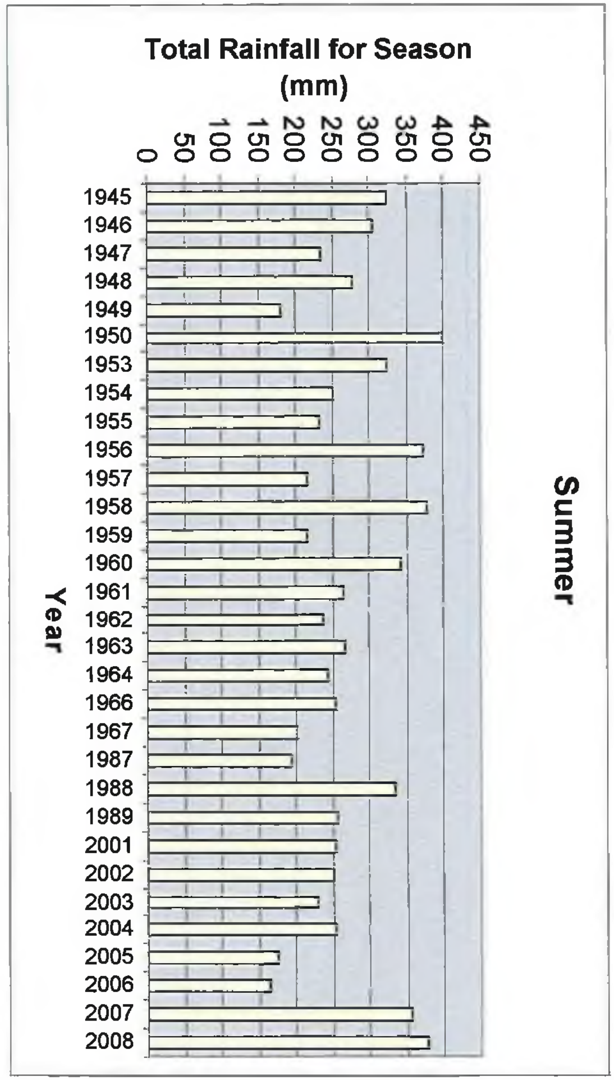
I RP.3

D–5 – Soil Classification for Runoff Potential [33]

Appendix E

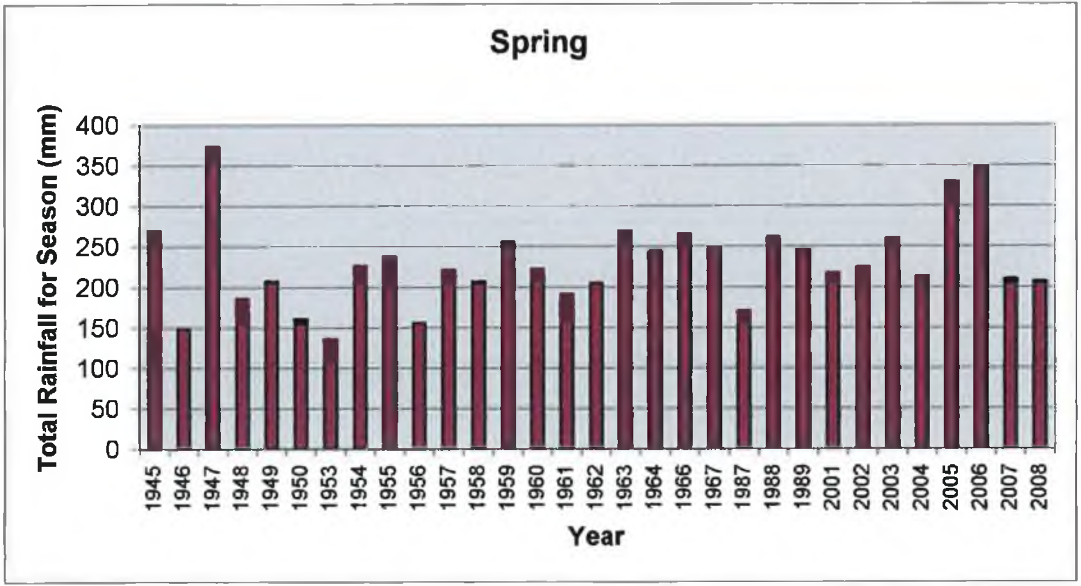
Rainfall Data:

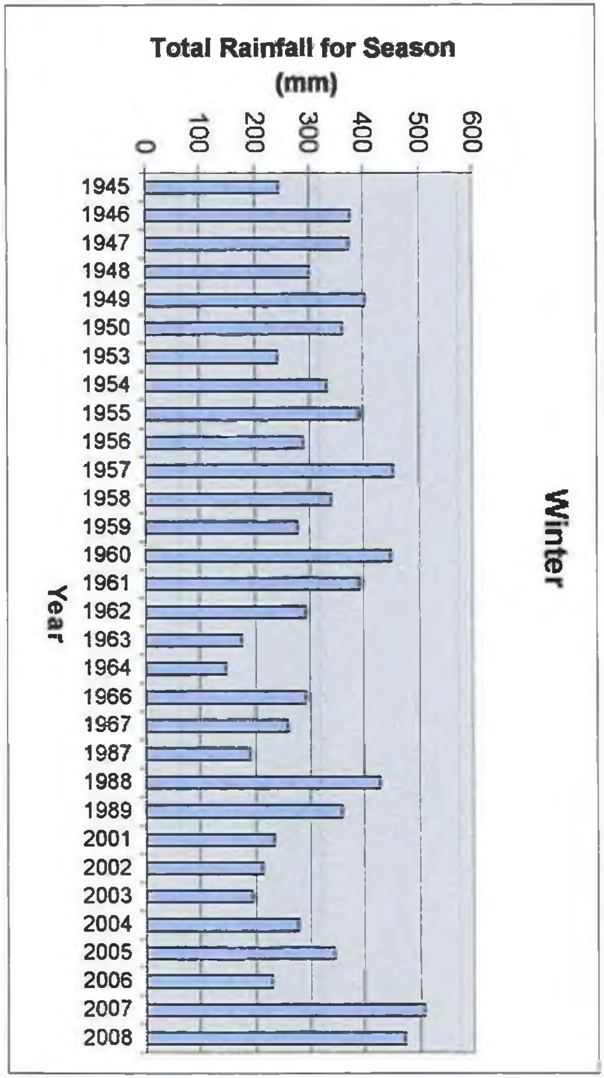
1. Seasonal Rainfall Totals
2. Maximum 3, 5 and 10-day Rainfall Totals



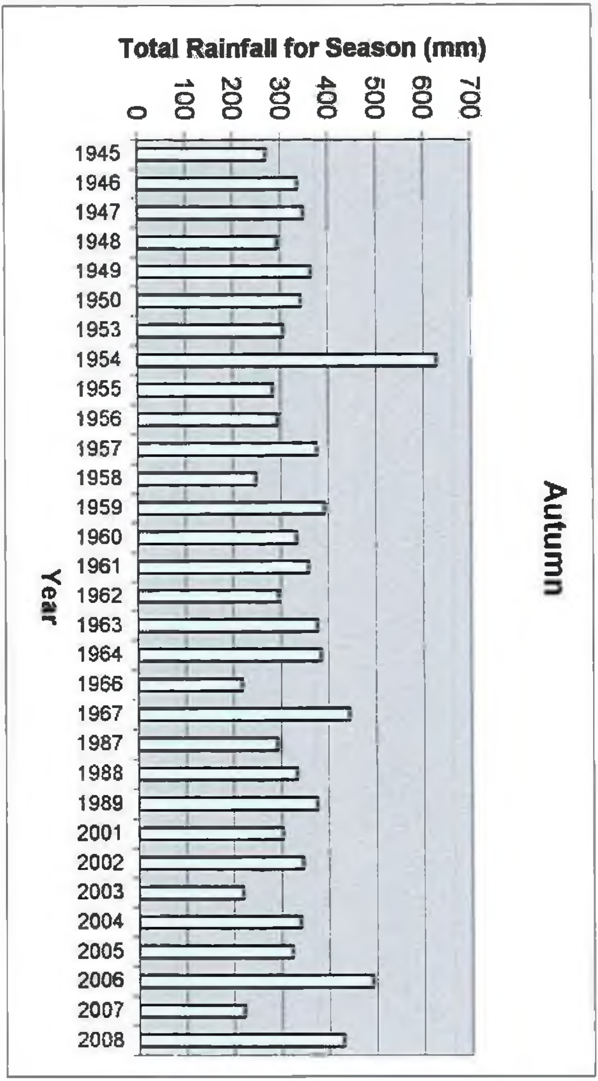
Total Rainfall for Season (mm)
0
50
100
150
200
250
300
350
400
450
1945
1946
1947
1948
1949
1950
1953
1954
1955
1956
1957
1958
1959
1960
1961
1962
1963
1964
1966
1967
1987
1988
1989
2001
2002
2003
2004
2005
2006
2007
2008
Year
Summer

E–1.1 – Milltown: Seasonal Rainfall Totals

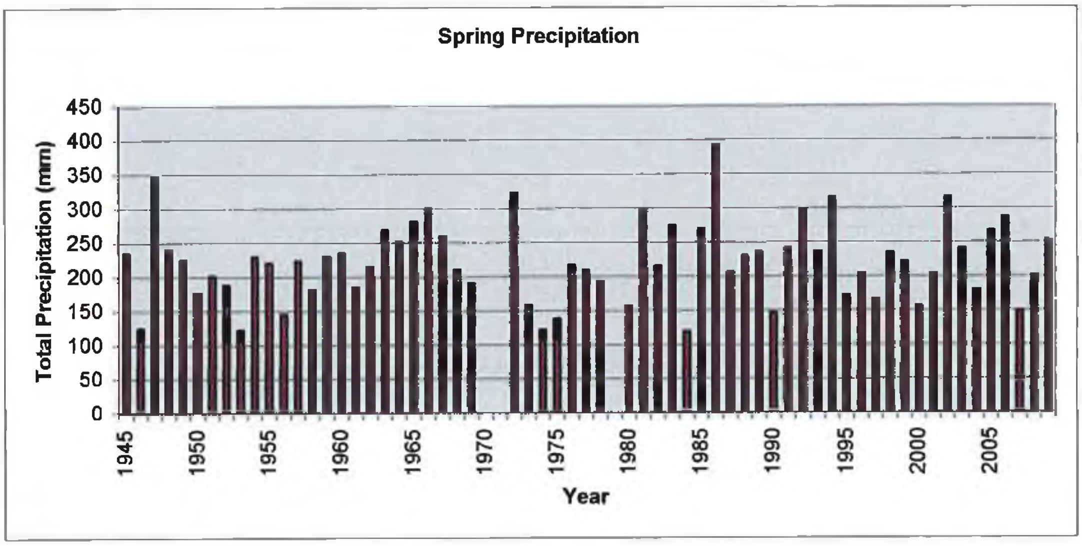


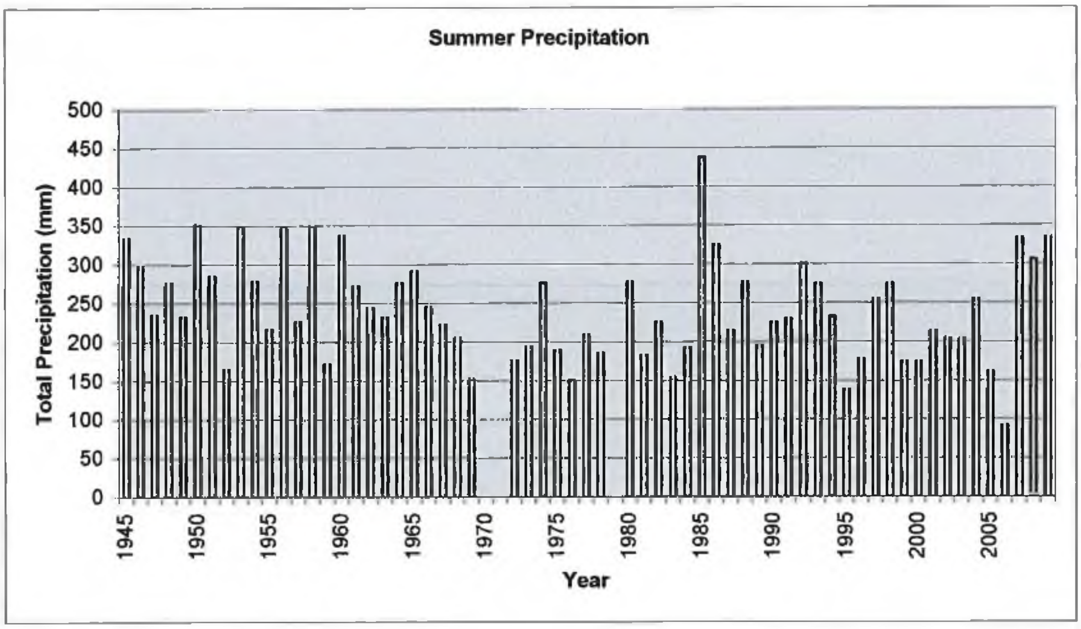


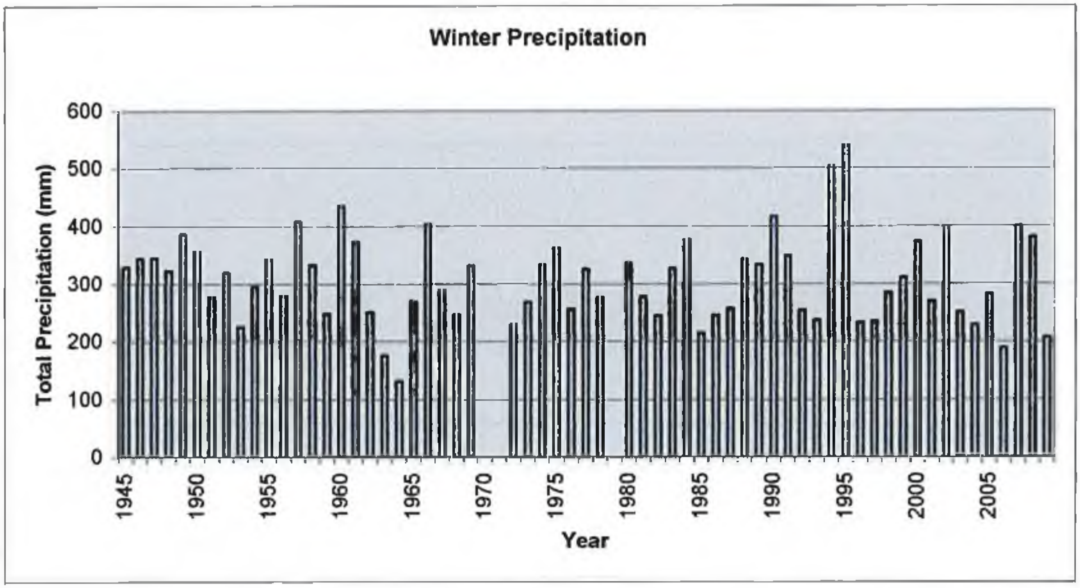
Winter
Total Rainfall for Season
(mm)
0
100
200
300
400
500
600
Year
1945
1946
1947
1948
1949
1950
1953
1954
1955
1956
1957
1958
1959
1960
1961
1962
1963
1964
1966
1967
1987
1988
1989
2001
2002
2003
2004
2005
2006
2007
2008

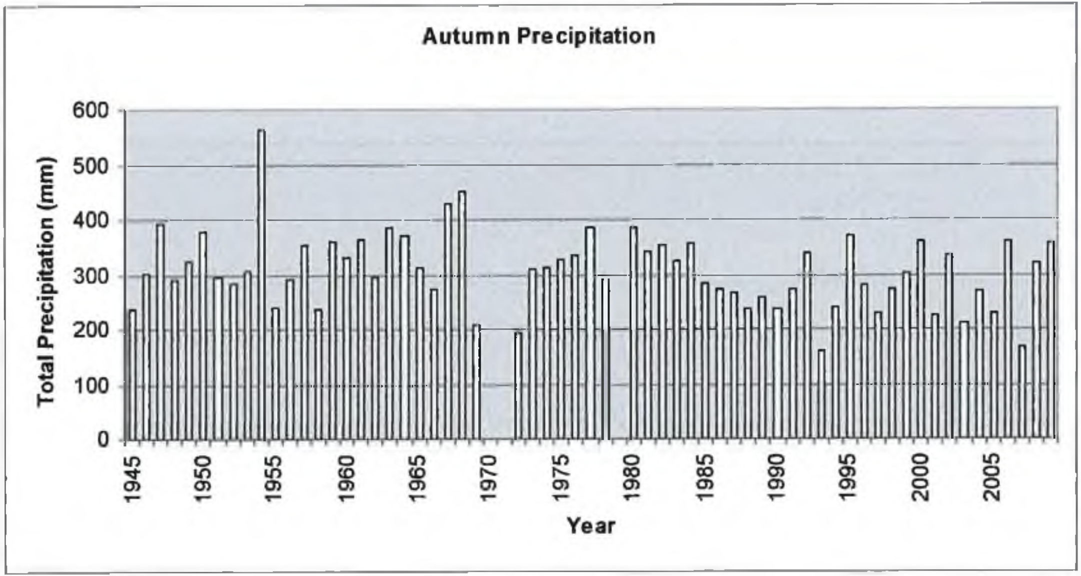

Total Rainfall for Season (mm)
0
100
200
300
400
500
600
700
1945
1946
1947
1948
1949
1950
1953
1954
1955
1956
1957
1958
1959
1960
1961
1962
1963
1964
1966
1967
1987
1988
1989
2001
2002
2003
2004
2005
2006
2007
2008
Year
Autumn

E–1.2 – Glenamaddy: Seasonal Rainfall Totals

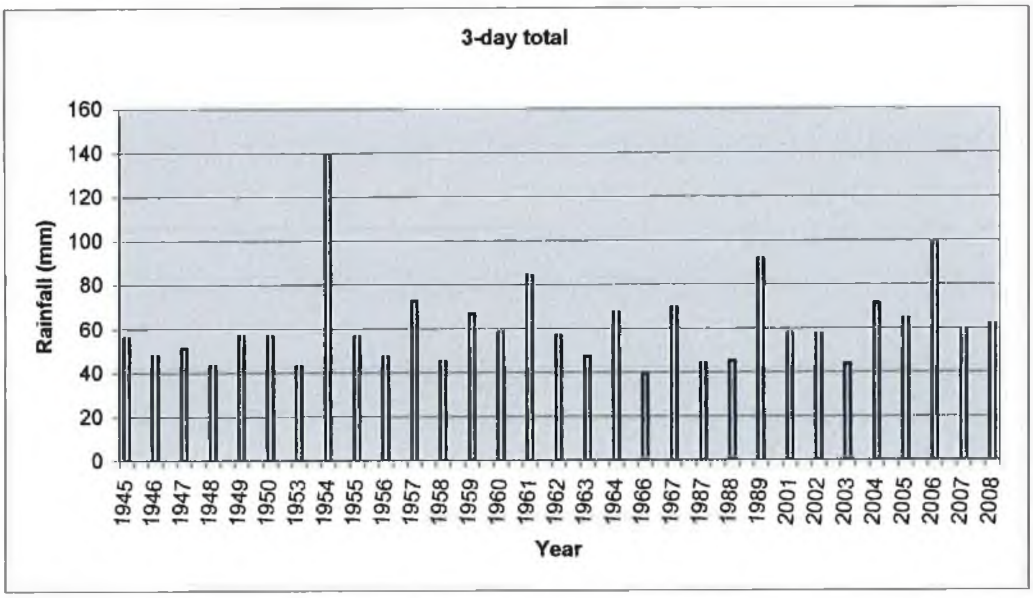


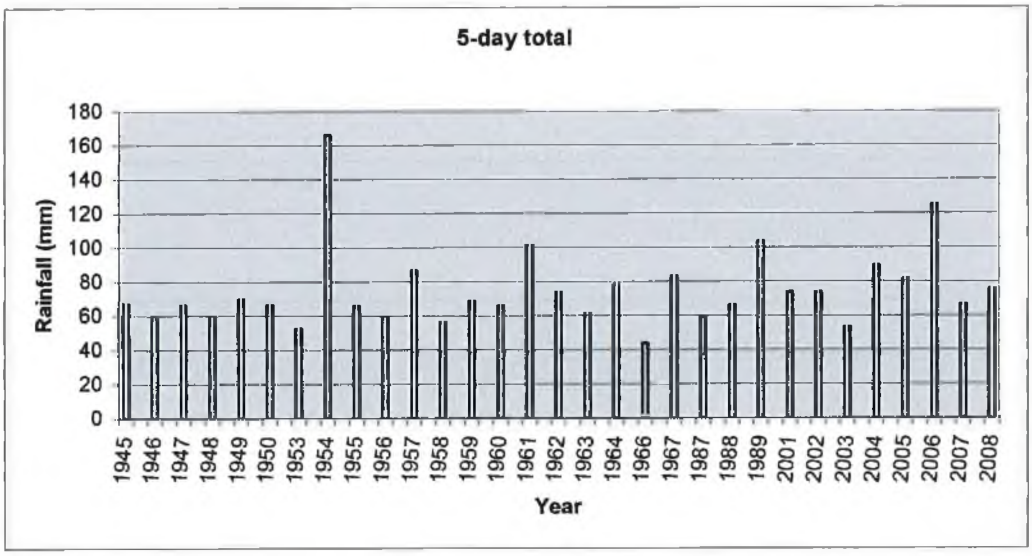


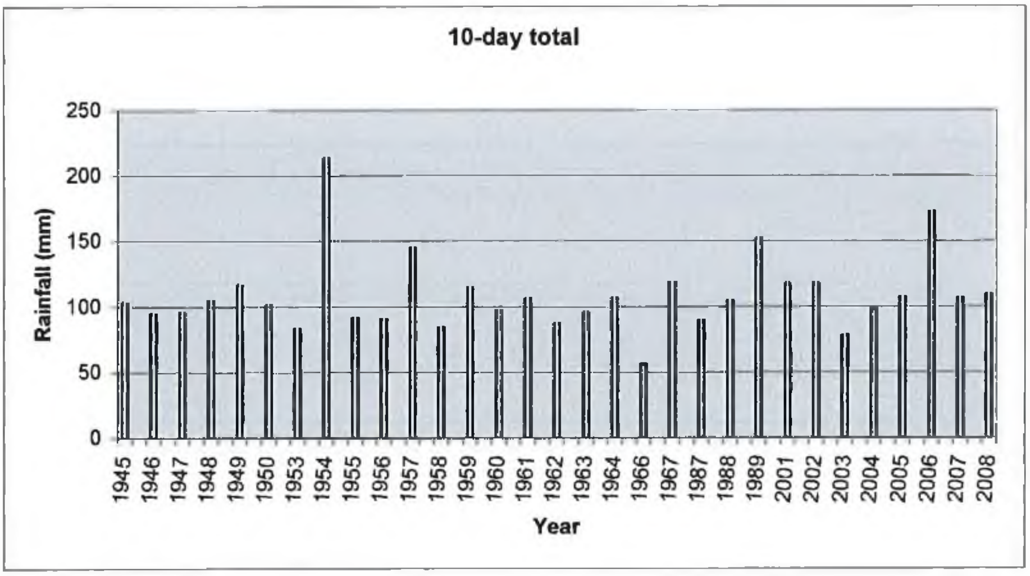

Winter Precipitation
Total Precipitation (mm)
0
100
200
300
400
500
600
1945
1950
1955
1960
1965
1970
1975
1980
1985
1990
1995
2000
2005
Year


Autumn Precipitation
Total Precipitation (mm)
0
100
200
300
400
500
600
1945
1950
1955
1960
1965
1970
1975
1980
1985
1990
1995
2000
2005
Year

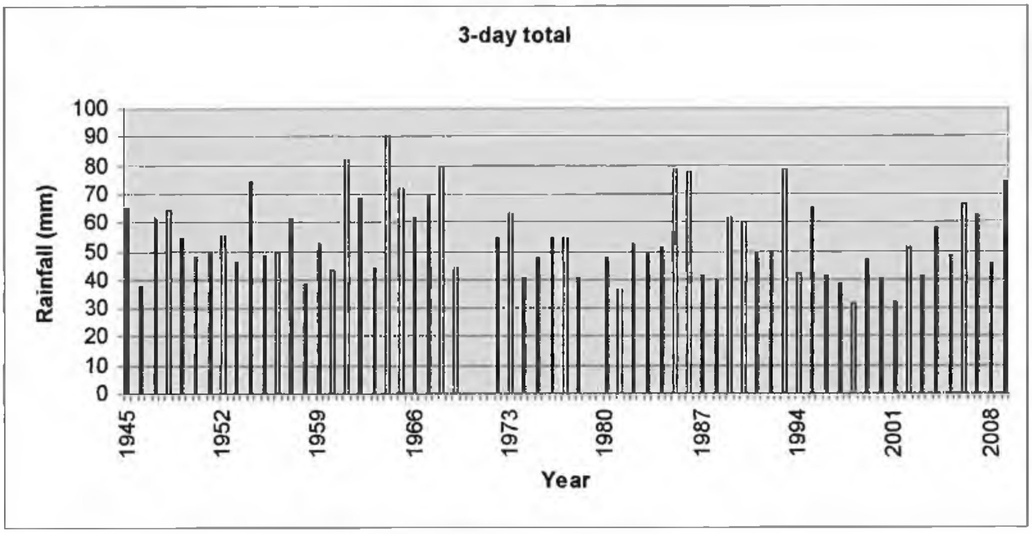
E–2.1 – Milltown: Maximum 3, 5 and 10-day Rainfall Totals

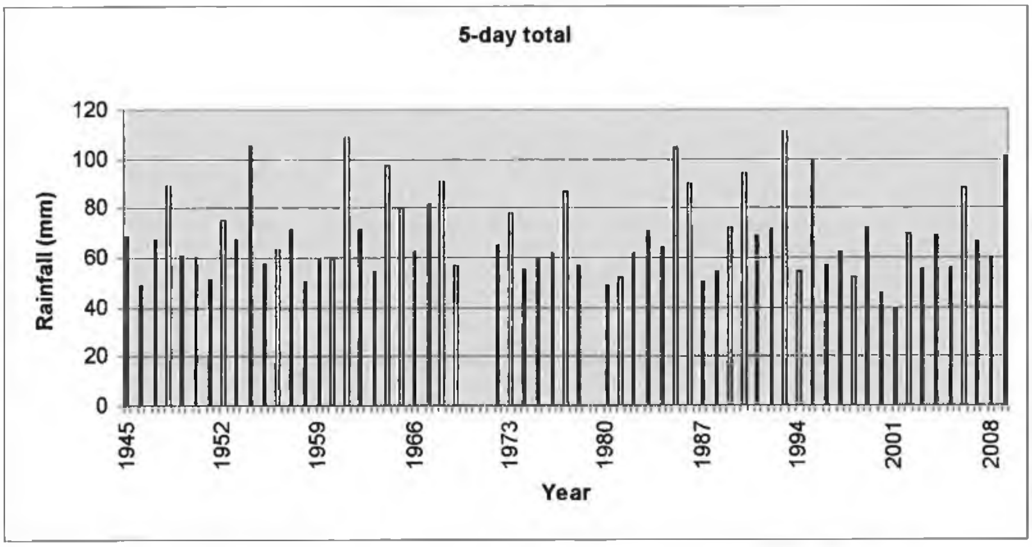


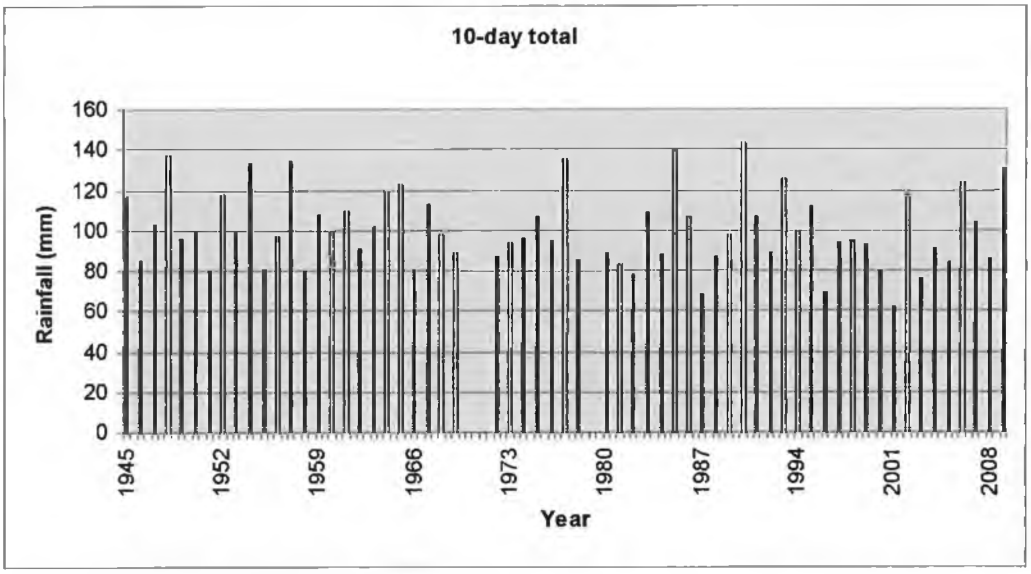




E–2.2 – Glenamaddy: Maximum 3, 5 and 10-day Rainfall Totals







Appendix F

Synthetic Unit Hydrograph (FSR Method):

1. Convolution of the 1-hr Unit Hydrograph with Net Rain for URBAN = 1.3%
2. Convolution of the 1-hr Unit Hydrograph with Net Rain for URBAN = 0.0%

F-1 - Convolution of the 1-hr Unit Hydrograph with Net Rain for URBAN = 1.3%
Scenario A

Hour	Net Rain (cm)	Hours from Origin in row 1 — 1-hr Unit Hydrograph Ordinates in m3/s [illegible] in row 2																																
		0	3	6	9	12	15	18	21	24	27	30	33	36	39	42	45	48	51	54	57	60	63	66	69	72	75	78	81	81	81	81	81	81
		0.00	16.13	32.27	48.40	64.53	80.67	96.80	112.93	102.67	92.40	82.13	71.87	61.60	51.33	41.07	30.80	20.53	10.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0.04	0.00	0.58	1.16	1.74	2.31	2.89	3.47	4.05	3.68	3.31	2.94	2.58	2.21	1.84	1.47	1.10	0.74	0.37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.07		0.00	1.16	2.31	3.47	4.63	5.78	6.94	8.10	7.36	6.63	5.89	5.15	4.42	3.68	2.94	2.21	1.47	0.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	0.09			0.00	1.50	3.01	4.51	6.02	7.52	9.02	10.53	9.57	8.61	7.66	6.70	5.74	4.79	3.83	2.87	1.91	0.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12	0.13				0.00	2.08	4.16	6.25	8.33	10.41	12.49	14.58	13.25	11.93	10.60	9.28	7.95	6.63	5.30	3.98	2.65	1.33	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15	0.18					0.00	2.89	5.78	8.68	11.57	14.46	17.35	20.25	18.41	16.57	14.72	12.88	11.04	9.20	7.36	5.52	3.68	1.84	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18	0.27						0.00	4.28	8.56	12.84	17.12	21.40	25.68	29.96	27.24	24.52	21.79	19.07	16.34	13.62	10.90	8.17	5.45	2.72	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
21	0.41							0.00	6.59	13.19	19.78	26.38	32.97	39.57	46.16	41.97	37.77	33.57	29.38	25.18	20.98	16.79	12.59	8.39	4.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
24	0.50								0.00	8.10	16.20	24.30	32.39	40.49	48.59	56.69	51.54	46.38	41.23	36.08	30.92	25.77	20.61	15.46	10.31	5.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
27	0.41									0.00	6.59	13.19	19.78	26.38	32.97	39.57	46.16	41.97	37.77	33.57	29.38	25.18	20.98	16.79	12.59	8.39	4.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30	0.27										0.00	4.28	8.56	12.84	17.12	21.40	25.68	29.96	27.24	24.52	21.79	19.07	16.34	13.62	10.90	8.17	5.45	2.72	0.00	0.00	0.00	0.00	0.00	0.00
33	0.18											0.00	2.89	5.78	8.68	11.57	14.46	17.35	20.25	18.41	16.57	14.72	12.88	11.04	9.20	7.36	5.32	3.68	1.84	0.00	0.00	0.00	0.00	0.00
36	0.13												0.00	2.08	4.16	6.25	8.33	10.41	12.49	14.58	13.25	11.93	10.60	9.28	7.95	6.63	5.30	3.98	2.65	1.33	0.00	0.00	0.00	0.00
39	0.09													0.00	1.50	3.01	4.51	6.02	7.52	9.02	10.53	9.57	8.61	7.66	6.70	5.74	4.79	3.83	2.87	1.91	0.96	0.00	0.00	0.00
42	0.07														0.00	1.16	2.31	3.47	4.63	5.78	6.94	8.10	7.36	6.63	5.89	5.15	4.42	3.68	2.94	2.21	1.47	0.74	0.00	0.00
45	0.04															0.00	0.58	1.16	1.74	2.31	2.89	3.47	4.05	3.68	3.31	2.94	2.58	2.21	1.84	1.47	1.10	0.74	0.37	0.00
	2.87	0.00	0.58	2.31	5.55	10.88	19.09	31.58	50.67	76.92	107.86	140.62	172.87	202.46	226.56	241.02	242.81	233.81	217.80	197.06	173.28	147.77	121.33	95.27	71.05	49.55	32.25	20.10	12.15	6.92	3.53	1.47	0.37	0.00
		34.02	34.60	36.33	39.57	44.89	53.11	65.60	84.69	110.93	141.88	174.64	206.89	236.48	260.38	275.04	[illegible]	267.83	251.82	231.08	207.30	181.79	155.35	129.29	105.07	83.57	66.27	54.12	46.17	40.94	37.55	35.49	34.39	34.02

Total Net Rain (cm)

Peak Flow of the Flood of 100-year Return Period

CWI	125	← Section 6 6 Part 13
RSMD	38.59	← Section 6 6 Part 4
ANSF	0.032	← ANSF = (3 26 x 10-4) (CWI – 125) + (7.4 x 10-4) RSMD + (3 x 10-3)
AREA	1078	← Section 6 6 Part 1
Baseflow	34.02	← Baseflow = ANSF * AREA

F-2 - Convolution of the 1-hr Unit Hydrograph with Net Rain for URBAN = 0.0%

Scenario B

Hour	Net Rain (cm)	Hour from Origin in row 1; 1-hr Unit Hydrograph Ordinates in m3/s (Figure 6.23) in row 2																																
		0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64
		0.00	9.80	19.60	29.40	39.20	49.00	58.80	68.60	78.40	88.20	98.00	107.80	101.46	95.12	88.78	82.44	76.09	69.75	63.41	57.07	50.73	44.39	38.05	31.71	25.36	19.02	12.68	6.34	0.00	0.00	0.00	0.00	0.00
2	0.03	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	2.59	2.43	2.27	2.11	1.94	1.78	1.62	1.46	1.30	1.13	0.97	0.81	0.65	0.49	0.32	0.16	0.00	0.00	0.00	0.00	0.00
4	0.03		0.00	0.31	0.62	0.93	1.24	1.55	1.85	2.16	2.47	2.78	3.09	3.40	3.20	3.00	2.80	2.60	2.40	2.20	2.00	1.80	1.60	1.40	1.20	1.00	0.80	0.60	0.40	0.20	0.00	0.00	0.00	0.00
6	0.04			0.00	0.40	0.79	1.19	1.59	1.99	2.38	2.78	3.18	3.58	3.97	4.37	4.11	3.86	3.60	3.34	3.09	2.83	2.57	2.31	2.06	1.80	1.54	1.29	1.03	0.77	0.51	0.26	0.00	0.00	0.00
8	0.06				0.00	0.54	1.09	1.63	2.18	2.72	3.27	3.81	4.36	4.90	5.45	5.99	5.64	5.29	4.93	4.58	4.23	3.88	3.52	3.17	2.82	2.47	2.11	1.76	1.41	1.06	0.70	0.35	0.00	0.00
10	0.07					0.00	0.71	1.41	2.12	2.83	3.53	4.24	4.95	5.65	6.36	7.07	7.77	7.32	6.86	6.40	5.94	5.49	5.03	4.57	4.11	3.66	3.20	2.74	2.29	1.83	1.37	0.91	0.46	0.00
12	0.08						0.00	0.74	1.47	2.21	2.94	3.68	4.42	5.15	5.89	6.62	7.36	8.10	7.62	7.14	6.67	6.19	5.72	5.24	4.76	4.29	3.81	3.33	2.86	2.38	1.91	1.43	0.95	0.48
14	0.11							0.00	1.09	2.18	3.27	4.36	5.45	6.54	7.63	8.71	9.80	10.89	11.98	11.28	10.57	9.87	9.16	8.46	7.75	7.05	6.34	5.64	4.93	4.23	3.52	2.82	2.11	1.41
16	0.12								0.00	1.19	2.38	3.58	4.77	5.96	7.15	8.35	9.54	10.73	11.92	13.12	12.34	11.57	10.80	10.03	9.26	8.49	7.72	6.94	6.17	5.40	4.63	3.86	3.09	2.31
18	0.16									0.00	1.60	3.21	4.81	6.42	8.02	9.63	11.23	12.84	14.44	16.05	17.65	16.61	15.57	14.54	13.50	12.46	11.42	10.38	9.34	8.31	7.27	6.23	5.19	4.15
20	0.20										0.00	2.00	4.00	6.01	8.01	10.01	12.01	14.01	16.02	18.02	20.02	22.02	20.73	19.43	18.14	16.84	15.55	14.25	12.95	11.66	10.36	9.07	7.77	6.48
22	0.28											0.00	2.72	5.45	8.17	10.89	13.62	16.34	19.06	21.79	24.51	27.23	29.96	28.20	26.43	24.67	22.91	21.15	19.38	17.62	15.86	14.10	12.34	10.57
24	0.32												0.00	3.17	6.33	9.50	12.66	15.83	18.99	22.16	25.32	28.49	31.65	34.82	32.77	30.72	28.67	26.62	24.58	22.53	20.48	18.43	16.38	14.34
26	0.32													0.00	3.17	6.33	9.50	12.66	15.83	18.99	22.16	25.32	28.49	31.65	34.82	32.77	30.72	28.67	26.62	24.58	22.53	20.48	18.43	16.38
28	0.28														0.00	2.72	5.45	8.17	10.89	13.62	16.34	19.06	21.79	24.51	27.23	29.96	28.20	26.43	24.67	22.91	21.15	19.38	17.62	15.86
30	0.20															0.00	2.00	4.00	6.01	8.01	10.01	12.01	14.01	16.02	18.02	20.02	22.02	20.73	19.43	18.14	16.84	15.55	14.25	12.95
32	0.16																0.00	1.60	3.21	4.81	6.42	8.02	9.63	11.23	12.84	14.44	16.05	17.65	16.61	15.57	14.54	13.50	12.46	11.42
34	0.12																	0.00	1.19	2.38	3.58	4.77	5.96	7.15	8.35	9.54	10.73	11.92	13.12	12.34	11.57	10.80	10.03	9.26
36	0.11																		0.00	1.09	2.18	3.27	4.36	5.45	6.54	7.63	8.71	9.80	10.89	11.98	11.28	10.57	9.87	9.16
38	0.08																			0.00	0.74	1.47	2.21	2.94	3.68	4.42	5.15	5.89	6.62	7.36	8.10	7.62	7.14	6.67
40	0.07																				0.00	0.71	1.41	2.12	2.83	3.53	4.24	4.95	5.65	6.36	7.07	7.77	7.32	6.86
42	0.06																					0.00	0.54	1.09	1.63	2.18	2.72	3.27	3.81	4.36	4.90	5.45	5.99	5.64
44	0.04																						0.00	0.40	0.79	1.19	1.59	1.99	2.38	2.78	3.18	3.58	3.97	4.37
46	0.03																							0.00	0.31	0.62	0.93	1.24	1.55	1.85	2.16	2.47	2.78	3.09
48	0.03																								0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25
	3.00	0.00	0.25	0.81	1.77	3.27	5.48	8.42	12.45	17.68	24.51	33.34	44.90	59.21	76.17	95.21	115.34	135.92	156.48	176.34	194.96	211.65	225.59	235.44	240.39	240.37	235.87	228.07	217.62	205.22	191.18	176.12	160.17	143.66
		34.02	34.27	34.83	35.79	37.29	39.50	42.44	46.47	51.70	58.53	67.36	78.92	93.23	110.19	129.23	149.36	169.94	190.50	210.36	228.98	245.67	259.61	269.46	274.41	274.39	269.89	262.09	251.64	239.23	225.20	210.14	194.18	177.68

Total Net Rain (cm)

Peak Flow of the Flood of 100-year Return Period

Scenario B - continued

Hour	Net Rain (cm)	Hour from Origin in row 1; 1-hr Unit Hydrograph Ordinates in m3/s (Figure 6.23) in row 2																		
		66	68	70	72	74	76	78	80	82	84	86	88	90	92	94	96	98	100	102
		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	0.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12	0.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
14	0.11	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	0.12	1.54	0.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18	0.16	3.11	2.08	1.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20	0.20	5.18	3.89	2.59	1.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
22	0.28	8.81	7.05	5.29	3.52	1.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
24	0.32	12.29	10.24	8.19	6.14	4.10	2.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
26	0.32	14.34	12.29	10.24	8.19	6.14	4.10	2.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
28	0.28	14.10	12.34	10.57	8.81	7.05	5.29	3.52	1.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30	0.20	11.66	10.36	9.07	7.77	6.48	5.18	3.89	2.59	1.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
32	0.16	10.38	9.34	8.31	7.27	6.23	5.10	4.15	3.11	2.08	1.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34	0.12	8.49	7.72	6.94	6.17	5.40	4.63	3.86	3.09	2.31	1.54	0.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
36	0.11	8.46	7.75	7.05	6.34	5.64	4.31	4.23	3.52	2.82	2.11	1.41	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00
38	0.08	6.19	5.72	5.24	4.76	4.29	3.81	3.33	2.86	2.38	1.91	1.43	0.95	0.48	0.00	0.00	0.00	0.00	0.00	0.00
40	0.07	6.40	5.94	5.49	5.03	4.57	4.11	3.66	3.20	2.74	2.29	1.83	1.37	0.91	0.46	0.00	0.00	0.00	0.00	0.00
42	0.06	5.29	4.93	4.58	4.23	3.88	3.52	3.17	2.82	2.47	2.11	1.76	1.41	1.06	0.70	0.35	0.00	0.00	0.00	0.00
44	0.04	4.11	3.86	3.60	3.34	3.09	2.83	2.57	2.31	2.06	1.80	1.54	1.29	1.03	0.77	0.51	0.26	0.00	0.00	0.00
46	0.03	3.40	3.20	3.00	2.80	2.60	2.40	2.20	2.00	1.80	1.60	1.40	1.20	1.00	0.80	0.60	0.40	0.20	0.00	0.00
48	0.03	2.50	2.73	2.59	2.43	2.27	2.11	1.94	1.78	1.62	1.46	1.30	1.13	0.97	0.81	0.65	0.49	0.32	0.16	0.00
	3.00	126.96	110.23	93.79	78.12	63.49	50.15	38.58	29.05	21.58	15.86	11.44	8.06	5.45	3.54	2.11	1.14	0.52	0.16	0.00
		160.98	144.25	127.81	112.14	97.51	84.17	72.60	63.07	55.59	49.88	45.46	42.08	39.47	37.56	36.13	35.16	34.54	34.18	34.02

Total Net Rain (cm)

CWI	125	Section 6.6 Part 13
RSMD	38.591892	Section 6.6 Part 4
ANSF	0.031558	ANSF = (3.26 x 10-4) (CWI – 125) + (7.4 x 10-4) RSMD + (3 x 10-3)
AREA	1078	Section 6.6 Part 1
Baseflow	34.019524	Baseflow = ANSF * AREA