TRALEE FLOOD RELIEF SCHEME

Hydrology Report

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1 INTRODUCTION

Tralee Town and its surrounding area are within the Lee sub-catchment. The Lee sub-catchment is within the Tralee Bay-Feale catchment, part of the Unit of Management (UoM) 23 in the Shannon River Basin District. The Lee sub-catchment defines the study area in this report which is shown in **Figure 1.1**.

Following hydrological and hydraulic analysis in the Shannon CFRAM Study (henceforth referred to as the CFRAM Study), the main source of flood risk to Tralee was found to originate from the River Lee and its two main tributaries, the Big River and Ballonagh, which flow through Tralee Town and the study area. There are other tributaries of the Lee which also need to be considered, such as the Ballyvelly, Ballybeggan, Cloghers, and Ballinorig. Hydrological analysis for the proposed Tralee Flood Relief Scheme will consider the flood risk emanating from the River Lee and its tributaries and any other drainage channels which may act as a source of flooding.

Tralee Town and the surrounding area have been subject to multiple flood events in the past, often due to interaction between fluvial flows, coastal water levels, drainage network flooding, and surface water run-off in urbanised areas. The topography of the area is also a contributing factor with the River Lee, Big River, Ballonagh, Ballinorig, and Ballybeggan draining the steep sloping ground of the Stack's Mountains to the north and northeast of the study area, with the Slieve Mish Mountain range to the south being drained by tributaries of the River Lee, namely Ballyvelly and Cloghers. A major flood relief scheme was completed in 1992 to reduce flood frequency via trunk sewers and culverts, however, the scheme may now be outdated and may not provide an adequate standard of protection.

1.1 CFRAM Study Hydrological Analysis

The hydrological analysis undertaken as part of the CFRAM Study utilised the latest Flood Studies Update (FSU) methodologies and the best gauge data available at the time (up to end of hydrological year 2012). The gauging stations within the study area provided poor data during the CFRAM Study, leading to the recommendation and subsequent installation of five gauging stations (Lisloose 23044, Ballyseedy 23045, Spring Water Lane 23046, Oakpark Road 23047, and Manor West 23049).

The CFRAM Study considered the soil type and geology in UoM 23 when deriving flood estimates. It concluded that the combination of soils and aquifer geology (karst) does not play a significant role in flood response of the modelled catchments affecting Tralee.

The CFRAM Study found that no significant flood events had been recorded during the Study. The CFRAM Study only considered flood data up to the summer of 2012 and that further hydrological analysis must consider the impact of additional years of data. It was noted in the CFRAM Study Hydraulics report that anecdotal evidence following significant flood events in 2015 suggests there are areas within and surrounding Tralee which were not represented on the flood maps produced during the CFRAM Study. Such events when considered during at-site flood frequency analysis may alter the understanding of flood risk for the scheme area. These events may also inform understanding of the timing of hydrographs and Joint Probability assumptions.

Other improvements could be made to enhance the robustness of the analysis at scheme level including:

- Consideration of the flood risk from other small watercourses and drains affecting Tralee which were gauged post-CFRAM study.
- Increase in the resolution of the hydrological analysis.
- Routing of recent flood flow data through the model.
- Using recent improvements in the FSU methods when determining design flows at hydrological estimation points (HEPs). These improvements are yet to be approved for use by the OPW. They will be considered in analysis but not relied upon until such a time as the new methodology is approved by the OPW.

1.2 Catchment Review

The Lee sub-catchment associated with Tralee covers an area approximately 98.6km^2 and is part of the larger Tralee Bay-Feale catchment (approx. 1708km^2). The Tralee Bay-Feale is drained by the River Feale and the many other watercourses entering tidal waters from the mouth of the Shannon Estuary to the western tip of the Dingle Peninsula, including the River Lee which drains into Tralee Bay through the Lee sub-catchment. The most upstream extents of the Lee sub-catchment are in the Stack’s Mountains where the source of the River Lee and many of its tributaries rise. The study catchment will include the watercourses considered to affect the study area; the River Lee, Big River, River Ballonagh, and any major tributaries, as shown in **Figure 1.1**.

The individual catchments (and watersheds) are defined within the FSU datasets. These will be checked through re-delineation using the automated catchment delineation tools available in Arc Hydro and the best available digital terrain models covering the entire catchment. Arc Hydro is a suite of GIS based tools that use raster processing algorithms to determine watershed and flow paths from rasterised digital terrain datasets such as LiDAR and Digital Height Models.

The Lee sub-catchment is in general moderately steep to steep, with Tralee Town and its hinterland to the east being less steep than the higher ground to the north. In the town and to the east, soils can be well-drained, but their extent is highly limited with poorly drained soils and extensive peat deposits dominating the catchment to the north and south of the study area. The two mountain ranges to the north and south, Stack’s and Slieve Mish, respectively, create steep upland and thus the Lee catchment and its smaller sub-catchments can be flashy in nature. The catchment is predominantly rural with the only urbanised area being Tralee Town at the lower reaches of the Lee. The catchment is relatively impermeable and moderately wet to wet.

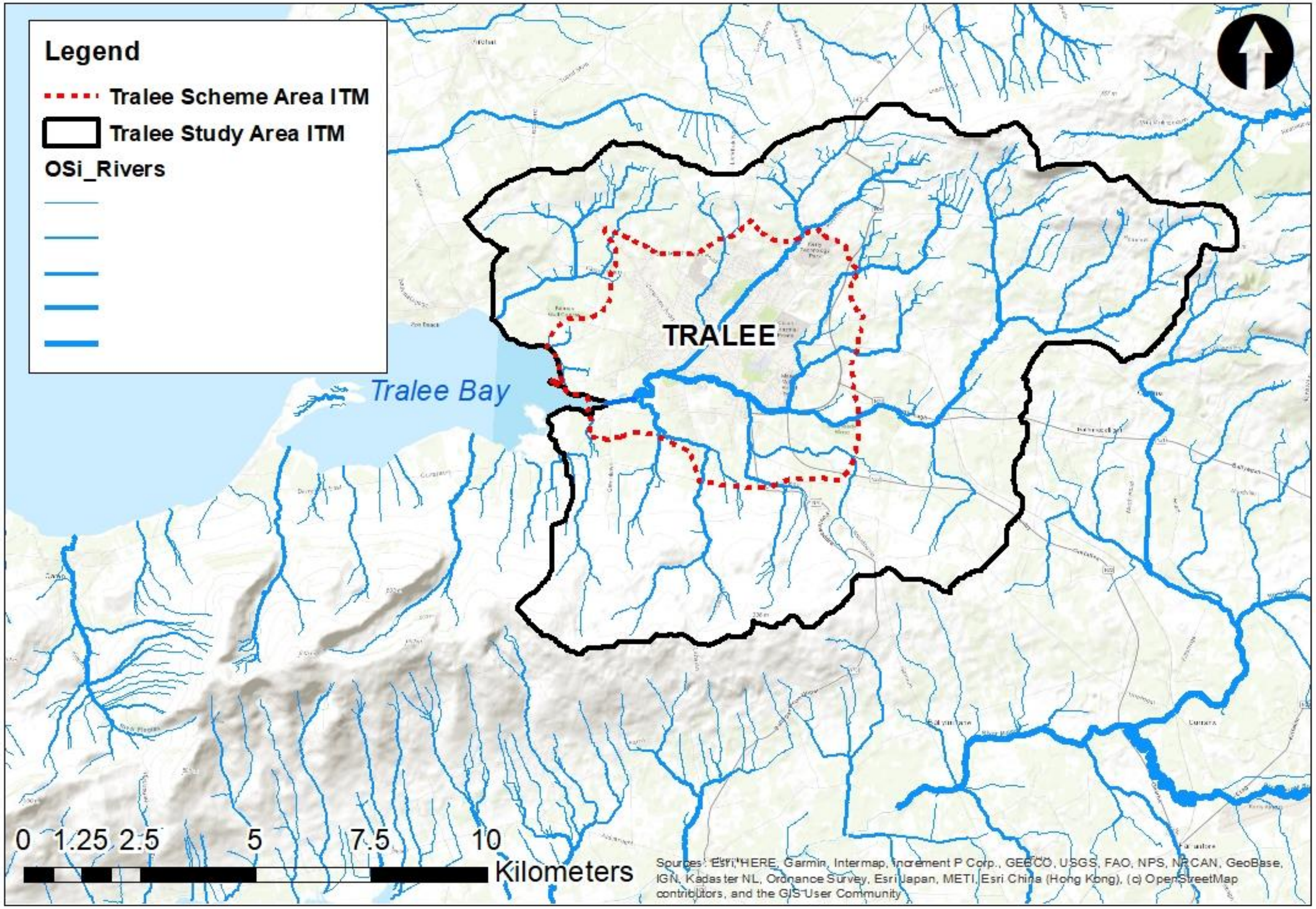


Figure 1.1: Tralee Study and Scheme Area

1.3 Available Hydrometric Data

Hydrological analysis will take into consideration all available data from past and present gauging stations. The main data which will underpin the hydrological analysis is the hydrometric data available on the River Lee and Big River, upon which statistical flood frequency analysis can be based.

The OPW and Environmental Protection Agency (EPA) were consulted for the most up to date hydrometric data for the catchment area. Out of the nine gauging stations identified (**Table 1.1**)**,** five were installed as recently as 2020 while three of the remaining four had only water level data available.

Table 1.1: Hydrometric Gauging Station Record Considered for Hydrological Analysis (WL = Water Level)

Station No.	Station Name	Waterbody	Catch Area Km^2	Operator	Record Start	Record End	Data
23011	Ballycarty	Lee	23.34	OPW	01/10/1959	01/07/1991	WL only
23012	Ballymullen	Lee	61.63	OPW	08/07/2004	Present	WL & Flow
23022	Tralee Clonalour	Big River	10.00	EPA	15/11/1985	16/01/2012	WL & Flow
23044	Lisloose	Ballonagh	1.80	OPW	14/05/2020	Present	WL & Flow
23045	Ballyseedy	Lee	31.90	OPW	12/06/2020	Present	WL & Flow
23046	Spring Water Lane	Lissardboola	13.50	OPW	25/06/2020	Present	WL & Flow
23047	Oakpark Road	Big River	9.00	OPW	25/06/2020	Present	WL & Flow
23049	Manor West	Ballybeggan	12.00	OPW	25/11/2020	Present	WL & Flow
23062	Blennerville	Lee Estuary	n/a	OPW	23/02/2004	Present	WL only
23063	Ballyard	Lee Estuary	17.96	OPW	29/05/2004	Present	WL only

While the spatial coverage of gauging stations in the scheme area (**Figure 1.2**) is significantly improved since 2020, only Tralee Clonalour (23022 – now inactive) and Ballymullen (23012) has a substantial (> 2 years) continuous flow record which could form the basis of flood frequency analysis and design flow estimation, highlighting the poor temporal coverage of long-term hydrometric data for the study area.

Ballymullen was considered tidally influenced following the CFRAM Study. No rating review for Ballymullen has been developed since 1996 following a significant change in rating between 1984/85 and flood relief works between 1992 and 1996. No continuous flow record is available for Ballymullen post 1996. An AMAX series for flow from 1974 to 1991 was provided by the OPW, however, uncertainty in using this record is high due to the flood relief works which may have significantly altered the catchment meaning an AMAX flow record between 1974 and 1991 at Ballymullen may not be representative of present-day flood flow conditions. A complete water level AMAX series is available for Ballymullen, as well as a number of spot flow gaugings. There is significant variation in the recorded spot flow gaugings at Ballymullen since 1996. The OPW carried out further studies to determine whether tidal conditions influenced the observed variation in spot flow gaugings, however, no relationship was found. This report details a rating review undertaken at the station (Section2.1) as well as flood frequency analysis carried out on the record (Section3.1.2). Rating reviews have also been undertaken for the five newly installed gauging stations (23044, 23047, 23049, 23045 and 23046), shown in **Figure 1.2** and presented in Chapter 2.

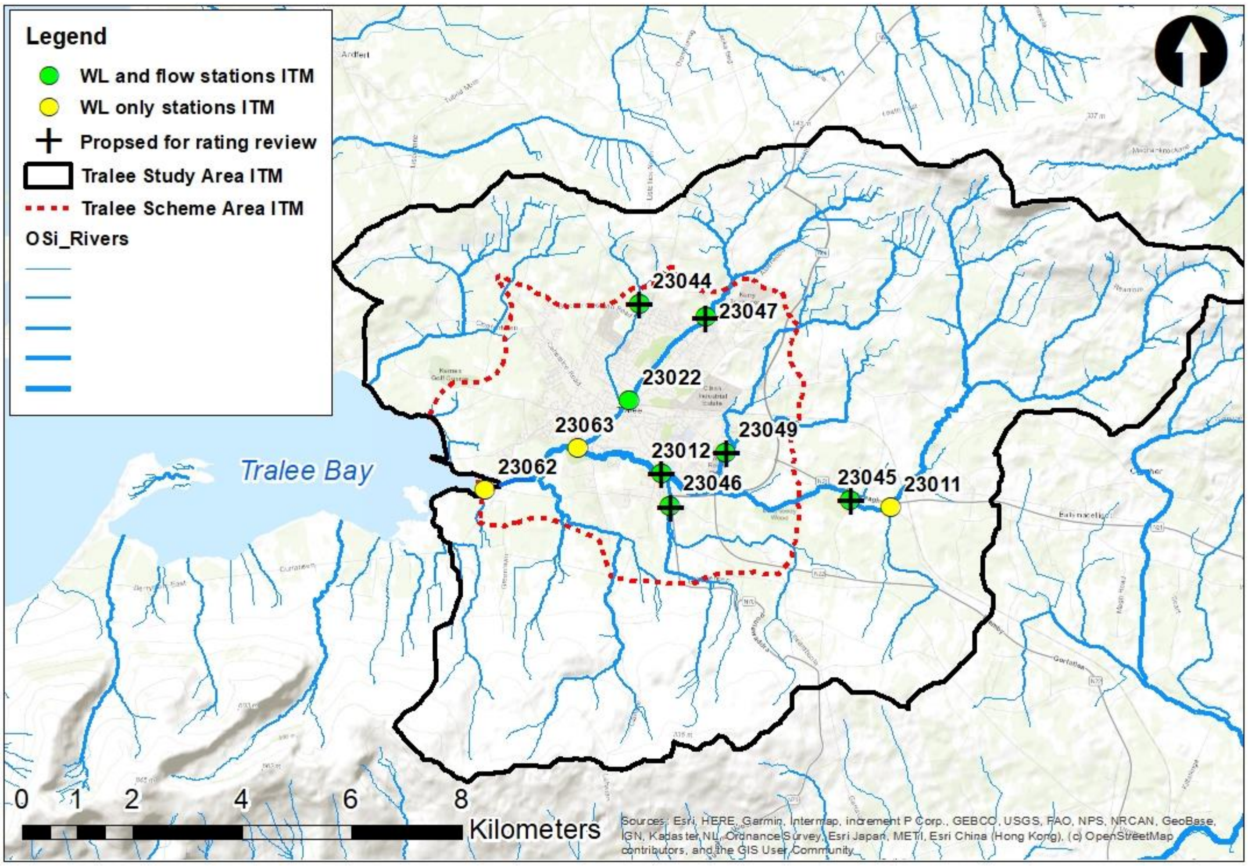


Figure 1.2: Gauging Stations Considered in this Study

1.4 Flood Event History

The study and scheme area has experienced damaging flood events driven by fluvial, pluvial, and coastal mechanisms. A significant flood event on 5th August 1986 led to the development and implementation of the Tralee Flood Relief Scheme (TFRS). Prior to the TFRS, the town centre of Tralee experienced recurring flood events, such as the 1973 event which greatly impacted businesses, residential properties, and infrastructure. Since the completion of the TFRS in 1992, flood events in Tralee Town have decreased in frequency and severity but still occur. A summary of the flood history affecting Tralee is presented in **Table 1.2**.

Table 1.2: Flood History at Tralee and Within the Scheme Area

Event Date	Source	Summary of Reports
20th/21st Aug 2020	Fluvial / Coastal	The coinciding factors of high tides, storm surges, and heavy rainfall led to localised flooding in the areas of Ballyard, Blennerville, Ballyvelly, and the centre of Tralee town with the basement flooding at 13 Princes Quay as well as properties in Days Place being impacted.
8th/9th Feb 2020	Fluvial / Pluvial / Coastal	Flooding took place in multiple locations around Tralee. Clash Road experienced flooding due to exceptionally heavy rain. The Ballybeggan racecourse was noted as a significant source of run-off, similar to that on the 24th of August 2012. The Ballyard area of Tralee was impacted by fluvial and tidal flooding, with James Street, Staughtons Row, and Godfrey Place specifically being impacted.
13th Oct 2019	Pluvial	Heavy rainfall led to the flooding of multiple homes in residential areas of Tralee Town, as reported by online sources.[1] Residential homes in the Boherbue area of Tralee town were assisted by the Kerry Civil Defence AFS Unit.[2]
3rd Jan 2018	Coastal	A combination of high seas and strong winds due to Storm Eleanor led to the tidal flooding of the Blennerville area of Tralee town. Online sources stated that the N86 Tralee/Dingle road was impassable due to coastal flooding.[3,4]
22nd Nov 2017	Pluvial	Heavy rainfall and subsequent flooding led to dangerous road surface conditions across Tralee Town.[5]
5th Dec 2015	Pluvial	Heavy rainfall accompanying Storm Desmond led to extensive flooding in Tralee with areas such as Manor West, Ballymullen and the town centre suffering the most. Many news reports, social media posts and images of the extensive flooding across Tralee are available online.[6,7,8,9]
13th/14th Oct 2015	Fluvial	Transport routes such as Manor West, the Bypass, and Dan Spring Road were all flooded due to heavy rainfall.
3rd Jan 2014	Coastal	Strong winds coincided with a high Spring Tide of 5.0m. A possible tidal surge led to flooding at Ballyvelly. Estimated 200mm depth of water above the tow path along canal with local residents of 20 years providing anecdotal evidence that they had never seen such a flooding event.[10]

[1] http://traleetoday.ie/brassil-calls-for-emergency-funding-for-families-after-flooding-in-tralee/

[2] https://www.civildefence.ie/kerry-flooding/

[3] https://www.independent.ie/irish-news/storms/storm-eleanor-aftermath-massive-clean-up-operation-underway-as-country-braces-for-more-gale-force-winds-36450295.html

[4] https://www.thesun.ie/news/1998277/met-eireann-warns-of-no-reprieve-from-the-wind-and-rain-as-two-more-storms-sweep-across-the-country-in-wake-of-storm-eleanor/

[5] http://traleetoday.ie/flooding-roads-heavy-rain-last-night/

[6] https://fergusdennehy.com/2015/12/05/photos-social-media-pictures-show-the-full-extent-of-storm-desmond/

[7] https://www.irishmirror.ie/news/irish-news/army-deployed-cork-kerry-help-6962186

[8] https://www.rte.ie/news/2015/1205/751447-weather/

[9] https://www.civildefence.ie/kerry-response-to-storm-desmond/

[10] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310 Data Collection/004 OPW Mungret/002 Reports/opm_re_JB_0000012881.pdf

Event Date	Source	Summary of Reports
24th Aug 2012	Pluvial	Flooding at Clash Road, Tralee, where exceptionally heavy rainfall led to the flooding of several properties, roads, and agricultural land, with flood depths recorded up to 1m. Additional information recorded by Kerry County Council highlighted the role of Tralee Racecourse in the flooding event whereby land-use increased the impact of the event.[11] The gates to the racecourse appeared to have impounded a significant volume of water which was then released once the gates gave way. 1.5 tonnes of debris were reportedly removed from gulley traps following the flood event.
29th Nov 2011	Pluvial	Storm sewers became overwhelmed following heavy rainfall which led to the flooding of Main Street, Ballymullen in Tralee. It was noted this location experiences frequent flooding events.[12]
18th Feb 2011	Coastal	Kearney's Road in Blennerville, Tralee, experienced tidal flooding over a 3-day period. High tide and surge conditions coincided to cause the flooding event. It was noted the location typically experiences 4 to 5 flooding events/year.[13]
7th Feb 2011	Pluvial	Heavy rainfall over a multi-day period led to the flooding of Curragraigue in Blennerville, Tralee. The cause was determined to be inadequate drainage or culvert capacity to deal with surface water run-off due to the exceptionally heavy rain. Flooding persisted until the 11th of February, but reports say it peaked on the 6th/7th.[14]
19th Nov 2009	Coastal / Pluvial	The Ballymullen area of Tralee experienced flooding due to heavy rain, high tides, and drainage systems becoming overwhelmed. Extensive damage was caused to properties and transport infrastructure, such as the N70 from Army Barracks to Ballymullen roundabout. 6 residential and 1 commercial property were damaged, while the local GAA clubhouse at Curragraigue in Blennerville flooded to a depth of 300mm. The road access to Blennerville was also interrupted. The flooding persisted until the 26th of November but peaked on the 19th.[15]
13th Aug 2008	Fluvial / Pluvial	Heavy rainfall resulting in surface water run-off, as well as the bursting of riverbanks, led to a flooding event lasting 2 days at Caherweesheen TD in Ballyard, Tralee. 1 house and a farm building were impacted by flooding.[16]
26th Dec 2004	Pluvial	Ballinorig area experiences on average one flood per year. A particularly bad event occurred on the 26th December 2004 which was assumed to be caused by inadequate capacity at a culvert.[17]
31st August 1997	Fluvial / Pluvial	Severe flooding of the Tralee Killarney road (N22) and the Earl of Desmond Hotel occurred. Residential properties in the Ballinorig area were also flooded to depths of 3 feet. Ballyseedy area of Tralee was impacted by flooding and an engineer's report found overgrowth on the banks of the Caherweesheen/Ballyseedy water routes to have exacerbated the fluvial flooding mechanism.[18]

[11] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310 Data Collection/016 Kerry County Council/002 Reports/kry_re_MN_0000012310.pdf

[12] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310 Data Collection/016 Kerry County Council/002 Reports/kry_re_OT_0000011880.pdf

[13] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/016%20Kerry%20County%20Council/002%20Reports/kry_re_OT_0000011684.pdf

[14] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/016%20Kerry%20County%20Council/002%20Reports/kry_re_MN_0000011682.pdf)

[15] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/016%20Kerry%20County%20Council/002%20Reports/kry_re_MN_0000011380.pdf

[16] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/016%20Kerry%20County%20Council/002%20Reports/kry_re_OT_0000011681.pdf

[17] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/016%20Kerry%20County%20Council/004%20Minutes%20Verbal%20Report/kry_mm_th_0000002449.pdf

[18] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310 Data Collection/004 OPW Mungret/002 Reports/opm_re_tg_0000001524.pdf

Event Date	Source	Summary of Reports
5th Aug 1986	Fluvial / Pluvial	High tides and heavy rainfall associated with the remnants of Hurricane Charley cased extensive flooding in Tralee town as the Big River burst its banks. Flooding lasted for several hours the entire business centre of the town was impacted leading to significant damage. Recording station at Ballycarty on the outskirts of the town exceeded the flood level recorded during 1973 by 0.25m.[19]
2nd Nov 1980	Fluvial / Pluvial	Heavy and prolonged rainfall led to surface flooding because of inadequate or blocked drains and/or culverts. The River Lee was also reported to have broken its banks and flooding was reported at Ballyseedy, Ballyard, and at other locations on the Big River.[20]
30th Nov 1973	Fluvial / Pluvial	Surface flooding occurred throughout the town of Tralee and was deemed to be caused by heavy rainfall and surface water run-off. Surface flooding in streets ranged from 0.3m to 0.6m in depth.[21] News reports from multiple sources suggested that the Big River and culvert flooded leading to significant damage in the town to infrastructure and property.[22,23]
25th to the 26th Dec 1968	Pluvial	It was reported in the Cork Examiner (regional newspaper) that over 2.5 inches, or around 63.5mm, of rainfall fell within a 48-hr period resulting in the flooding of many transport routes around Tralee town. Flood depths, at the time, were estimated to be 2 to 4 feet, or roughly 0.6 to 1.3m, in places.[24]
22nd to the 24th Oct 1961	Fluvial / Pluvial / Coastal	Heavy rain, high tides and a storm system brought significant tidal surge led to the flooding event.[25]
16th Nov 1916	Pluvial	The event of 1916 was noted as a common occurrence in a consultant's report at the time. Heavy rainfall and insufficient drainage networks resulted in the overwhelming of the drainage system and subsequent flooding.[26]

[19] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/003%20OPW%20Headford/002%20Reports/oph_re_ab_0000000671.pdf

[20] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/002%20OPW%20Hatch%20St/002%20Reports/opd_re_pm_0000000422.pdf

[21] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/003%20OPW%20Headford/002%20Reports/oph_re_ab_0000000585.pdf

[22] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/PressSearchArchives/4351_1.jpg

[23] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/003%20OPW%20Headford/002%20Reports/oph_re_ab_0000000675.pdf

[24] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/PressSearchArchives/2484_1.jpg

[25] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/PressSearchArchives/4609.jpg

[26] https://s3-eu-west-1.amazonaws.com/floodmaps.floodinfo.ie/Reports/F310%20Data%20Collection/002%20OPW%20Hatch%20St/002%20Reports/opd_re_pm_0000000424.pdf

2 RATING REVIEWS

RPS has conducted six rating reviews on hydrometric gauges within the Tralee study area (coordinates in Irish Transverse Mercator (ITM)):

- Ballymullen (23012) gauging station on the River Lee, located 1.4 km south-east of the town of Tralee @ X: 484483, Y: 613390
- Lisloose (23044) gauging station on Ballonagh River, located in the townland of Lisloose 1.8 km north of the town of Tralee @ X: 484092, Y:616485
- Ballyseedy (23045) gauging station on the River Lee, located in the townland of Ballyseedy 4.4 km east of the town of Tralee @ X: 487929, Y: 612875
- Spring Water Lane (23046) gauging station on the Lissardboola River, located in the townland of Caherweesheen 2 km south-east of the town of Tralee @ X: 484627, Y: 612780
- Oakpark Road (23047) gauging station on the Big River, located in the townland of Killeen 2 km north of the town of Tralee @ X: 485237, Y: 616171
- Manor West (23049) gauging statin on Ballybeggan River, located in the townland of Manor West 2 km east of the town of Tralee @ X: 485635, Y: 613759

The rating reviews have been conducted as part of the hydrological and hydraulic analysis for the FRS at Tralee to validate (or otherwise) the Index Flood Flow (Q_{med}) and design flow estimation on both the respective watercourses.

The general form of any rating equation is:

$$Q = C(h + a)^{\beta}$$

Where:

Q = river flow (m^3/s)

C, a, β = rating equation constants

h = stage height (m)

The rating equation constants have physical significance and are originally derived from hydraulic theory. Rating equation constants have been derived using an iterative process and setting a number of constraint conditions on these physical constants. This helps to generate optimal parameters such that it limits the sum of the squared residuals (Chi Squared) of the observed and predicted flows.

2.1 Ballymullen (23021)

An assessment of the rating at Ballymullen on the River Lee was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record for the staff gauge zero level is 3.49 m above ordnance datum (mAOD Poolbeg). The survey undertaken as part of this project found the staff gauge zero level to be 0.78 mAOD Malin. The two levels are in good agreement with a difference of 10 mm when the OPW value is converted to the latest datum (Malin). The staff gauge zero level was taken as 0.78 mAOD Malin for this project.

2.1.1 Raw Data Analysis

A review of the available data at the gauging station was conducted. The station records water levels at 15-minute intervals. The station has recorded data available for analysis from the 8th July 2004 to the 1st October 2021, shown in **Figure 2.1**. There are significant data gaps within the record period, highlighted by the green shading. The majority of year 2005, 2006 and 2007 is missing, likely due to the water level measurement instrument being absent over this period.

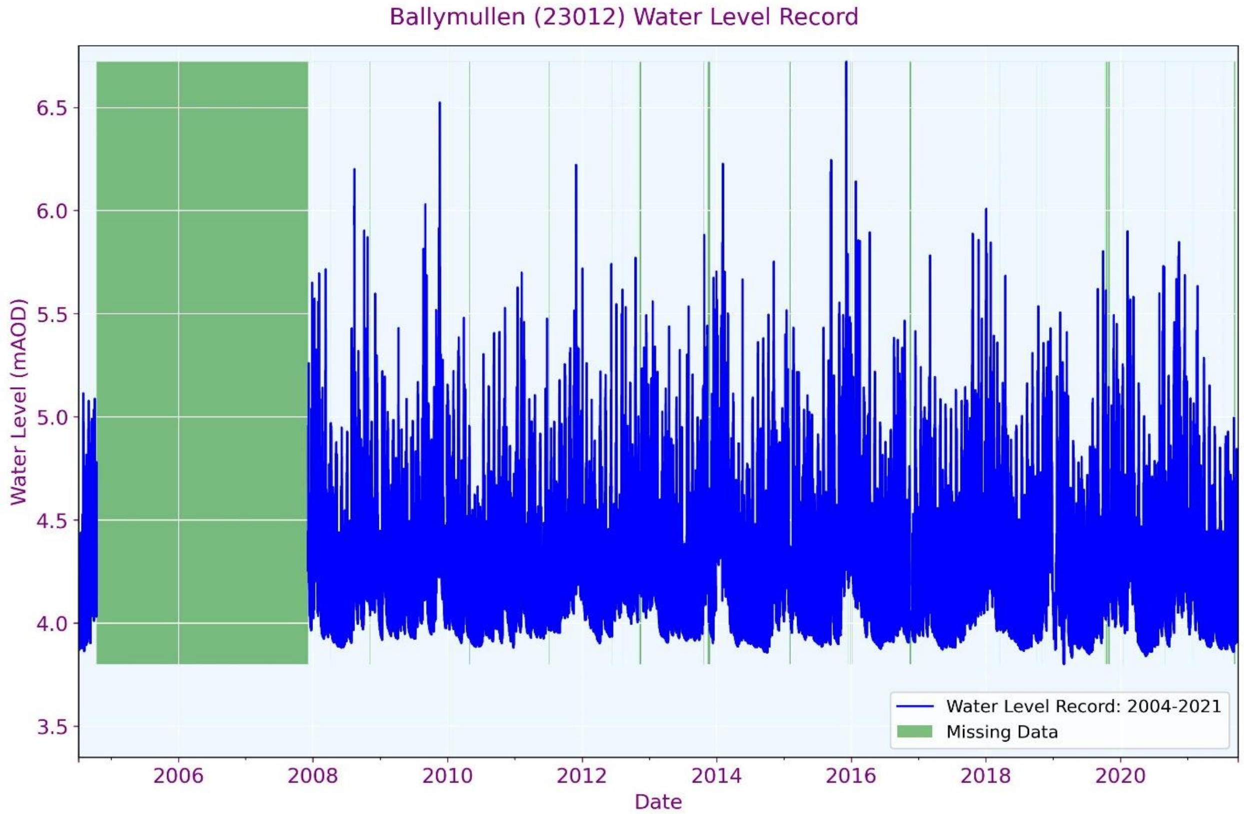


Figure 2.1: River Lee at Ballymullen (23012) Water Level Record from 2004 to 2021

From 2008 onwards, there are 6,914 missing values, which corresponds to roughly 72 days of missing data. A closer look at the water level record from 2008 onwards show periodic gaps in the data, seen in **Figure 2.2**. This may be a result of instrument malfunction, or the instrument being shut down for maintenance. Missing data could be detrimental to understanding Qmed at Ballymullen and adds uncertainty to the analysis. For example, if the instrument is malfunctioning at high water levels at or over Qmed, such water levels may not be captured which could lead to an underestimation of Qmed.

An in-depth analysis of the entire water level record from 2008 onwards provided by the OPW does not suggest that the instrument is failing at high water levels. While missing data does add uncertainty to the rating curve and Qmed analysis, the overall proportion of missing data to the complete water level record at Ballymullen is low.

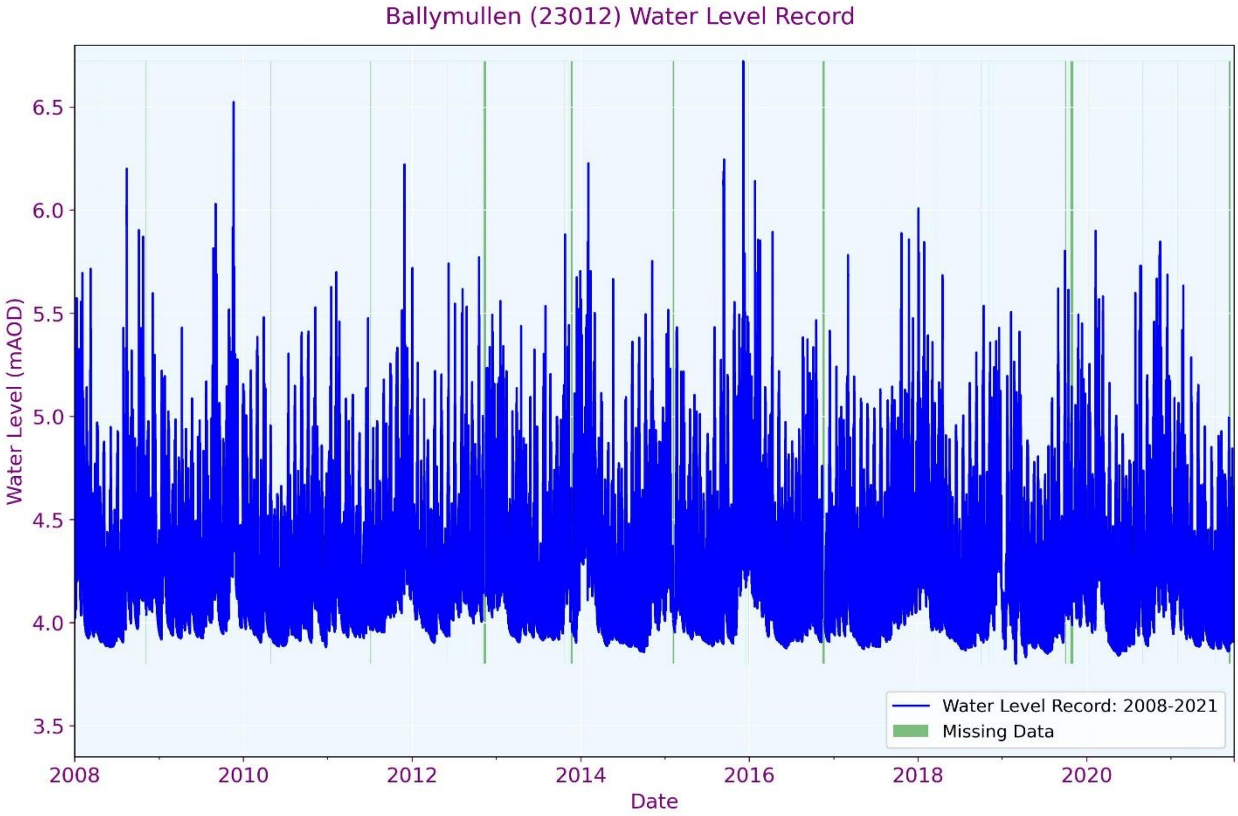


Figure 2.2: River Lee at Ballymullen (23012) Water Level Record from 2008 to 2021

2.1.2 Spot Gauging History

RPS has analysed the spot gauging history at the Ballymullen gauging station (23012). **Figure 2.3** shows the full available spot gauge record from 1974–2021, as received from the OPW. Initial analysis suggested there was a wide spread of spot gaugings across the record period. There is also a shift in the trend of the spot gaugings from 1992 onwards owing to flood relief works between 1992-1996 which may have significantly altered the hydraulic characteristics of the river reach. No rating review has been developed since 1996 and so a new rating review of the Ballymullen gauging station was required in light of the new spot gaugings. Spot gaugings taken prior to 1992 were excluded from the analysis as they are not believed to be representative of present-day hydraulic conditions.

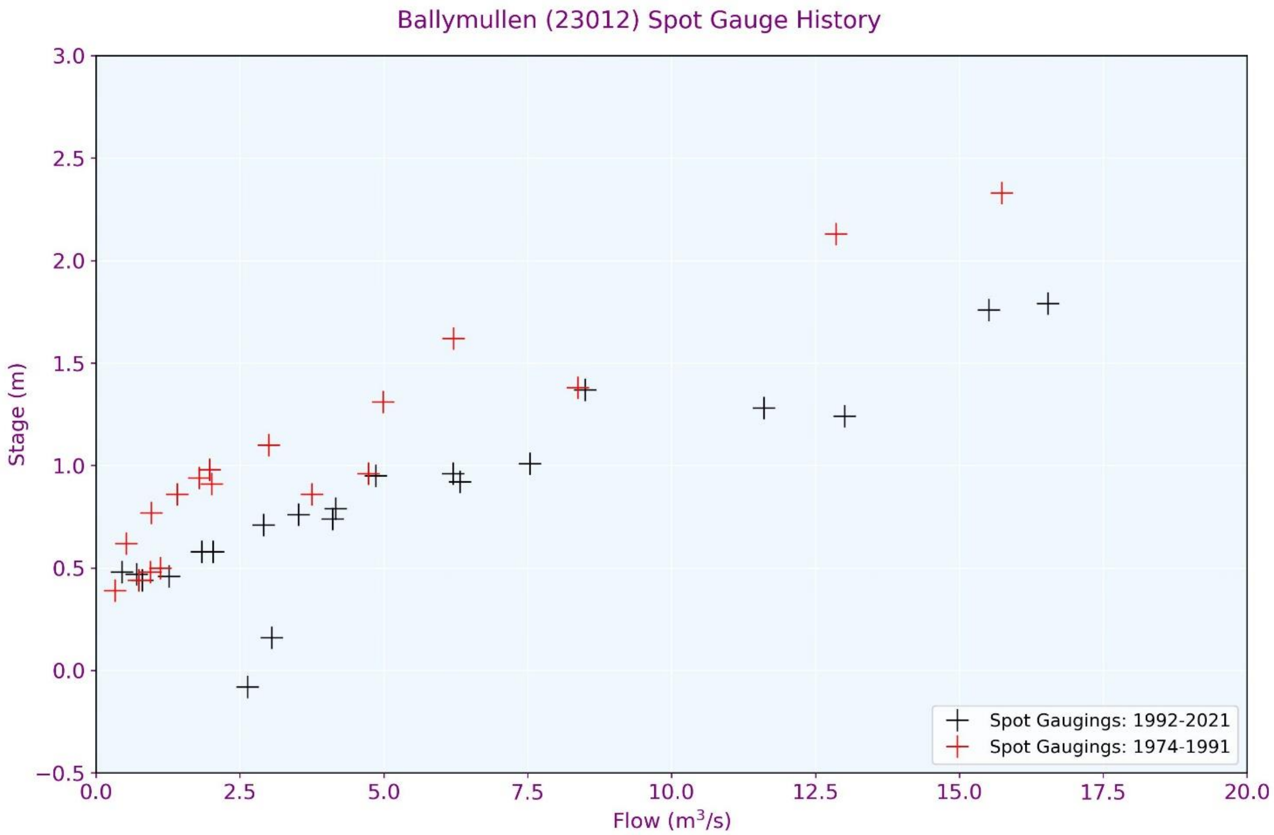


Figure 2.3: Total Spot Gauge History for Ballymullen 23012. Black crosses indicate readings taken during the time period 1974-1991 (pre–Tralee Flood Relief Works), while red crosses indicate readings taken during the time period 1992-2021 (post Tralee Flood Relief Works)

2.1.3 Hydraulic Model Development

The modelled River Lee reach for rating review extends approximately 5 km upstream of the gauging station and 4.9 km downstream of the gauging station, to its outfall at Tralee Bay. The gauging station lies 10 m downstream of a small footbridge and 70 m downstream of Castlemaine Rd Bridge, seen in **Figure 2.4**. The gauge has no downstream structural controls on flow. The land surrounding the gauge is urbanised with residential and commercial buildings.

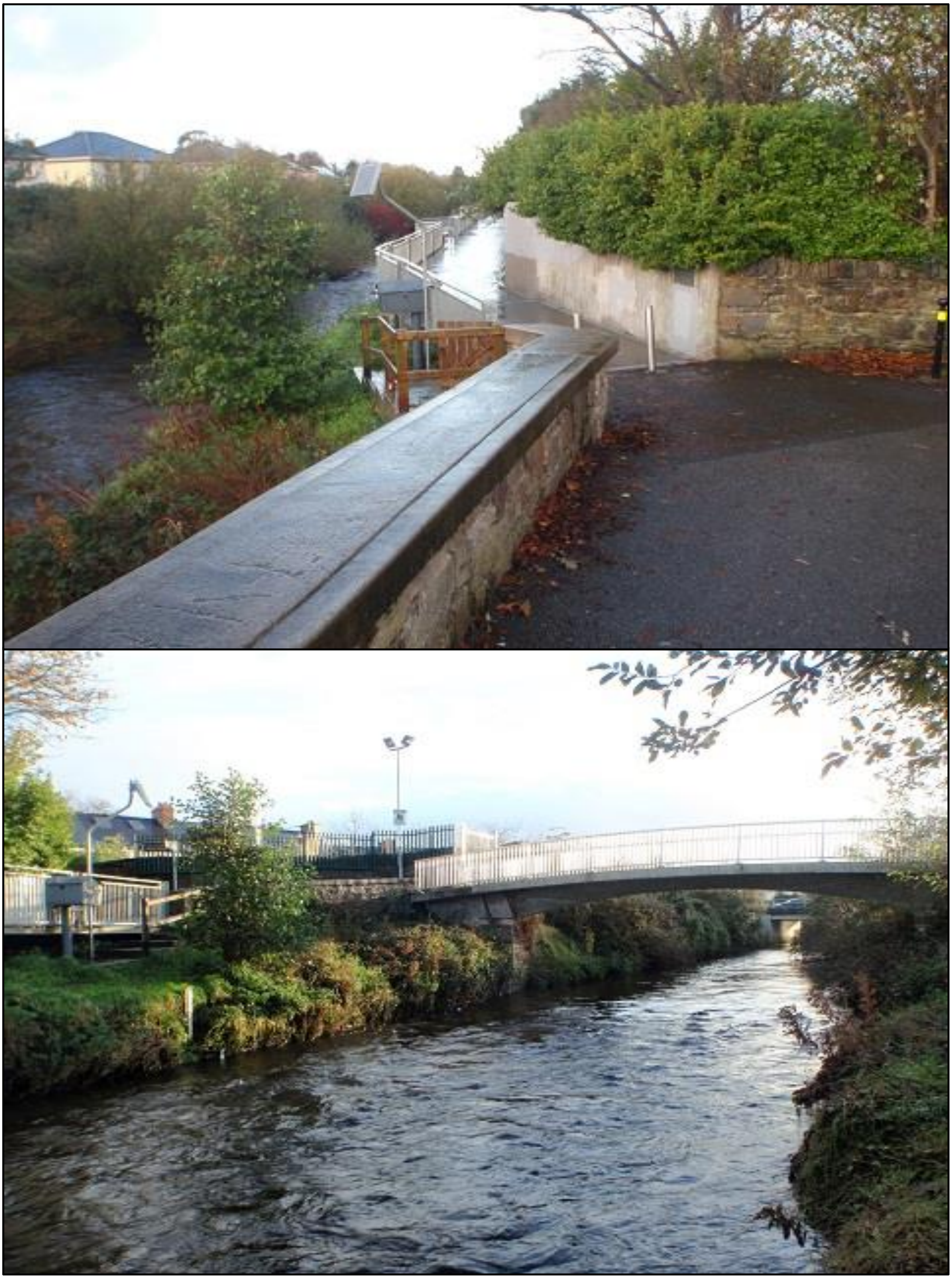


Figure 2.4: (Top) Overview of Ballymullen 23012 Gauging Station. (Bottom) Footbridge and Castlemaine Road Bridge Upstream of the Gauge

The gauged cross-section measures approximately 22 m wide (**Figure 2.5**). As the bridges were considered a potential control on flow at the gauging station, resurvey of both faces of the footbridge and the downstream face of the Castlemaine Rd Bridge were carried out alongside additional cross-sections along the channel to improve data resolution. Survey data obtained during the CFRAM Study of the upstream face of the Castlemaine Rd Bridge was deemed suitable in representing the bridge profile for the purpose of this study and new data was subsequently not obtained for the upstream face of this structure.

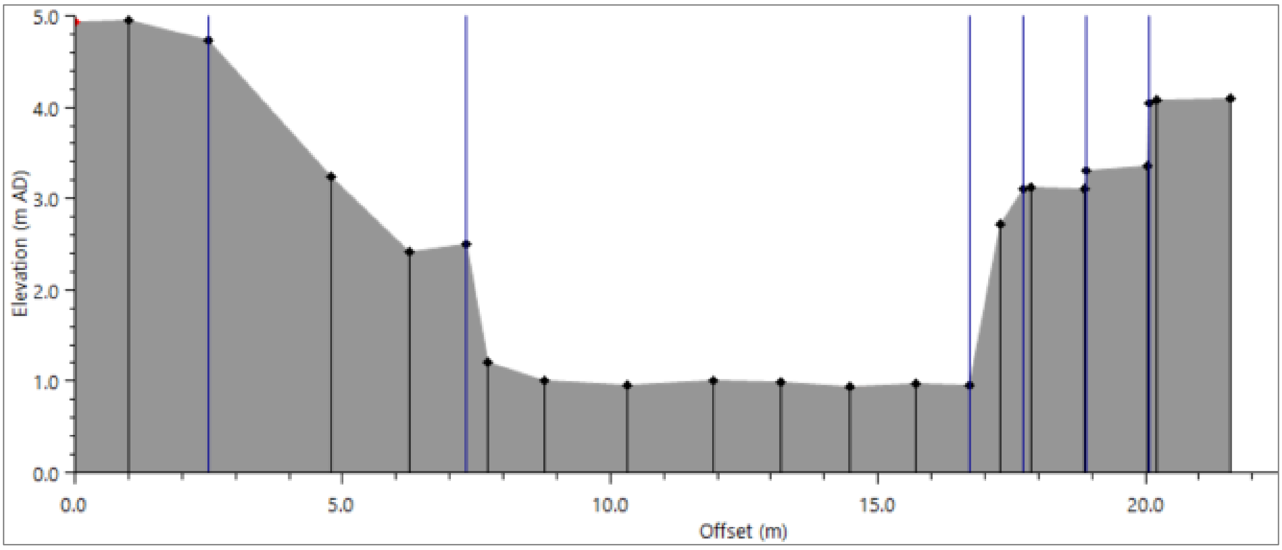


Figure 2.5 Modelled gauge cross-section 23RLEE00220

2.1.4 Hydraulic Model Calibration and Sensitivity

Model calibration involves comparing the existing model Q-h relationship (flow and total water level) with the range of spot gaugings and adjusting model parameters as required, within appropriate bounds, until the Q-h and spot gaugings align. After initial hydraulic modelling, sensitivity tests were carried out on the modelled reach to determine if flow and level outputs closely correspond with OPW spot gauge records. Review of these outputs found that the Q-h relationship at the gauge had a good match to those indicated in the spot gaugings, though some minor adjustments were necessary to develop a realistic and representative model of the channel and flow conditions at Ballymullen.

Calibration is an iterative process that identifies which parameters and physical characteristics are most and least hydraulically significant. By modifying parameters individually and in line with all available information it is possible to identify the impact on the river reach hydraulics of a specific modification through observation of the change in Q-h relationship relative to the observed spot gauging record.

Upper and lower bound scenarios were produced to assess the sensitivity of the final model to changes in parameters. Change in parameters for the lower and upper bound scenarios are relative to the final model.

In the upper bound scenario, the following parameters were adjusted:

- 1D and 2D roughness: Manning's n values were increased by 25%.
- Contraction and expansion coefficients were increased by 50% at the footbridge and Castlemaine Rd Bridge.
- Headloss was increased to 1 at the footbridge and Castlemaine Road Bridge.

In the lower bound scenario, the following parameters were adjusted:

- 1D and 2D roughness: Manning's n values were decreased by 25% at all cross sections.
- Contraction and expansion coefficients were decreased by 50% at the footbridge and Castlemaine Rd Bridge.

For Ballymullen, iterative testing found that neither of the bridges upstream of the gauge were hydraulically significant. Though the footbridge is situated within ~10 m of the gauge board, adjustment of the headloss, discharge coefficient and contraction/expansion were found to have negligible impact on flows at the gauge. This was similarly observed when parameters at the Castlemaine Rd bridge were modified, and these were consequently left unaltered in the final model network. Calibration was primarily achieved by adjusting channel and overbank roughness (Manning's *n*) values within the 1D domain to better reflect the conditions on-site. Increasing the *n* values served to imply a rougher and less-efficient reach condition, steepening the resulting Q-h plot across the range of flows, and generating a slightly higher stage for a given flow. Conversely, by reducing the *n* value (representative of a smoother and more-efficient reach) the steepness of the curve was reduced, with a lower stage observed for the same flow. Modification of the *n* value to both extremes of the permissible range (based on survey images of the bed and banks) across the rating review reach gave some ability to influence the Q-h relationship. Modification of the 2D roughness had a similar effect, though its impact was restricted to the top portion of the rating curve during high flows.

In the final model network, 1D roughness values were increased slightly to replicate the weedy, rocky channel surrounding the gauge. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the River Lee at the gauging station are shown in **Table 2.1**.

Table 2.1: Model Parameters in Each Scenario for the Ballymullen 23012 Rating Review. Change in Manning's (*n*) value and bridge parameters for the upper and lower bound scenarios are relative to the final model

Scenario	Manning's *n* value	Bridge Parameters (23RLEE00223_US_BR.1 and 23RLEE00407D_us_br)	
		Contraction and Expansion Coefficients	Headloss
Upper Bounds	**1D Roughness:** Increased by 25% at all cross sections **2D Roughness:** Increased by 25% to 0.075	**Increased by 50%:** - Contraction loss 0.6 - Reverse flow expansion loss 1.0 - Expansion loss 1.0 - Reverse flow contraction loss 0.6	1
Final Model (Model Q-h)	**1D Roughness:** Section 23RLEE00235: 0.045 (channel) 0.06 (banks) Section 23RLEE00221A**:** 0.045 (channel) 0.05-0.06 (banks) **2D Roughness:** 0.06	**Default:** - Contraction loss 0.3 - Reverse flow expansion loss 0.5 - Expansion loss 0.5 - Reverse flow contraction loss 0.3	0
Lower Bounds	**1D Roughness:** Decreased by 25% at all cross sections **2D Roughness:** Decreased by 25% to 0.045	**Decreased by 50%:** - Contraction loss 0.15 - Reverse flow expansion loss 0.25 - Expansion loss 0.25 - Reverse flow contraction loss 0.15	0

The rating curves for each scenario are presented in **Figure 2.6**, with the upper and lower bound scenario curve represented by a range. A very close match to recorded water level and flow data is evident in the finalised Model Q-h curve, providing strong confidence in model outputs.

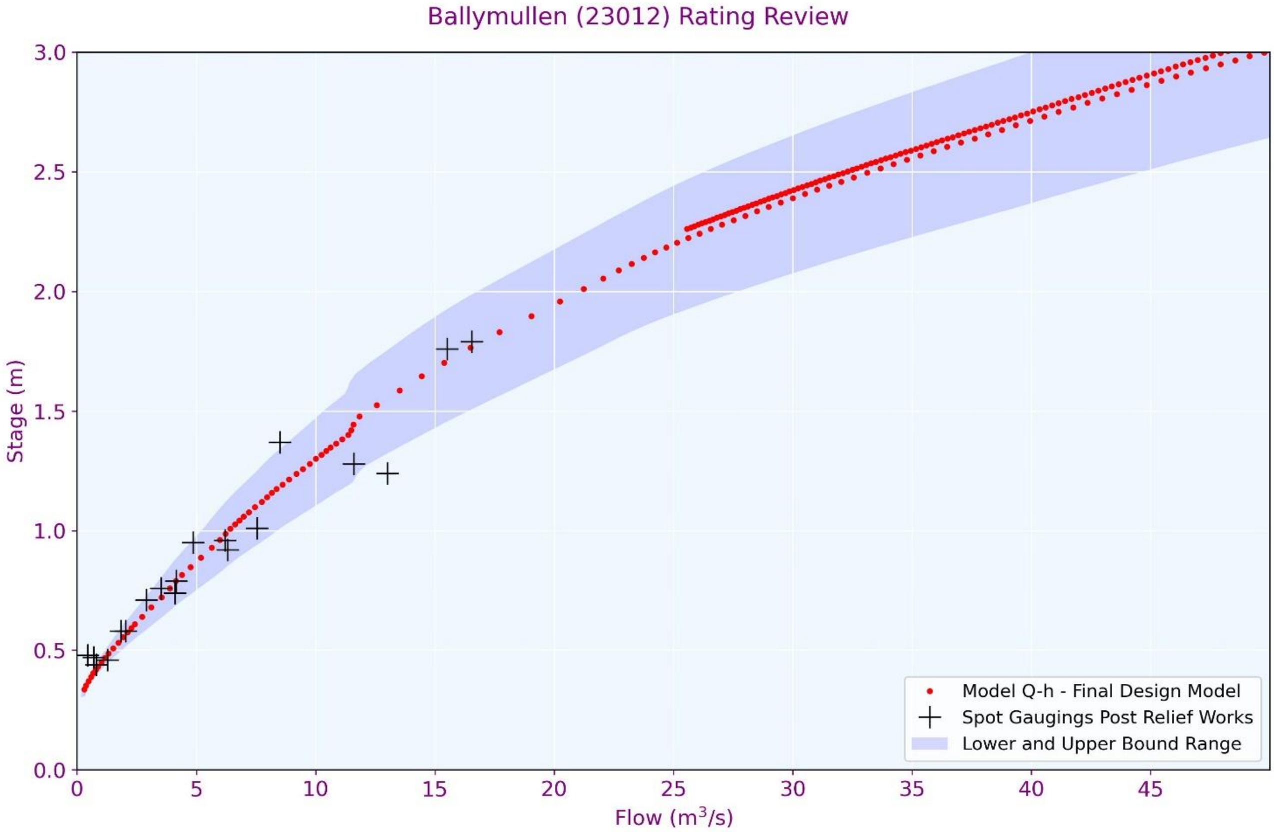


Figure 2.6: Ballymullen (23012) Model Q-h with Upper and Lower Bound Range Included. Spot gaugings post Tralee Flood Relief Works are also shown from 1995-2021, inclusive

2.1.5 Rating Curve Development

Following data collection in conjunction with the OPW Hydrometrics Team, and further analysis of the full spot gauging history, the data suggests that there is a significant shift in rating at the Ballymullen gauging station. It is clear that the Tralee Flood Relief Works which took place between 1992-1996 caused a significant shift in spot gaugings, thus any rating curve developed prior to these works should be considered redundant. As a result, RPS have developed a new rating curve as part of the Tralee FRS from water level data 2008 to 2021. The rating curve is fitted to the model Q-h data, the final design model as described in Section 2.1.4.

2.1.5.1 OPW Rating

The most recently approved OPW rating at the gauging station was derived in 1988. The physical constant rating parameters of the rating equation are shown in **Table 2.2** while the rating is illustrated in **Figure 2.7** along with the RPS final model data to highlight the shift in Q-h relationship at the gauging station.

Table 2.2: OPW Rating Physical Constant Parameters, Ballymullen (1988-1992)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.25	9.00	-0.14	2.22
02	0.25	0.59	9.00	-0.14	2.22
03	0.59	1.12	5.60	-0.14	1.624
04	1.12	1.44	5.60	-0.14	1.624
05	1.44	3.00	5.60	-0.14	1.624

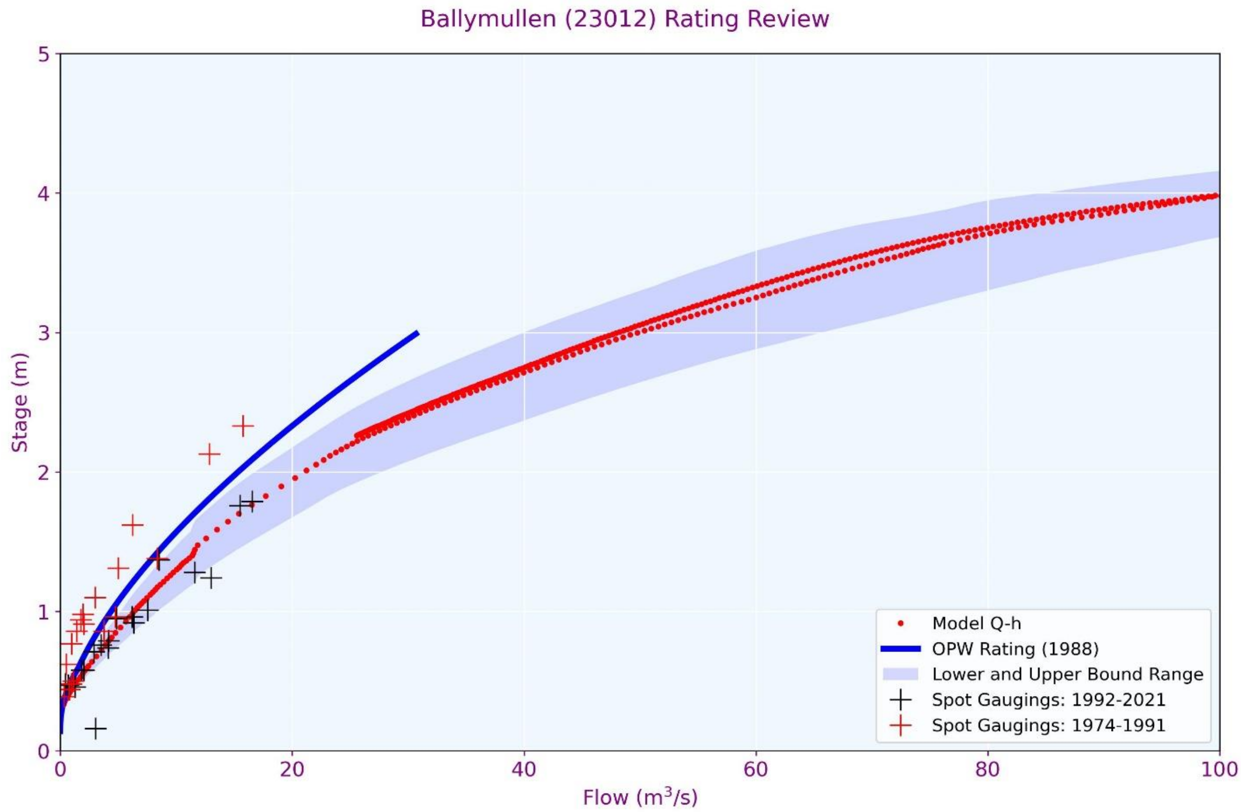


Figure 2.7: Existing OPW Rating Curve (blue) which was Developed in 1988 for Ballymullen (23012). Also shown is the Final Model Design Q-h, including the lower and upper bound scenario range, which highlights the significant shift expected in rating curve

2.1.5.2 RPS 2022 Rating Curve

The proposed RPS rating curve physical constant parameters derived for each limb is shown in **Table 2.3**. The newly proposed rating curve, **Figure 2.8**, shows a good fit to the current hydraulic model of the River Lee at the Ballymullen gauging station. It also provides a good fit to the spot gauging history from 1992-2021.

Table 2.3 Proposed RPS Rating (2022) Physical Constant Parameters, Ballymullen (23012)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	2.00	7.98	-0.15	1.57
02	2.00	3.03	6.9×10^{-4}	4.24	5.65
03	3.03	4.00	3.67×10^{-15}	14.95	12.86
04	4.00	4.48	3.91×10^{-15}	7.79	15.31

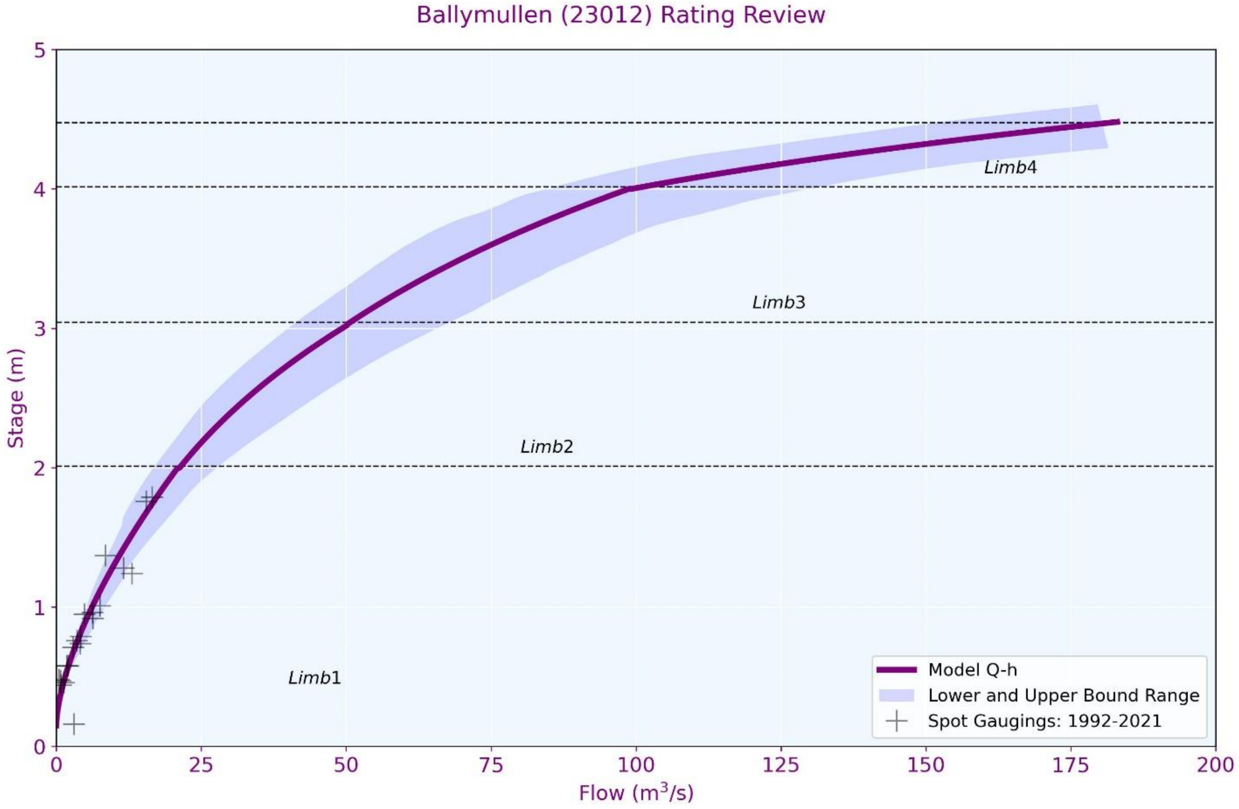


Figure 2.8: Proposed Rating Curve for Ballymullen 23012 Based on Final Model Design Q-h. The specific limbs associated with the RPS rating curve are also highlighted. Included is the upper and lower bound range and spot gauging record post Tralee Flood Relief Works

2.1.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Ballymullen gauging station to inform and validate the hydrological analysis and hydraulic model built for the Tralee FRS.

Initial analysis carried out during the Shannon CFRAM Study showed that water levels at Ballymullen (23012) may be affected by the tide, thus no hydraulic model was completed for the gauging site and no attempt was made to derive a set of rating equations. While RPS have concluded low water levels are influenced by tidal effects at Ballymullen gauging station, for high water levels this tidal fingerprint is effectively removed. As can be seen in **Figure 2.8**, spot gaugings taken at low water levels also appear to support a stable rating curve. While low water levels do have a much higher degree of uncertainty due to the tidal influence, stable spot gauging pattern and well fitted rating curve at lower water levels gives confidence in the rating curve developed. Analysis carried out for the Tralee FRS is concerned with water levels capable of flooding, i.e., high water levels. RPS are satisfied to present the rating curve developed for Ballymullen while stressing the inherit uncertainty associated with rating curves.

A significant shift in spot gaugings was seen pre- and post-Relief Works at Ballymullen, hence the RPS rating curve is based on spot gaugings post-Tralee Flood Relief Works (1996-2021) to ensure the true flood-flow conditions are captured. The OPW's most recent rating at Ballymullen was developed in 1988 and covered the period 1971-1992. To compare the two rating curves is appropriate to capture the physical change in the catchment due to the works and subsequent shift in Q_{med}.

Figure 2.9 compares the results of the proposed RPS rating against the OPW's rating from 1988 at the Ballymullen gauging station and the impact it has on the Q_{med} and subsequent re-processed AMAX series. It should be noted that the AMAX record period 1996-present day was reprocessed through the proposed RPS rating. The time period pre-Tralee Flood Relief Works (pre-1996) was not considered for comparison.

The RPS 2022 proposed rating has resulted in an increase in Q_{med} of +44.94%. This rating has been applied across the record period at the gauging station from 1996 – 2021. It was deemed appropriate to only apply the rating across this record period due to the obvious shift in the rating. The increase in Q_{med} is used to inform the Ballymullen gauging station adjustment factor which is used through the design flow estimation.

The proposed RPS rating has also resulted in much higher flows across the reprocessed AMAX series. The top three AMAX events have increased by 64%, 56%, and 54%, respectively. The OPW rating was only valid up to stage heights of 3.0 m, therefore for events where stage height was greater than 3.0 m the maximum possible flow from the OPW rating curve was used (30.68 m^3/s).

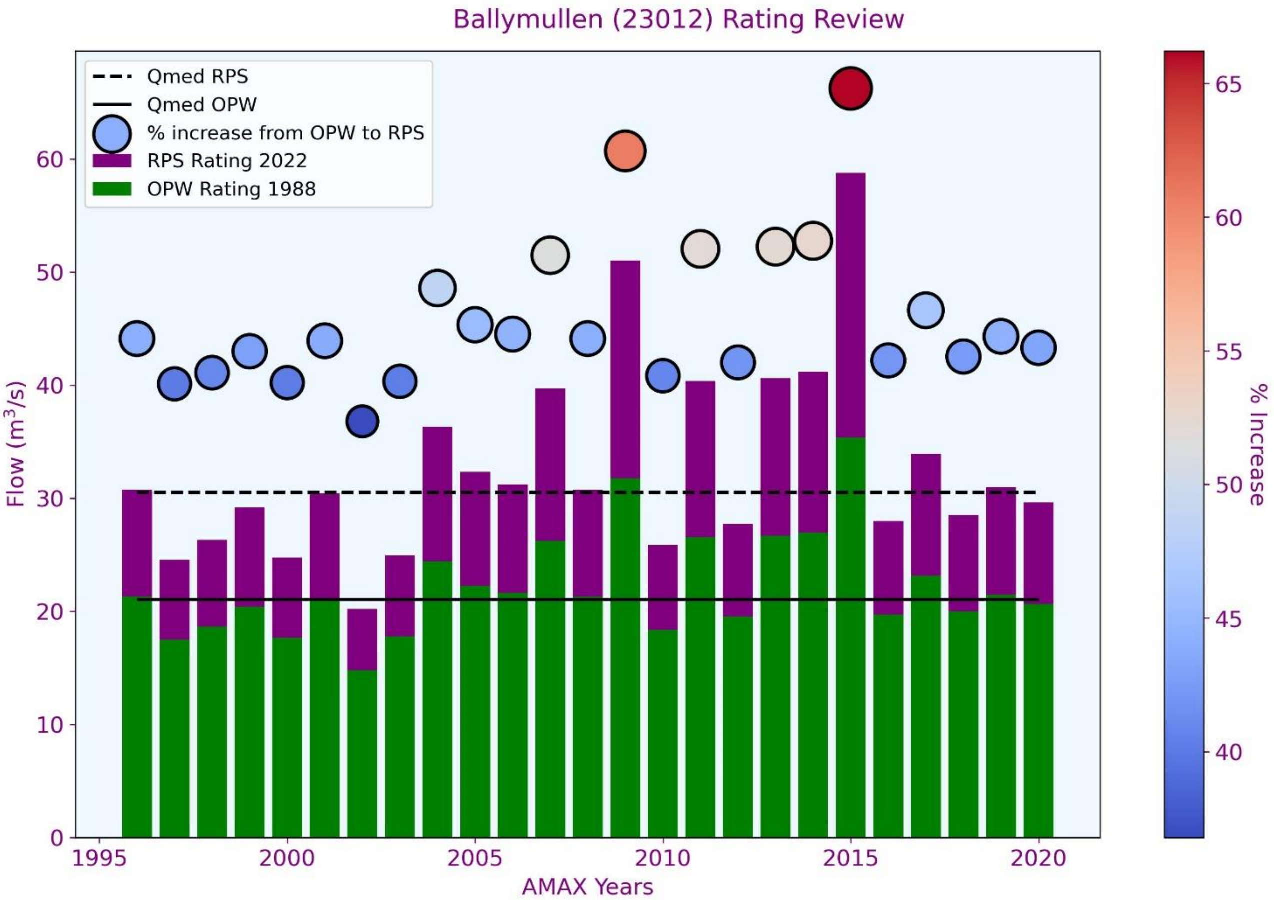


Figure 2.9: AMAX Flows Based on RPS Proposed Rating 2022 (purple) and OPW Rating from 1988. Included is the percentage increase in flows represented by size-dependent markers due to the change in rating, and subsequent Q_{med} from both the OPW and RPS rating for record period 1996-2021

The large percentage increase in flow of the AMAX events is due to a number of reasons. The proposed RPS rating has a less steep trajectory than the current OPW rating (see **Figure 2.10**) which results in the larger high flows at the top end of the proposed RPS rating.

Furthermore, the proposed RPS rating is based on a model of the hydraulic characteristics of the river reach including its floodplain performance. Flattening of the rating curve is expected when water breaks out of bank and flows across the floodplain. As a result, the flow cross-section becomes wider but is reflective of reality. The OPW rating beyond the top of bank is based on an extrapolation of the in-channel Q-h relationship, given that there are no spot gaugings taken during high flows / out of bank flow, and no modelled representation of the floodplain. The flattening effect of the rating curve is not captured. The RPS rating can therefore be considered the rating best anchored to the physical characteristics of the flow cross-section.

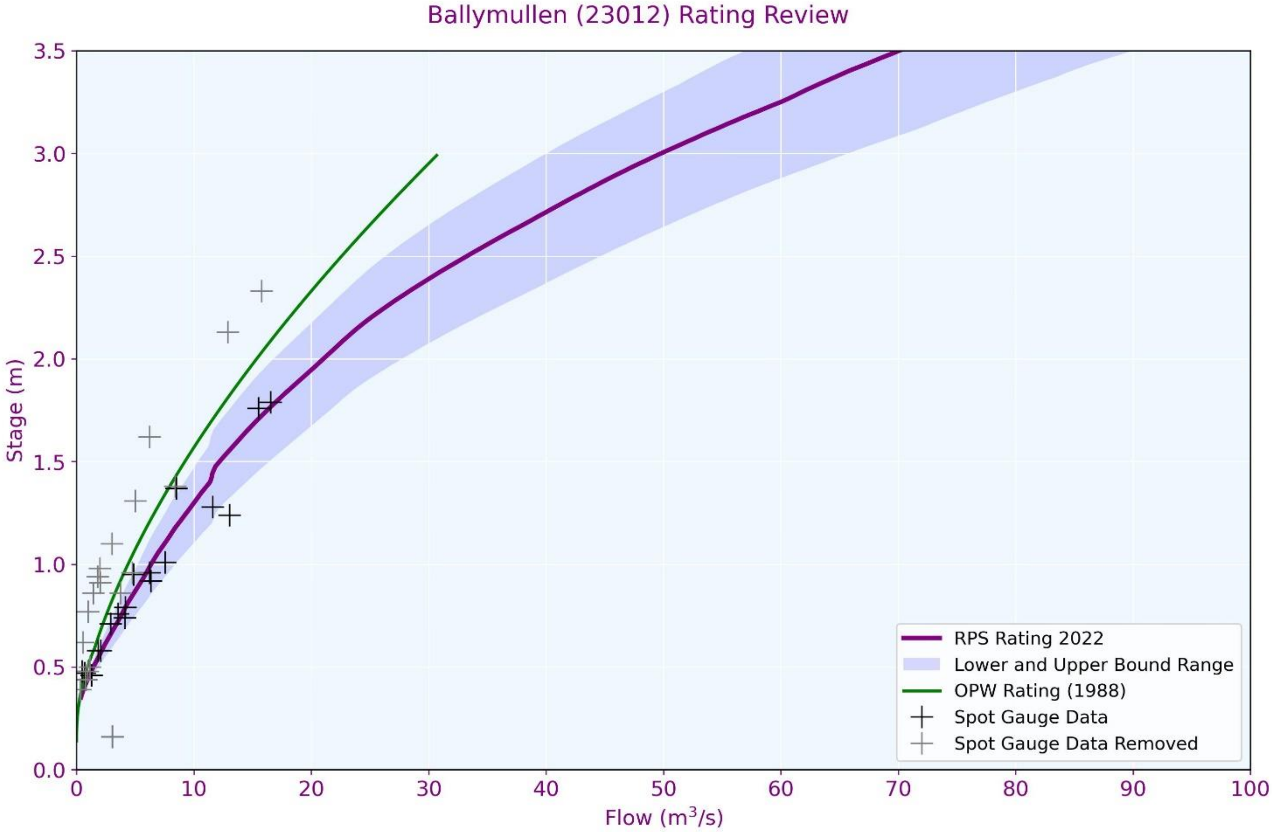


Figure 2.10: OPW Rating (1988) Compared to RPS Proposed Rating (2022) which shows a clear shift to a less steep rating curve which results in higher flows. Also included is the spot gaugings and those which were removed for the development of RPS proposed Rating (2022)

2.2 Lisloose (23044)

An assessment of the rating at Lisloose on the Ballonagh watercourse was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record (taken in May 2020) for the staff gauge zero level is 41.209 mAOD Malin. The survey undertaken as part of this project found the staff gauge zero level to be 41.18 mAOD Malin. The two levels are in moderate agreement with a difference of approximately 30mm. The staff gauge zero level was taken as 41.18 mAOD Malin for this project to ensure consistency with the survey data upon which the model is based.

2.2.1 Raw Data Analysis

Given this station is newly installed and currently has no OPW-derived rating, a review of the available data at the gauging station was conducted. The station records water levels at 15-minute intervals. The station has recorded data available for analysis from the 14th May 2020 to the 1st October 2022 shown in **Figure 2.11**.

There is a gap in the data at the beginning of the record. From the 19th June 2020 to the 21st of June 2020 there is no water level data for the station. While it is unclear what caused this data gap, the data gap occurs in June during which water levels appear low. It is not expected that any significant event was missed over this three-day period. Furthermore, the rest of the record lacks any significant data gaps. RPS deem the station data sufficient to carry out the necessary rating review analysis.

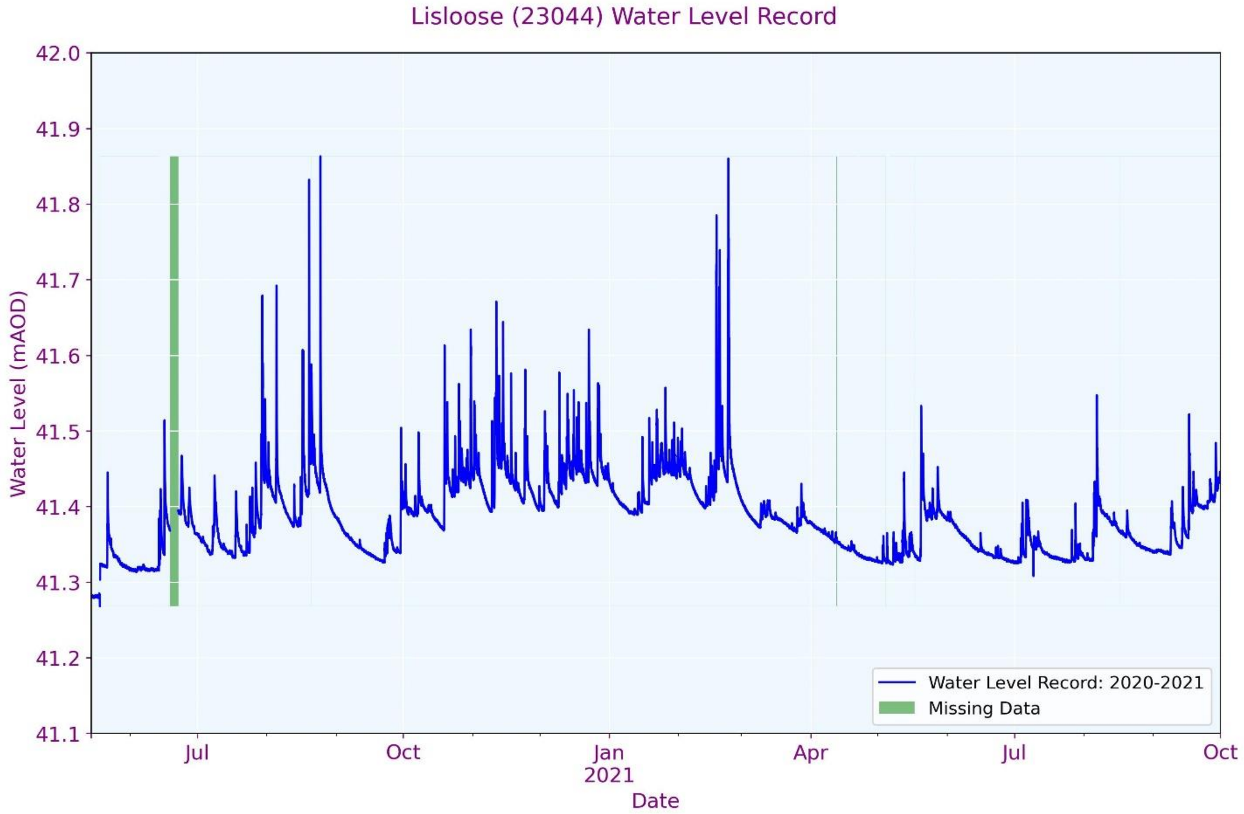


Figure 2.11: Balloonagh Watercourse at Lisloose Gauging Station (23044) Water Level Record from 2020 to 2021

2.2.2 Spot Gauging History

RPS has analysed the spot gauging history at Lisloose (23044). **Figure 2.12** shows the full available spot gauge record from 2020–2021, as received from the OPW. Historic data is limited, and initial analysis suggested there is some scatter in the gaugings across the record period.

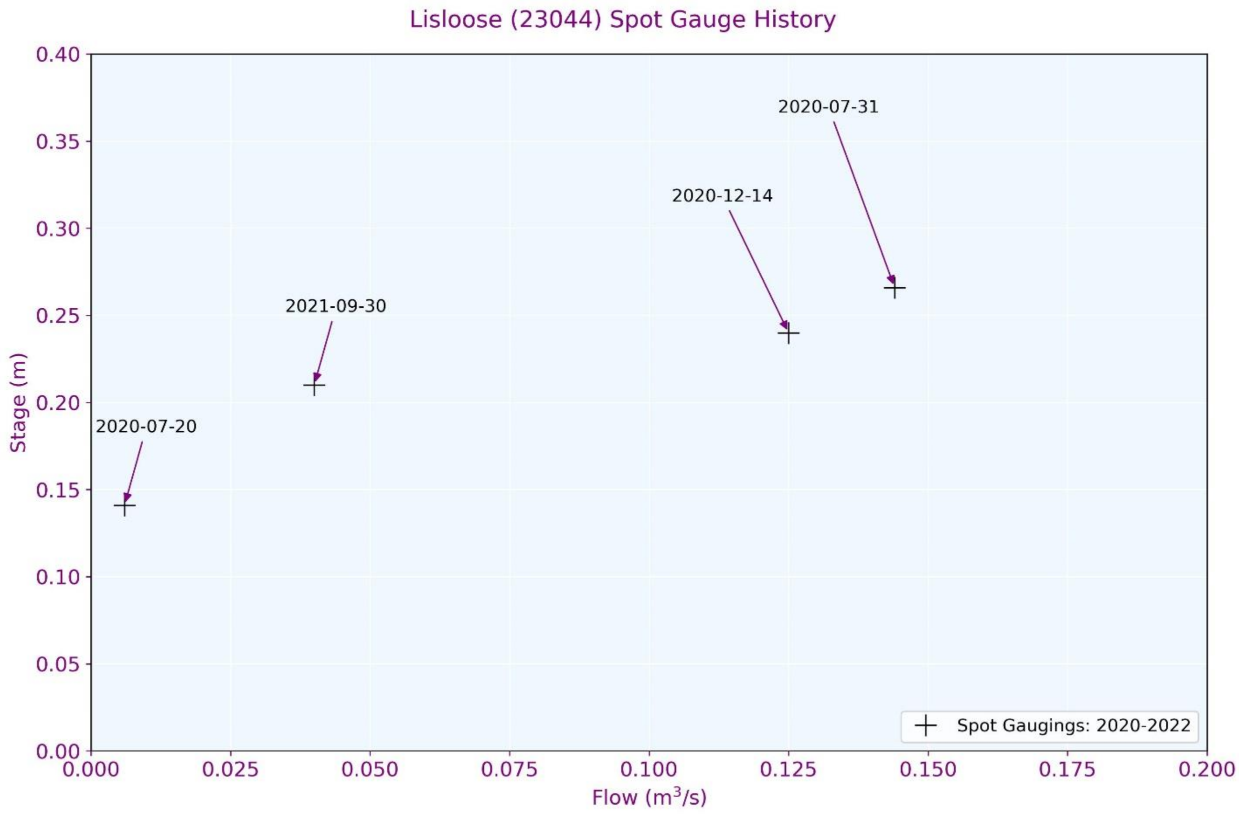


Figure 2.12: Spot Gauging History 2020-2021, Lisloose (23044)

2.2.3 Hydraulic Model Development

The rating review modelling for Lisloose was carried out within the Ballonagh network, which extends approximately 900 m upstream and ~3.5 km downstream from the gauging station to where the extensive Balloonagh culvert (which is approximately 2.7 km in length) discharges into the downstream end of Big River. Though the land upstream of the gauge is predominantly rural, the station is on the outskirts of the Shanakill estate, with residential and commercial buildings just downstream of the site. The gauge itself is situated on the downstream face of the Bullock Hill Bridge (see **Figure 2.13**).

The gauging station cross-section measures approximately 17 m wide, as shown in **Figure 2.14**. New survey data was obtained for the bridge which is considered a potential control on flow at the gauging station. Additional cross-sections along the channel were also captured to improve data resolution.

Figure 2.13: Lisloose Gauging Station at Bullock Hill Bridge Viewed from Downstream

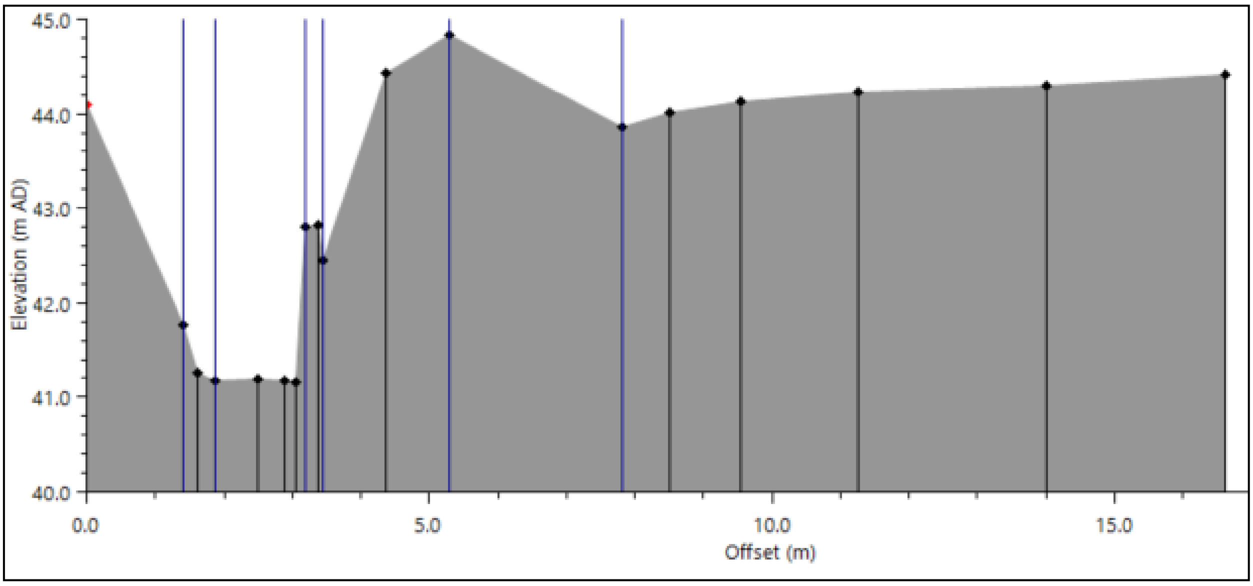


Figure 2.14: Modelled Gauge Cross-Section 23BIGA00220

2.2.4 Hydraulic Model Calibration and Sensitivity

Sensitivity tests were carried out on the Ballonagh to determine if flow and level outputs closely correspond with the OPW spot gauge records. Review of these outputs found that the modelled Q-h relationship at the gauge was around 60-80 mm lower than that of the OPW spot gaugings, and adjustments were necessary to develop a realistic and representative model of the channel and flow conditions at Lisloose.

Upper and lower bound scenarios were also produced to assess the sensitivity of the final model to changes in parameters. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the Ballonagh watercourse are shown in **Table 2.4**. Iterative testing found that Bullock Hill Bridge immediately upstream of the gauge is not hydraulically significant, and hence it was left unaltered in the final model network.

Table 2.4: Model Parameters Adjusted In Each Scenario For The Lisloose Rating Review

Scenario	Manning's *n* value	Bridge Parameters (23BIGA00222_US_BR)	
		Contraction and Expansion Coefficients	**Headloss**
Upper Bounds	Bank and channel Manning's *n* increased by 25% at all cross sections	**Increased by 50%:** - Contraction loss 0.6 - Reverse flow expansion loss 1.0 - Expansion loss 1.0 - Reverse flow contraction loss 0.6	1
Final Model (Model Q-h)	**Sections** **23BIGA00250D_DS_BR, 23BIGA00222_DS_BR.2:** 0.045 (channel) **Sections 23BIGA00188I_DS_BR.2, 23BIGA00193I_DS_CUL.2:** 0.05 (channel)	**Default:** - Contraction loss 0.3 - Reverse flow expansion loss 0.5 - Expansion loss 0.5 - Reverse flow contraction loss 0.3	0
Lower Bounds	Bank and channel Manning's *n* decreased by 25% at all cross sections	**Decreased by 50%:** - Contraction loss 0.15 - Reverse flow expansion loss 0.25 - Expansion loss 0.25 - Reverse flow contraction loss 0.15	0

Calibration was primarily achieved through slight modification of the channel topography. In this instance, the bed was altered to simulate spikes in the bed elevation that were visible on survey long-sections. The bed level was increased by 0.16 m at a cross section 1.5 metres downstream of the gauge to better represent the rocky terrain. This allowed the Q-h relationship to closely align with spot gaugings at low flows. In addition, channel and overbank roughness were increased to reflect the ineffective flow conditions arising due to the combined effects of low discharge and jagged terrain.

The final model data for each scenario (upper and lower scenarios represented by range) are presented in **Figure 2.15**. The final design model provides a reasonably close match to the spot gaugings, though is out by a small margin at two gaugings. This was deemed acceptable as iterative modelling of all surveyed data did not produce a closer fit to the limited historic data available.

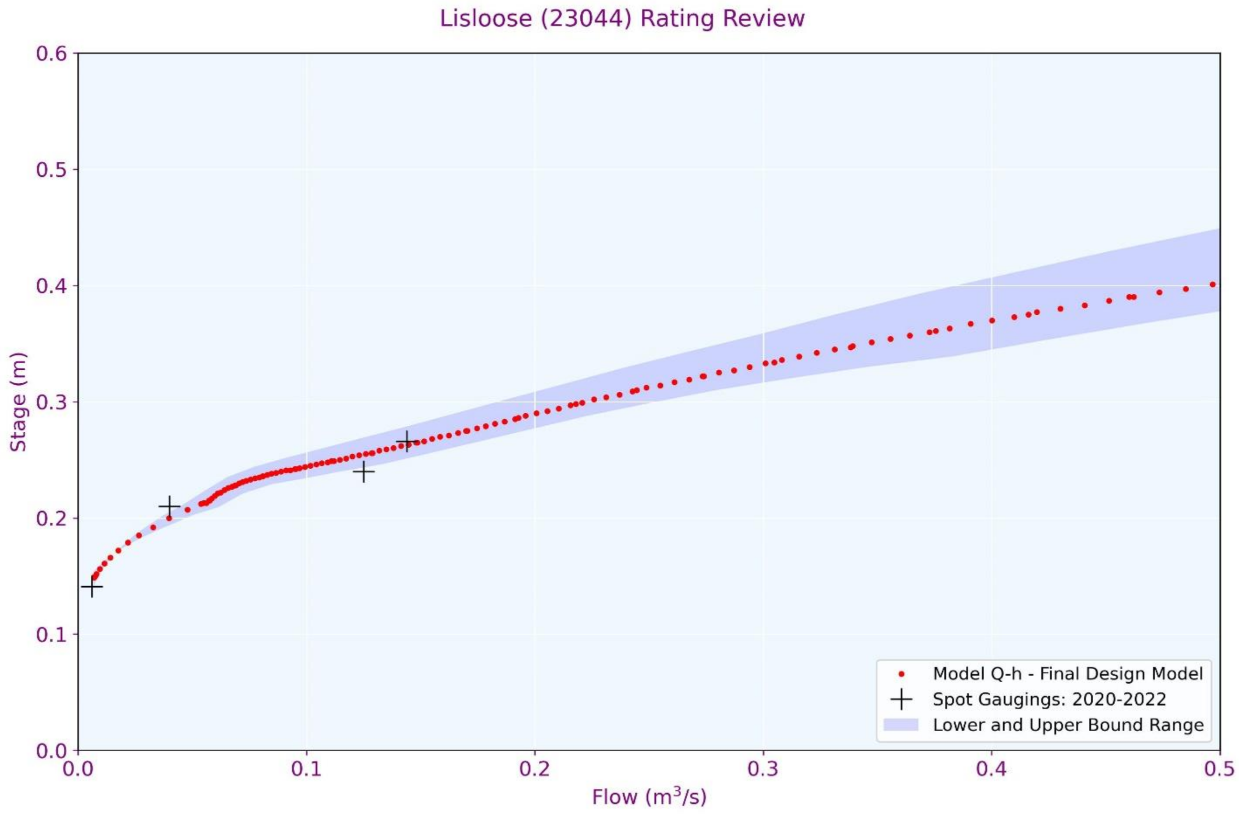


Figure 2.15 Lisloose (32044) model Q-h including lower and upper bound scenario range.

2.2.5 Rating Curve Development

A new rating curve was developed using an iterative process which defined the most appropriate physical parameters to describe the Q-h relationship at the Balloonagh watercourse at Lisloose gauging station. The proposed physical constant parameters derived for each limb of the rating curve are shown in **Table 2.5**, with the respective rating curve shown in **Figure 2.16**.

Table 2.5: Proposed RPS Rating (2022) Physical Constant Parameters, Lisloose (23044)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.23	2.38	-0.13	1.50
02	0.23	0.40	3.17	-0.19	1.21
03	0.40	0.60	3.59	-0.27	0.97
04	0.60	0.73	4.71	-0.17	1.63
05	0.73	0.83	0.22	0.76	5.35
06	0.834	1.05	6.64	-0.50	0.85

The newly proposed rating shows a good fit to the current hydraulic model of the Balloonagh watercourse at the Lisloose gauging station. It provides a good fit to the spot gauging history from 2020 to 2021, however, due ot the lack of historic data uncertainty in the rating curve is high.

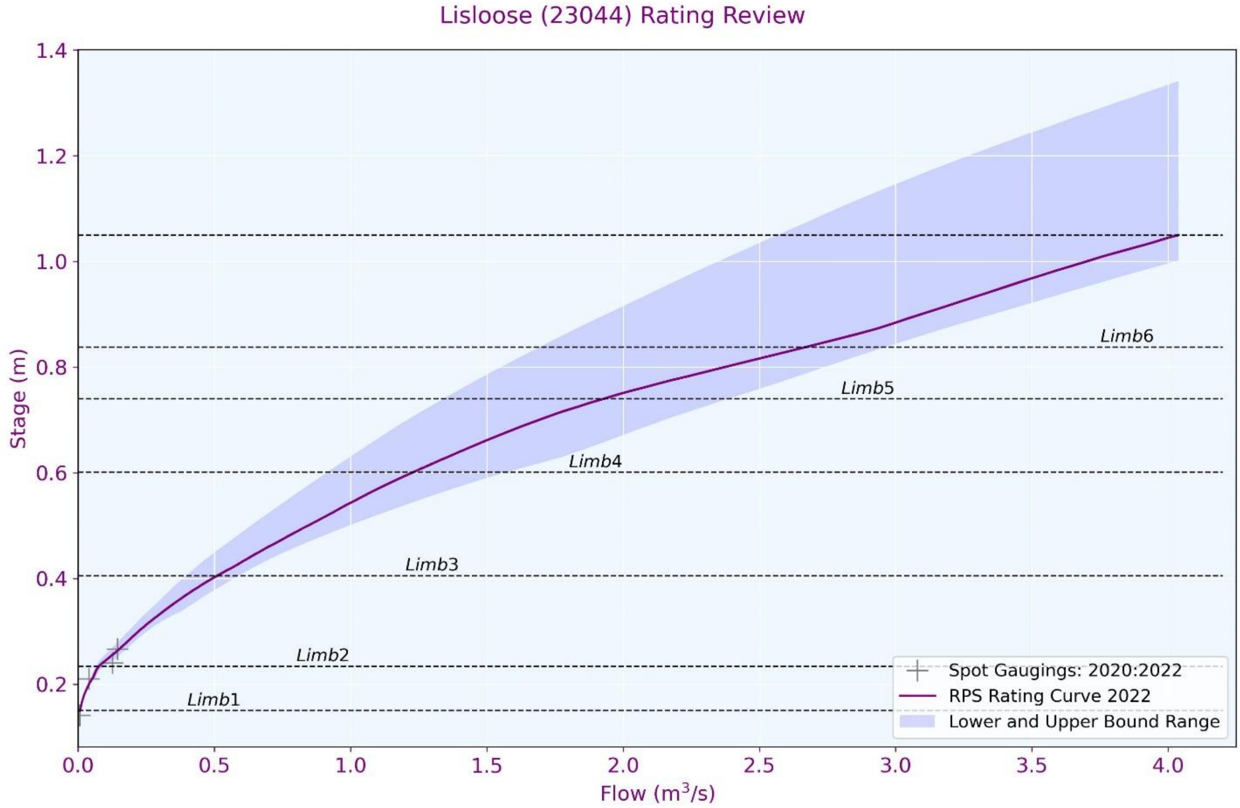


Figure 2.16: Proposed Rating Curve for Lisloose 23044 Based on Final Model Design Q-h. Included is the upper and lower bound range

The development of this rating equation allowed for the extraction of AMAX values for the two available hydrometric years (2019 and 2020). The two-year AMAX series that was processed is shown in **Table 2.6**.

Table 2.6: Processed Two-Year AMAX for Balloonagh Watercourse at Lisloose (23044) using Proposed RPS Rating Curve 2022

Hydrometric Year	AMAX Record Date	S.G. Reading (m)	Estimated Flow (m^3/s)
2019	25/08/2020	0.68	1.62
2020	23/02/2021	0.68	1.58

The processed two-year AMAX at Lisloose using RPS rating curve 2022 produces a Q_{med} estimate of 1.60 m^3/s which is approximately 8% below the initial design flow estimate (1.71 m^3/s) derived from the FSU 5-variable regression equation. The estimation of this Q_{med} based on AMAX data over a two-year data period is highly uncertain but offers an indication that the proposed RPS rating curve captures the physical behaviour of the gauged catchment at Lisloose.

2.2.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Lisloose gauging station to inform and validate the hydrological analysis and hydraulic model build for the Tralee FRS. While close agreement exists between Q_{med} derived from the rating and that obtained from the FSU5 variable equation, caution is advised due to the very short record.

2.3 Ballyseedy (23045)

Located roughly 3.9 km upstream of the Ballymullen (23012) gauging station, an assessment of the rating at Ballyseedy (23045) on the River Lee was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record for the staff gauge zero level is 10.928 mAOD Malin. The survey undertaken as part of this project found the staff gauge zero level to be 10.900 mAOD Malin. The two levels are in moderate agreement with a difference of less than 30mm. The staff gauge zero level was taken as 10.900 mAOD Malin for this project to ensure consistency with the survey data upon which the model is based.

2.3.1 Raw Data Analysis

Given this station is newly installed and currently has no OPW-derived rating, a review of the available data at the gauging station was conducted.

The station records water levels at 15-minute intervals. The station has recorded data available for analysis from 26th June 2020 to the 1st October 2022 shown in **Figure 2.17**. There are no significant data gaps within the record period.

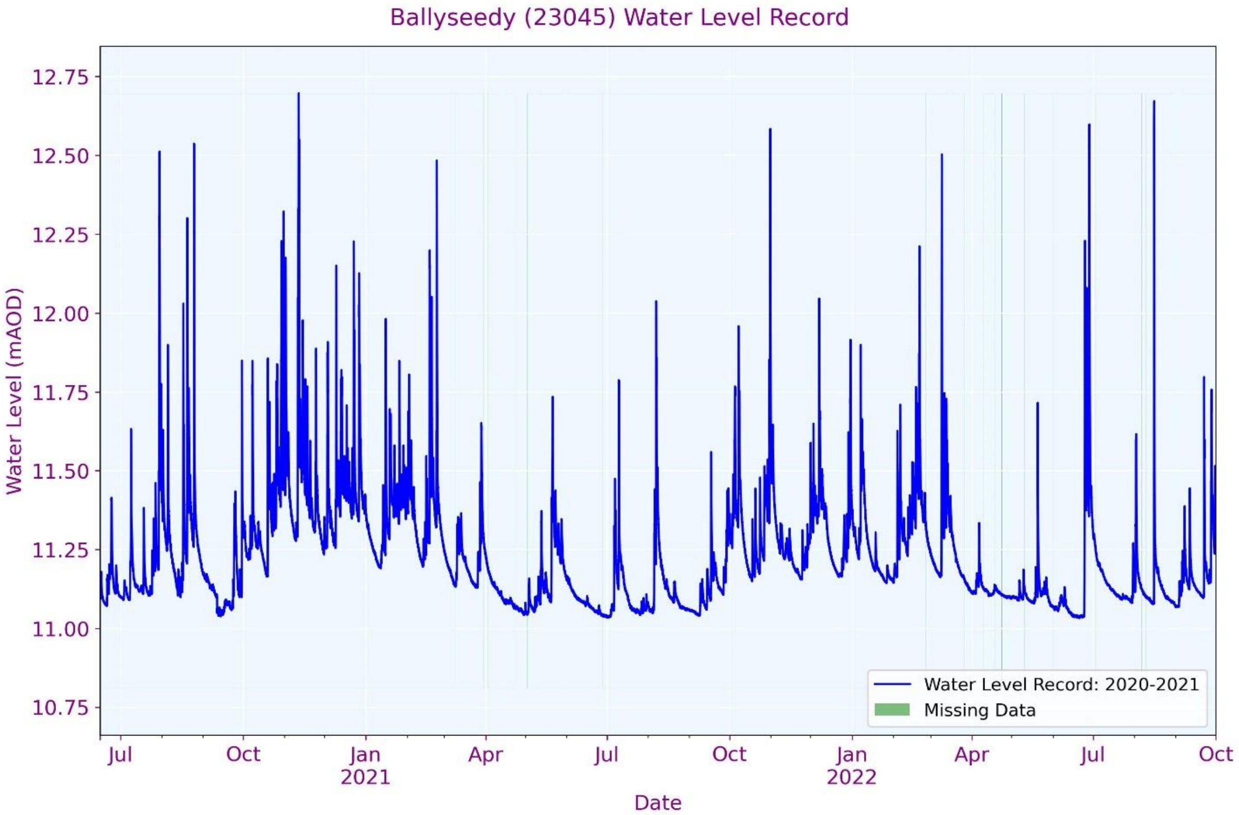


Figure 2.17: River Lee at Ballyseedy (23045) Water Level Record from 2020 to 2022

2.3.2 Spot Gauging History

RPS analysed the spot gauging history at the Ballyseedy gauging station. As the gauge was installed in June 2020, the OPW have not developed a rating curve at this site, with just five spot gaugings available. A new rating review of the Ballyseedy gauging station has been conducted by RPS in light of this. **Figure 2.18** shows the full available spot gauge record, dating from July 2020 to February 2022. Initial analysis suggests the highest flow gauging is anomalous, however, this cannot be said with strong confidence given the limited amount of gaugings to date.

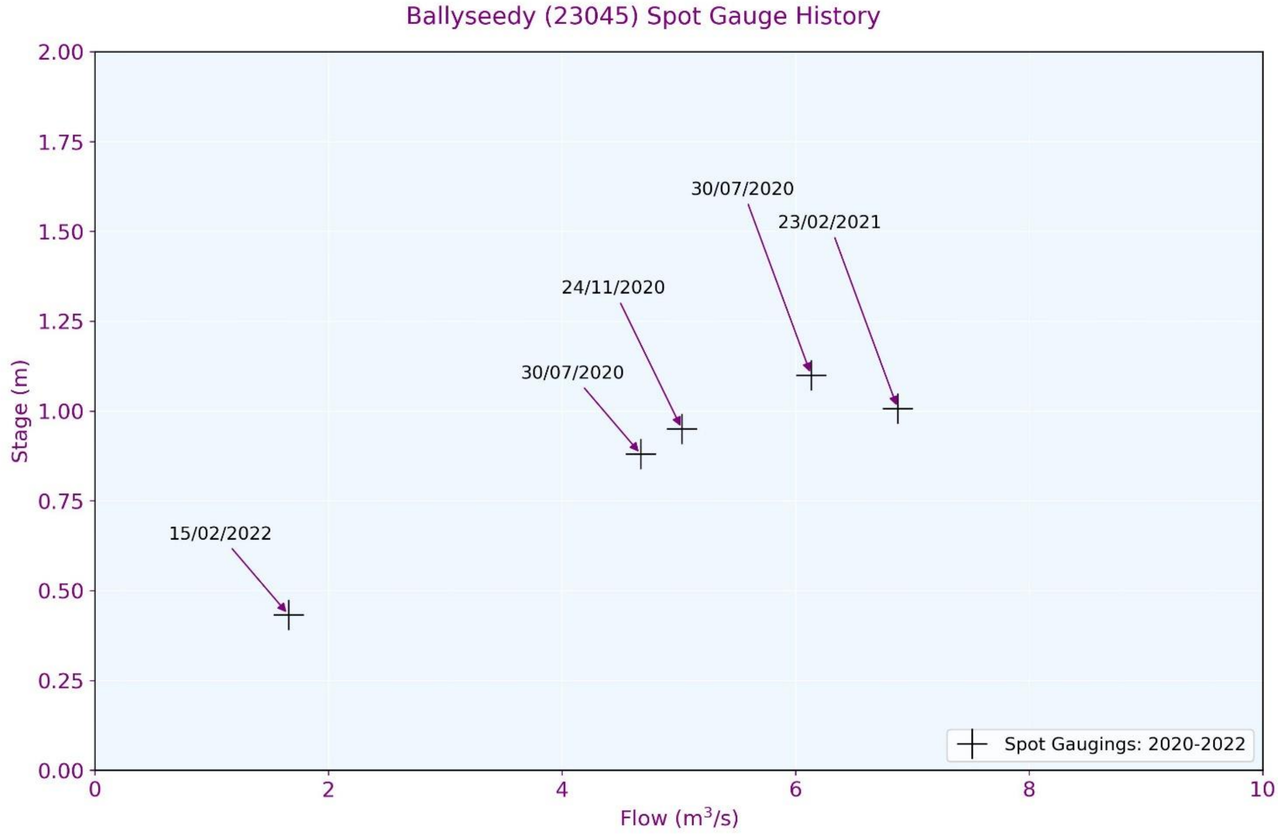


Figure 2.18: Spot Gauging History 2020-2022, Ballyseedy

2.3.3 Hydraulic Model Development

The Lee river reach for the rating review extends approximately 1.1 km upstream of the Ballyseedy gauging station and 9.2 km downstream of the gauging station, to its outfall at Tralee Bay. The gauge is situated on the upstream face of the Ballyseedy Rd (L6547) Bridge and is located approximately 40 m upstream of another small bridge on a private road (**Figure 2.19**). Though the land surrounding the gauge is largely rural, there are some residential and commercial buildings nearby.

Figure 2.19: Ballyseedy Road Bridge (left) and the Small Bridge Located 40 m Downstream (right)

The gauging cross-section measures approximately 13 m in width (see **Figure 2.20**). As the bridges were considered a potential control on flow at the gauging station, resurvey of both bridges was carried out alongside procurement of additional cross-sections along the channel to improve data resolution.

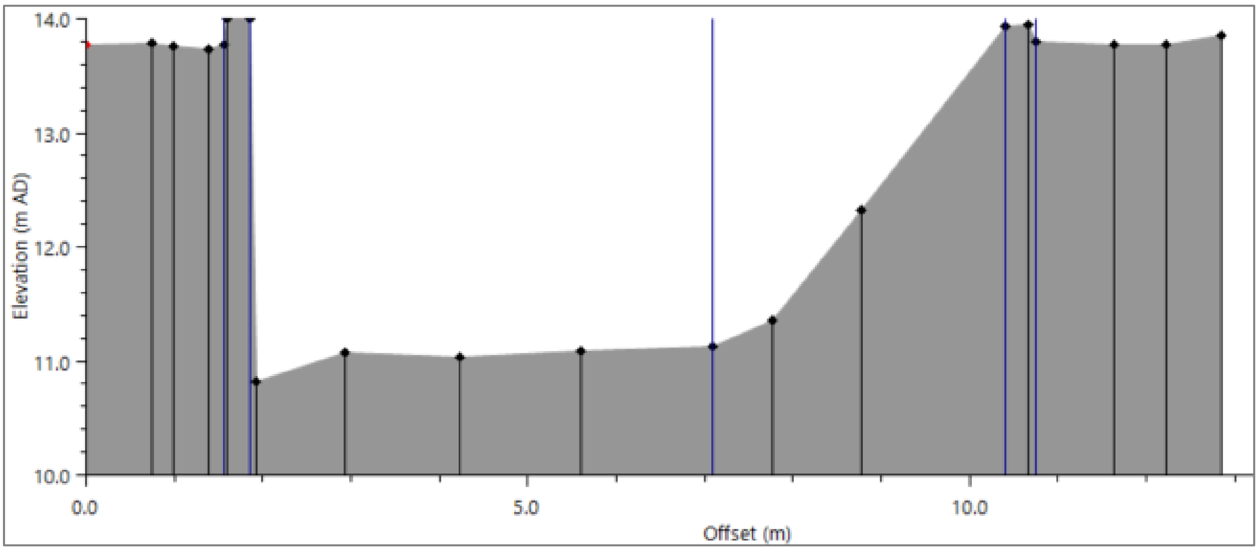


Figure 2.20: Modelled Gauge Cross-Section 23RLEE0100220

2.3.4 Hydraulic Model Calibration and Sensitivity

After initial hydraulic modelling, sensitivity tests were carried out on the Lee to determine if flow and level outputs closely correspond with OPW spot gauge records at Ballyseedy. Review of these outputs found that the Q-h relationship at the gauge had a good match to two out of the five spot gauges, and adjustments were necessary to develop a realistic and representative model of the channel and flow conditions at Ballyseedy.

As with Ballymullen, upper and lower bounds scenarios were also produced to assess the sensitivity of the final model to changes in parameters. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the River Lee at the gauging station are shown in **Table 2.27**. Iterative testing found that neither of the bridges downstream of the gauge were hydraulically significant at low flows. Though the gauge board is located on the upstream face of the Ballyseedy Rd Bridge, adjustment of the headloss, discharge coefficient and contraction/expansion were found to have negligible impact on flows at the gauge. This was similarly observed when parameters at the small bridge ~40 m downstream were modified, and these were consequently left unaltered in the final model network. Calibration was primarily achieved by increasing Manning's *n* values to better reflect the vegetation growth and develop a realistic model of the channel and flow conditions which is validated by the spot gaugings.

Table 2.7: Model Parameters Adjusted in Each Scenario for the Ballyseedy (23045) Rating Review

Scenario	Manning's *n* value	Bridge Parameters (23RLEE0100220_US_BR.1 and 23RLEE0100206_US_BR.1)	
		Contraction and Expansion Coefficients	Headloss
Upper Bounds	**1D Roughness:** Increased by 25% at all cross sections **2D Roughness:** Increased by 25% to 0.075	**Increased by 50%:** - Contraction loss 0.6 - Reverse flow expansion loss 1.0 - Expansion loss 1.0 - Reverse flow contraction loss 0.6	1
Final Model (Model Q-h)	**1D Roughness:** Section 23RLEE0100218: 0.055 (channel) Section RLEE0100104_2022: 0.045 (channel) **2D Roughness: 0.06**	**Default:** - Contraction loss 0.3 - Reverse flow expansion loss 0.5 - Expansion loss 0.5 - Reverse flow contraction loss 0.3	0
Lower Bounds	**1D Roughness:** Decreased by 25% at all cross sections **2D Roughness:** Decreased by 25% to 0.045	**Decreased by 50%:** - Contraction loss 0.15 - Reverse flow expansion loss 0.25 - Expansion loss 0.25 - Reverse flow contraction loss 0.15	0

The final model data for each scenario (upper and lower scenarios represented by range) are presented in **Figure 2.21**. A reasonably close match is found with three of the spot gaugings, though iterative modelling of all surveyed data proved that false alterations of channel parameters would be required to manipulate the gradient of the curve in order to meet the low flow spot gauging. Attempts to simulate a Q-h relationship with this lower point were consequently abandoned.

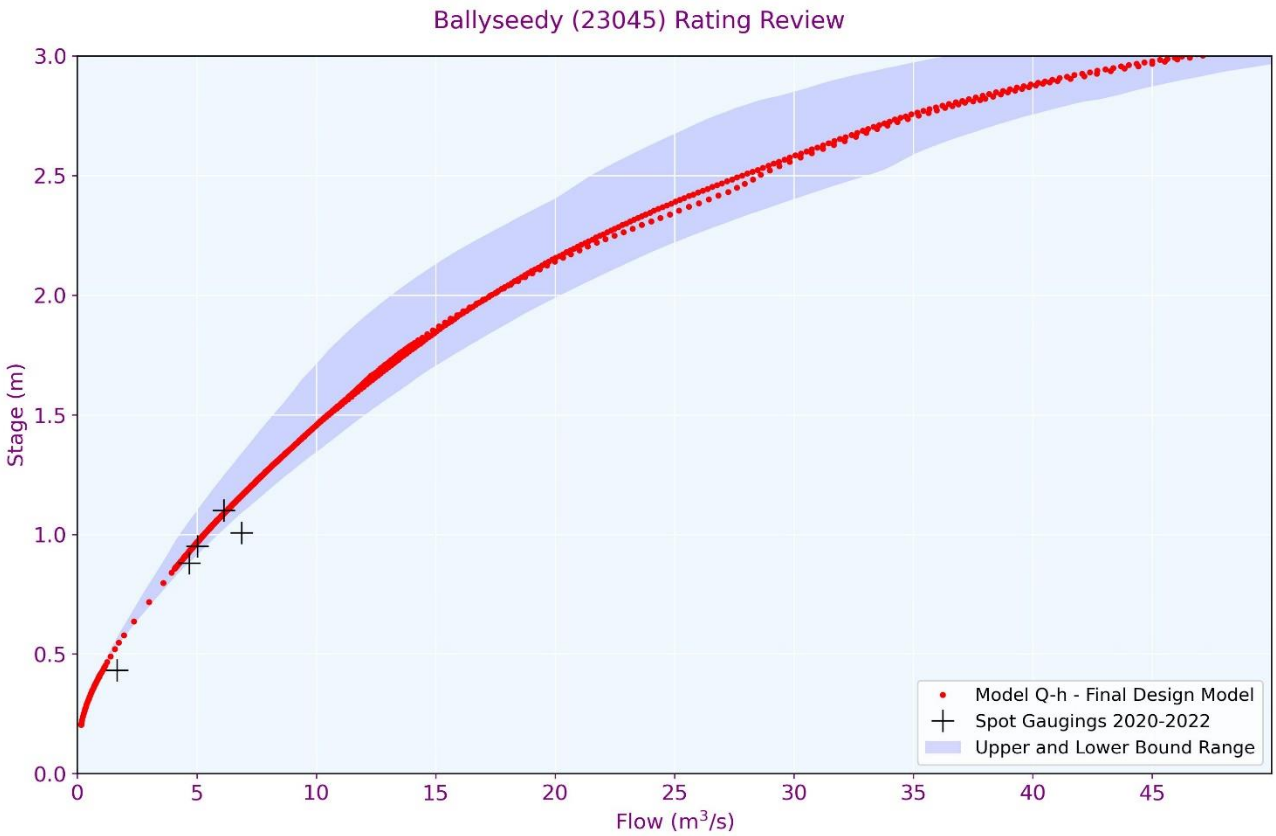


Figure 2.21: Ballyseedy (23045) Model Q-h with Upper and Lower Bounds Curves Included

2.3.5 Rating Curve Development

A new rating curve was developed using an iterative process which defined the most appropriate physical parameters to describe the Q-h relationship at the River Lee at Ballyseedy gauging station. The proposed physical constant parameters derived for each limb of the rating curve are shown in **Table 2.8**, with the respective rating curve shown in **Figure 2.22**. The newly proposed rating shows a good fit to the current hydraulic model of the River Lee at the Ballyseedy gauging station.

Table 2.8: Proposed RPS Rating (2022) Physical Constant Parameters, Ballyseedy (23045)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.91	6.64	-0.22	1.12
02	0.91	1.51	2.61	0.36	2.22
03	1.51	2.46	2.40×10^{-3}	3.35	5.30
04	2.46	2.87	3.85×10^{-15}	12.14	13.61
05	2.87	3.15	45.89	-2.20	1.10

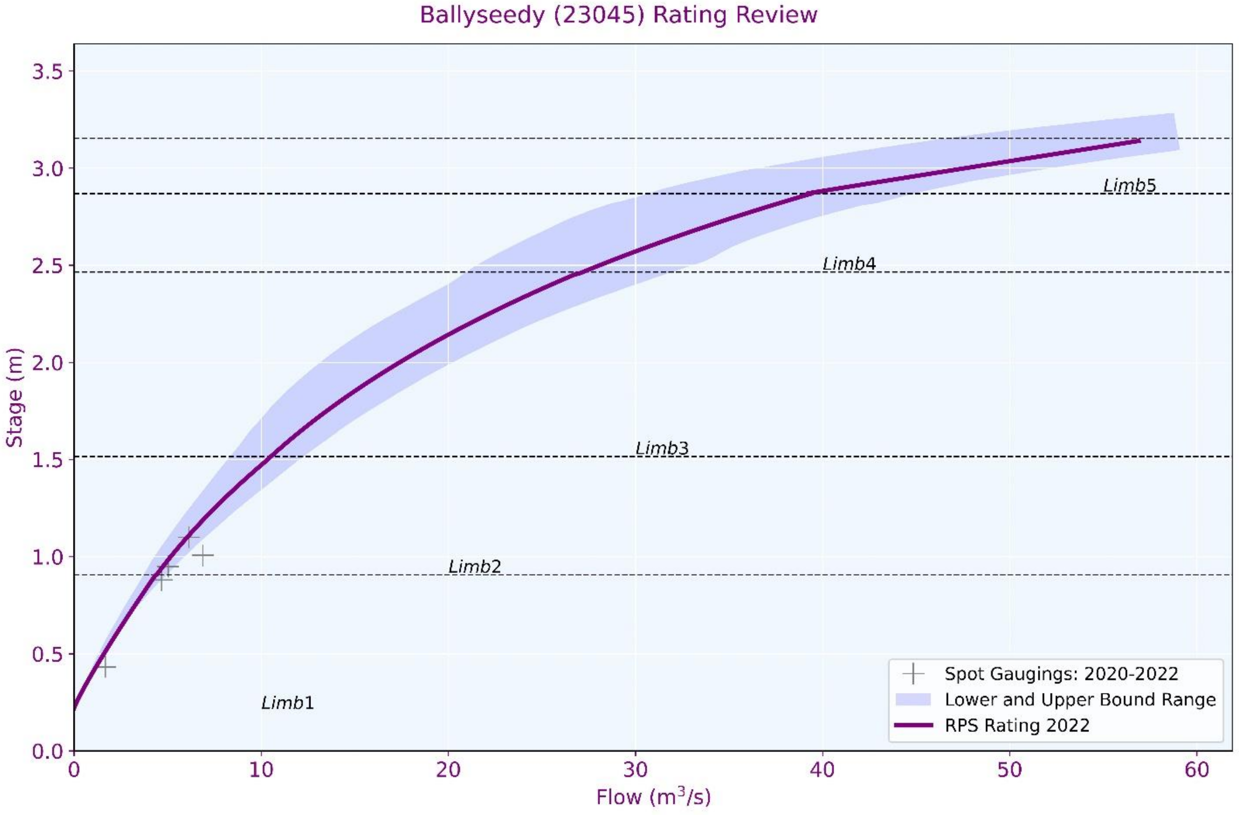


Figure 2.22: Proposed Rating Curve for Ballyseedy 23045 Based on Final Model Design Q-h. Included is the upper and lower bound range.

The development of this rating equation allowed for the extraction of AMAX values for the three available hydrometric years (2019, 2020, and 2021). The three-year AMAX series that was processed is shown in **Table 2.9**.

Table 2.9: Processed Three-Year AMAX for River Lee at Ballyseedy (23045) Using Proposed RPS Rating Curve 2022

Hydrometric Year	AMAX Record Date	S.G. Reading (m)	Estimated Flow (m^3/s)
2019	25/08/2020	1.64	12.03
2020	11/11/2020	1.79	14.22
2021	15/08/2022	1.77	13.78

The processed three-year AMAX at Ballyseedy using RPS rating curve 2022 produces a Q_{med} estimate of 13.34 m^3/s which is approximately 3.5% below the initial design flow estimate (13.83 m^3/s) derived from the FSU7 variable regression equation. The estimation of this Q_{med} based on AMAX data over a three-year data period is highly uncertain but offers an indication that the proposed RPS rating curve captures the physical behaviour of the gauged catchment at Ballyseedy.

2.3.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Ballyseedy gauging station to inform and validate the hydrological analysis and hydraulic model build for the Tralee FRS. While close agreement exists between Q_{med} derived from the rating and that obtained from the FSU7 variable equation, caution is advised due to the short AMAX record.

2.4 Spring Water Lane (23046)

An assessment of the rating at Spring Water Lane on the Lissardboola watercourse was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record (taken in June 2020) for the staff gauge zero level is 2.53 mAOD Malin. The survey undertaken as part of this project found the staff gauge zero level to be 2.58 mAOD Malin. There is a significant difference of 50 mm between the two surveyed levels. The most recently surveyed staff gauge zero level of 2.58 mAOD was used to complete the rating review as any discrepancy is thought to be relative and this level is consistent with the survey data from which the hydraulic model is constructed.

2.4.1 Raw Data Analysis

Given this station is newly installed and currently has no OPW-derived rating, a review of the available data at the gauging station was conducted.

The station records water levels at 15-minute intervals. The station has recorded data available for analysis from 26th June 2020 to the 1st October 2021 shown in **Figure 2.23**. There are no significant data gaps within the record period.

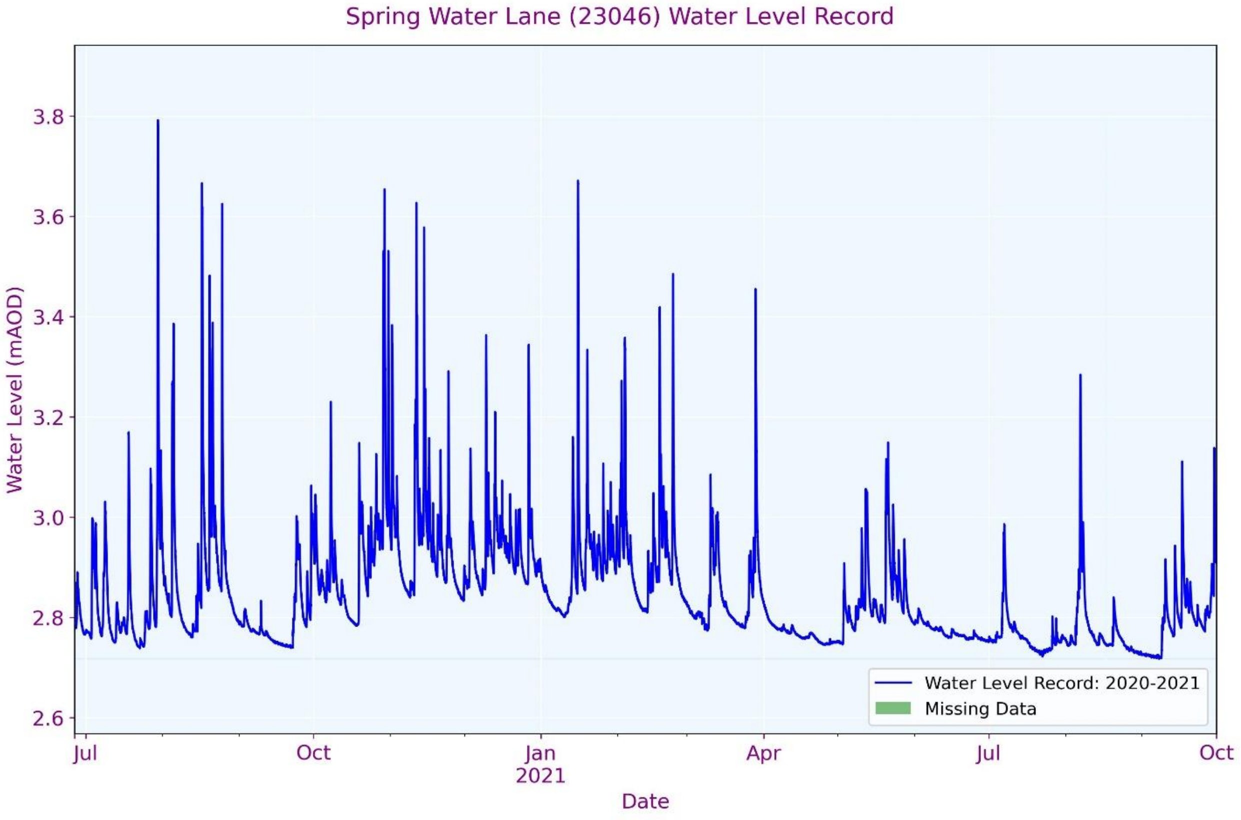


Figure 2.23: Lissardboola Watercourse at Spring Water Lane (23046) Water Level Record from 2020 to 2021

2.4.2 Spot Gauging History

RPS has analysed the spot gauging history at the Spring Water Lane gauging station (23046). **Figure 2.24** shows the full available spot gauge record from 2020–2022, as received from the OPW. Historic data is limited, and initial analysis suggested there is substantial scatter in the gaugings to date. Despite these data limitations, RPS has produced a rating curve for the site.

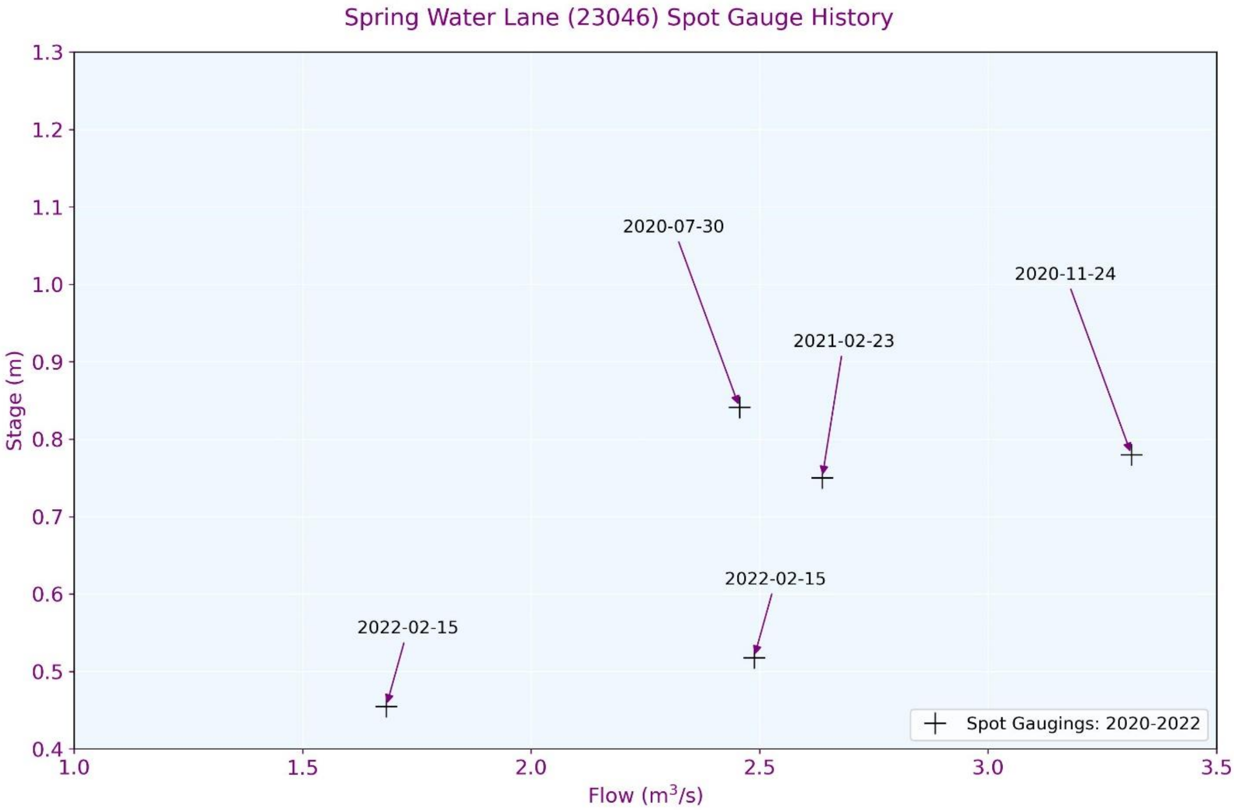


Figure 2.24: Spot Gauging History 2020-2022, Spring Water Lane

2.4.3 Hydraulic Model Development

The rating review modelling for Spring Water Lane was carried out within the Lissardboola network, which extends approximately 3.1 km upstream and 550 m downstream from the gauging station to the confluence with the River Lee. The gauging station is located in a suburban area, with predominantly residential buildings surrounding the site. The gauge itself is situated on the downstream face of the Spring Water Lane Bridge (see **Figure 2.25**).

Figure 2.25: Upstream (top) and Downstream (bottom) View of the Spring Water Lane Bridge, taken in 2012 Prior to Installation

The gauging station cross-section measures approximately 22 m wide, as shown in **Figure 2.26**. New survey data was obtained for the bridge which is considered a potential control on flow at the gauging station. Additional cross-sections along the channel were also captured to improve data resolution.

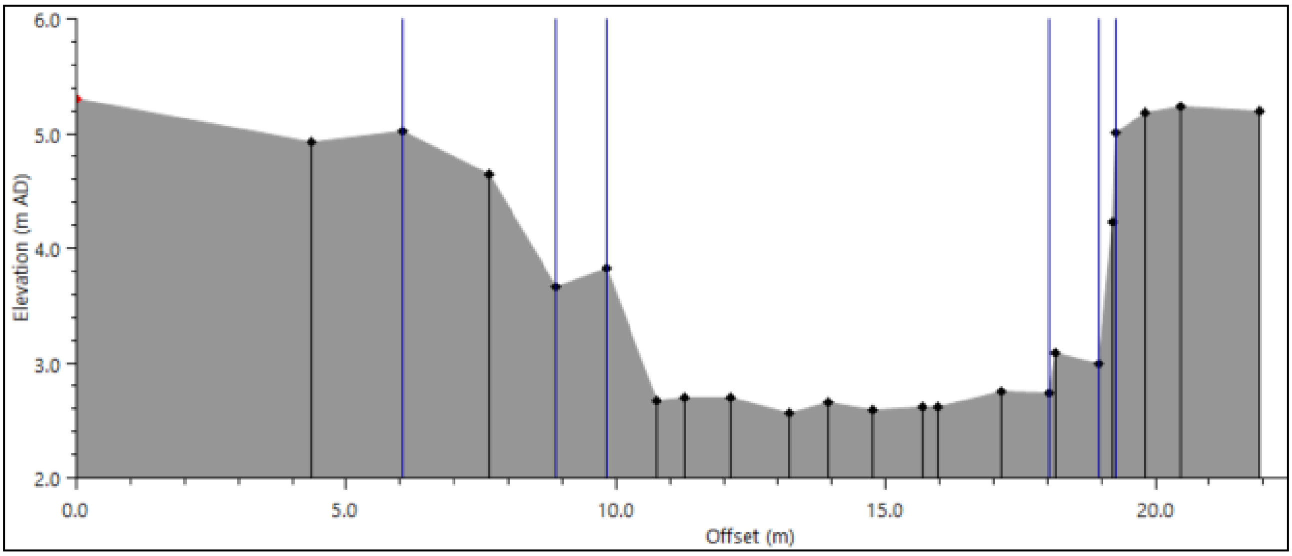


Figure 2.26: Modelled Gauge Cross-Section 23LEEF0100084

The two openings of Spring Water Lane Bridge change shape between the upstream and downstream face (**Figure 2.25**), with a large step inside each, shown in new survey images taken inside the bridge in **Figure 2.27**. An additional weir is located at the downstream end of the right opening.

Figure 2.27: Survey Photo of Weir Inside One of the Spring Water Lane Bridge Openings

Given software constraints, the bridge was modelled as a culvert to allow more accurate representation of the complexities of this structure. The weirs within the bridge openings were modelled by lowering the invert level of the downstream conduits, though it should be noted that it is not possible to set, or adjust, any weir parameters using this method.

2.4.4 Hydraulic Model Calibration and Sensitivity

Sensitivity tests were carried out on the Lissardboola to determine if flow and level outputs closely correspond with the OPW spot gauge records. Review of these outputs found that the modelled Q-h relationship had a good match to just one of the gaugings, and adjustments were therefore necessary to develop a realistic and representative model of the channel and flow conditions at Spring Water Lane.

Upper and lower bounds scenarios were also determined to assess the sensitivity of the final model to changes in parameters. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the Lissardboola watercourse are shown in **Table 2.10**. Iterative testing found that neither the Spring Water Lane Bridge nor the weir at the downstream face of the right bank opening were hydraulically significant, and these were consequently left unaltered in the final model network.

Calibration was instead primarily achieved through slight modification of the channel topography to simulate dips in the bed elevation that were visible on survey long-sections but not effectively captured in the cross sections, which were collected in 5 m intervals at the gauging station. The bed level was lowered by 0.15 m at a cross section 7 m downstream of the gauge to represent the uneven terrain more accurately. In addition to this, channel and overbank roughness were increased substantially to emulate the ineffective flow conditions arising due to overhanging vegetation growth.

Table 2.10: Model Parameters Adjusted in Each Scenario for the Spring Water Lane Rating Review

Scenario	Manning's *n* value	Conduit Parameters (23LEEF0100085_CUL_INL1, 23LEEF0100085_CUL1_BREAK, 23LEEF0100085_CUL_INL2, 23LEEF0100085_CUL2_BREAK)
		Headloss
Upper Bounds	Bank and channel Manning's *n* increased by 25% at all cross sections	Headloss of 6 (representative of a 60° bend)
Final Model (Model Q-h)	**Section 23LEEA0100235 and 23LEEA0100222**: 0.045 (channel) **Section 23LEEA0100218 and 23LEEA0100213:** 0.055 (channel)	Headloss of 3.3 (representative of a 30° bend)
Lower Bounds	Bank and channel Manning's *n* decreased by 25% at all cross sections	Headloss of 0 (representative of no bend)

The model data for each scenario are presented in **Figure 2.28**. The final model data, show in purple, does not align with the gaugings. Attempts to simulate a Q-h relationship calibrated to the four lower spot gaugings were unfeasible given the substantial scatter in the records to date. However, following some adjustments to model parameters, a close match is observed between the modelled data and the high flow gauging.

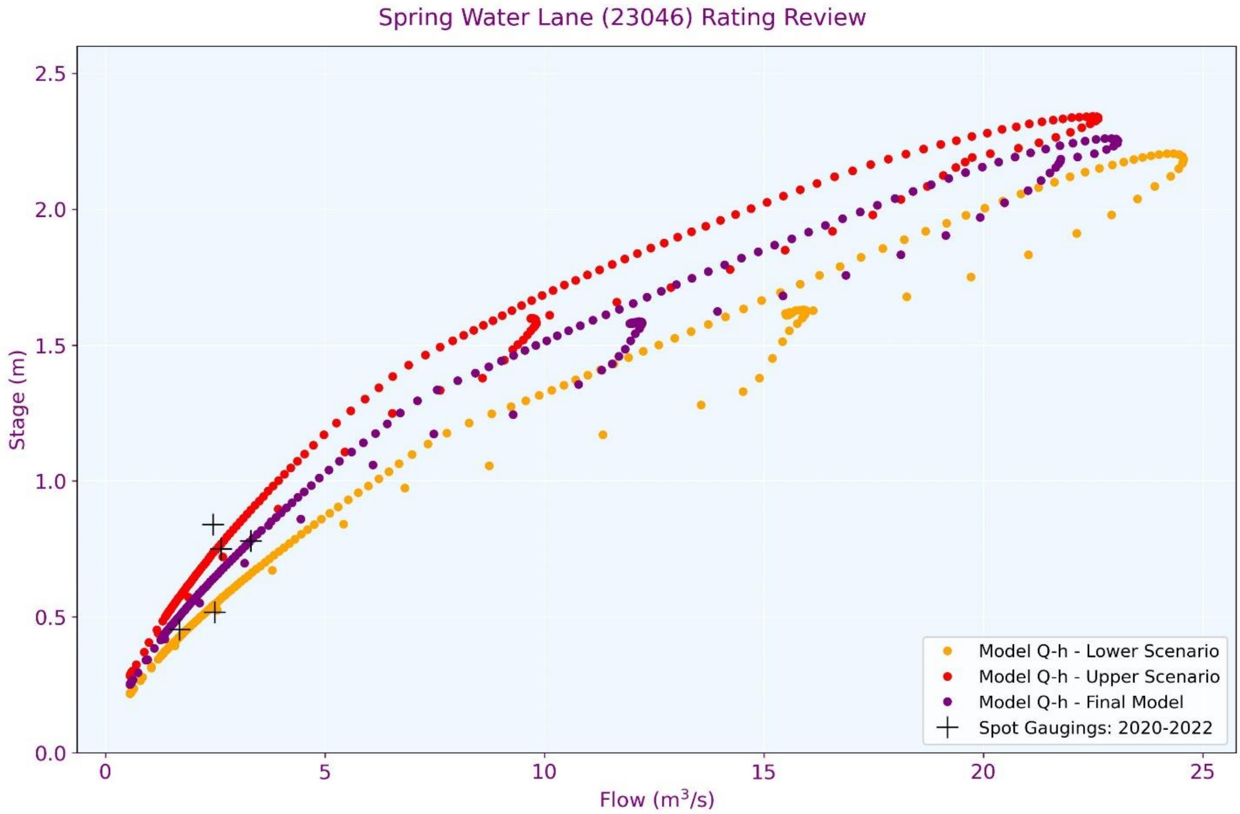


Figure 2.28: All Three Modelled Scenarios – Lower (orange), Medium (purple) and Upper (red) – Rating Curves for Spring Water Lane (23046). Included are the spot gaugings (black crosses).

It is also noted that the rating curve for each scenario displays a looping feature highlighting that the Spring Water Lane gauging station is affected by hysteresis. The looping of the rating curves is a common phenomenon where significant hydraulic structures are present and is representative of different channel hydraulics (Q-h) during the rising and receding limbs of the flood hydrograph. The medium scenario rating curve was selected as the final model that best represents the Q-h relationship at the Spring Water Lane gauging station on the Lissardboola watercourse. The upper and lower scenarios are retained and serve as an indication of the range in which the Q-h relationship could potentially exist, as shown by a lower and upper bound range in **Figure 2.29**.

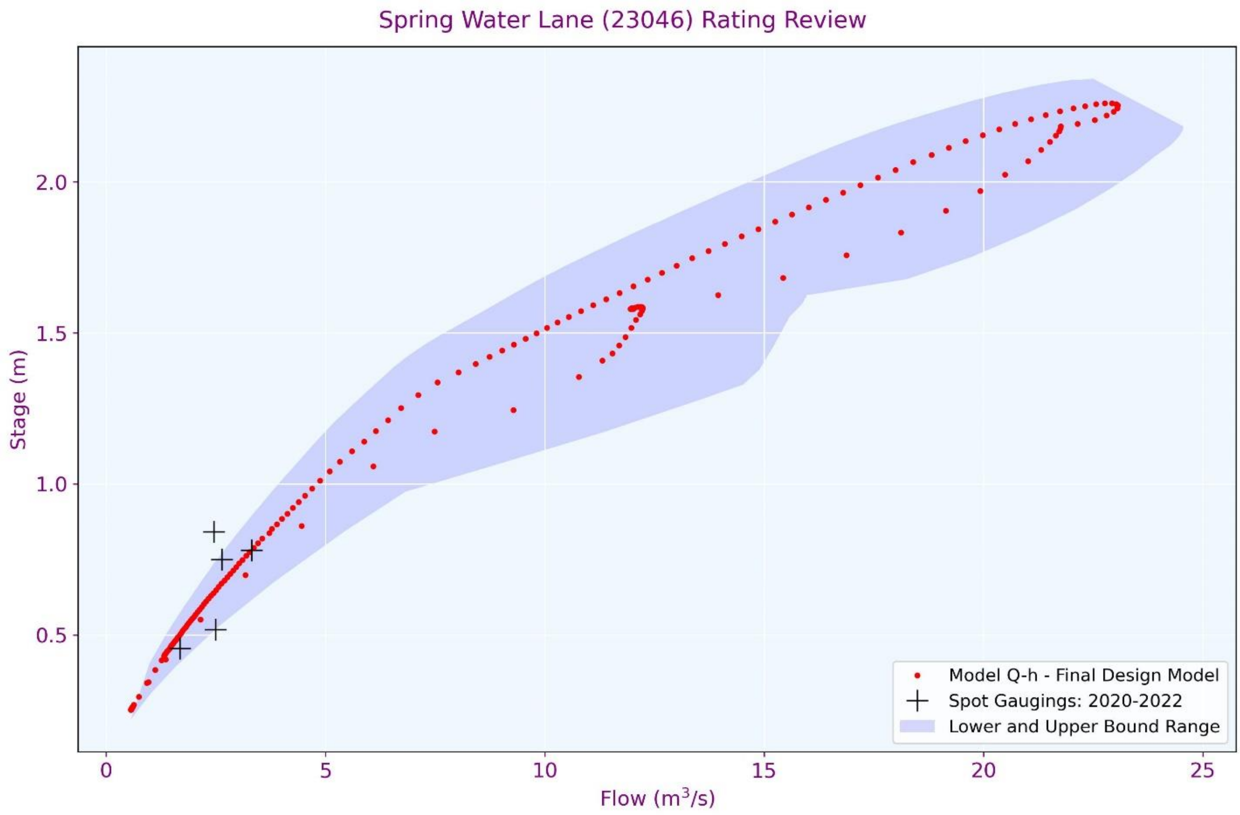


Figure 2.29: Spring Water Lane (23046) Model Q-h with the Upper and Lower Bound Range Included

2.4.5 Rating Curve Development

A new rating curve was developed using an iterative process which defined the most appropriate physical parameters to describe the Q-h relationship at the Lissardboola watercourse at Spring Water Lane gauging station. The proposed physical constant parameters derived for each limb of the rating curve are shown in **Table 2.11**, with the respective rating curve shown in **Figure 2.30**.

The newly proposed rating shows a good fit to the current hydraulic model of the Lissardboola watercourse at Spring Water Lane gauging station, although the lack of historic spot gaugings results in signiifcant uncertainrty in any rating curve developed. The rating curve, as shown in **Figure 2.30**, has been developed based on the peak water level of the hydydrograph, i.e., the apex of the looped features, following best practice guidance published by the UK Environemtal Protection Agency[27].

[27] https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/290416/sw6-061-m-e-e.pdf

Table 2.11: Proposed RPS Rating (2022) Physical Constant Parameters, Spring Water Lane (23046)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.79	5.07	-0.03	1.47
02	0.79	1.30	0.16	1.37	3.98
03	1.30	1.59	0.48	0.96	3.46
04	1.59	1.86	9.24	-0.37	1.46
05	1.86	2.26	1.55	0.83	2.40

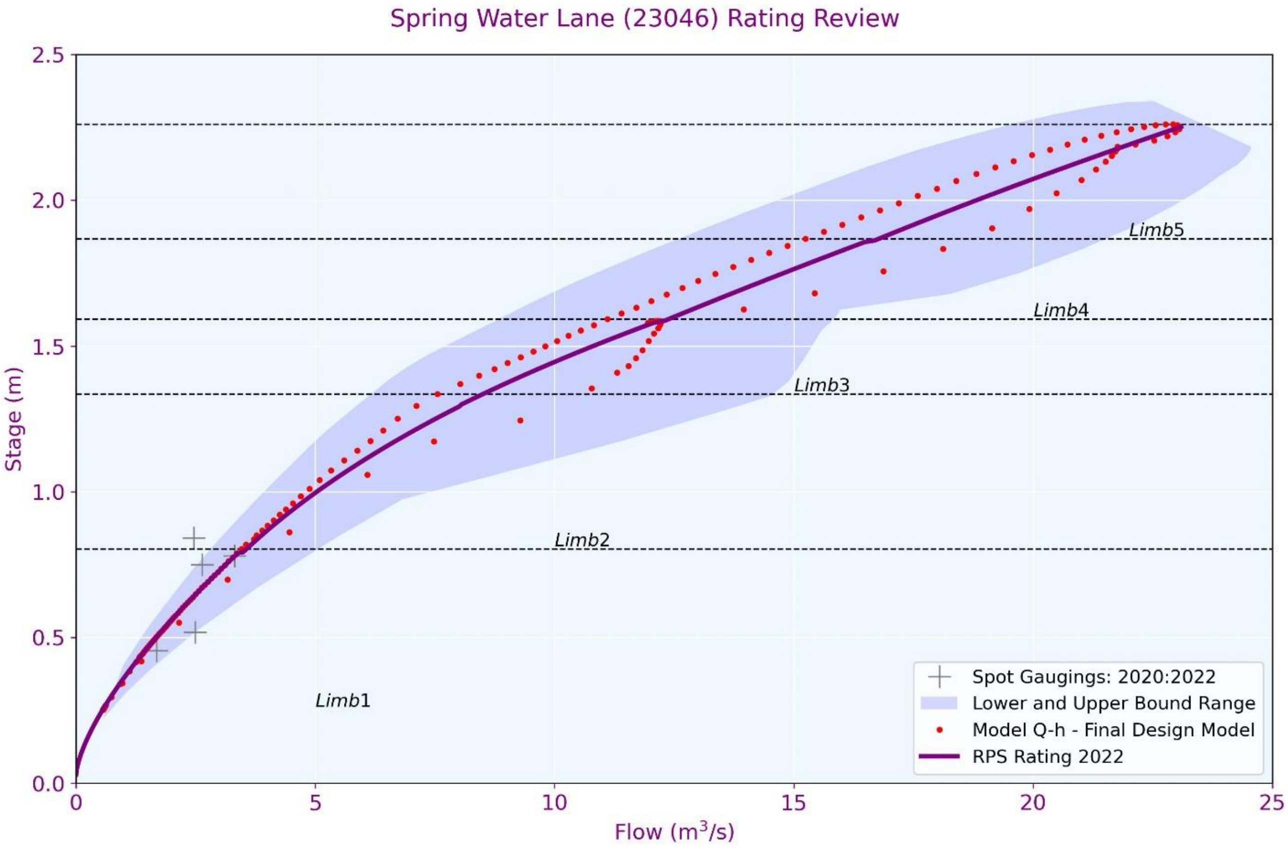


Figure 2.30: Proposed Rating Curve for Spring Water Lane (23046) Based on final Model Design Q-h. Included is the upper and lower bound range

The development of this rating equation allowed for the extraction of AMAX values for the two available hydrometric years (2019 and 2020). The two-year AMAX series that was processed is shown in **Table 2.12**.

Table 2.12: Processed Two-Year AMAX for Lissardboola Watercourse at Spring Water Lane (23046) Using Proposed RPS Rating Curve 2022

Hydrometric Year	AMAX Record Date	S.G. Reading (m)	Estimated Flow (m^3/s)
2019	30/07/2020	1.21	6.99
2020	11/11/2020	1.09	5.78

The processed two-year AMAX at Spring Water Lane using RPS rating curve 2022 produces a Q_{med} estimate of 6.39 m^3/s which is approximately 38% below the initial design flow estimate (10.36 m^3/s) derived from the FSU5 variable regression equation. This difference between Q_{med} derived from the rating curve and the FSU5 variable regression equation reflects the range of uncertainty in the rating curve due to the lack of historical spot gaugings at and around Q_{med}. Furthermore, the estimation of Q_{med} based on AMAX data over a two-year data period results in significant uncertainty. If the FSU7 variable equation was to be used, a Q_{med} estimate of around 7.5 m^3/s would be calculated. This is more aligned with the estimate obtained from the RPS rating curve, however, due to uncertainty associated with the rating curve and the standard factorial standard error associated with the FSU regression equations, RPS suggests the more conservative Q_{med} estimate derived from the FSU5 equation is retained.

2.4.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Spring Water Lane gauging station to inform and validate the hydrological analysis and hydraulic model build for the Tralee FRS. The uncertainty in the RPS rating curve derived is significantly high due to the lack of historic data. It is recommended that more spot gaugings are taken at and around Q_{med} to better constrain the Q-h relationship on the Lissardboola watercourse at Spring Water Lane gauging station. In the meantime, the RPS rating curve 2022 should be treated as an indication of the potential Q-h relationship and will be used to validate hydrograph shape at the station. Due to the lack of data, the rating curve is considered highly uncertain and is not recommended to be adopted.

2.5 Oakpark Road (23047)

An assessment of the rating at Oakpark Road on the Big River was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record (taken in June 2020) for the staff gauge zero level is 31.322 mAOD Malin. The survey undertaken as part of this project found the staff gauge zero level to be 31.35 mAOD Malin. The two levels are in moderate agreement with a difference of just under 30 mm. The most recently surveyed staff gauge zero level of 31.350 mAOD was used to complete the rating review.

2.5.1 Raw Data Analysis

Given this station is newly installed and currently has no OPW-derived rating, a review of the available data at the gauging station was conducted.

The station records water levels at 15-minute intervals. The station has recorded data available for analysis from 25th June 2020 to 1st October 2022 shown in **Figure 2.31**. There are no significant data gaps within the record period.

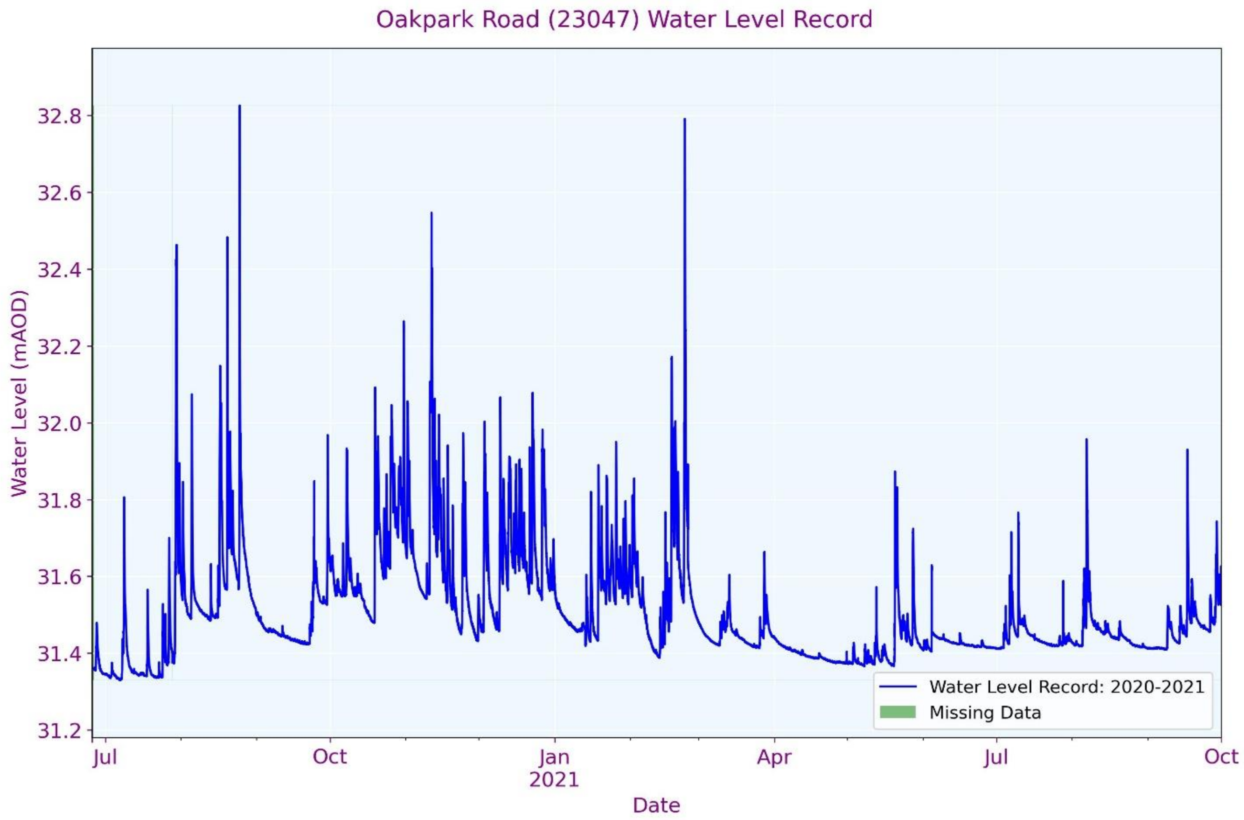


Figure 2.31: Big River at Oakpark Road (23047) Water Level Record from 2020 to 2021

2.5.2 Spot Gauging History

RPS has analysed the spot gauging history at Oakpark Road (23047). **Figure 2.32** shows the full available spot gauge record from 2020–2022, as received from the OPW. Oakpark Road has proved difficult to gauge and eight out of 15 gaugings were excluded by the OPW due to quality issues. The main challenges at the site are related to the gradient and channel characteristics.

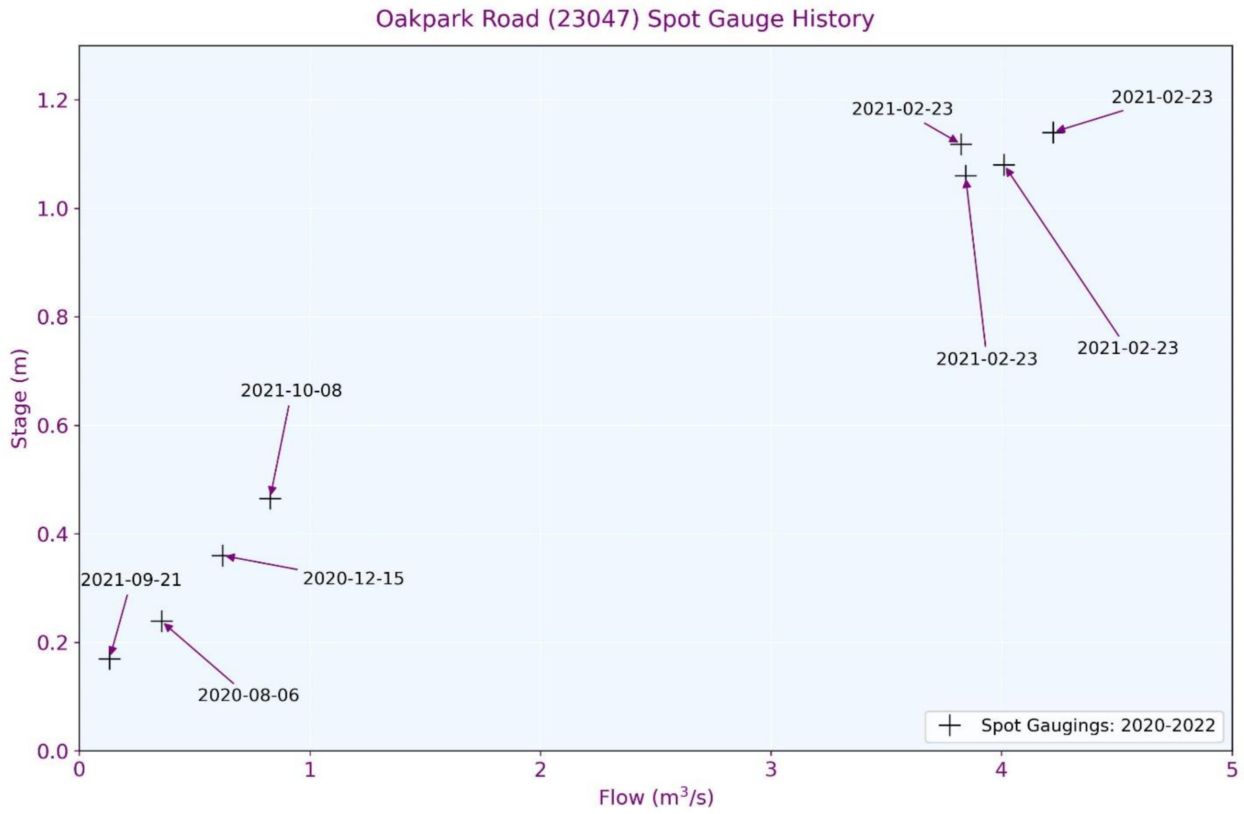


Figure 2.32: Spot Gauging History 2020-2022, Oakpark Road

2.5.3 Hydraulic Model Development

The rating review modelling was carried out within the Big River network. The modelled Big River reach extends approximately 1.95 km upstream and 3.6 km downstream of the gauge at the confluence between the Big River and River Lee. The gauge itself is located just upstream of a large 1.5 km diversion chamber connecting the Big River to the Ballybeggan (shown in **Figure 2.33**). In addition to this extensive culvert, the gauge board is also situated just upstream of an engineered channel constriction for flow control, depicted in **Figure 2.33**, and 25 m upstream of a small weir (see **Figure 2.34**).

Located upstream of the Kileen and Muing West suburban areas, the gauging station is in close proximity to residential houses and urbanised land. The gauge board cross-section measures approximately 7 m wide, as shown in **Figure 2.35**. New survey data was obtained for the weir and the flow constriction structure, alongside CCTV of the pipe system to inform of any blockages. Additional cross-sections along the channel were also captured to improve data resolution. Survey data of the inlet and outlet of the diversion chamber (including the trash screen) obtained during the CFRAM Study was deemed suitable in representing this structure.

Figure 2.33: Oakpark Road Gauging Station Location Viewed from Upstream, Taken in 2012 Prior to Installation

Figure 2.34: Weir Situated 25 m Downstream of the Oakpark Road Gauge

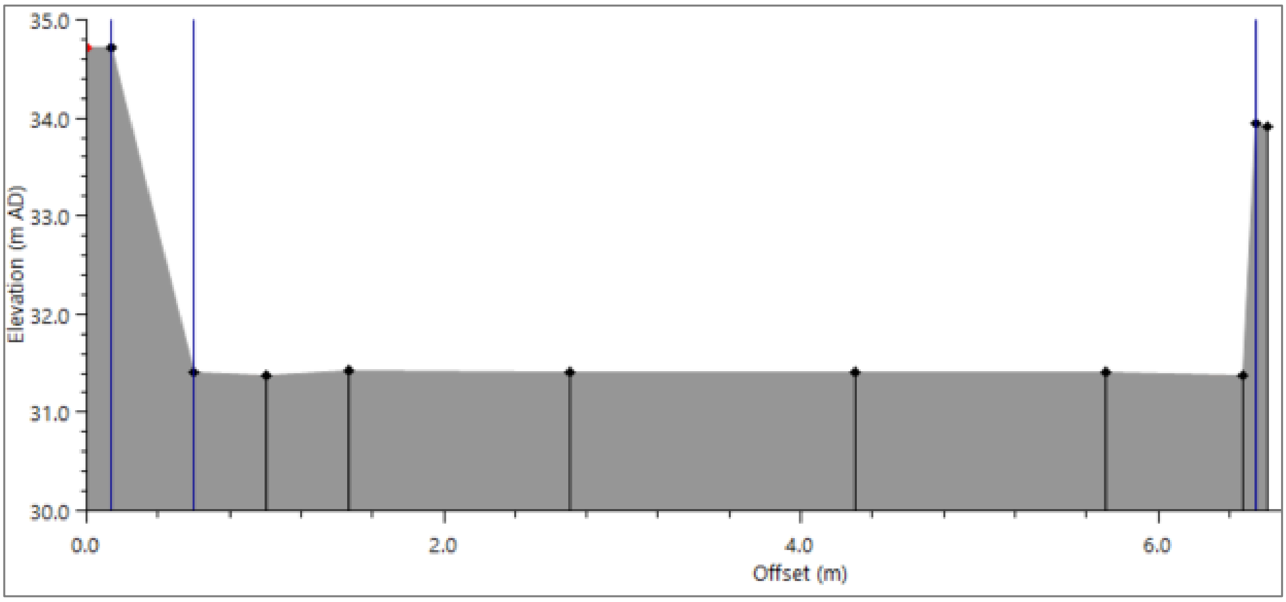


Figure 2.35: Modelled Gauge Cross-Section 23BIGR0100345A

2.5.4 Hydraulic Model Calibration and Sensitivity

Sensitivity tests were carried out on the Big River to determine if flow and level outputs closely correspond with the OPW spot gauge records. Review of these outputs found that while the modelled Q-h relationship at the gauge aligned well with the low flow gaugings, the model stage level was ~500 mm lower than the OPW high flow gaugings. Adjustments were necessary to develop a realistic and representative model of the channel and flow conditions at Oakpark Road.

Upper and lower bounds scenarios were also produced to assess the sensitivity of the final model to changes in parameters. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the Big River are shown in **Table 2.13** with the model data for each scenario depicted in **Figure 2.36**.

Unlike in the previous rating reviews, iterative testing found that nearby structures had a substantial impact on the hydraulic regime at Oakpark Road, with minor adjustments to both the diversion chamber and the weir downstream of the gauge resulting in a notable change to the rating curve. Given the significant influence of these two structures on the rating curve, it was not deemed necessary to adjust channel or bank roughness to meet the spot gaugings. Manning's *n* values were consequently left unaltered in the final model network. Instead, calibration was primarily achieved by increasing headloss at the upstream end of the diversion chamber. In the final model network headloss was set to 6 (representative of a 60° bend) to depict the loss of energy and reduction in the total head due to friction against the trash screen and walls within the pipe system.

Table 2.13: Model Parameters Adjusted in Each Scenario for the Oakpark Road Rating Review

Scenario	Manning's *n* value	Structure Parameters	
		Weir Discharge Coefficient (23BIGR0100341_US_Weir.1)	US Headloss of Conduit (23CUL_8_IN.1)
Upper Bounds	Bank and channel Manning's *n* increased by 25% at all cross sections	1.2	6.6 (representative of a 90° bend)
Final Model (Model Q-h)	Bank and channel Manning's n values allocated as per survey photos	1.7 (ICM default)	6 (representative of a 60° bend)
Lower Bounds	Bank and channel Manning's *n* decreased by 25% at all cross sections	2.2	3 (representative of a 30° bend)

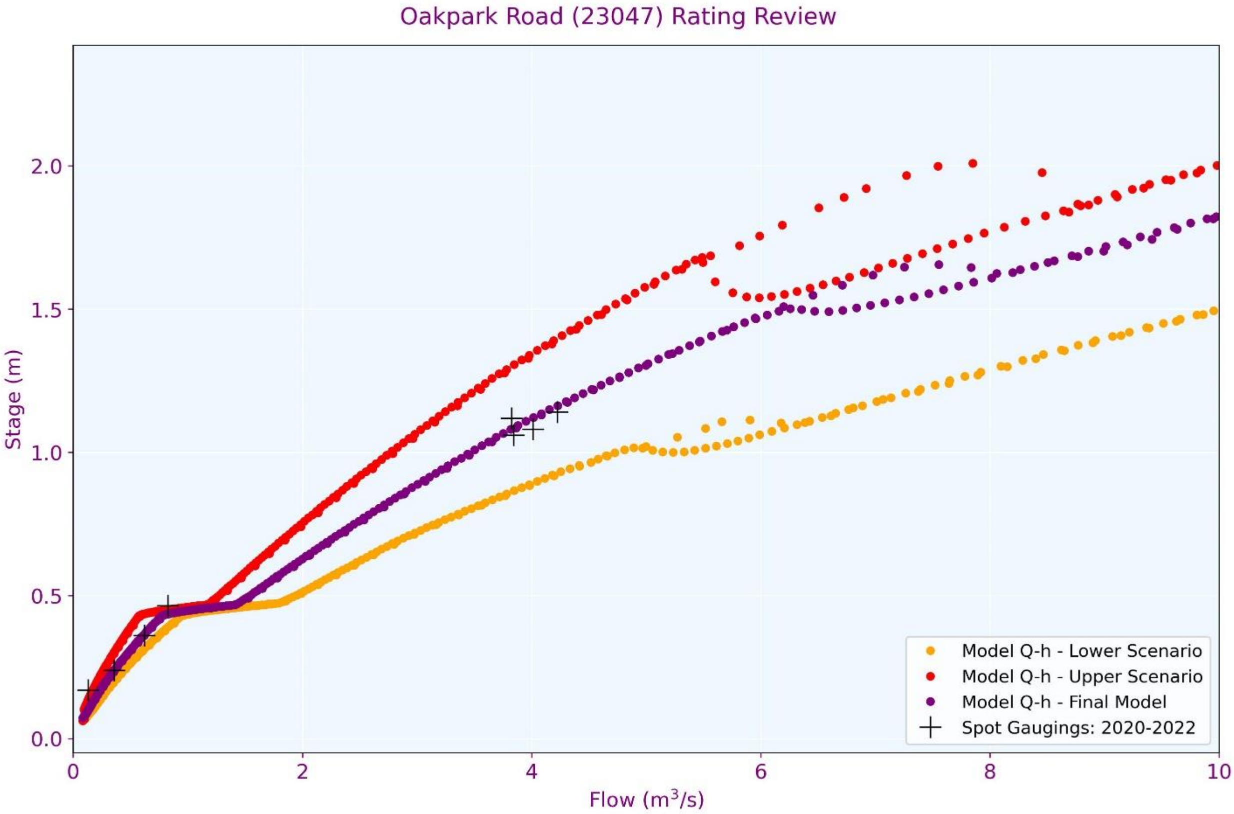


Figure 2.36: All Three Modelled Scenarios – Lower (orange), Final Model (purple) and Upper (red) – Rating Curves for Oakpark Road (23046). Included are the spot gaugings (black crosses)

The final design model depicting Q-H relationship is shown in **Figure 2.37**. The upper and lower bound scenarios are included, visualised as a range bound. Two dips are visible in the final rating curve at around 0.5m and 1.5m. The former can be attributed to the diversion chamber, with the stage height briefly plateauing here as the section level exceeds 31.8 mAOD, overtopping the crest of the trash screen and draining into the culvert. The decline at 1.5m marks the point at which flow overtops the concrete structure walls, causing a hydraulic jump to supercritical flow conditions.

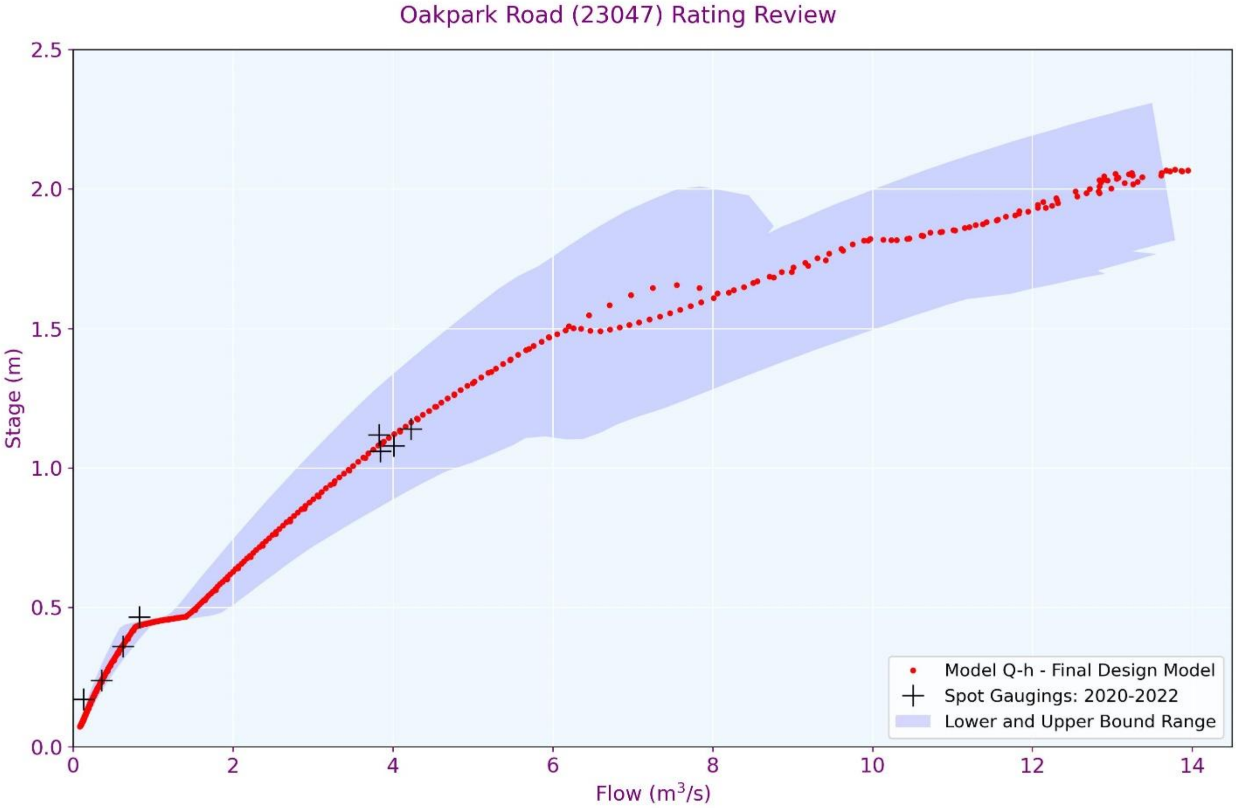


Figure 2.37: Oakpark Road (23047) Model Q-h with Upper and Lower Bounds Included

A discharge coefficient value of 1.7 was deemed appropriate for the weir in the final model network as it is indicative of smooth channel, adequately representing the concrete bed conditions depicted in **Figure 2.34**. By contrast, a discharge coefficient of 1.2 in the upper bounds scenario increased model instability and did not effectively represent the efficiency of flow over the weir.

2.5.5 Rating Curve Development

A new rating curve was developed using an iterative process which defined the most appropriate physical parameters to describe the Q-h relationship for Big River at Oakpark Road gauging station. The proposed physical constant parameters derived for each limb of the rating curve are shown in **Table 2.14**, with the respective rating curve shown in **Figure 2.38**.

The newly proposed rating, considering the hydraulic features on the reach, shows a good fit to the current hydraulic model of the Big River at the Oakpark Road gauging station. The modelled data further fits the spot gaugings well, however, the uncertainty must be stressed in the proposed RPS rating curve due to the sparse number of spot gaugings.

Table 2.14: Proposed RPS Rating (2022) Physical Constant Parameters, Oakpark Road (23047)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.44	2.46	0.05	1.55
02	0.44	0.47	40.44	-0.37	1.47
03	0.47	1.07	3.58	-0.03	1.14
04	1.07	1.50	3.58	-0.03	1.35
05	1.50	1.63	9.84	-1.48	0.12
06	1.63	1.84	14.08	-1.18	0.72
07	1.84	2.07	16.67	-1.77	0.18

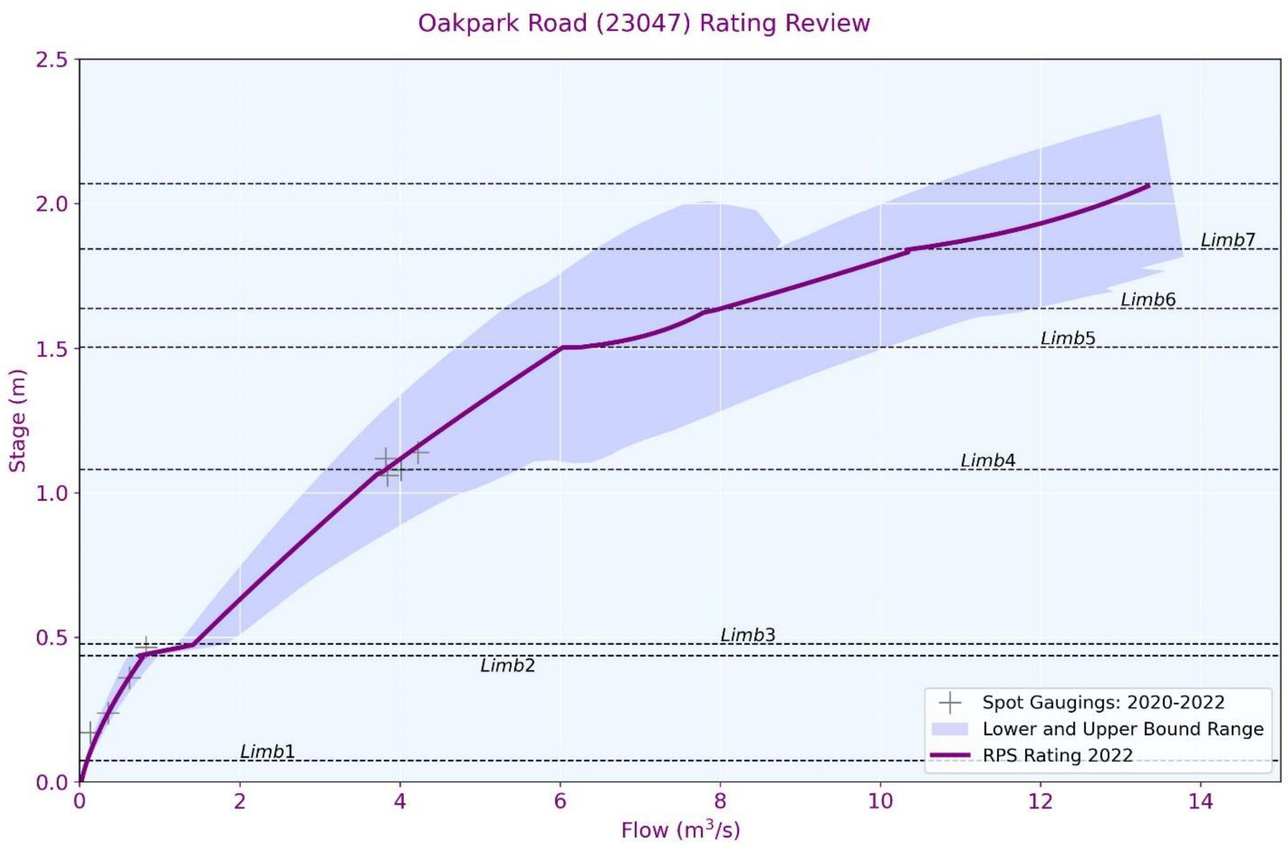


Figure 2.38: Proposed Rating Curve for Oakpark Road 23047 Based on Final Model Design Q-h. Included is the upper and lower bound range.

The development of this rating equation allowed for the extraction of AMAX values for the two available hydrometric years (2019 and 2020). The two-year AMAX series that was processed is shown in **Table 2.15**.

Table 2.15: Processed Two-Year AMAX for Big River at Oakpark Road (23047) Using Proposed RPS Rating Curve 2022

Hydrometric Year	AMAX Record Date	S.G. Reading (m)	Estimated Flow (m^3/s)
2019	25/08/2020	1.47	5.87
2020	23/02/2021	1.44	5.70

The processed two-year AMAX at Oakpark Road using RPS rating curve 2022 produces a Q_{med} estimate of 5.79 m^3/s which is approximately 27% below the initial design flow estimate (7.91 m^3/s) derived from the FSU5 variable regression equation. This difference between Q_{med} derived from the rating curve and the FSU5 variable regression equation reflects the range of uncertainty in the rating curve due to the lack of historical spot gaugings at and around Q_{med}. Furthermore, the estimation of Q_{med} based on AMAX data over a two-year data period results in significant uncertainty. If the FSU7 variable equation was to be used a Q_{med} estimate of around 6.65 m^3/s would be calculated. This is still 13% above the Q_{med} estimate obtained from the RPS 2022 rating curve which may be indicative of low confidence in the rating curve produced.

2.5.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Oakpark Road gauging station to inform and validate the hydrological analysis and hydraulic model build for the Tralee FRS. The significant differences between estimated Q_{med} from the RPS rating curve and the FSU equations highlights the uncertainty in the proposed RPS rating curve due to the lack of spot gaugings.

2.6 Manor West (23049)

An assessment of the rating at Manor West on the Ballybeggan watercourse was conducted as part of the hydrological analysis for the Tralee FRS.

Rating equation parameters have been derived using the stage heights relative to the staff gauge zero level at the gauging station. The most recent OPW record (taken in June 2020) for the staff gauge zero level is 7.145 mAOD Malin. The survey undertaken as part of this project found the staff gauge zero level to be 7.17 mAOD Malin. The two levels are in reasonable agreement with a difference of 25 mm. The surveyed staff gauge zero level was used to complete the rating review as any discrepancy is thought to be relative and the surveyed staff gauge zero level is consistent with the survey data from which the hydraulic model is constructed.

2.6.1 Raw Data Analysis

Given this station is newly installed and currently has no OPW-derived rating, a review of the available data at the gauging station was conducted.

The station records water levels at 15-minute intervals. The station has recorded data available for analysis from 26th June 2020 to the 1st October 2022 shown in **Figure 2.39**. There are no significant data gaps within the record period.

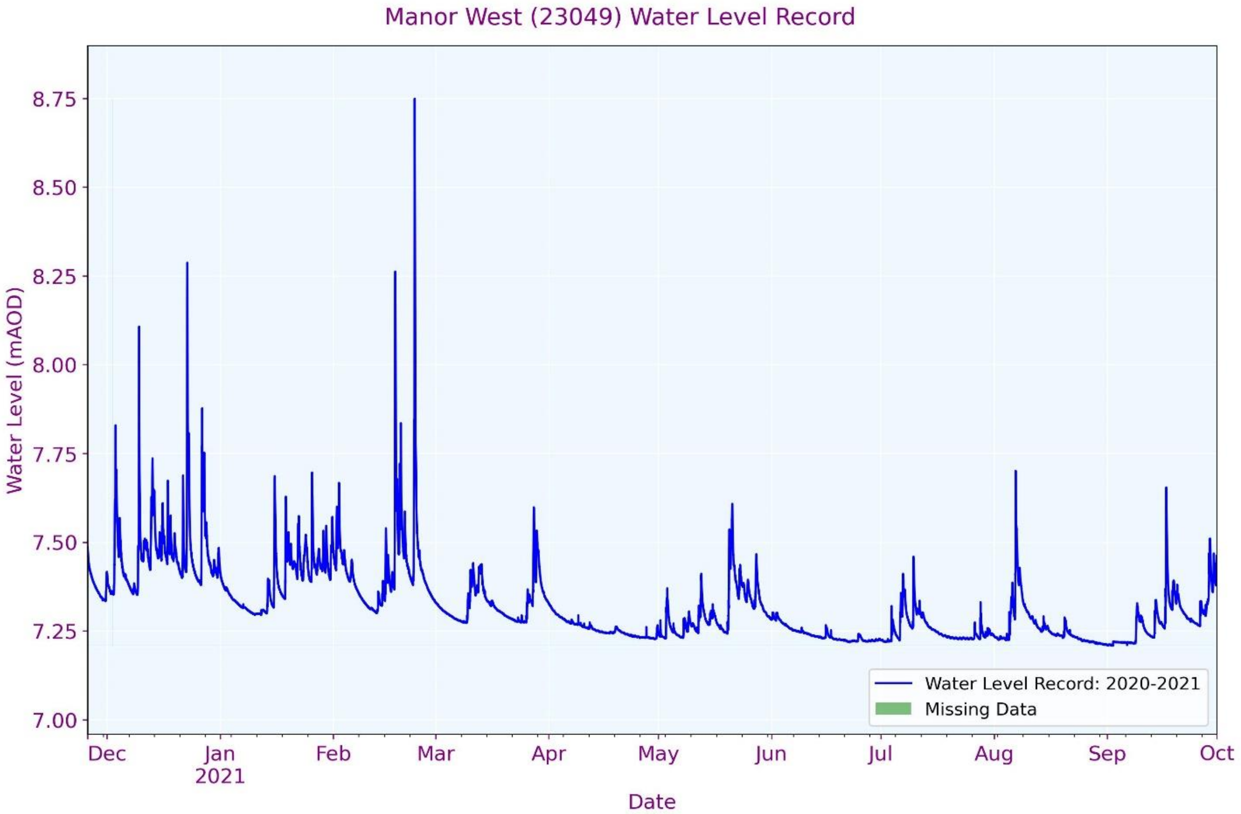


Figure 2.39: Ballybeggan Watercourse at Manor West (23049) Water Level Record from 2020 to 2021

2.6.2 Spot Gauging History

RPS has analysed the spot gauging history at the Manor West gauging station (23049). Installed in November 2020, only one spot gauge record is available for this gauge to date (0.78 m stage reading on the 23/02/2021) and consequently no rating curve has been developed by the OPW.

2.6.3 Hydraulic Model Development

The rating review modelling was carried out within the Ballybeggan network. The modelled Ballybeggan reach extends approximately 2.7 km upstream and 750 m downstream from the gauging station to the confluence with the River Lee. In addition to the bridge that the gauge is situated on (**Figure 2.40**), there are three bridges within 75 m of the gauge, two upstream and one downstream (see **Figure 2.41**). Situated in the Manor Park region of Tralee, the land surrounding the gauge is highly urbanised, in close proximity to commercial buildings including Mile Height retail park and Manor West shopping centre. The gauging station cross-section measures approximately 17 m wide, as shown in **Figure 2.42**. Survey data obtained during the CFRAM study was deemed suitable in representing the bridge 75 m upstream of the gauge as it is not believed to have significant control on flow at the gauge. However, new survey data was obtained for the other three bridges, which are located within 25 m of the gauging station. Additional cross-sections along the channel were also captured to improve data resolution.

Figure 2.40: Manor West Gauging Station Location Viewed from Upstream

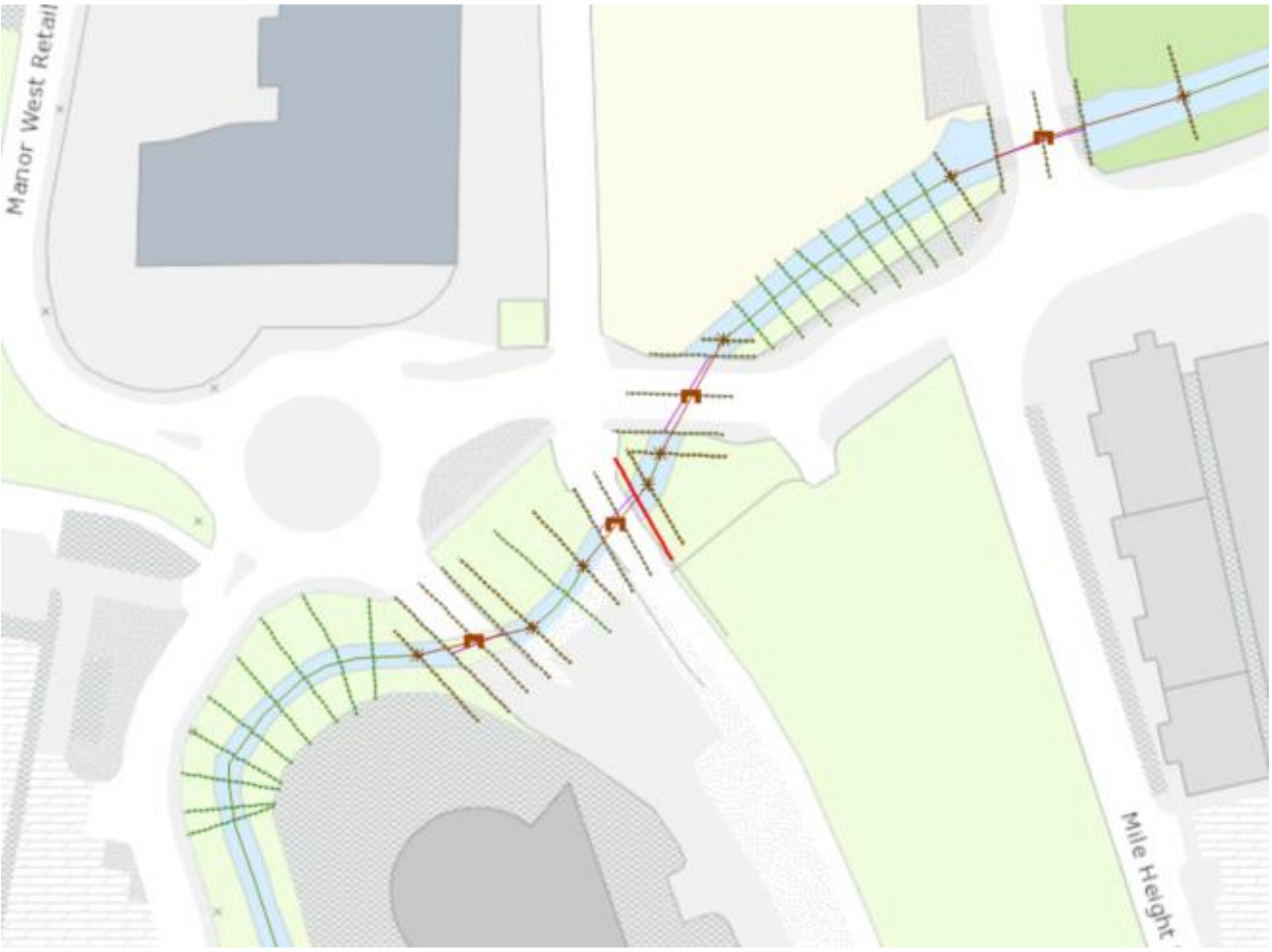


Figure 2.41: Location of Gauge Cross-Section (shown in red) and Nearby Bridges within Ballybeggan Network

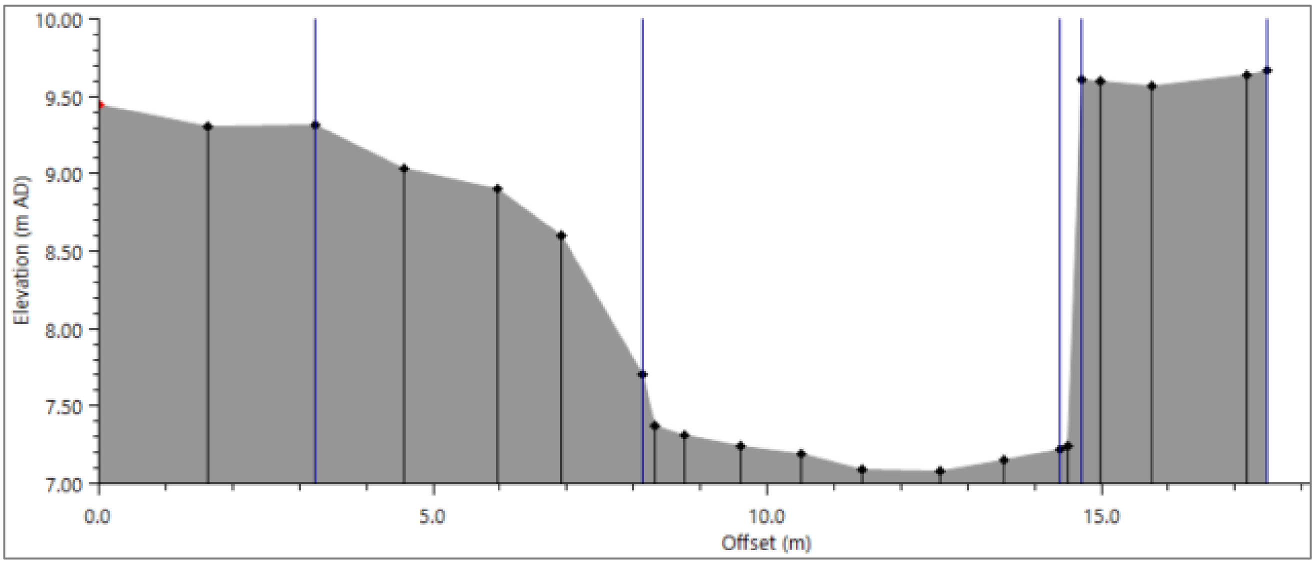


Figure 2.42: Modelled Gauge Cross-Section 23LEEA0100220

2.6.4 Hydraulic Model Calibration and Sensitivity

Sensitivity tests were carried out on the Ballybeggan to determine if flow and level outputs closely correspond with the OPW spot gauge records. Review of these outputs found that the modelled Q-h relationship at the gauge was around 200 mm lower than that of the OPW spot gauging, and adjustments were necessary to develop a realistic and representative model of the channel and flow conditions at Manor West.

Upper and lower bounds scenarios were also produced to assess the sensitivity of the final model to changes in parameters. Model outputs of each of the hydraulic sensitivity runs were analysed and the final hydraulic model parameters used on the Ballybeggan watercourse are shown in **Table 2.16**. Iterative testing found that none of the bridges surrounding the gauge were hydraulically significant, and these were consequently left unaltered in the final model network. As with Ballymullen and Ballyseedy, calibration was primarily achieved by increasing channel and overbank roughness within the 1D domain to reflect the ineffective flow conditions arising due to steep banks and overhanging vegetation growth.

Table 2.16: Model Parameters Adjusted in Each Scenario for the Manor West Rating Review

Scenario	Manning's *n* value	Bridge Parameters (23LEEA0100224_US_BR.1, 23LEEA0100220_US_BR.1 and 23LEEA0100215_US_BR.1)	
		Contraction and Expansion Coefficients	Headloss
Upper Bounds	Bank and channel Manning's *n* increased by 25% at all cross sections	**Increased by 50%:** Contraction loss 0.6 - Reverse flow expansion loss 1.0 - Expansion loss 1.0 - Reverse flow contraction loss 0.6	1
Final Model (Model Q-h)	**Section 23LEEA0100235 and 23LEEA0100222**: 0.045 (channel) **Section 23LEEA0100218 and 23LEEA0100213:** 0.055 (channel)	**Default:** - Contraction loss 0.3 - Reverse flow expansion loss 0.5 - Expansion loss 0.5 - Reverse flow contraction loss 0.3	0
Lower Bounds	Bank and channel Manning's *n* decreased by 25% at all cross sections	**Decreased by 50%:** - Contraction loss 0.15 - Reverse flow expansion loss 0.25 - Expansion loss 0.25 - Reverse flow contraction loss 0.15	0

The rating review curves for each scenario (upper and lower represented by range bound) are presented in **Figure 2.43** with the final modelled Q-h relationship showing a close match to the spot gauge. While this provides some confidence in model outputs, it should be noted that there are inherent uncertainties in the Manor West rating given the limited spot gaugings available.

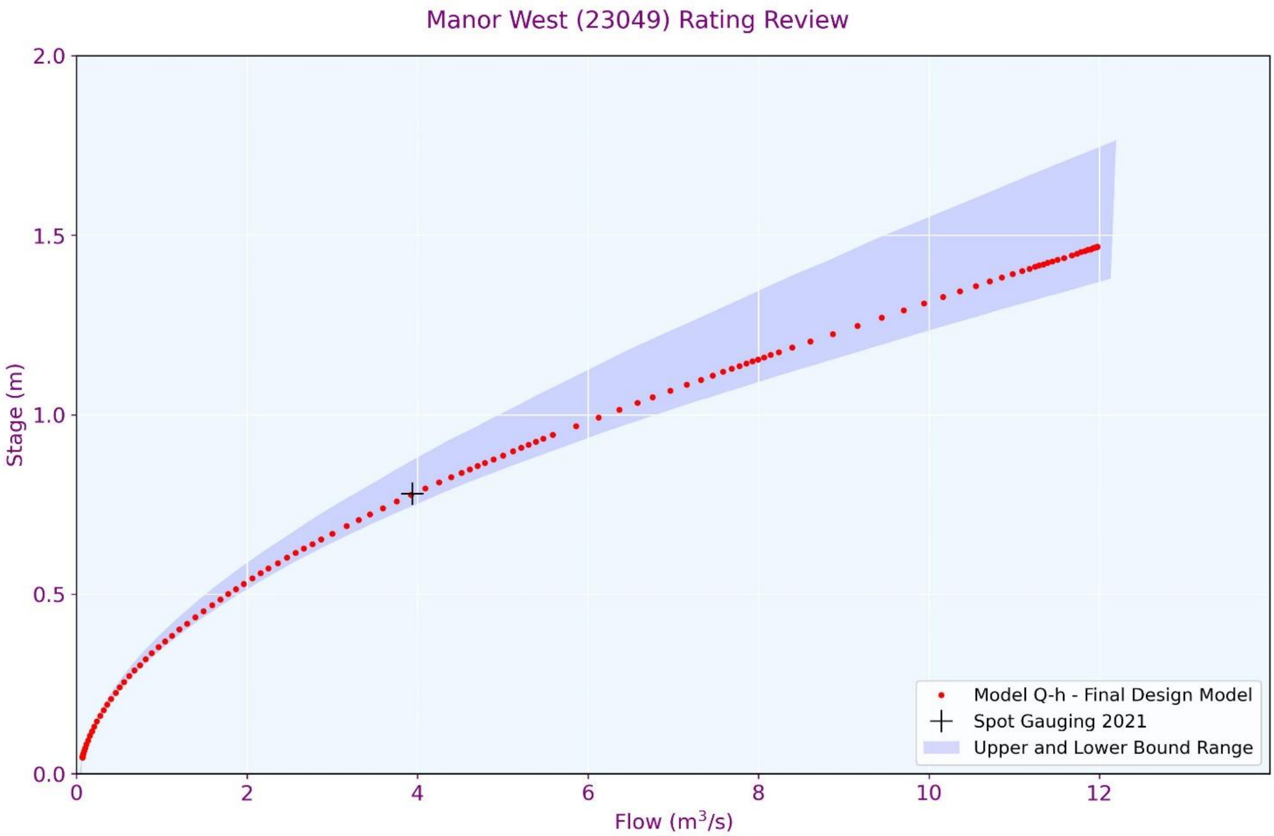


Figure 2.43: Manor West (23049) model Q-h with Upper and Lower Bound Curves Included

2.6.5 Rating Curve Development

A new rating curve was developed using an iterative process which defined the most appropriate physical parameters to describe the Q-h relationship for Ballybeggan at Manor West gauging station. The proposed physical constant parameters derived for each limb of the rating curve are shown in

Table 2.17, with the respective rating curve shown in **Figure 2.44**. The newly proposed rating shows a good fit to the current hydraulic model of the Ballybeggan stream at the Manor West gauging station, however, the uncertainty must be stressed in the proposed RPS rating curve due to the sparse number of spot gaugings.

Table 2.17: Proposed RPS Rating (2022) Physical Constant Parameters, Manor West (23049)

Limb	Minimum Stage (m)	Maximum Stage (m)	C	a	β
01	0.00	0.41	5.39	0.05	1.93
02	0.41	0.80	5.42	0.06	1.94
03	0.80	1.15	9.19	-0.26	1.31
04	1.15	1.47	11.90	-0.46	1.09

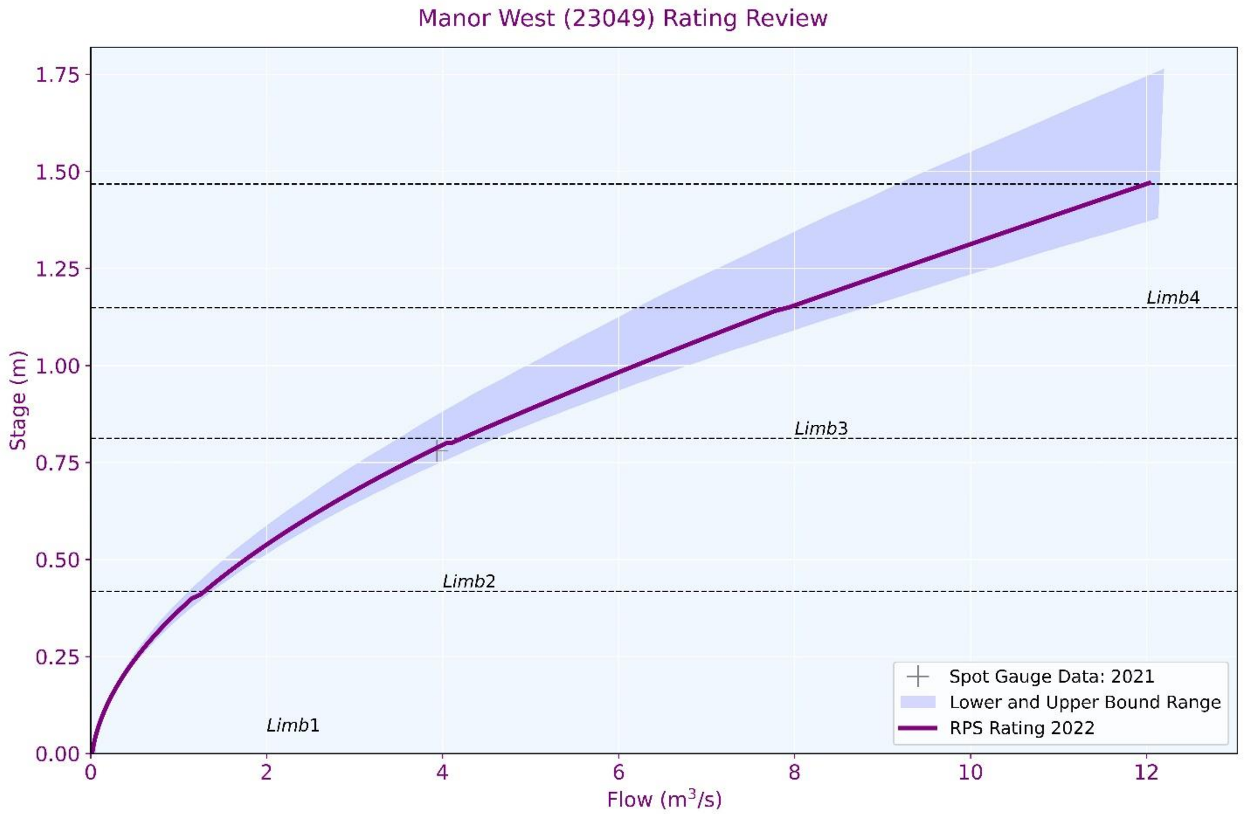


Figure 2.44: Proposed Rating Curve for Manor West 23049 Based on Final Model Design Q-h. Included is the upper and lower bound range.

The development of this rating equation allowed for the extraction of AMAX values for the one available hydrometric year (2020) The one-year AMAX series that was processed is shown in **Table 2.18**.

Table 2.18: Processed One-Year AMAX for River Lee at Ballyseedy (23045) Using Proposed RPS Extrapolated Rating Curve 2022

Hydrometric Year	AMAX Record Date	S.G. Reading (m)	Estimated Flow (m^3/s)
2020	23/02/2021	1.58	13.46

The processed one-year AMAX at Manor West has a stage value which exceeds the RPS Rating curve shown in **Figure 2.44**. **Figure 2.45** shows the RPS rating curve extended beyond modelled stage heights to allow estimation of flow from the one-year AMAX event at Manor West.

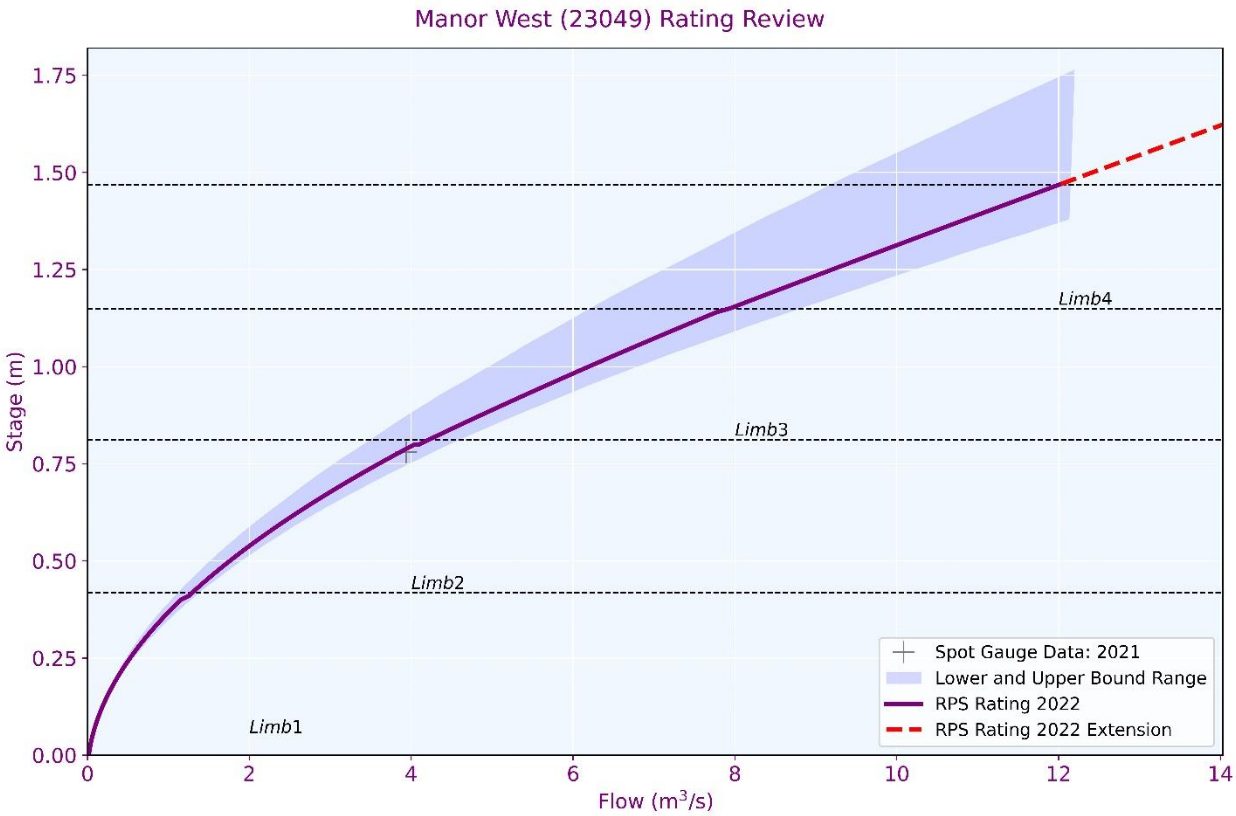


Figure 2.45: Proposed Rating Curve for Manor West 23049 Based on Final Model Design Q-h Extended Beyond Modelled Stage Heights. Included is the upper and lower bound range for the modelled stage heights

Using the extended RPS rating curve 2022 and the one-year extracted AMAX event at Manor West, a flow estimate of 13.46 m^3/s was obtained. As there is only one complete hydrological year, a Q_{med} estimate based on one year of data is highly uncertain and would be considered inaccurate. Taking the AMAX flow as a Q_{med} proxy, this estimate is approximately 52% above the initial design flow estimate (8.84 m^3/s) derived from the FSU5 variable regression equation. If the FSU7 variable equation was to be used a Q_{med} estimate of around 7.60 m^3/s would be calculated. The observed value based on the new rating RPS 2022 rating curve is 77% above the FSU7 Q_{med} estimate which may be indicative of low confidence in the rating curve produced. The estimation of this Q_{med} based on AMAX data over a one-year data period is highly uncertain. Furthermore, the extension of the rating curve beyond modelled stage heights compounds the uncertainty. More data is required to be confident in any rating to be developed at Manor West.

An explanation as to why the Q_{med} obtained from the rating review and Q_{med} estimate are significantly different may be due to the Balloonagh culvert diversion which flows into the upstream extent of Ballybeggan, upstream of the Manor West gauging station. For the purposes of the hydrological analysis, only the natural watercourses are considered within a catchment. Man-made structures are ignored, such as the Balloonagh culvert diversion. For this reason, the FSU estimate is based on catchment descriptors solely for the natural Ballybeggan catchment, which is likely the reason for the underestimation compared to that obtained from the rating review at Manor West gauging station. Regardless, the lack of data at the station results in too high uncertainty to anchor analysis to the gauging station.

2.6.6 Rating Review Conclusions

RPS conducted the rating review using available hydrometric data at the Manor West gauging station to inform and validate the hydrological analysis and hydraulic model build for the Tralee FRS. The large difference between estimated Q_{med} from the RPS rating curve and the FSU methods highlights the large uncertainty in the proposed RPS rating curve due to the lack of spot gaugings.

3 FLOOD EVENT ANALYSIS

The first step to understanding the flood frequency behaviour in relation to the subject catchment and its sub-catchments is conducting statistical analysis of the at site flood flow records for the watercourses which represent the main drivers for flood risk in the study area. Extreme Value Analysis (EVA) has been carried out in line with procedures set out in the FSU[28] using the Annual Maximum (AMAX) series records for estimated flow (m^3/s) supplied by the OPW and the EPA. Where the AMAX series for flows are not available, water levels have been used to estimate the frequency of past extreme events which can then be linked to past flood events in the study area.

3.1 Fluvial Flood Events

This section will focus on fluvial flood events and the behaviour of extreme flow events in the Tralee study area.

3.1.1 Tralee Clonalour (23022)

Tralee Clonalour is an inactive gauging station which was located in Tralee Town at the lower extents of the Big River, upstream of its confluence with the River Lee. It was operated by the EPA from 1985 to 2012. An AMAX series for peak flows (shown in **Figure 3.1**) was calculated from water level data using the EPAs revised rating equations from 2007 shown below.

$$0.057m \leq Stage \leq 0.093m : River\ Discharge(Stage) = 2.19841\ x\ (Stage - 0)^{1.5039}\ m^3/s$$

$$0.093m < Stage \leq 0.126m : River\ Discharge(Stage) = 255.96\ x\ (Stage - 0)^{3.50333}\ m^3/s$$

$$0.126m < Stage \leq 0.404m : River\ Discharge(Stage) = 29.314\ x\ (Stage - 0)^{2.45884}\ m^3/s$$

An upper limit of 0.404m was applied to the rating curve equation which capped EPA estimated max flows at 5.33 m^3/s. This upper limit was applied by the EPA as readings were rarely taken of “out of bank flows” which occurred when water levels exceeded 0.404 m. For this initial analysis, the upper limit was removed to better understand and capture peak flows of past flood events, or at least their relative ranking within the AMAX series which is the basis of flood frequency estimation.

At-site flood frequency analysis was performed on the AMAX series with several distributions commonly found in EVA tested to find the best-fit. The Generalised Extreme Value (GEV) distribution was found to fit the dataset the best and was used to produce the at-site flood frequency curve shown in **Figure 3.2**.

[28] FSU Technical Research Reports Volume II, Flood Frequency Estimation available at http://opw.hydronet.com/default.aspx?page=11

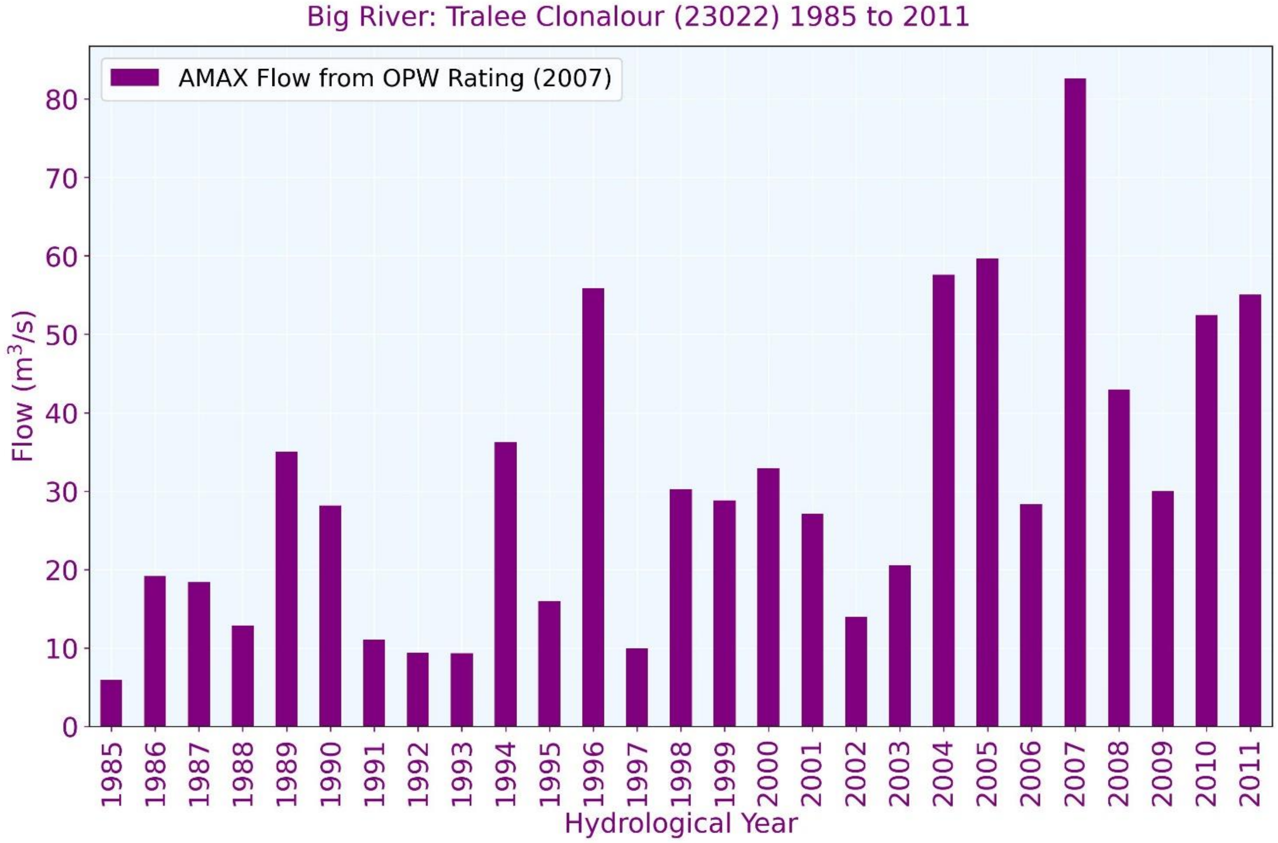


Figure 3.1: Big River at Tralee Clonalour 23022 AMAX series (peak flow estimates Q)

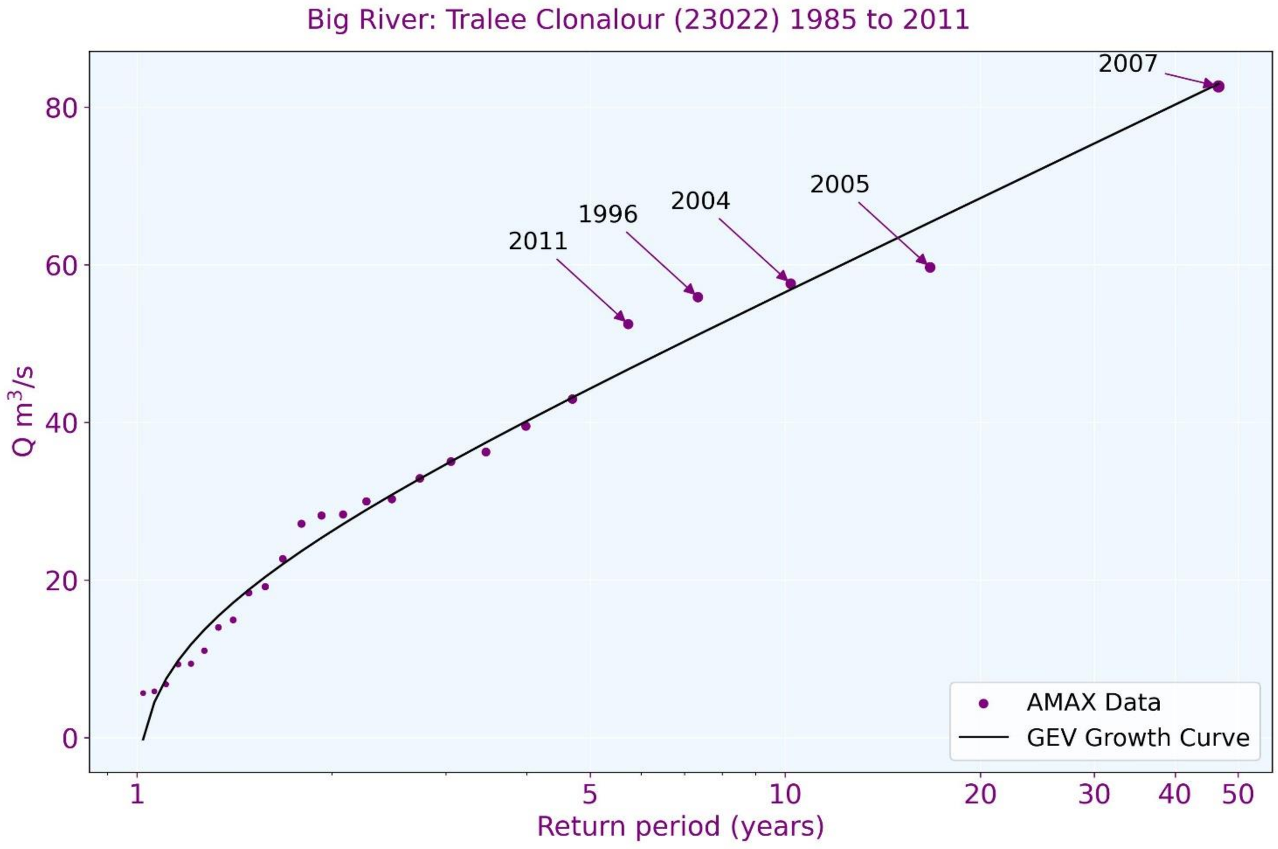


Figure 3.2: Flood Frequency Curve Fitted to AMAX Data for Big River at Tralee Clonalour 23022. The top five highest flow events are also highlighted with associated AMAX year attached.

The flood events identified as having impacted Tralee are shown in **Table 3.1** with the peak flow, rank of the event within the AMAX series and the estimated return period given. Also included are the top five highest flow events recorded with their respective estimated return periods. Note that flood events which have been highlighted after 2012 were not captured by the Tralee Clonalour 23022 station. Only flood events between 1985 and 2011 (hydrological years) are considered.

Table 3.1: Estimated Return Period for Five Largest Peak Flows and Flood Events Affecting Tralee at 23022

Event Date	Hydrological Year	Peak Flow (m^3/s)	Rank	Estimated Return Period (Years)
18th Feb 2011	2010-2011	0.5 (reflects tidal flooding event)	-	< 1
7th Feb 2011	2010-2011	52.5	5	5 – 10
19th Nov 2009	2009-2010	26.3	-	1 – 5
13th Aug 2008	2007-2008	82.6	1	40 – 50*
2nd Dec 2005	2005-2006	59.7	2	10 – 20
7th Jan 2005	2005-2006	57.6	3	≈ 10
26th Dec 2004	2004-2005	1.3 (reflects pluvial-driven event)	-	< 1
31st Aug 1997	1996-1997	55.9	4	5 – 10
5th Aug 1986	1985-1986	0.2	-	< 1

* From Extreme Value Analysis, return periods greater than two times the length of a record results in significant uncertainty in the estimated return period.

3.1.2 Ballymullen (23012)

The Ballymullen station is situated to the south-east of Tralee Town and is downstream of where the Lissardboola Stream joins the River Lee. An AMAX flow record was available for Ballymullen from 1974 to 1991, however, this record may not represent present day flows at the station (see Section 1.3).

An AMAX water level series was provided by the OPW and is shown in **Figure 3.3** which have been displayed to OD Malin (OSGM15). The change in water level between the pre- and post-works period, highlighted in **Figure 3.3**, further supports the decision not to rely on the AMAX flow record from 1974 to 1991 to represent catchment flood flow conditions for the present day.

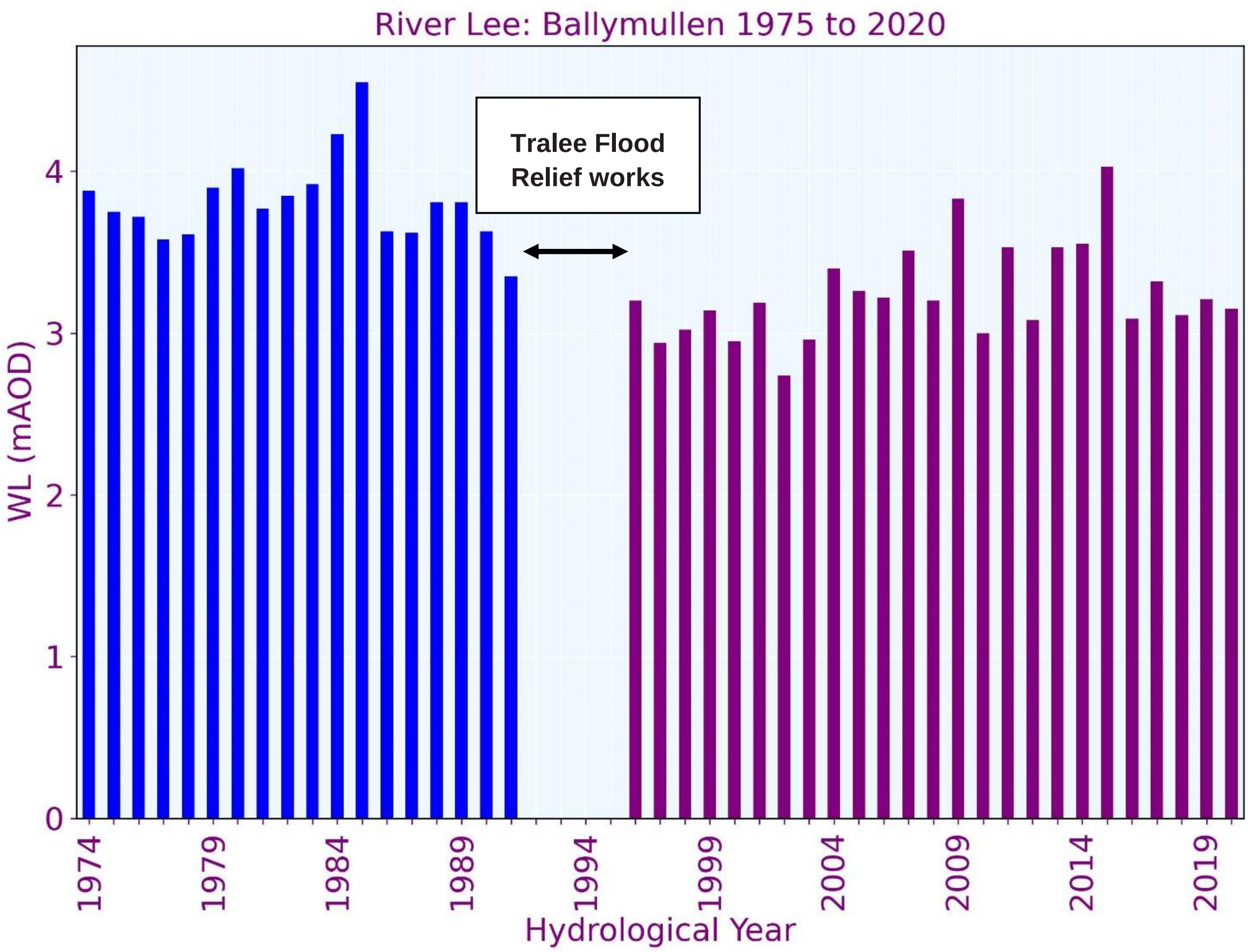


Figure 3.3: River Lee at Ballymullen (23012) AMAX series (peak water level WL) corrected from OD Poolbeg to OD Malin

The AMAX series for Ballymullen is split into two time periods: pre- and post-Tralee Flood Relief works which were carried out between 1991 and 1995 (hydrological year). As a result of the works, data is missing over a four-year period. At-site flood frequency analysis was performed on the AMAX water level series from the period prior to the works taking place, with the at-site flood frequency curve shown in shown in **Figure 3.4(a)**. The GEV distribution was found to fit the dataset the best. It is important to note that the CFRAM Study found Ballymullen to be affected by the tide which means any records may not represent a “fluvial influence alone”. While tidal influence may be present in the records, it appears from visual analysis that for peak water levels the tidal signal is overwhelmed by fluvial driven water levels. The water level record post works was converted to flow using RPS’ newly proposed rating curve equation developed in Section 2.1. The at-site flood frequency curve for post-Tralee Flood Relief works is shown in **Figure 3.4(b)** where the GLO distribution was found to fit the dataset the best.

The flood events pre- and post-Tralee Flood Relief works identified as having impacted Tralee are shown in **Table 3.2** with the peak water level, rank of the event within the AMAX series, and the estimated return period given.

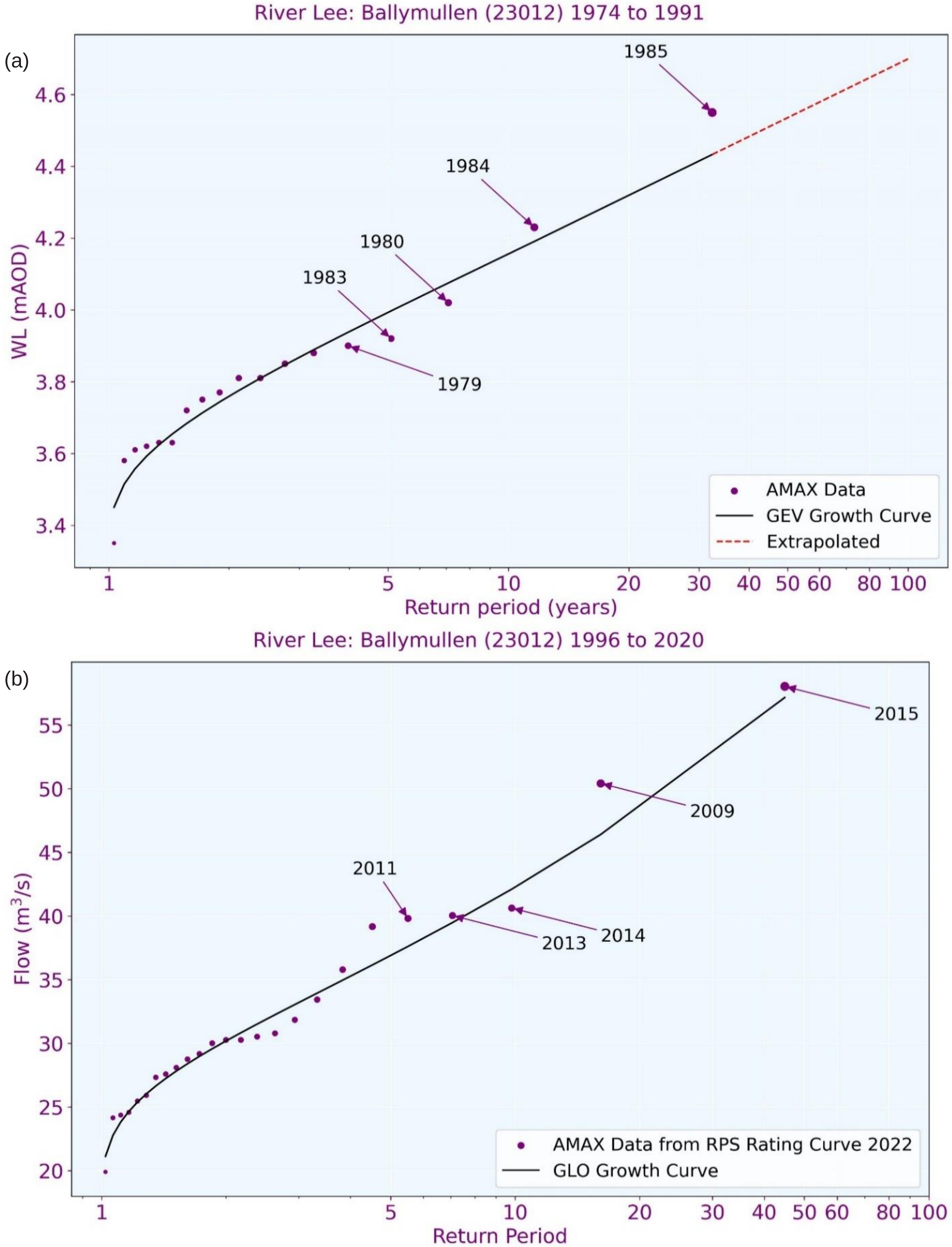


Figure 3.4: Flood Frequency Curve Fitted to AMAX data (a) pre-and (b) post-Tralee Flood Relief Works for River Lee at Ballymullen 23012. The top five highest water level events are also highlighted with associated dates attached

Table 3.2: Estimated Return Period for Five Largest Peak Water Levels and Flood Events Affecting Tralee at Ballymullen 23012

Event Date	Hydrological Year	Peak Water Level (mAOD Malin)	Rank	Estimated Return Period (Years)
Pre-Flood Relief works				
5th Aug 1986	1985	4.5 (on the 4th Aug 1986)	1	≈ 30 – 60*
14th Aug 1985	1984	4.2	2	10 – 20
2nd Nov 1980	1980	4.0	3	≈ 5
16th Jan 1984	1983	3.9	4	≈ 4 – 5
15th Dec 1979	1979	3.9	5	≈ 4 – 5
Post Flood Relief works				
20th/21st Oct 2020	2020	2.7 / 2.2	-	< 1 (both)
8th/9th Feb 2020	2019	2.9 / 3.2	-	≈ 1 / 1 – 5
14th Oct 2019	2019	2.9	-	≈ 1
3rd Jan 2018	2017	2.9 (3.3 on the 2nd Jan 2018)	8	≈ 1 (1 – 5)
22nd Nov 2017	2017	3.2	-	1 – 5
26th Jan 2016	2015	3.4	-	≈ 4 – 5
29th/30th Dec 2015	2015	Missing data / 2.5	-	< 1
23rd Dec 2015	2015	2.7	-	< 1
5th Dec 2015	2015	4.0	1	40 – 50*
13/14th Sep 2015	2014	3.6 (14th Sept)	3	5 – 10
3rd Jan 2014	2013	2.9	-	≈ 1
24th Aug 2012	2011	2.8	-	≈ 1
29th Nov 2011	2011	3.5	5	5 – 10
18th Feb 2011	2010	2.5	-	< 1
7th Feb 2011	2010	3.0	21	1 – 2
19th Nov 2009	2009	3.8	2	20 – 30
13th Aug 2008	2007	3.5	12	5 – 10
26th Dec 2004	2004	no data	-	-
31st Aug 1997	1996	3.2	13	1 – 5

Note* From Extreme Value Analysis, return periods greater than two times the length of a record results in significant uncertainty in the estimated return period. Events are noted in the table to highlight heightened uncertainty in estimates.

3.2 Pluvial Event Analysis

Flood risk appears to be strongly influenced by both fluvial and heavy rainfall events, i.e., pluvial mechanisms, informed by the review of historical flood events (Table 1.2). Description of past flood events derived from multiple sources suggest that many may have been driven directly by rainfall resulting in pluvial and/or drainage system flooding.

There are three daily and one high intensity (15 min) rainfall gauges within the study area, as shown in **Figure 3.5**. The nearest daily rainfall gauge for which data is available from 1980 to present day is the Lisaboula station maintained by Met Éireann located approximately 6km south-east of Tralee town centre (latitude: 52.229, longitude: -9.649). Met Éireann also maintained two stations in the town of Tralee: Tralee (Clash) station (latitude: 52.269, longitude: -9.689) from January 1941 to February 1996, and Tralee UDC station (latitude: 52.267, longitude: -9.711) from May 1975 to September 2017. All stations have been considered when investigating the role of daily rainfall amounts in the occurrence of past flood events.

The location of Lisaboula, Tralee (Clash), and Tralee UDC stations could result in their daily rainfall records being representative of the rainfall frequency conditions in the scheme area. However, due to the small size of the catchment and the relatively short anticipated critical storm duration (i.e., high intensity), there is fair uncertainty in the application of the data when used as a proxy for flood frequency at the scheme area. Furthermore, the lack of hourly rainfall data at both stations means the data is limited in terms of estimating accurate rainfall frequency for high intensity events which are known to drive pluvial flooding. The nearest hourly rainfall gauge with a substantial record length is the Valentia Observatory station near Cahersiveen located nearly 51km to the south-west of Tralee. This meteorological station is too far away from the study area and is therefore not suitable for estimation of short duration high intensity events at Tralee.

The newly installed high intensity rainfall gauge at Lisloose Reservoir, station number 82302, captures rainfall amounts every 15 minutes since 17/08/2020. While this rainfall gauge is capable of capturing high intensity rainfall events, the relatively short record length and lack of recorded flood events during the time means understanding of pluvial driven flood events in Tralee is limited. As stated in Section 3.4.2.3 of the Project Brief, while station 82302 is temporally limited for use in this initial analysis, it’s data will be used to reduce uncertainty in any pluvial joint-probability approach, to calibrate hydraulic models, and to assists in the estimation of design flood parameters at a later stage.

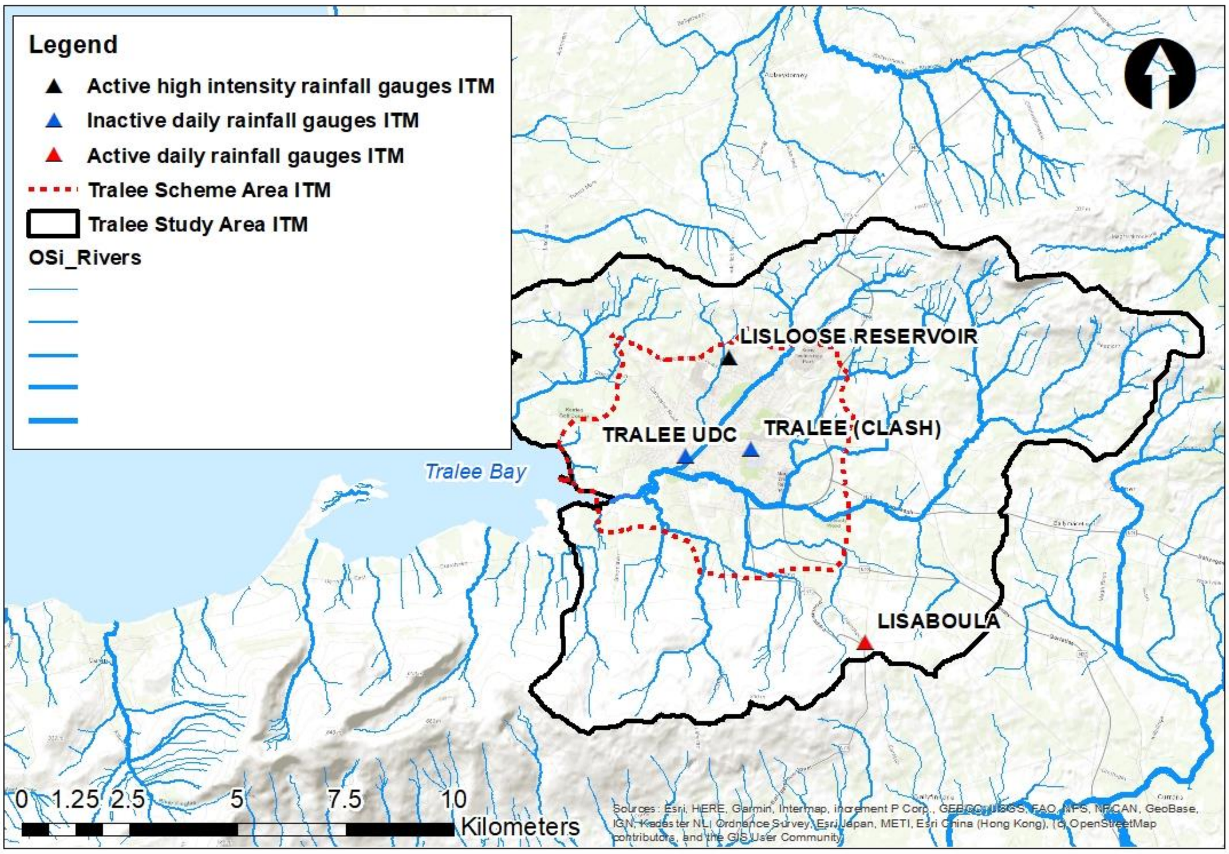


Figure 3.5 Daily and High Intensity Rainfall Gauges Considered Within this Initial Study

Analysis of daily rainfall events was undertaken by considering the recorded rainfall depth and duration against the FSU Depth, Duration, Frequency (DDF) model. The FSU model covers the study area with 2 km grid points and is derived from long term rainfall records (1941-2004)[29]. The FSU DDF was undertaken on an all-island dataset and can provide estimated return periods for defined daily rainfall amounts on a fixed (24 hr) or sliding (< 24 hr) timeframe. Return periods for daily rainfall events at Lisaboula, Tralee (Clash) and Tralee UDC stations are shown in **Table 3.3** for each flood event driven, or part-driven, by pluvial flood mechanisms.

Table 3.3: Rainfall Return Periods of Flood Events Affecting Tralee Noted to have had a Pluvial Influence in their Occurrence

Event Date	Daily Rainfall Sum (mm)	Daily Rainfall Return Period Years (AEP %)
Lisaboula Station		
8th/9th Feb 2020	8th: 50.7	2 (66.7%)
14th Oct 2019	13th: 22.9 14th: 14.8	< 1 for both
22nd Nov 2017	21st: 39.2	1 (90.9%)
5th Dec 2015	4th: 82.1 5th: 49 2-day combined event: 131.1	16 (6.4%) 1 (71.4%) 64 (1.6%)
24th Aug 2012	24th: 18.2 (Other reports say > 40mm fell in a 2.5hr period)	< 1
29th Nov 2011	28th: 52.0 29th: 5.9	8 (12.7%) < 1
6th Feb 2011	3-day period leading up to event (4th – 6th): 79.6	8 (12.3%)
19th Nov 2009	19th: 50 (approx. from reports)	7 (15.4%)
	5-day period leading up to event (15th – 19th): 140.8	117 (0.9%)
	15-day period (15th – 30th): 299.6	> 1000
13th Aug 2008	12th: 48.6	1 (71.4%)
31st Aug 1997	30th: 35.8 31st: 14.9 2-day combined event: 50.7	< 1 < 1 2 (52.5%)
5th Aug 1986	5th: 67.7	5 (20.8%)
2nd Nov 1980	1st: 71.3	7 (15.4%)
Tralee UDC station		
5th Dec 2015	4th: 34.0 5th: 43.2 Combined 2-day total: 77.2	2 (62.5%) 3 (29.4%) 15 (6.6%)
24th Aug 2012	24th: 40.0	3 (40%)
29th Nov 2011	28th: 51.8	8 (12.8%)
6th Feb 2011	3-day period leading up to event (4th – 6th): 79.7	8 (12.3%)
19th Nov 2009	5-day period leading up to event (15th – 19th): 100.4 15-day period (15th – 30th): 206.8	10 (10.4%) 42 (2.4%)
13th Aug 2008	3-day period leading up to event (11th – 13th): 66.8	3 (32.3%)

[29] Fitzgerald, D.L. (2007) Estimation of Point Rainfall Frequencies, Met Éireann, Irish Meteorological Service, Technical Note 61 ISSN 1393-905X

Event Date	Daily Rainfall Sum (mm)	Daily Rainfall Return Period Years (AEP %)
31st Aug 1997	2-day period leading up to event (30th – 31st): 50.3	2 (52.6%)
Tralee Clash		
30th Nov 1973	28th: 22.7 29th: 13.5 30th: 62.5 Combined 3-day event: 98.7	 21 (4.7%) 33 (3.1%)
23rd/24th Dec 1968	(from station) 48-hr period: 55.8 (from reports) 48-hr period: 63.5	3 (37%) 5 (20%)

Extreme rainfall events on both a daily and multi-day timeframe have been considered on or near to past flood events where pluvial processes were expected to play a role.

It is difficult to infer with confidence whether daily extreme rainfall events did contribute to the recorded flood events. **Figure 3.6** below shows the hydrological response of the River Lee and Big River during two flood events on (1) 5th December 2015, and (2) 19th November 2009. The gauging stations located on both rivers register a rise in water level/flow after or on the day of the heavy rainfall event, most clearly seen in **Figure 3.6(a)**.

Figure 3.6(c) shows a delay in response of river flow at Tralee Clonalour to heavy rainfall. The poor visual correlation in rainfall/river flow between Lisaboula rainfall station and Ballymullen may be explained by the gauging station being located 6 km away from the rainfall gauge. Furthermore, human error in recording daily rainfall values as well as the issue of multi-day sums could also cause discrepancy and errors in the data. However, the CFRAM Study noted that the flow behaviour at Tralee Clonalour on the 19th of November 2009 may have been a result of a drowned weir as it displayed behaviour inconsistent with rainfall data and other hydrographs in the area. The importance of considering localised rainfall is also supported with the good correlation between peak in daily rainfall at Tralee UDC and peak in flow at Ballymullen during the 19th November 2009 flood event.

While daily extreme rainfall events ranging from 1 to 20-year return periods were recorded, as noted before, sub-daily and hourly extreme rainfall events are known to be the main driver of pluvial flooding. Interestingly, several extreme rainfall sums were considered against a range of durations highlighting extreme multi-day rainfall events, such as the combined two-day event on 4th and 5th December 2015 (64-year return value). It is important to further note that these longer rainfall events may not coincide with shorter duration, high intensity rainfall events meaning they may not translate to the same flood return period for the small to medium size catchments affecting Tralee. For example, on 24th August 2012 the Lisaboula station recorded 18.9 mm of rainfall over the 24hr period, whereas reports say > 40 mm fell at some locations in Tralee over a 2.5 hr period. This highlights the localised nature of extreme rainfall, as well as highlighting the driving factor of short duration, high intensity rainfall in flooding events at Tralee.

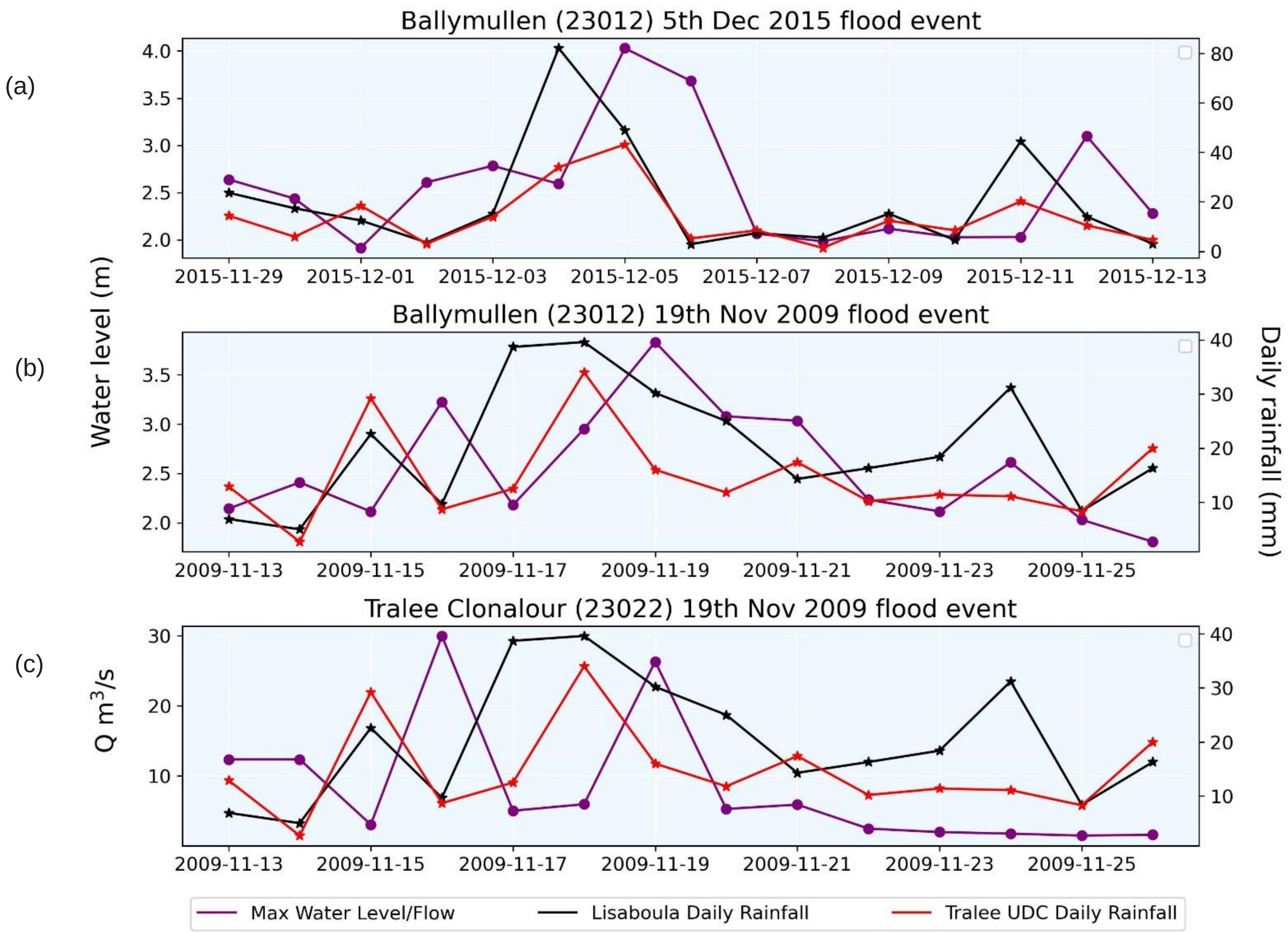


Figure 3.6: Daily Rainfall Amounts for Flood Events with Expected Pluvial Mechanism, (a) 5th December 2015, (b) and (c) 19th November 2009

4 DESIGN FLOW ESTIMATION

The design flow estimation requires a review of all available hydrological / hydrometeorological data up to the complete hydrometric year 2020 and employing FSU and Flood Studies Report (FSR) techniques to predict flood discharges at various locations across the modelled extents.

The FSU methods follow the steps of estimating peak flows using an index-flood method, growth curve development and later determining a design hydrograph around this peak flow using the Hydrograph Width Analysis (HWA) approach. The estimated hydrograph is then linked/justified with observed hydrograph shape data from the study area where it is available.

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

Following the rating reviews carried out for all hydrometric stations in the study area (see Section 2), Ballymullen gauging station downstream on the River Lee was the only station which offered a robust long-term hydrometric record; thus, a rating review was produced with relevant confidence that it accurately captures the catchment behaviour. While Tralee Clonalour has a long-term hydrometric record up to 2012 it is not recommended to use this record for design flow estimates as the catchment behaviour may have changed significantly since 2012.

Generally, the techniques employed in Tralee are largely reliant on FSU statistical methods given the lack of robust long-term hydrometric data on the majority of watercourses in the study area. Due to the presence of heavily urbanised areas within the Tralee FRS scheme area, particularly within the city, RPS may also use a rainfall runoff modelling based approach for simulating the design flood hydrographs. This will be undertaken using the InfoWorks ICM modelling suite. The model will be calibrated/verified using observed rainfall records at the locations discussed in Section 0 and also using the short-term rainfall, water level and flow monitoring data/records. Peak flows for all HEPs located on the smaller river/stream catchments will also be estimated using the FSU recommended statistical methods for comparison purposes.

4.1 HEP Physical Catchment Descriptors

A detailed hydrological analysis was undertaken to determine the flood flow hydrographs and peak design flows for the Tralee study area. The design flow estimation requires a comprehensive understanding of the physical catchment characteristics/descriptors (PCDs) throughout the Tralee study area. Hydrological Estimation Points (HEPs) have been specified at the locations where peak design flows have been estimated.

HEPs have been established at the most upstream and downstream extremities of the Tralee hydraulic model, i.e., at the upstream modelled extents of the River Lee and other watercourses in the study area.

There are intermediate HEPs along the modelled watercourses to fully capture the changing flood frequency conditions as the catchment land-use and physical characteristics change moving from the upstream to downstream of each of the modelled watercourses. Intermediate HEPs have also been specified upstream and downstream of significant incoming tributary catchments to capture the changing flood frequency conditions on the main channel watercourse as a result of the incoming tributary flows. Intermediate HEPs have been located at each point along the watercourse where the area of the catchment increases by roughly 10%, This was to avoid unnecessary analysis as where a watercourse catchment does not significantly change over a distance greater than 1000m. Reporting nodes can be added post modelling in addition to HEPs if required. The intermediate HEPs act as checkpoints along the modelled watercourses to ensure that the Tralee hydraulic model is anchored to the hydrological analysis.

Tributary HEPs have been specified at the downstream extents of modelled tributary watercourses and where significant unmodelled tributary watercourses enter the modelled watercourse.

HEPs have also been placed at significant identified discharge points from the surface water sewer network.

Subject to the availability of appropriate catchment data, PCDs have been updated from the FSU-defined PCDs where the datasets needed to do so were available. For example, the extent of urbanisation (URBEXT), drainage density (DRAIND), and mean average slope (S1085) were updated for each individual sub-catchment throughout the Tralee study area. DRAIND was re-calculated following the review of the catchment delineation. S1085 was re-calculated using the LiDAR coverage for the Tralee study area. Finally, a PCD that can significantly influence the hydrological analysis is URBEXT. This was updated through an analysis of the most up to date CORINE Land Use Mapping. This allowed the URBEXT PCD to be updated to capture the most recent changes in urban extent in the Tralee study area.

The specification of HEPs is presented in **Figure 4.1**. **Table 4.1** provides a summary of the PCDs assigned at the upstream and downstream extent of the main modelled watercourses within the Tralee scheme area.

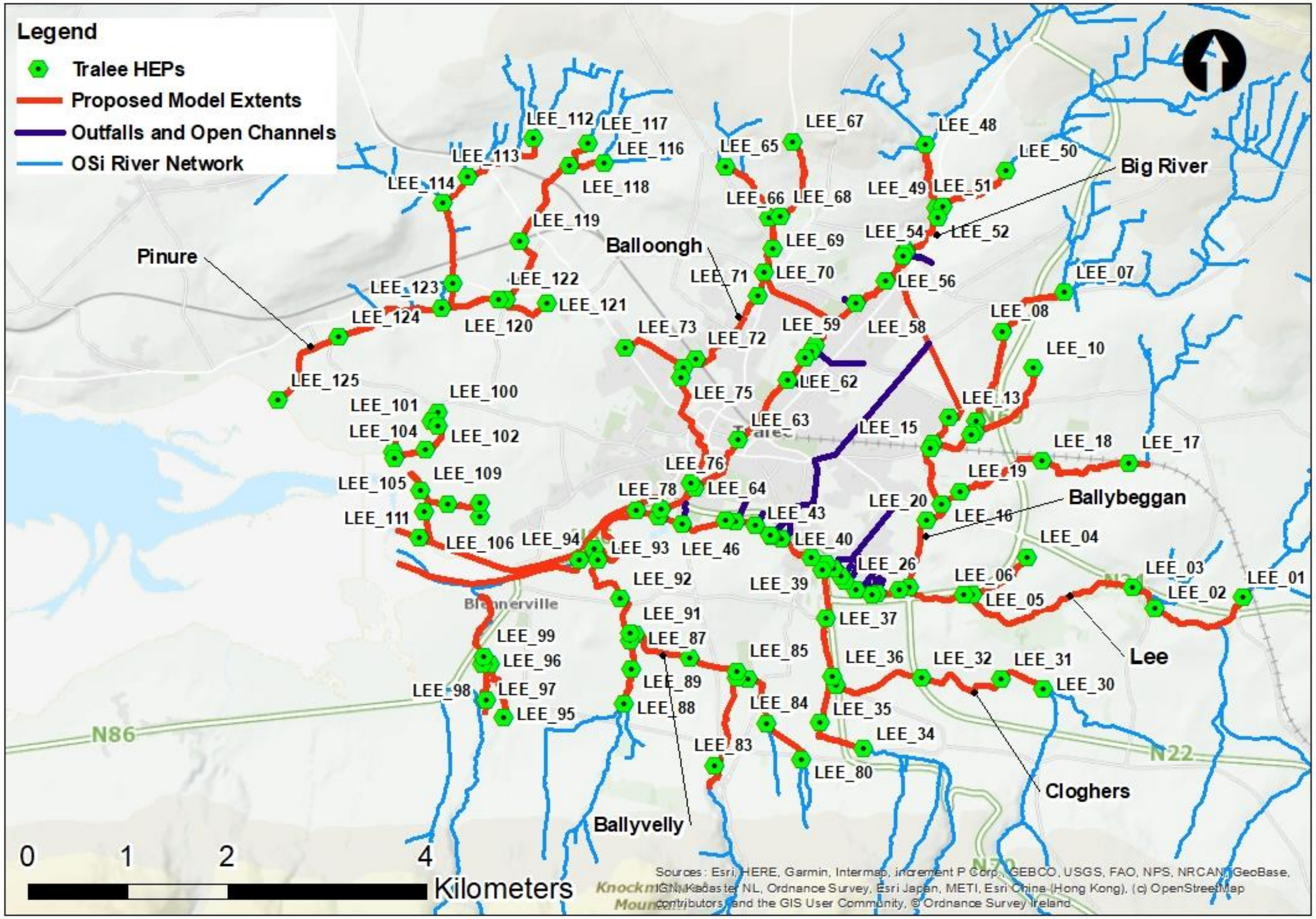


Figure 4.1: Hydrological Estimation Points (HEPs) and Schematisation for Tralee

Table 4.1: Physical Catchment Descriptors for Tralee

HEP	Watercourse (EPA-Defined Name)	Area (km^2)	Centroid X	Centroid Y	SAAR (mm)	FARL	BFISoil	URBEXT (%)
Main Channel								
LEE_02	Lee (Ballyseedy GS)	23.08	488811	612984	1227	0.997	0.461	0.00
LEE_39	Lee (Ballymullen GS)	60.81	484483	613390	1264	0.999	0.465	2.42
LEE_94	Lee	97.55	482151	613367	1252	0.999	0.422	9.48
Tributaries								
LEE_07	Ballybeggan	4.86	487009	616047	1175	1.000	0.349	0.00
LEE_20	Ballybeggan (Manor West GS)	11.48	485635	613758	1198	1.000	0.350	3.82
LEE_21	Ballybeggan	11.97	485461	613084	1200	1.000	0.350	4.41
LEE_30	Cloghers	6.09	486804	612072	1354	1.000	0.458	0.00
LEE_37	Cloghers (Spring Water Lane GS)	13.28	484627	612780	1348	1.000	0.454	0.23
LEE_38	Cloghers	13.95	484588	613264	1343	1.000	0.455	3.19
LEE_48	Big River	4.40	485620	617528	1102	1.000	0.351	4.40
LEE_56	Big River (Oakpark Road GS)	9.08	485218	616158	1125	1.000	0.351	8.11
LEE_69	Balloonagh (Lisloose GS)	1.78	484091	616484	1116	1.000	0.351	0.00
LEE_76	Balloonagh	6.45	483270	614129.4	1116	1.000	0.320	6.73
LEE_77	Big River (Ballyard GS)	17.96	482972	613871	1132	1.000	0.348	25.68
LEE_83	Ballyvelly	6.19	483510	611303.4	1424	1.000	0.347	0.00
LEE_93	Ballyvelly	13.99	482337	613360	1370	1.000	0.369	0.56
LEE_115	Pinure	5.69	480879	616138	1106	1.000	0.457	5.66
LEE_125	Pinure	14.03	479131	614963	1107	1.000	0.463	7.58

Where:

FARL – Index of flood attenuation by reservoirs and lakes;
SAAR – Standard average annual rainfall (mm);
URBEXT – Proportional extent of catchment area mapped as urbanised;
AREA – Individual area of each catchment computed based on GIS based catchment delineation (km^2);
BFI_{SOIL} – Soil Base Flow Index (estimate of BFI derived from soils, geology, and climate data).

4.2 Index Flood Flow (Qmed) Estimation

The first step in determining the design flows for the various HEPs will be the estimation of the Index Flood Flow (Q_{med}) at each HEP. Q_{med} is a crucial flood statistic as it can be robustly determined from suitable gauged locations with a significant record length. Seen in **Figure 4.2**, the AMAX and POT (Peaks Over Threshold) techniques yield Q_{med} estimates with low standard factorial error after 2 to 5 years of data compared to estimates based on PCD equations. This should be the minimum record length considered to produce an estimate of Q_{med} from observed data.

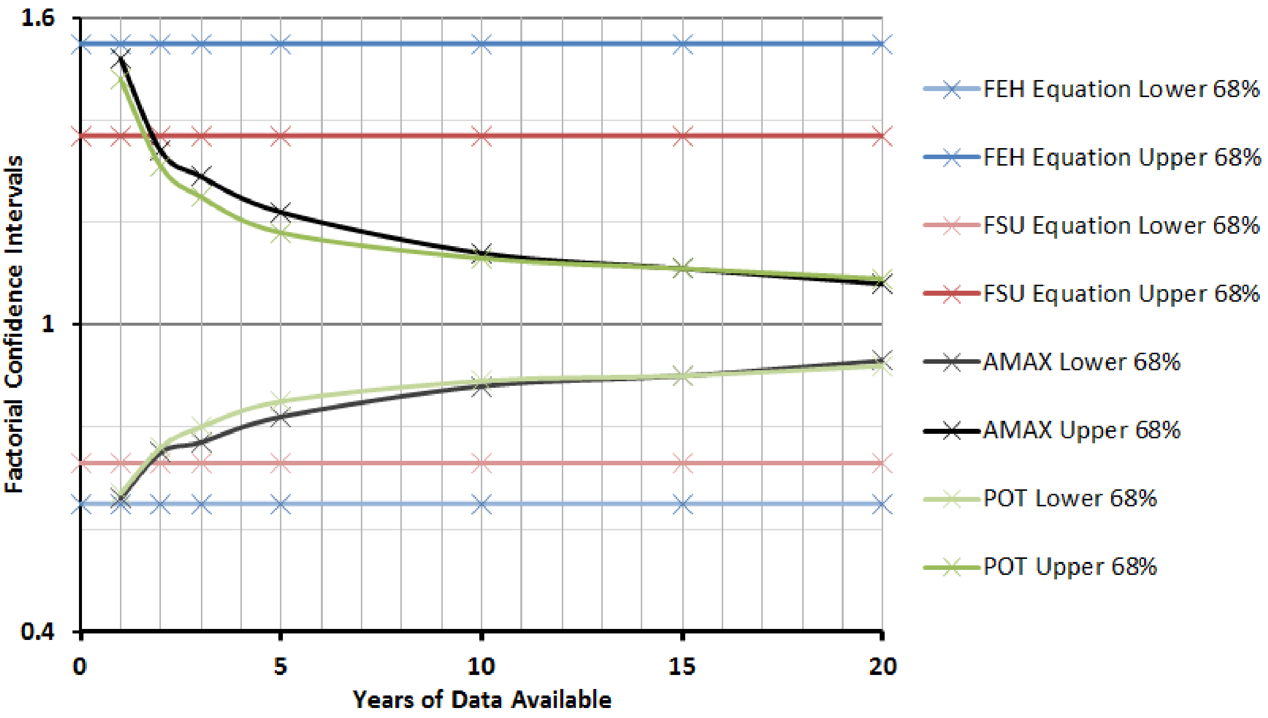


Figure 4.2: Standard Factorial Error Based on Different Techniques Used to Estimate Q_{med}

Hydrometric data is available across the Tralee FRS study area. Following rating reviews carried out for all gauging stations, confidence in rating curves were determined. Due to small record lengths at many stations, confidence was too low and uncertainty high in Q_{med} estimates. At such stations, PCD equations were employed.

Ballymullen gauging station was the exception with a rating curve being produced with high confidence. An observed Q_{med} was available on the River Lee due to long-term hydrological record at Ballymullen gauging station. This observed Q_{med} has been interrogated as part of the rating review at the station (Section 2.1). This allows for the anchoring of the hydrological analysis on the River Lee to robust observed data within the model extents. This is a noted improvement on the CFRAM Study which had no observed estimate of Q_{med} on the River Lee due to concern the station was tidally influenced.

Design flow estimates of Q_{med} on all other watercourses are estimated using the ungauged approach to Q_{med} estimation under FSU.

4.2.1 Q_{med} at gauged locations

Table 4.2 shows the observed Q_{med} following rating review and the available record period for hydrometric data at the Ballymullen gauging station.

Table 4.2: Observed Q_{med} at Ballymullen (23012)

River	Gauge Location	Number	Q_{med} (m^3/s)	Q_{med} Record Period
Lee	Ballymullen	23012	30.26	1996-2021

This represents a robust estimation of Q_{med} in the River Lee. The rating review has increased the confidence in the rating at the gauging station at higher flows and validated the observed Q_{med} to be used during the current hydrological analysis.

4.2.2 Qmed at Ungauged Locations

The estimation of Q_{med} using ungauged methods is required on the majority of watercourses and all HEP checkpoints along modelled watercourses. These estimates are initially treated as ungauged but receive a pivotal site adjustment to ensure the peak design flows are informed by the gauged data within the Tralee study area and from appropriate hydrologically similar gauged sites regionally / nationally.

Estimation of Q_{med} for ungauged catchments in their rural form, referred to henceforth as $Q_{med\ rural}$, will initially be based on the FSU methods for ungauged catchments, i.e., using regression equations derived from both FSU Work Packages 2.3 and 4.2 to calculate the $Q_{med\ rural}$. FEH methods for the UK will also be used for comparative purposes while catchments less than 25km^2, as defined in the Project Brief Section 3.4.3.3 Design Event Flow Estimation Methods, four methods will be considered as shown in **Table 4.3**.

The FSU 7 Variable equation has been derived through regression analysis of the observed records and associated various PCDs from 190 stations in Ireland. The equation has a Factorial Standard Error (FSE) of 1.37. This method is the standard FSU equation to be considered across all catchments, but it has been found to have significant limitations when applied across some small and / or urbanised catchments.

To deal with the above-mentioned limitations using the 7-variable equation, an attempt will be made to improve Q_{med} estimates for the smaller catchments using the 5-variable regression-based method recommended by the FSU work package 4.2. The FSU 5 Variable equation is an alternative method to the FSU 7 Variable equation and is suitable for small catchments, particularly those where the physical catchment descriptors, such as drainage density, are poorly defined. FSU Work Package 4.2 highlights that the FSU5 Variable equation has the lowest factorial standard equation in terms of estimation at the small catchment scale.

Furthermore, the Rational Method[30] will be applied to smaller catchments where an equation to estimate critical storm duration for certain flood events will be applied to determine a rainfall intensity. This rainfall intensity will then be used with the FSU Rainfall Depth Duration Frequency (DDF) model found online at the FSU web portal to derive an estimate of the peak run-off.[31] A final decision on which method to be applied may be subject to review of the flood frequency as simulated through the hydraulic model.

Table 4.3: Methodologies to be Employed when Determining Design Flow Estimations at Ungauged Locations

Methodologies for $Q_{med\ rural}$
FSU 7 Variable (standard method): $$Q_{med\ rural} = 1.27x10^{-5} * AREA^{0.937} * BFI_{SOIL}^{-0.922} * SAAR^{1.306} * FARL^{2.217} * DRAIND^{0.341} * S1085^{0.185} * (1 + ARTDRAIN2)^{0.408}$$
FEH: $$Q_{med\ rural} = 8.3062 * AREA^{0.851} * 0.153^{\frac{1000}{SAAR}} * FARL^{3.4451} * 0.0460^{BFIHOST^2}$$
Methodologies for Q_{med} in catchments < 25km^2
FSU 5 Variable (small catchments): $$Q_{med\ rural} = 2.0951x10^{-5} * AREA^{0.9245} * BFI_{SOIL}^{-0.9030} * SAAR^{1.2695} * FARL^{2.3163} * S1085^{0.2513}$$
Rational Method: $$Q_{med\ rural} = CiA$$ where C is the runoff coefficient, i is the rainfall intensity (mm/hr) based on intensity duration frequency data, and A is the area (km^2)

[30] Microsoft Word - FSU Work Package 4.2 draft final.doc (hydronet.com)

[31] FSU Web Portal - Applications - Rainfall and Flood Estimation Applications (hydronet.com)

4.2.3 Pivotal Site Adjustment

FSU recommends that the ungauged Q_{med} estimates (i.e., $Q_{med\ rural}$) are adjusted using a 'pivotal site'; a geographically close or hydrologically similar gauged catchment that can be used to reduce the error and level of uncertainty in the estimates. The basis of this adjustment is grounded in the fact that Q_{med} estimation based on the physical catchment descriptor regression equations can be highly uncertain as it has been generalised for application across a wide range of catchments.

However, this estimation can be improved, and the uncertainty reduced by using data transfer methods from other nearby or hydrologically similar gauged catchments. This approach borrows observed flood flow data from an appropriately gauged subject catchment, which is known as the 'pivotal site' in FSU methodologies. Preference can be given to hydrologically similar gauges that are geographically close to the area of interest. Sites brought forward for use as pivotal sites must be sufficiently robust in terms of their rating (confidence at Q_{med}) and the Q_{med} value used will be derived from the most up to date AMAX data.

A comprehensive review of nearby and hydrologically similar gauging stations was conducted in relation to the Tralee study area. At the time of the CFRAM Study, the main channel and tributary reaches were adjusted based on the top 5 hydrologically similar sites for each reach due to no suitable gauge for Q_{med} adjustment being found within the study area. A weighted average of the top 5 hydrologically similar sites was taken.

The Ballymullen gauging station is considered appropriate as a pivotal site for the River Lee. Adjusting the peak design flows based on a site that has a significant record length and rating curve improves the statistical confidence significantly.

The adjustment is considered in relation to the catchment in its rural state such that the effects of urbanisation from the pivotal site are not transferred. In light of this, the estimated effect of urbanisation may need to be stripped from the observed Q_{med} value by dividing the observed value by the Urban Adjustment Factor (UAF).

In the case of the Ballymullen gauging station, urbanisation is approximately 2.4% and therefore is not considered to have a significant effect on the observed Q_{med} value. The observed and FSU7 variable equation predicted Q_{med} values, and the resulting pivotal site adjustment factor (AdjFac), is shown in **Table 4.4**.

Table 4.4: Pivotal Site Adjustment Factor (AdjFac) for River Lee

River	Gauge Location	Number	HEP	Q_{med} Observed (m^3/s)	$Q_{med\ rural}$ Predicted (m^3/s) – FSU7	Adjustment Factor (AdjFac)
Lee	Ballymullen	23012	LEE_37	30.26	24.59	1.23

Table 4.4 suggests that the FSU7 variable catchment descriptor equation is underestimating the Q_{med} on the River Lee. This suggests that the $Q_{med\ rural}$ estimates require an upwards adjustment of approximately 23%, given the adjustment factor of 1.23.

The AdjFac obtained from the Ballymullen gauging station could be used for the other watercourses in the Tralee FRS study area, however, there is uncertainty in applying this adjustment factor to the smaller watercourses and tributaries. The catchment represented by the Ballymullen gauging station is significantly larger than the tributaries of the River Lee, however, where hydrologically similar, it may be included in the group of pivotal sites.

A further review of available small catchment hydrometric gauging stations which are geographically close and hydrologically similar for use as a pivotal site for the tributaries was conducted for each major tributary of the River Lee and for the Pinure watercourse in the north-west of the study area.

A similar approach was adopted for all tributary and other major reaches as in the CFRAM Study whereby the top five nearby or hydrologically similar gauging stations with a robust Q_{med} were selected to obtain a pivotal site adjustment factor. If five stations were not available, a minimum of two stations were required.

The tributary catchments were grouped into two main categories:

1. Small tributary catchments (area ≤ 25 km^2),
2. Large tributary catchments (area > 25 km^2).

For small and large tributary catchments, pivotal sites were determined for the HEPs at the downstream extent of every major tributary. Where smaller tributaries exist, i.e., tributaries of major tributaries, an average of the small tributary adjustment factor was taken and applied in this case.

The adjustments are again considered in relation to the catchments in their rural state such that the effects of any urbanisation from the pivotal site are not transferred. Where urbanisation for a catchment was greater than 2.5%, the effect of urbanisation was stripped from the observed Q_{med} value. The observed and FSU5 variable equation predicted Q_{med} values, and the resulting pivotal site adjustment factor for small catchment tributaries are shown in **Table 4.5**.

Table 4.5: Pivotal Site Adjustment Factors (AdjFacs) for Small Catchment Tributaries

River	Gauge Location	Number	Q_{med} Observed (m^3/s)	$Q_{med\ rural}$ Predicted (m^3/s) – FSU5	Adjustment Factor (AdjFac)
Oakpark Road Gauging Station (Big River) – LEE_56					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Weighted Average AdjFac					0.89
Lisloose Gauging Station (Balloonagh) – LEE_69					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Slang	Frankport	9011	1.19	1.86	0.64*
Weighted Average AdjFac					0.80
Balloonagh – LEE_76					
Swilly	New Mills	39001	47.70	57.16	0.84
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Glenamoy	Glenamoy	33001	59.30	67.09	0.88
Weighted Average AdjFac					0.89
Unknown Tributary – LEE_99					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Slang	Frankport	9011	1.19	1.86	0.64*
Carrickmines	Cabinteely	10022	2.62	3.26	0.80*
Weighted Average AdjFac					0.80
Unknown Tributary – LEE_111					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Carrickmines	Cabinteely	10022	2.62	3.26	0.80*
Slang	Frankport	9011	1.19	1.86	0.64*
Weighted Average AdjFac					0.76
Manor West Gauging Station (Ballybeggan) – HEP_20					

River	Gauge Location	Number	Q_{med} Observed (m^3/s)	$Q_{med\ rural}$ Predicted (m^3/s) – FSU5	Adjustment Factor (AdjFac)
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Mountmellick	Owenass	14033	29.70	34.15	0.87
Lee	Ballymullen	23012	30.26	29.94	1.01
Weighted Average AdjFac					0.91
Spring Water Lane Gauging Station (Cloghers) – HEP_37					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Lee	Ballymullen	23012	30.26	29.94	1.01
Weighted Average AdjFac					0.93
Big River – LEE_77					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Lee	Ballymullen	23012	30.26	29.94	1.01
Glenamoy	Glenamoy	33001	59.30	67.09	0.88
Weighted Average AdjFac					0.92
Ballyvelly – LEE_93					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Yellow	Kiltybardan	36021	23.37	24.91	0.94
Lee	Ballymullen	23012	30.26	29.94	1.01
Weighted Average AdjFac					0.93
Pinure – LEE_125					
Deenagh (Laune)	White Bridge	22009	10.55	12.62	0.84*
Mountmellick	Owenass	14033	29.70	34.15	0.87
Lee	Ballymullen	23012	30.26	29.94	1.01
Weighted Average AdjFac					0.91
Weighted Average AdjFac for Small Catchments					0.79

* This adjustment factor considers the effects of urbanisation and removes it from the observed Q_{med}

Table 4.5 suggests the FSU5 variable equation is overestimating Q_{med} on small catchment tributaries in the Tralee study area. An average adjustment factor of 0.79 would result in a downward adjustment of the FSU5 estimation by 21%. There does exist high uncertainty in Q_{med} on small catchments in Ireland due to the lack of hydrometric gauge data for small catchments. This is reflected in the small number of suitable pivotal sites that were available for this analysis. There is therefore significant uncertainty in adjustment factors on a catchment-by-catchment basis.

A large catchment in the study area was defined as a catchment > 25 km^2. The River Lee catchment was the only catchment in the Tralee FRS that met the area threshold for large catchments.

4.2.4 Urban Adjustment

This $Q_{med\ rural}$ value does not consider the effects of urbanisation which is considered separately through an Urban Adjustment Factor (UAF) calculated as follows:

$$UAF = (1+URBEXT)^{1.482}$$

The final Q_{med} which considers the effect of urbanisation is then calculated:

$$Q_{med\ urban} = UAF^{*}\ Q_{med\ rural}$$

The urban adjustment has been applied to all HEPs on both the main channel and the tributaries in the Tralee FRS study area where the level of urbanisation is deemed to have a potential impact on run-off and the peak design flows. In general, any HEP that has an URBEXT value of greater than 2.5% (0.025) has had the urban adjustment factor applied.

4.2.5 Final Q_{med} Design Flow Estimates

In addition to the FSU methods, the FEH method and a rainfall run-off method were employed for catchments less than 25 km^2 to estimate Q_{med}. The FEH method resulted in Q_{med} estimates which were on average 47% and 17% greater than the FSU7 and FSU5 estimates, respectively. Rational method resulted in Q_{med} estimates 12% and 16% greater than FSU5 for catchments less than 25 km^2 and less than 5 km^2, respectively.

It is RPS' preference to use the FSU5 estimate for catchments less than 25 km^2 and FSU7 for larger catchments. Where necessary and potentially in small, urbanised catchments, the hydraulic model will be driven by rainfall directly.

Following the urban adjustment procedure, the $Q_{med\ urban}$ estimates for each sub-catchment then had their pivotal site adjustment applied to give the final Q_{med} estimate ($Q_{med\ urban\ adj.}$). **Table 4.6** provides the final Q_{med} design flow estimates for the Tralee FRS study area.

Table 4.6 Final Q_{med} Design Flow Estimates, Tralee

HEP	Flood Estimation Method	Q_{med} rural (m^3/s)	Pivotal Site (Gauging Station No.)	Urban Adjustment Factor	Pivotal Site Adjustment Factor	Final Q_{med} (m^3/s)
Main Channel						
LEE_02	FSU7	13.49	23012	1.00	1.23	16.60
LEE_39	FSU7	24.59	23012	1.00	1.23	30.26
LEE_94	FSU7	39.03	23012	1.14	1.23	54.94
Tributaries						
LEE_21	FSU5	9.23	22009, 36021, 14033, 23012	1.07	0.91	8.99
LEE_38	FSU5	10.08	22009, 36021, 23012	1.05	0.93	9.80
LEE_76	FSU5	4.85	39001, 36021, 33001	1.10	0.89	4.73
LEE_77	FSU5	11.92	22009, 36021, 23012, 333001	1.34	0.92	14.62
LEE_93	FSU5	14.37	22009, 36021, 23012	1.00	0.93	13.34
LEE_99	FSU5	1.37	22009, 36021, 9011, 10022	1.00	0.80	1.10
LEE_111	FSU5	0.97	22009, 10022, 9011	1.11	0.76	0.82
LEE_125	FSU5	6.64	22009, 36021, 23012	1.11	0.91	6.70

4.3 Growth Curve Development

To derive design event peak flows for the range of event probabilities / return periods (50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP), a growth curve must be developed. The growth curve defines the relationship between Q_{med} and the various event probability peak flows (Q_T).

With observed data, a growth curve can be defined from available AMAX data from a reliable gauging station. This applies to the River Lee given the presence of the Ballymullen gauging station. However, in the absence of observed hydrometric data of significant length the development of a growth curve relies on pooled analysis. This involves forming an estimated growth curve using key statistical characteristics of observed AMAX data from a pooled group of hydrologically similar gauged catchments.

4.3.1 CFRAM Study Growth Curve Development

Growth curve development was previously undertaken as part of the CFRAM Study based on the FSU methodologies to determine growth factors for a range of design events. The growth factors derived for the Tralee hydraulic model from the CFRAM Study are presented in **Table 4.7**.

Table 4.7: CFRAM Study Growth Curves

Tralee Model Reach	Growth Factors at AEP (Return Period)							
	50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
River Lee	1.00	1.24	1.39	1.54	1.73	1.88	2.02	2.36
Big River	1.00	1.26	1.43	1.59	1.80	1.96	2.12	2.49
Ballybeggan	1.00	1.25	1.41	1.56	1.77	1.92	2.07	2.42
Cloghers	1.00	1.26	1.42	1.59	1.80	1.95	2.11	2.47
Ballyvelly	1.00	1.26	1.44	1.60	1.82	1.98	2.14	2.51

4.3.2 At-Site Growth Curve

A single site analysis was conducted at the Ballymullen gauging station post-rating review (see Section 3). The at-site growth curve is extracted from the flood frequency curve by taking the ratio of estimated peak flow to the observed Q_{med} value. The GLO was the best performing distribution during the at-site analysis. The at-site GLO growth curve for the Ballymullen gauging station is shown in **Figure 4.3** with the CFRAM adopted growth curve for the Lee.

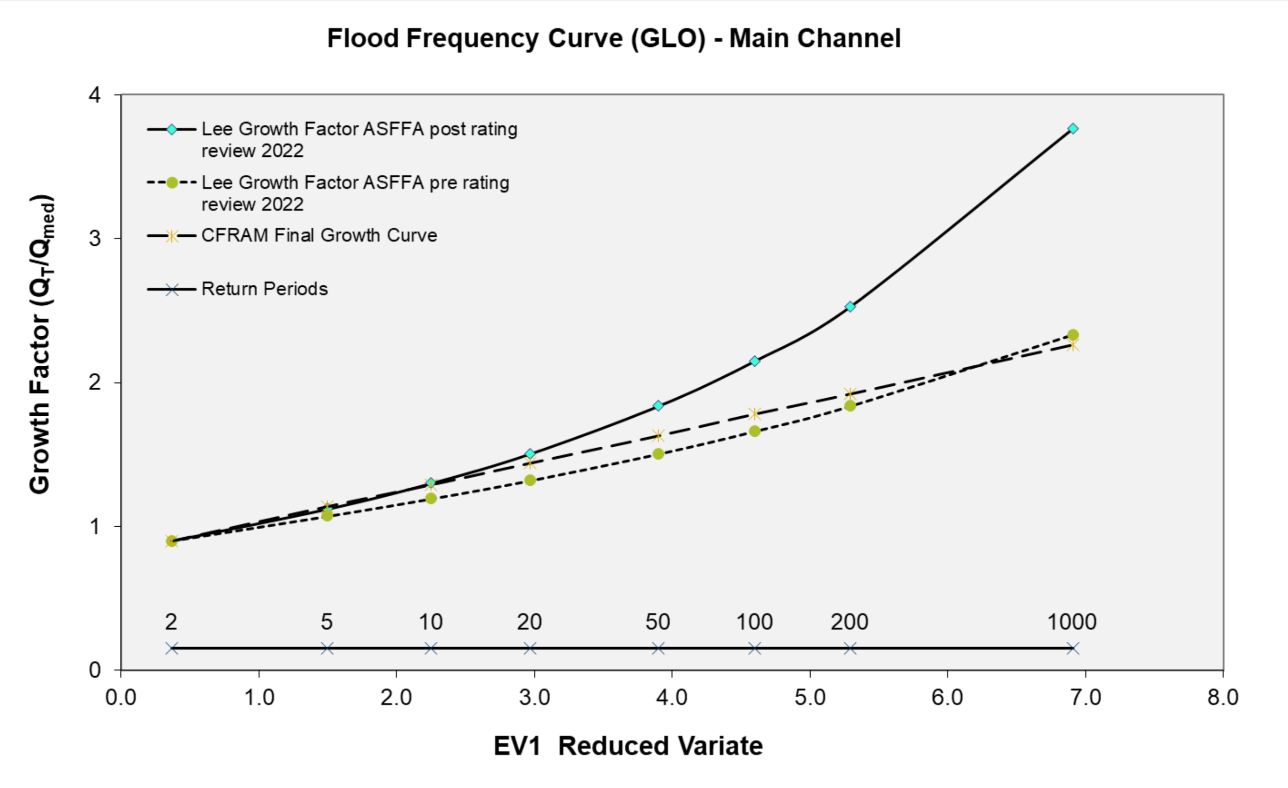


Figure 4.3: At-Site Growth Curve Pre- and Post-Rating Review, Ballymullen (23012) (ASFFA = At-Site Flood Frequency Analysis)

The at-site growth curve developed post-rating review (2022) is much steeper than the pre-rating review growth curve resulting in higher growth factors. The growth curve adopted in CFRAM followed that of the pre-rating review growth curve, thus the growth curve adopted in this analysis is also much steeper than that adopted on the River Lee in the CFRAM Study.

The at-site growth curve developed for the Ballymullen gauging station is based on 26 years of AMAX flood flow data. Following the rating review, the AMAX values are considered to be more robust, and the flood frequency curve is suitable for the basis of flood flow estimation. However, as noted in **Table 4.8**, it is only recommended that the at-site analysis is used as the basis for design flow estimation if there are more than *T* years of AMAX data, where *T* is the design return period. In other words, the Ballymullen at-site analysis is only recommended for use as the basis for design flow estimation up to the 26-year return period (i.e., for Q2, Q5, Q10, Q20).

Therefore, for design flow estimation greater than 26-year return period, a pooled analysis will need to be carried out to obtain the most robust growth curve for Ballymullen and the River Lee.

An at-site growth curve was not constructed for any other gauging station in the Tralee study area as few had sufficient record length which also led to high uncertainty in the rating reviews carried out. For that reason, growth curves for all other major tributaries and reaches in the Tralee study area will need to be developed by pooled analysis techniques.

Table 4.8: Selection of Growth Curves Methodology, Adapted from FEH Volume 3 Tables 8.2 and 8.3

Record Length	At-Site Analysis	Pooled Analysis	Preferred Method
< T/2 or 14 years	No	Yes	Pooled
T/2 to T years	For confirmation	Yes	Pooled
T to 2T years	Yes	Yes	Joint
> 2T years	Yes	For Confirmation	At-Site

4.3.3 Development of Pooled Growth Curves

The pooled analysis has been updated and re-analysed for the Tralee study area. There were limitations to the current FSU database from which pooled growth curves are derived. An update of the FSU database was conducted to include all hydrometric data and AMAX years up to the latest hydrometric year (end of the hydrometric year 2019 / 2020). This update was conducted on sites that were included in the pooling group for Tralee FRS (i.e., based on hydrological similarity).

This update was applied to FSU sites which have recently been determined to be robust for pooling group analysis by the OPW. This was to ensure the updated pooled analysis included the most recent, robust hydrometric data to inform the development of new growth curves. FSU sites not included within the list were assumed to be robust up to 2004 (as per the original FSU classification) and were retained.

Pooled analysis and pooled growth curves were developed at multiple locations across the Tralee study area to fully capture the changing flood frequency conditions. A 100-year target return period was used.

Pooled growth curves were developed at three points along the main channel to capture the changing frequency conditions along the modelled reaches:

- Upper – LEE_02
- Middle – LEE_39
- Lower – LEE_94

Pooled growth curves were then developed at each downstream extent of every major tributary of the main channel:

- Ballybeggan – LEE_20
- Cloghers – LEE_37
- Balloonagh – LEE_76
- Big River – LEE_77
- Ballyvelly – LEE_93
- Unknown Tributary – LEE_99
- Unknown Tributary – LEE_111
- Pinure – LEE_125

The final pooled stations for each location on the main channel are provided in **Table 4.9** and for tributaries in **Table 4.10**.

Table 4.9: Final Pooled Stations for Main Channel

LEE_002	LEE_39	LEE_94
BALLYMULLEN (23012) MOUNTMELLICK (14033) CURRAGHBONAUN (34009) BILLA BR. (35002) AGIVEY (3028) AUGHNAGROSS (16005) CAPPAMORE (25158) BIG BRIDGE (35004) BURN DENNET (1007) FOURMILEWATER (16013) LAGAN (5008) ENLER (5020) LARAGH (10004) FAULKLAND (3051) BALLYHAUNIS (30020)	BALLYMULLEN (23012) MOUNTMELLICK (14033) CURRAGHBONAUN (34009) BILLA BR. (35002) BURN DENNET (1007) CAPPAMORE (25158) AGIVEY (3028) BIG BRIDGE (35004) KILTIMAGH (34024) AUGHNAGROSS (16005) FAULKLAND (3051) FOURMILEWATER (16013) BLACKWATER (3022) WILLSBROOK (26006)	AGIVEY (3028) CURRAGHBONAUN (34009) BALLYMULLEN (23012) MOUNTMELLICK (14033) BURN DENNET (1007) SANDY MILLS (1041) BLACKWATER (3022) FAULKLAND (3051) BIG BRIDGE (35004) LAGAN (5008) KNOCKADANGAN (34029) COLEBROOKE (6005) WILLSBROOK (26006) RIVERVILLE (22003) CAPPAMORE (25158) MAIN (3011)

Table 4.10: Final Pooled Stations for Tributaries

LEE_20	LEE_37	LEE_76
KILTYBARDAN (36021) INCH BR. (27001) BALLYMULLEN (23012) LAGAN (5008) AGIVEY (3028) LOUISBURG WEIR (32011) MOUNTMELLICK (14033) GLENAMOY (33001) SANDY MILLS (1041) RAVERNET (5005) DUNCANNON (18016) ENLER (5020) NEW MILLS (39001) CURRAGHBONAUN (34009)	BALLYMULLEN (23012) LARAGH (10004) BALLYHAUNIS (30020) MOUNTMELLICK (14033) BILLA BR. (35002) BALLYGOLY (6030) AGIVEY (3028) KILTYBARDAN (36021) CURRAGHBONAUN (34009) FOURMILEWATER (16013) CAPPAMORE (25158) AUGHNAGROSS (16005) INCH BR. (27001) SANDY MILLS (1041)	KILTYBARDAN (36021) INCH BR. (27001) GLENAMOY (33001) BALLYMULLEN (23012) LAGAN (5008) LOUISBURG WEIR (32011) RAVERNET (5005) ENLER (5020) NEW MILLS (39001) AGIVEY (3028) MOUNTMELLICK (14033) BALLYBOGHIL (8012) SANDY MILLS (1041) DUNCANNON (18016) ASHBOURNE (8007)

LEE_77	LEE_93	LEE_99
INCH BR. (27001) KILTYBARDAN (36021) BALLYMULLEN (23012) LAGAN (5008) AGIVEY (3028) RAVERNET (5005) MOUNTMELLICK (14033) SANDY MILLS (1041) ENLER (5020) GLENAMOY (33001) DUNCANNON (18016) LOUISBURG WEIR (32011) CUSHER (3024) INCH BR. (23001)	KILTYBARDAN (36021) INCH BR. (27001) BALLYMULLEN (23012) LOUISBURG WEIR (32011) NEW MILLS (39001) AGIVEY (3028) SANDY MILLS (1041) DUNCANNON (18016) GLENAMOY (33001) MOUNTMELLICK (14033) LAGAN 5008) CURRAGHBONAUN (34009) LARAGH (10004) BALLYCARROON (34007)	BALLYGOLY (6030) KILTYBARDAN (36021) BALLYHAUNIS (30020) LARAGH (10004) BALLYMULLEN (23012) BALLYBOGHIL (8012) INCH BR. (27001) ROCHFORT (25034) MOUNTMELLICK (14033) ENLER (5020) CURRALHIR (6031) CLOBANNA (16051) RAVERNET (5005) LAGAN (5008) AGIVEY (3028)

LEE_111	LEE_125
BALLYGOLY (6030)	ROSCREA (25040)
ROCHFORT (25034)	WHITE BRIDGE (22009)
BALLYHAUNIS (30020)	BALLYMULLEN (23012)
ROSCREA (25040)	ASHBOURNE (8007)
WHITE BRIDGE (22009)	ENLER (5020)
CLOBANNA (16051)	MOUNTMELLICK (14033)
HOSPITAL (24022)	BALLYHAUNIS (30020)
CURRALHIR (6031)	BALLYBOGHIL (8012)
LARAGH (10004)	CURRALHIR (6031)
NAUL (8002)	LISDARN (36031)
FOULK'S MILL (13002)	BALLYGOLY (6030)
STATION ROAD (19046)	RAVERNET (5005)
BALLYBOGHIL (8012)	LAGAN (5008)
BALLINACLOGH (16006)	AUGHNAGROSS (16005)
KILMORE (26022)	CLOBANNA (16051)
BALLINRINK BR. (26058)	BIG BRIDGE (35004)
ASHBOURNE (8007)	BELLANTRA BR. (26009)

Table 4.11 provides a summary of the pooling group analysis and summary statistics for each of the locations where pooled growth curves were developed.

Table 4.11: Pooling Summary Statistics, Tralee

HEP	Final Pooling Group Total Years	Number of Stations Removed from Original Pooling Group	L-Skewness	L-Kurtosis	Preferred Distribution (GLO or GEV)
Main Channel					
LEE_02	502	4	0.218	0.194	**GLO**
LEE_39	503	0	0.199	0.180	**GLO**
LEE_94	543	0	0.146	0.229	**GLO**
Tributaries					
LEE_20	508	0	0.134	0.149	**GLO**
LEE_37	506	7	0.170	0.166	**GLO**
LEE_76	509	0	0.110	0.129	**GLO**
LEE_77	528	0	0.113	0.161	**GLO**
LEE_93	510	0	0.119	0.139	**GLO**
LEE_99	509	1	0.185	0.194	**GLO**
LEE_111	510	1	0.155	0.146	**GLO**
LEE_125	530	0	0.226	0.192	**GLO**

The pooling groups for the main channel and tributaries did not require major adjustment in terms of removing stations from the pooling group. Many pooling groups required no stations to be removed. The GLO was the preferred growth curve distribution across the Tralee FRS study area.

4.3.4 Enhanced Single Site Analysis

Given that the River Lee is well gauged at the Ballymullen gauging station a growth curve can be obtained from at-site flood frequency analysis, however as discussed in Section 4.3.2, confidence in the growth curve beyond 25-year (4% AEP) decreases due to the record length.

Therefore, an enhanced single site (ESS) analysis growth curve for Ballymullen was carried out and adopted as the growth curve for the River Lee. ESS combines pooled analysis with at-site flood frequency analysis. Extra weighting is given to the target gauging station in the pooled analysis, in this instance, Ballymullen gauging station (23012). ESS helps to capture the flood frequency conditions accurately for higher return periods where station data is less than the target return period.

4.3.5 Selection of Final Growth Curves

The final growth factors/curves for the locations on both the main channel and tributaries are presented in **Table 4.12** and illustrated in **Figure 4.4** to **Figure 4.9**. It is noted that the growth curve adopted for the main channel is the ESS growth curve.

Table 4.12: Finalised Growth Factors, Tralee

Tralee Model Reach (HEP)	Growth Factors at AEP (Return Period)							
	50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
Main Channel – Lee								
Upper – LEE_02	1.00	1.31	1.55	1.80	2.19	2.53	2.93	4.12
Middle – LEE_39	1.00	1.25	1.42	1.62	1.90	2.15	2.44	3.28
Lower – LEE_94	1.00	1.21	1.35	1.50	1.71	1.88	2.08	2.61
Ballymullen GS At-Site Growth Curve	1.00	1.22	1.40	1.60	1.94	2.25	2.63	3.86
Lee Enhanced Single Site Growth Curve	1.00	1.23	1.41	1.61	1.92	2.20	2.54	3.57
Tributaries								
Ballybeggan – LEE_20	1.00	1.23	1.38	1.53	1.76	1.94	2.14	2.68
Cloghers – LEE_37	1.00	1.23	1.40	1.57	1.82	2.03	2.27	2.95
Balloonagh – LEE_76	1.00	1.26	1.43	1.60	1.84	2.04	2.24	2.79
Big River – LEE_77	1.00	1.25	1.41	1.57	1.80	1.99	2.19	2.71
Ballyvelly – LEE_93	1.00	1.23	1.39	1.55	1.77	1.95	2.14	2.66
Unknown Tributary – LEE_99	1.00	1.28	1.49	1.70	2.03	2.30	2.62	3.52
Unknown Tributary – LEE_111	1.00	1.34	1.57	1.81	2.17	2.46	2.79	3.70
Pinure – LEE_125	1.00	1.29	1.50	1.74	2.10	2.43	2.80	3.94

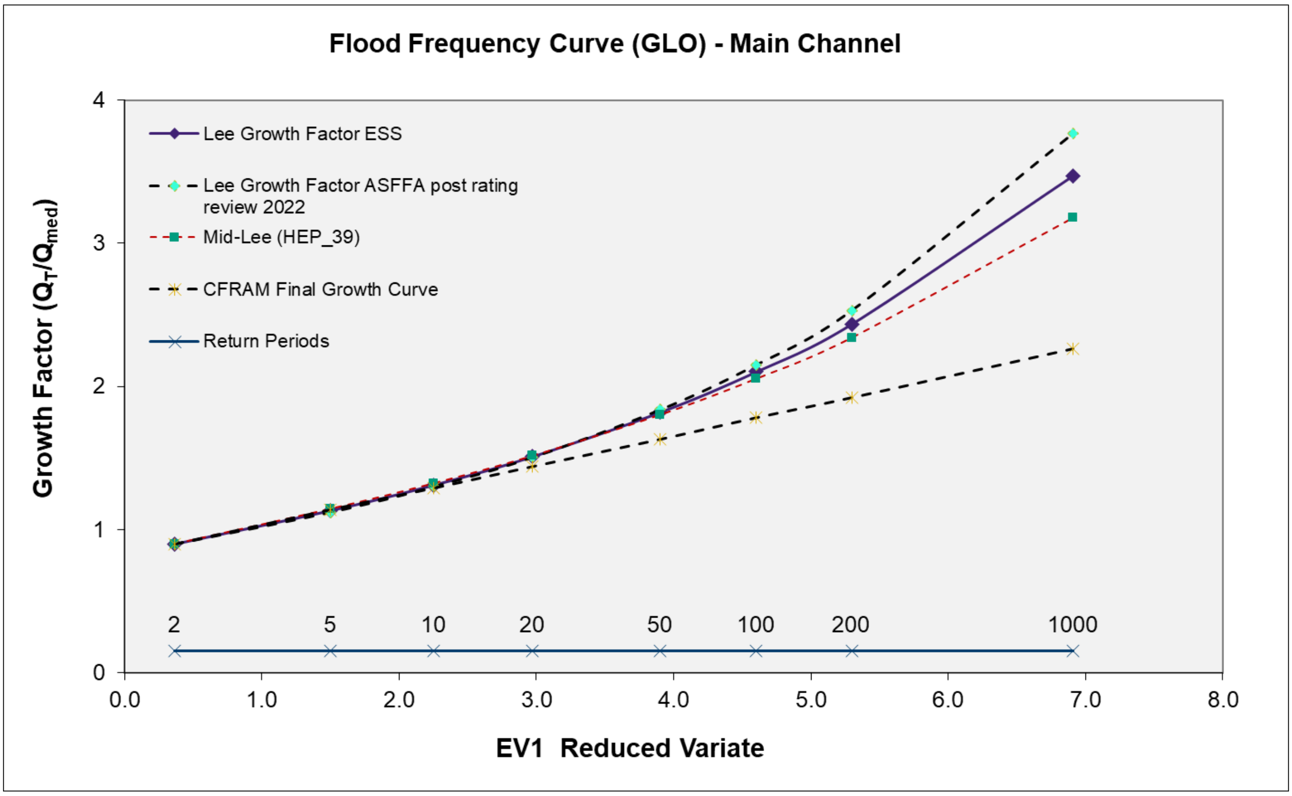


Figure 4.4: Final Growth Curve, Lee

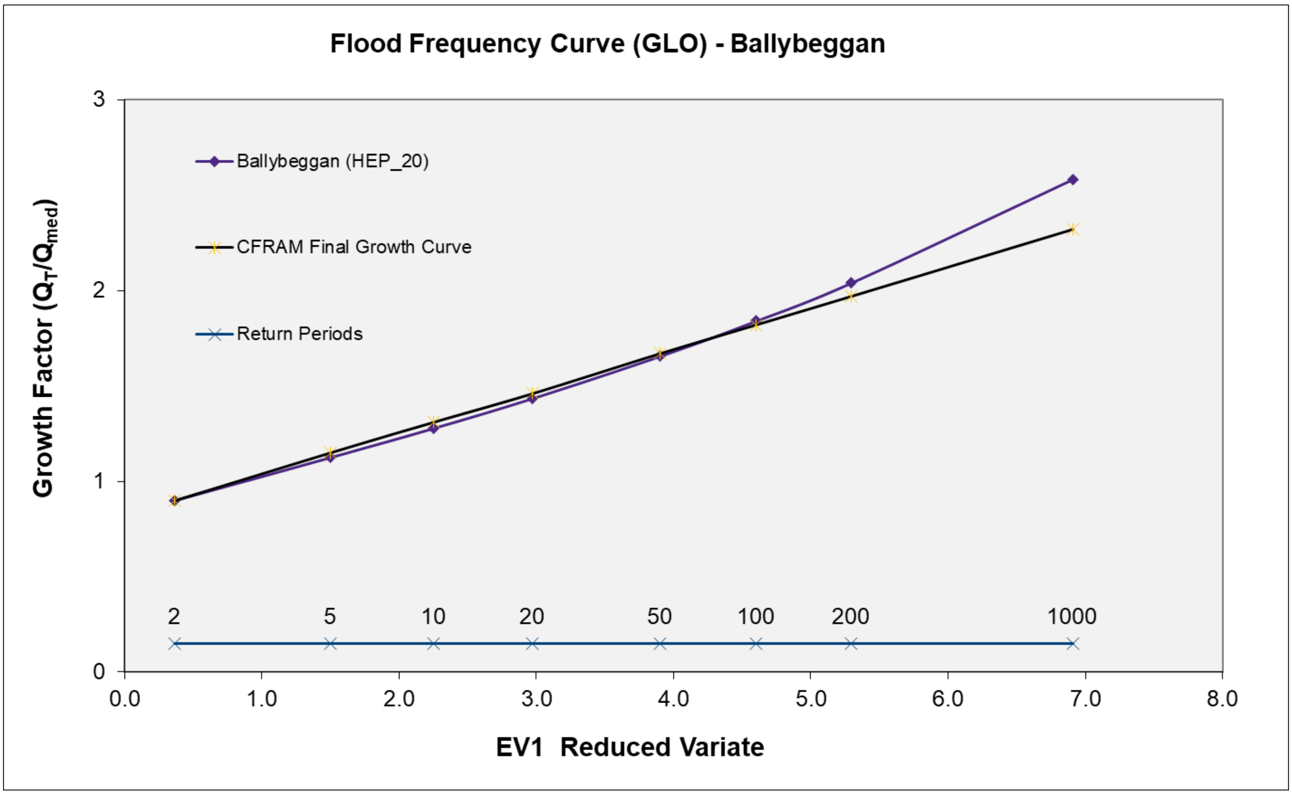


Figure 4.5: Final Growth Curve, Ballybeggan

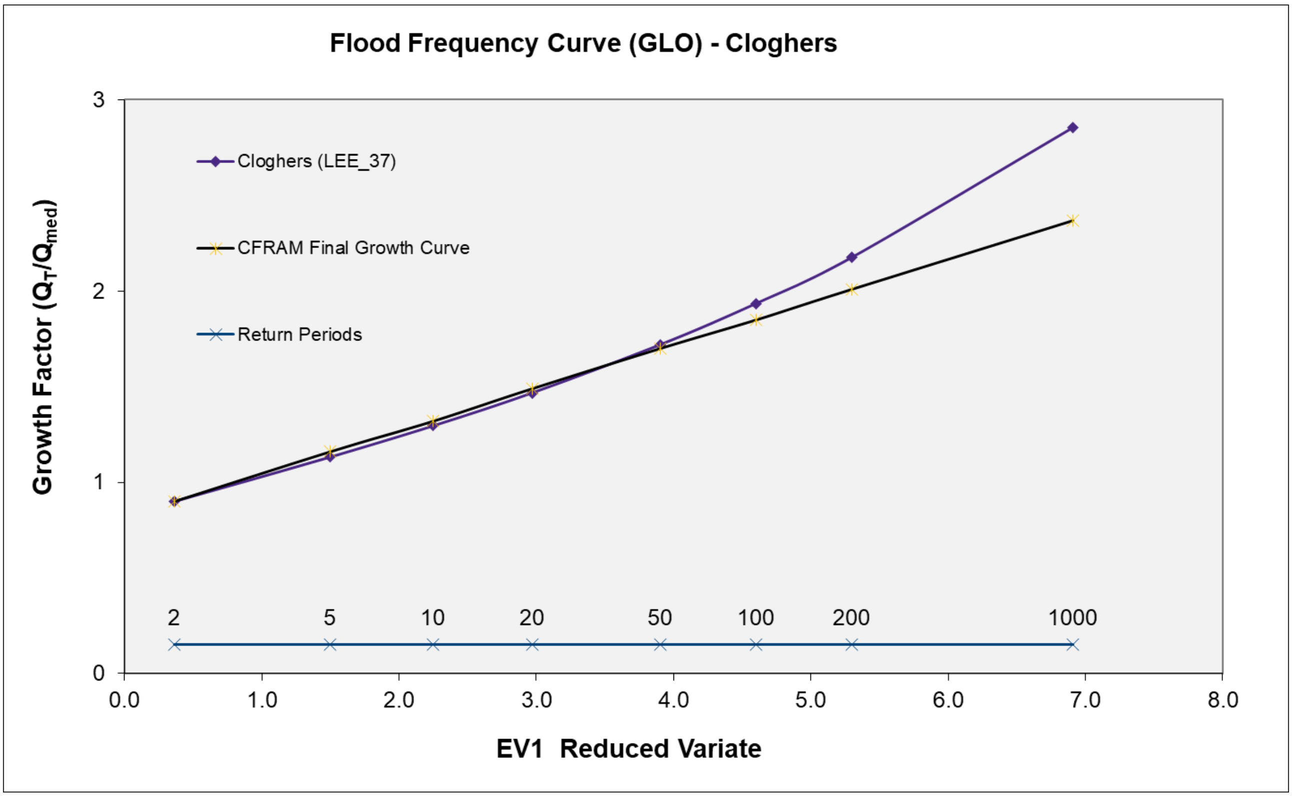


Figure 4.6: Final Growth Curve, Cloghers

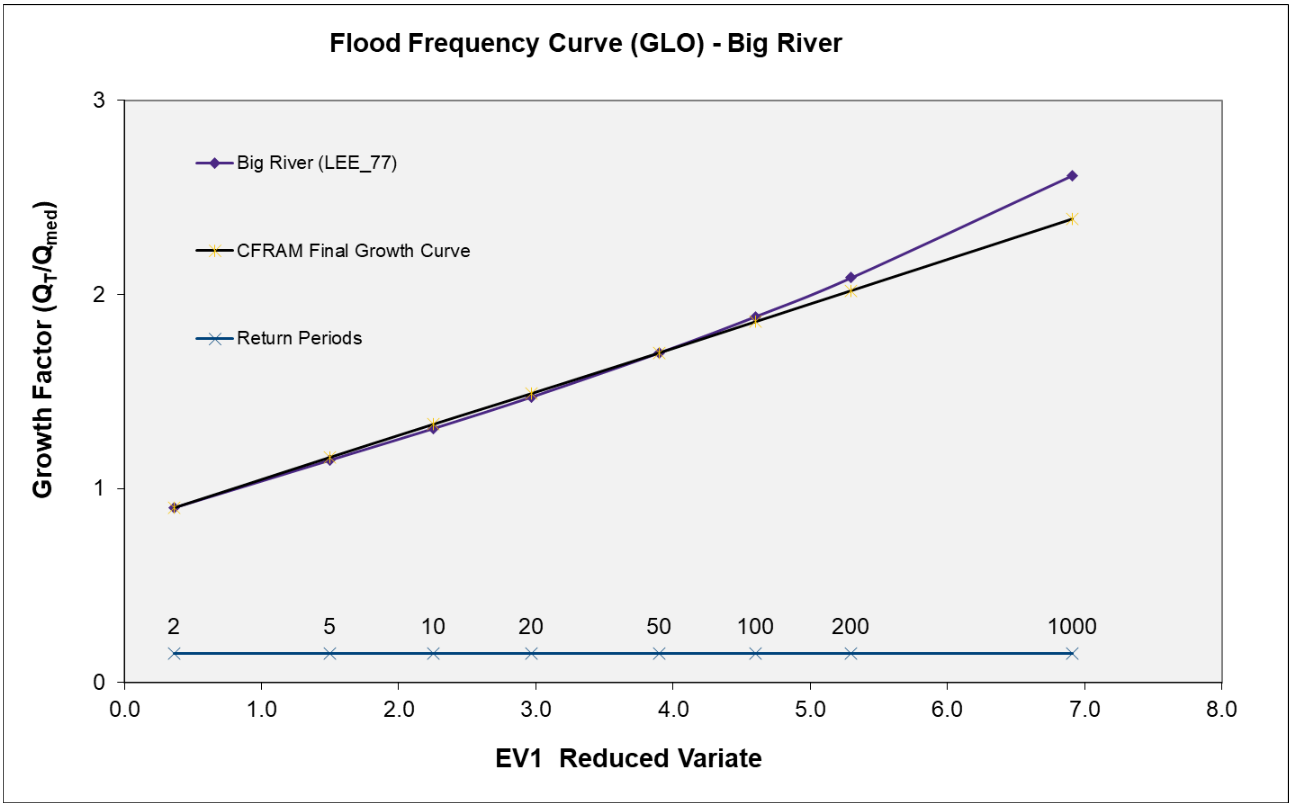


Figure 4.7: Final Growth Curve, Big River

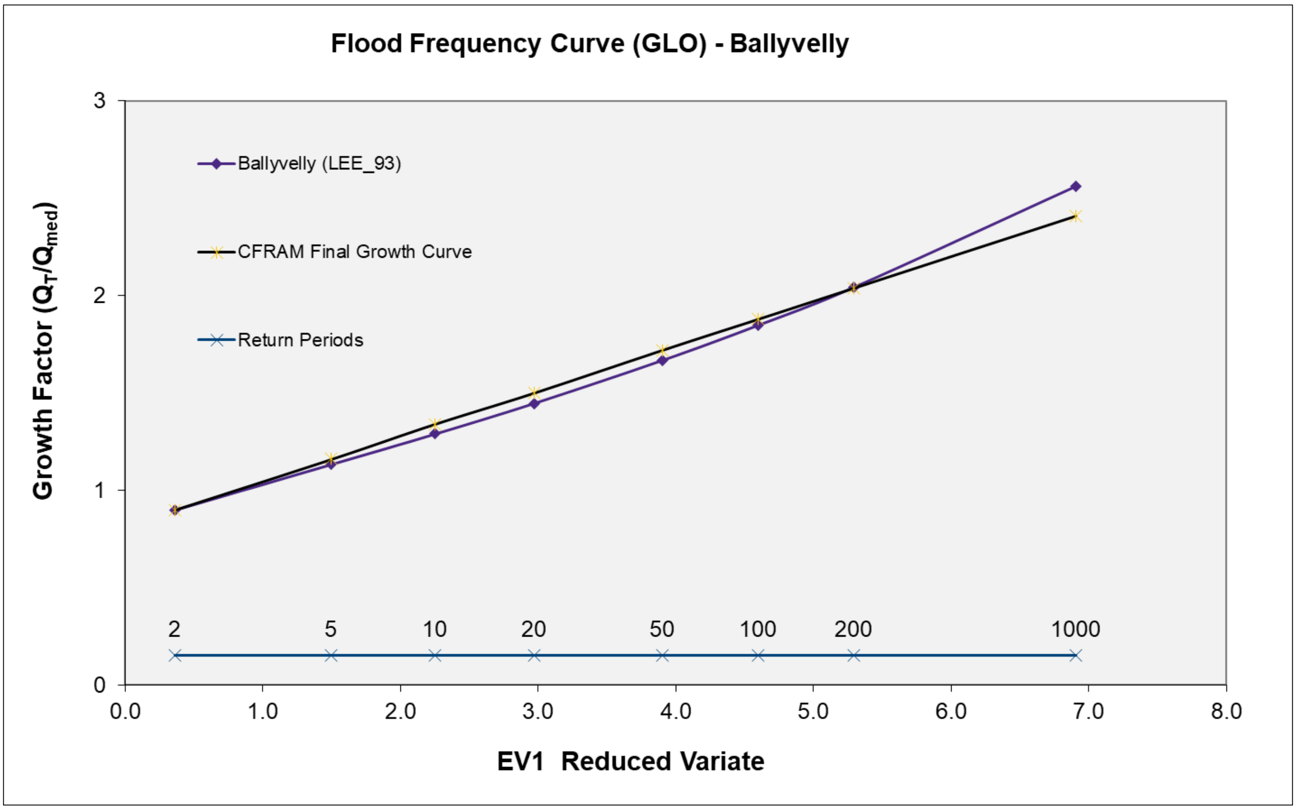


Figure 4.8: Final Growth Curve, Ballyvelly

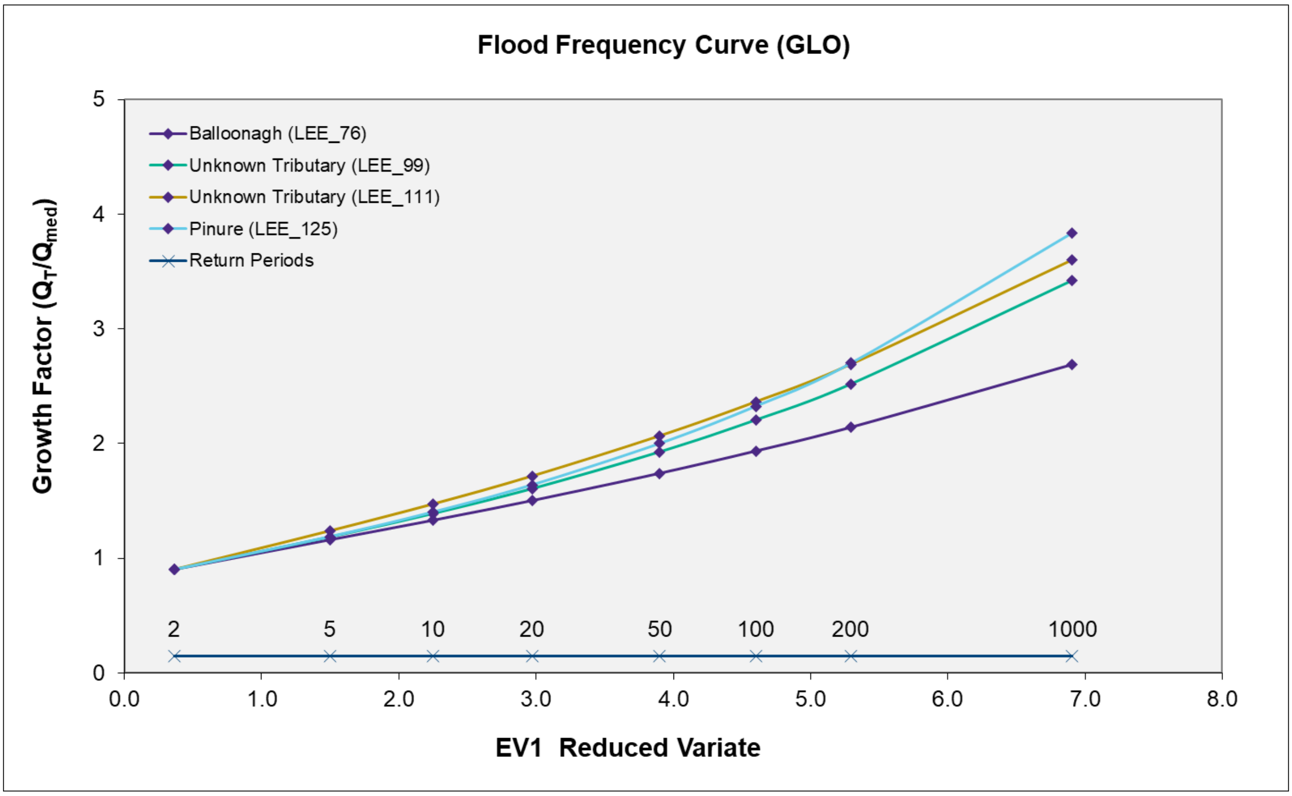


Figure 4.9: Final Growth Curve, Watercourses with Previously Undetermined Growth Curves

4.4 Tralee FRS Final Design Flows

The final peak design flow estimates for a selection of the HEPs in the Tralee study area are presented in **Table 4.13**.

Table 4.13: Final Peak Design Flow Estimates, Tralee

Tralee Model Reach (HEP)	Flows for AEP (Return Period)							
	50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
Main Channel								
Upper – LEE_02	16.60	20.29	23.24	26.64	32.16	37.35	43.65	64.14
Middle – LEE_39	30.27	37.35	42.74	48.74	58.12	66.66	76.75	108.10
Lower – LEE_94	54.94	67.79	77.57	88.46	105.50	120.99	139.31	196.21
Tributaries								
Ballybeggan U/S – LEE_07	3.72	4.56	5.12	5.70	6.53	7.21	7.96	9.97
Ballybeggan D/S – LEE_21	8.99	11.02	12.39	13.80	15.79	17.45	19.25	24.12
Cloghers U/S – LEE_30	5.57	6.87	7.78	8.73	10.14	11.34	12.68	16.46
Cloghers D/S – LEE_38	9.80	12.08	13.68	15.37	17.84	19.95	22.30	28.96
Balloonagh U/S – LEE_65	0.45	0.57	0.65	0.72	0.83	0.92	1.01	1.26
Balloonagh D/S – LEE_76	4.73	5.96	6.77	7.58	8.72	9.64	10.62	13.22
Big River U/S – LEE_48	3.49	4.35	4.92	5.49	6.29	6.94	7.63	9.48
Big River D/S – LEE_77	14.62	18.20	20.58	22.97	26.30	29.02	31.95	39.66
Ballyvelly U/S – LEE_80	0.52	0.67	0.78	0.89	1.06	1.20	1.37	1.84
Ballyvelly D/S – LEE_93	13.34	17.13	19.84	22.73	27.02	30.72	34.91	46.95
Unknown Tributary U/S – LEE_95	0.86	1.11	1.28	1.47	1.74	1.98	2.25	3.03
Unknown Tributary D/S – LEE_99	1.10	1.41	1.64	1.87	2.23	2.53	2.88	3.87
Unknown Tributary U/S – LEE_100	0.23	0.31	0.37	0.42	0.50	0.57	0.65	0.86
Unknown Tributary D/S – LEE_111	0.82	1.09	1.28	1.48	1.77	2.01	2.28	3.02
Pinure U/S – LEE_112	0.63	0.81	0.95	1.10	1.33	1.53	1.77	2.49
Pinure D/S – LEE_125	6.70	8.63	10.07	11.65	14.08	16.25	18.78	26.39

4.4.1 Comparison of Design Flows with CFRAM Study

Table 4.14 presents a summary of the comparison between the final design flows in the Tralee FRS hydrological analysis against the CFRAM Study at comparable HEPs across the Tralee study area.

Table 4.14: Comparison between Tralee FRS and CFRAM Final Design Flow Estimates

CFRAM Study HEP	Qmed (m^3/s)	Q100 (m^3/s)	Tralee FRS HEP	Qmed (m^3/s)	Q100 (m^3/s)	% Change Qmed	% Change Q100
Main Channel							
23_1521_2	16.10	30.20	**LEE_06**	17.23	37.94	**7.0%**	**25.6%**
23_2921_2	28.80	54.00	**LEE_39**	30.27	66.66	**5.1%**	**23.4%**
23_2919_2	49.90	93.70	**LEE_94**	54.94	120.99	**10.1%**	**29.1%**
Tributaries							
23_2750_2	10.30	19.80	**LEE_21 (Ballybeggan)**	8.99	17.45	**-12.7%**	**-11.9%**
23_1563_2	8.30	16.20	**LEE_38 (Cloghers)**	9.80	19.95	**18.1%**	**23.1%**
23_2612_3	17.30	33.90	**LEE_77 (Big River)**	14.62	29.02	**-15.5%**	**-14.4%**
23_2648_3	15.70	31.10	**LEE_93 (Ballyvelly)**	13.34	30.72	**-15.0%**	**-1.2%**

There are significant differences with the current hydrological analysis when compared to the design flows at the time of the CFRAM Study. In general, there has been an increase in the resolution of the analysis in terms of HEPs, catchment delineation, pooled analysis, growth curve development, and rating reviews associated with the gauging stations within the Tralee study area.

Catchment delineation was reviewed using 2m LiDAR where available in comparison with the widespread use of the 5m National Digital Height Model (NDHM) during the CFRAM Study. It is also noted that more watercourses were included in the Tralee FRS hydrological analysis which were not considered in the CFRAM Study. Therefore, no comparison could be provided for such watercourses.

The changes shown on the main channel River Lee are likely due to the knock-on effect of the rating review at Ballymullen. This has resulted in an uplift in Q_{med} of around 5.1%. The changes at the Q100 are due to a significant increase in the growth factors (**Figure 4.4**) between the two analyses.

Significant differences also exist on the major tributaries of the Lee compared to CFRAM. Differences at Q_{med} range from -15% to +19%, while differences at Q100 range from -14% to +23%. These differences are likely due to an over-generalisation of the PCDs used on these river reaches during the CFRAM Study. The application and assignment of PCDs were revisited and updated during the current hydrological analysis. The revision of PCDs likely has led to a better characterisation of the catchment. In effect, the revised flows taken forward for this project are due to improved resolution and accuracy of the analysis.

4.5 Design Hydrograph Estimation

The previous sections discussed the process of arriving at peak design flow estimates. Before being applied within a hydraulic model these must be coupled to a hydrograph such that they provide a true representation of the rising and falling of a flood flow over time. The approach adopted is the FSU-based approach set out in the FSU Technical Research Report Volume III[32]. The method is similar in principle to the estimation of the Q_{med} in that it uses PCDs to arrive at an initial estimate of the hydrograph shape, defined in three parameters, and then uses a pivotal site to adjust the shape based on observed data.

4.5.1 Hydrograph Width Analysis

The approach used leans heavily on the method developed by Archer *et al.* (2000) and as set out in FSU under Technical Research Report Volume III. The method used is based on principles of Hydrograph Width Analysis (HWA). HWA aims to define the shape of the design flood hydrograph based on observed flood hydrographs which have occurred. The premise is that hydrographs of floods occurring at a particular station are broadly similar in character and that the typical shape of the hydrograph of a future flood - embodied in the design flood hydrograph – can be expected to reflect the general features of those which have already occurred.

The approach considers all the observed hydrographs represented within the AMAX series for the gauged catchment. Firstly, hydrographs are analysed to isolate the individual event shape with data from before and after this point removed. Hydrograph width is then determined for each 10th percentile of the peak flow and median widths are determined for each percentile. This is then used to determine a median width hydrograph reflecting the AMAX data based on three shape parameters:

n – Shape parameter of Gamma distribution (and of UPO-ERR-Gamma model)

Tr – Rise time (=translation parameter) of UPO-ERR-Gamma model)

C - Recession parameter of UPO-ERR-Gamma model (hours)

For the Tralee analysis this approach has been applied to a number of nearby gauging stations which were not included in the original FSU Hydrograph Width Analysis or have since been updated with a longer record period that may impact the Tralee study area significantly. These stations include:

- Ballymullen (23012) – River Lee
- Ballyseedy (23045) – River Lee
- Spring Water Lane (23046) – Cloghers River
- Manor West (23049) – Ballybeggan River
- Lisloose (23044) – Balloonagh River
- Oakpark Road (23047) - Big River

The HWA hydrographs when applied at each of the HEPs may not accurately capture the baseflow characteristics for each catchment. This will be estimated based on catchment descriptors from the Flood Studies Supplementary Report No. 16 and as restated in the FEH Volume 4, equation 2.19 and is as follows:

$$BF = \{33(CWI - 125) + 3.0\ SAAR + 5.5\}10^{-5}AREA$$

Where:

BF = Baseflow (m^3/s)
CWI = Catchment Wetness Index
SAAR = Standard Average Annual Rainfall (mm)

4.5.2 Design Hydrographs at Gauged Locations

The HWA was undertaken at a number of gauging stations in the Tralee study area to potentially form the basis for the design inflow hydrographs on the reach where the gauging station is located.

[32] http://opw.hydronet.com/data/files/Technical%20Research%20Report%20-%20Volume%20III%20-%20Hydrograph%20Analysis%281%29.pdf

As many gauging stations in the Tralee study area have been recently installed, the HWA undertaken was to validate hydrograph shapes obtained from the FSU hydrograph shape generator. The outputs of the HWA are compared to the design hydrograph derived with the focus solely on shape in terms of key hydrograph characteristics (ascending and receding limb and time to peak). The FSU hydrograph shape generator was used at all ungauged locations to derive statistically informed catchment responses. Design hydrographs were also obtained from the FSU hydrograph shape generator for the downstream extent of each ungauged major tributary.

To validate the HWA output hydrographs at gauged locations, a data analysis process of event extraction, normalisation and superimposition was undertaken. This process involved the following steps:

1. Locate all AMAX events in the continuous records,
2. Extract the events by taking a window of 24hrs preceding and succeeding the event peak flow,
3. Overlay events with their mean and median traces to generalise complexities in observed hydrographs affected by compounded rainfall events and antecedent conditions,
4. Superimpose the design outputs from the HWA exercise.

4.5.2.1 Hydrograph Width Analysis for River Lee at Ballymullen 23012 Gauging Station (LEE_39)

The HWA validation process was conducted on the entire 15-minute flow record (Q15) at Ballymullen gauging station on the River Lee. The flow record is derived from the water level record at the gauge. Therefore, the Q15 flow record was reprocessed following the rating review. This was done by applying the new rating to the 15-minute water level file to produce an updated flow record.

A total of 15 AMAX years were extracted from the continuous flow record from 2008-2022. The water level record for 2004-2022 was obtained, however, a significant amount of data between 2004 and 2008 was missing, therefore, the HWA was carried out on a reduced timeseries dataset. Water year 2016 was also removed due to an anomaly in the water level data.

The AMAX events from the remaining data were extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.10** shows the outputs of the HWA at Ballymullen.

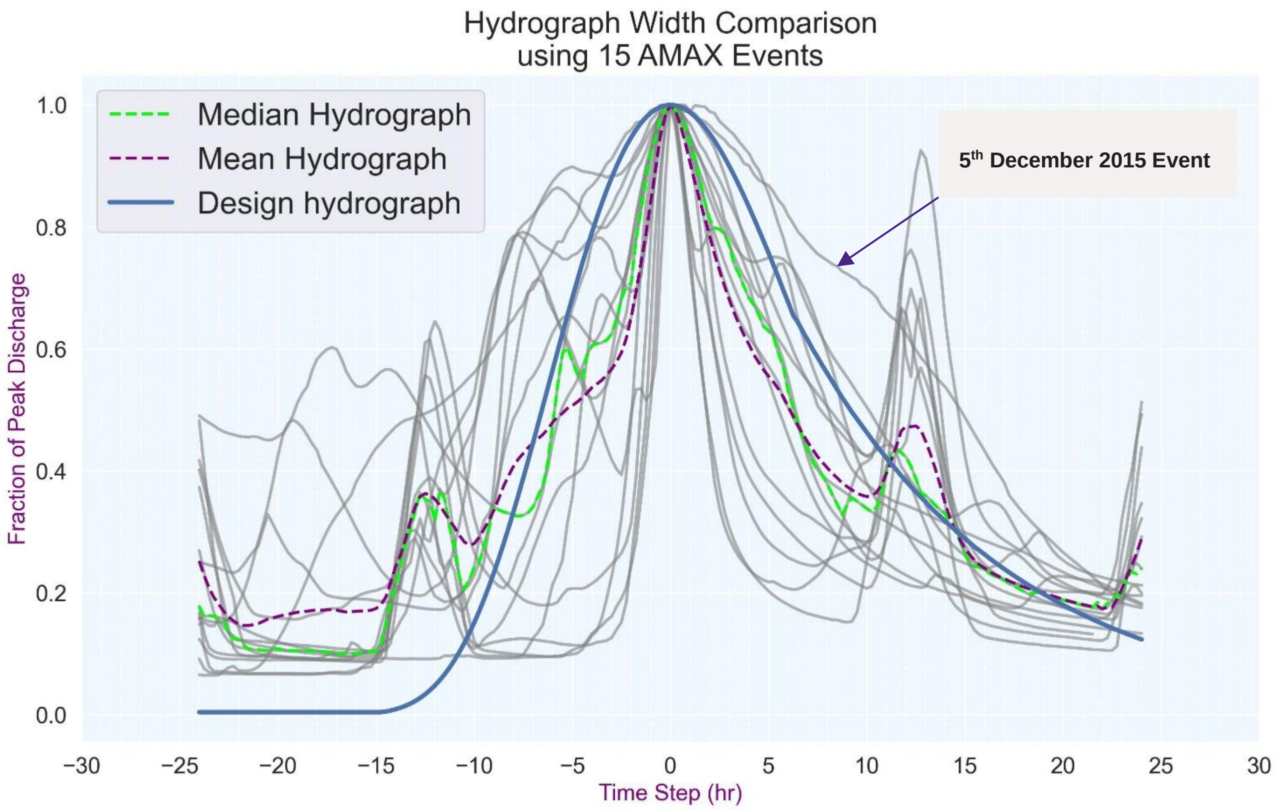


Figure 4.10: Ballymullen Hydrograph Shape Validation

The design hydrograph appears to be slightly wider than the average event hydrographs (mean and median hydrographs) and their peaks. The event hydrographs are complex with artefacts in both the rising and receding limb. Theses artefacts may be caused by tidal influence. It is noted that some events do not have a possible tidal signal in the rising and receding limb. These events were found to be the largest AMAX events recorded suggesting the tidal influence may be drowned out during large events, in line with analysis carried out in Section 2.1. The average event hydrographs are more heavily influenced by the more complex observed hydrographs, which is noticeable on the receding limb when compared to both the rising and receding limb of the design hydrograph.

The 5th December 2015 flood event experienced in Tralee was one of the largest to date in terms of water level and subsequent flow recorded at Ballymullen gauging station. The hydrograph shape, the most bulbus shown in **Figure 4.10**, has a distinct and anomalous shape compared to other AMAX events. As can be seen in **Figure 4.11**, sustained high coastal water levels over a two-day period (5th and 6th December 2015) was recorded at Blennerville coastal gauging station downstream of Ballymullen and at the point of discharge of the Lee into Tralee Bay. To determine whether or not the sustained coastal water level at Blennerville was due to fluvial mechanisms, coastal water levels at Fenit Pier station were analysed over the same time period.

Fenit Pier is situated within Tralee Bay and located circa 3.3 km west from Blennerville, meaning it is not influenced by fluvial mechanisms from the River Lee yet is close enough to represent the coastal mechanism influencing the area.

Sustained high coastal water levels are noted at Fenit Pier leading up to and during 5th December 2015 flood event. This may be representative of natural variation in coastal water level or a surge in coastal water levels, either way it appears that the coastal water levels played a role in the bulbus hydrograph shape recorded at Ballymullen gauging station.

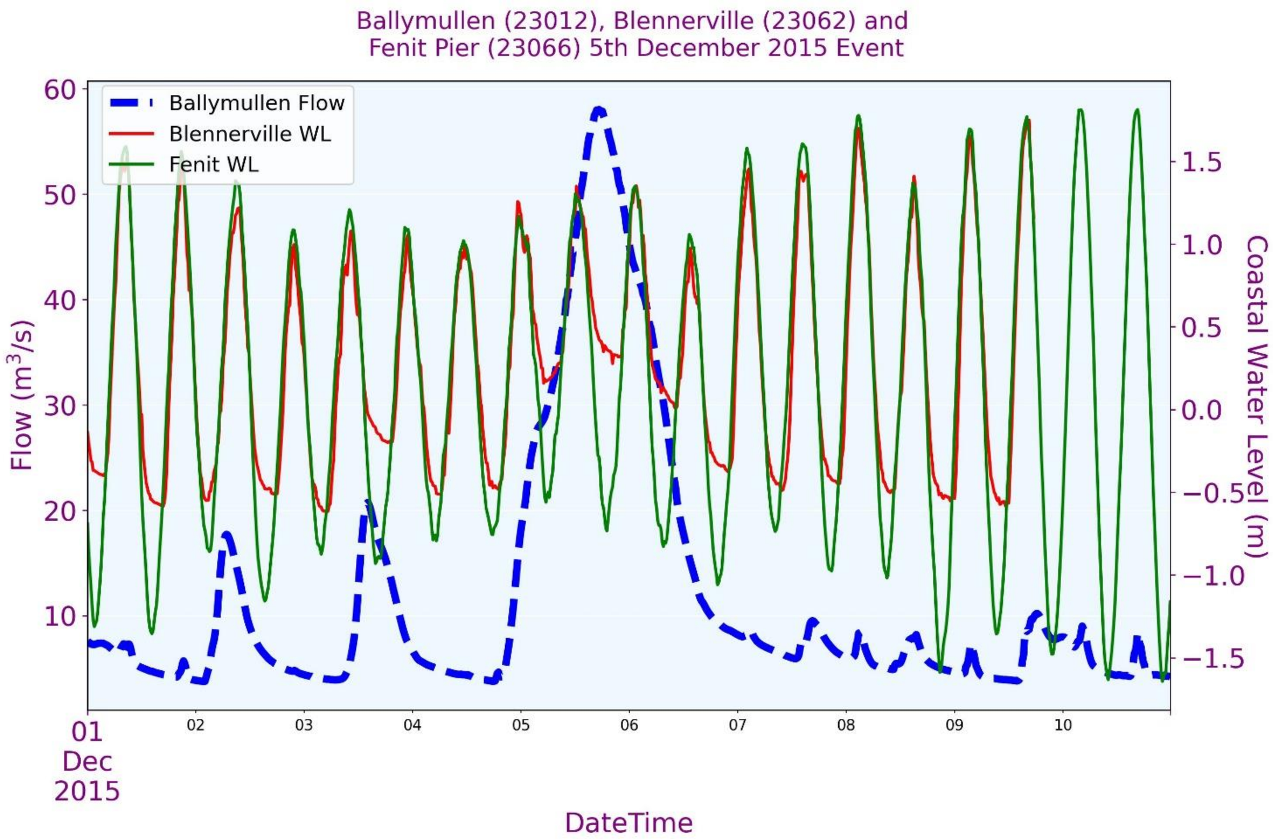
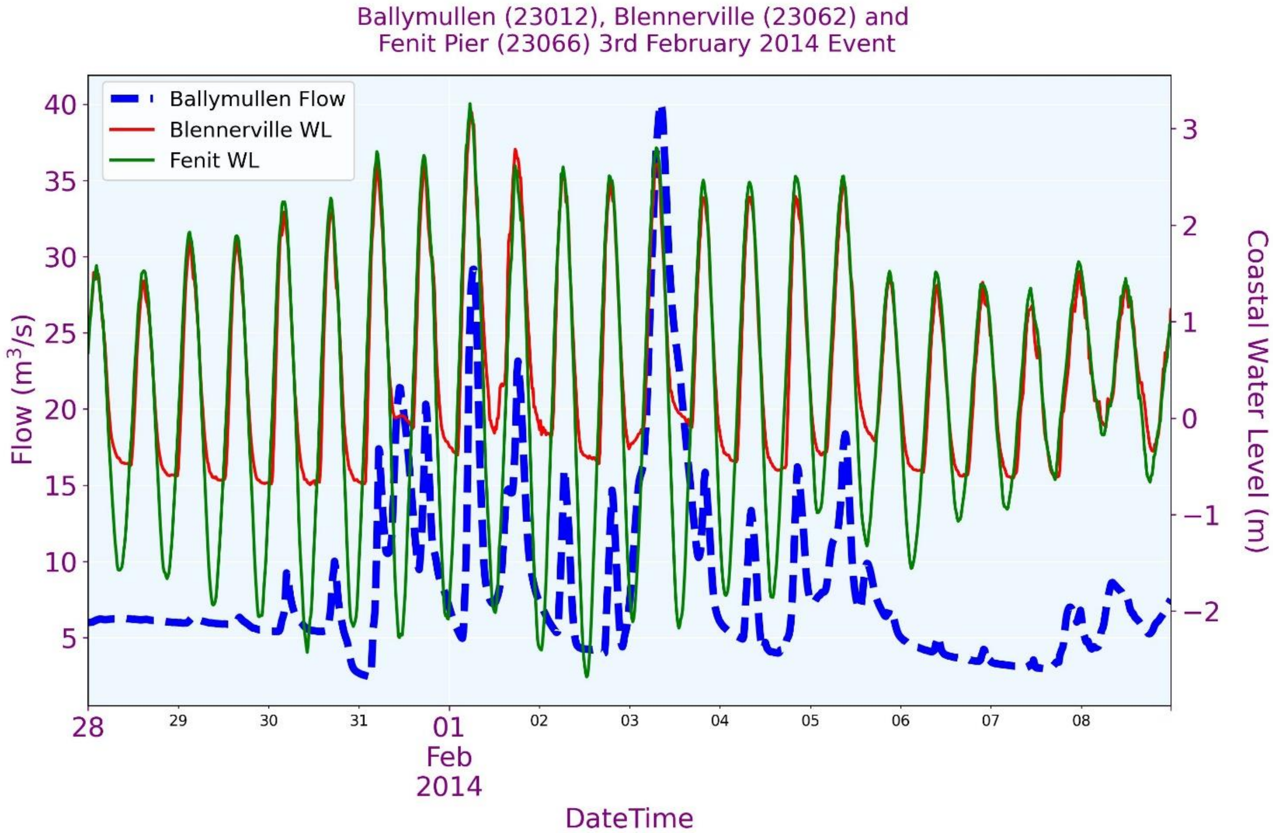


Figure 4.11: 5th December 2015 Flood Event, Tralee, Recorded Flow at Ballymullen (23012) and Coastal Water Levels at Blennerville (23062) and Fenit Pier (23066) over a time-window. WL = Water Level

Figure 4.12(a) and **Figure 4.12(b)** shows a sample of other AMAX events recorded at Ballymullen. One can see from the figures that the downstream coastal mechanisms appear to have little influence the hydrograph shape at Ballymullen compared to 5th December 2015 event when compared to Fenit Pier. This further highlights the anomalous 2015 event. When considering Blennerville coastal water level record and AMAX events other than the 2015 event, it appears this record is fluvially influenced by peak flood flows in the River Lee.

The overall shape and timing associated with the adopted design hydrograph (**Figure 4.10**) are considered an acceptable representation of the observed behaviour, albeit leaning towards a conservative interpretation of the receding limb and volume. The design hydrograph was used for design flow estimation at the station and downstream HEPs on the River Lee, as well as hydraulic modelling (as a check flow hydrograph). A hydrograph width sensitivity analysis will be carried out at the modelling stage to further refine the understanding of the Lee catchment behaviour during peak flood flow events and ensure an inflated design hydrograph is not adopted.

(a)



(b)

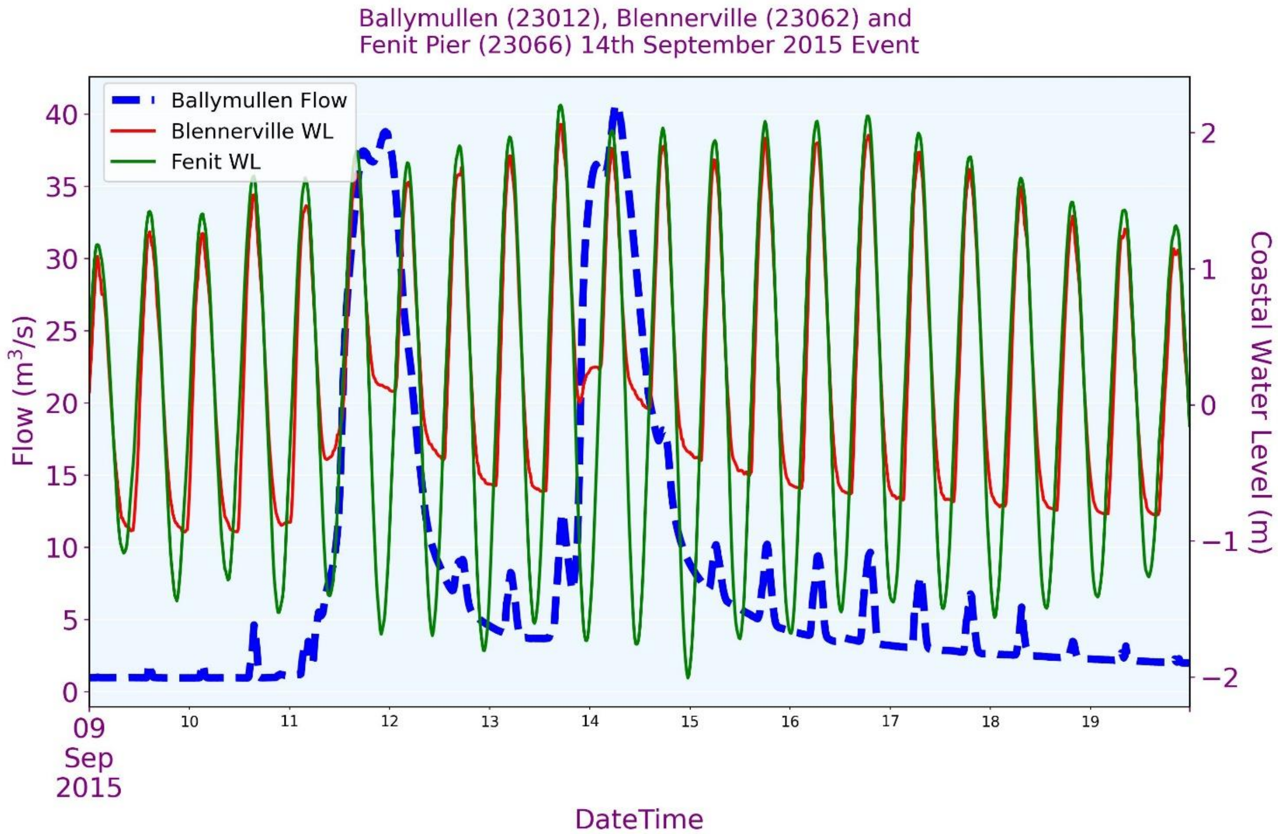


Figure 4.12: (a) 3rd February 2014 and (b) 14th September 2015 Flood Event, Tralee, Recorded Flow at Ballymullen (23012) and Coastal Water Levels at Blennerville (23062) and Fenit Pier (23066) over a time-window. WL = Water Level

4.5.2.2 Hydrograph Width Analysis of River Lee at Ballyseedy 23045 Gauging Station (LEE_02)

HWA validation was conducted on the River Lee at the Ballyseedy gauging station (LEE_02). Similar to the Ballymullen HWA, the flow record is derived from the water level record at the gauge. The input Q15 flow record was processed post-rating review by applying the new derived rating to the Q15 water level record for the period 2019-2021.There are a number of spot gaugings at Ballyseedy which gives some robustness to the derived rating equation, however, uncertainty exists in the rating review carried out for Ballyseedy given the short record period.

A total of two AMAX years were extracted from the continuous flow record from 2019-2020. The water year 2021 was removed from the analysis as it was incomplete. The AMAX events at Ballyseedy were extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.13** shows the outputs of the HWA at Ballyseedy gauging station on the River Lee.

The design hydrograph output following HWA appears to be slightly wider than the average event hydrographs (mean and median hydrographs) but follows the general shape of the mean hydrograph for the AMAX events. The shape of the two observed hydrographs are clearly different highlighting the uncertainty in the HWA at Ballyseedy.

The overall shape and timing associated with the design hydrograph displays a rough representation of the mean observed AMAX events, however, the difference in shape between the two observed hydrographs lends little confidence in validating the design hydrograph. In light of this, the design hydrograph at Ballymullen gauging station (LEE_39) was adopted for the upstream extent of the River Lee.

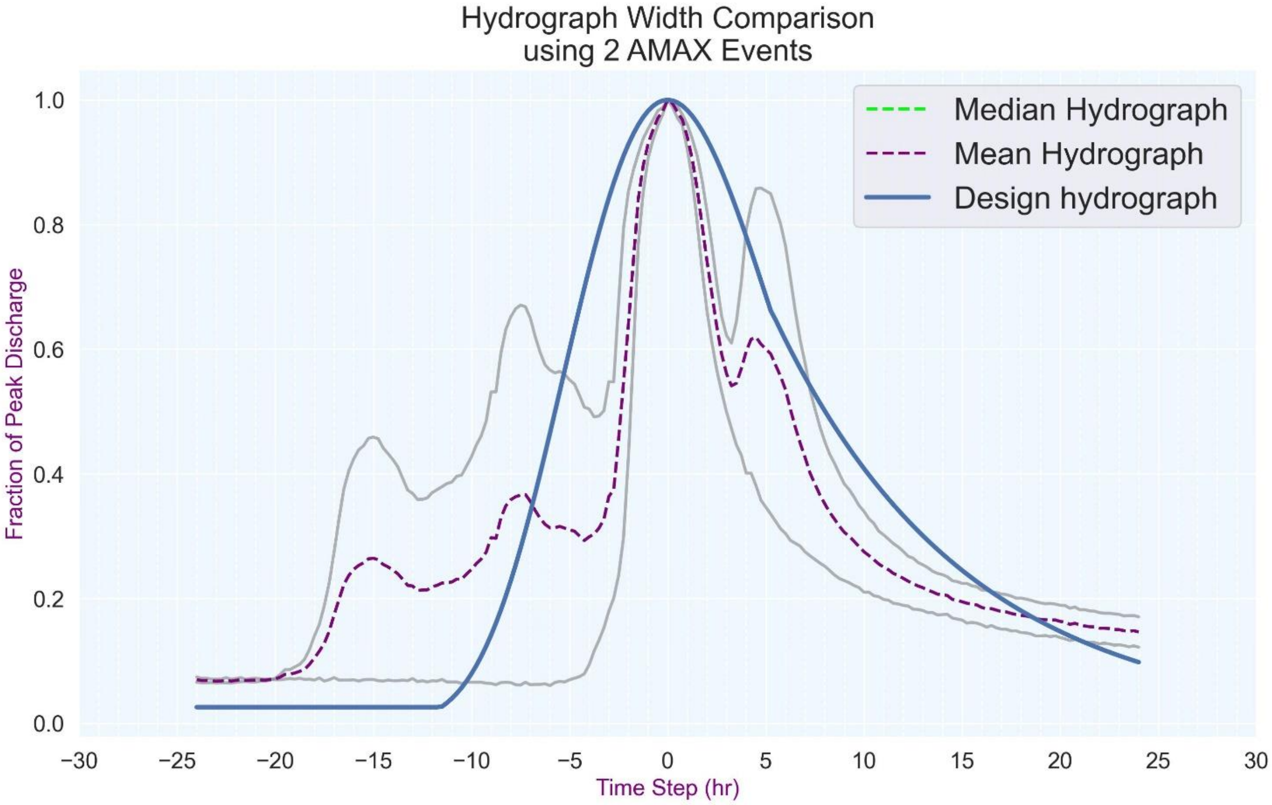


Figure 4.13: Ballyseedy Hydrograph Shape Validation

4.5.2.3 Hydrograph Width Analysis of Balloonagh River at Lisloose 23044 Gauging Station (LEE_69)

HWA validation was also conducted on the Balloonagh River at the Lisloose gauging station (LEE_69). The flow record is derived from the water level record at the gauge. The input Q15 flow record was processed post-rating review by applying the new derived rating to the Q15 water level record for the period 2019-2021. However, significant uncertainty exists in the rating review carried out for Lisloose given the short record period and small number of spot gaugings recorded.

A total of two AMAX years were extracted from the continuous flow record from 2019-2020. The water year 2021 was removed from the analysis as it was incomplete. The AMAX events at Lisloose were extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.14** shows the outputs of the HWA at Lisloose gauging station on the Balloonagh River.

The design hydrograph output following HWA appears to be slightly wider than the average event hydrographs (mean and median hydrographs) but follows the general shape of the AMAX events. The rising and receding limb of the two event hydrographs do display complexity and the small number of recorded events makes it difficult to validate the design hydrograph with confidence.

The overall shape and timing associated with the design hydrograph displays a rough representation of the mean observed AMAX events, however, the difference in shape between the two observed hydrographs lends little confidence in validating the design hydrograph.

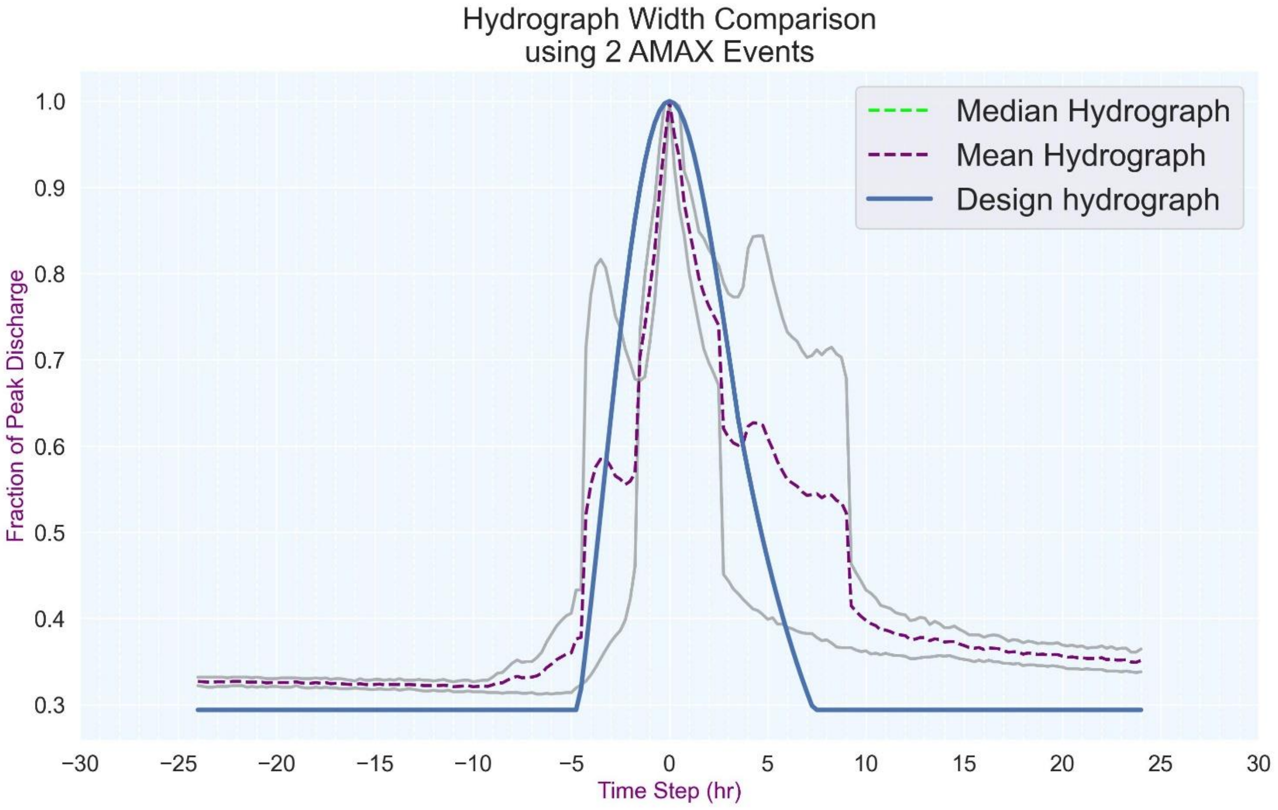


Figure 4.14: Lisloose Hydrograph Shape Validation

4.5.2.4 Hydrograph Width Analysis of Cloghers watercourse at Spring Water Lane 23046 Gauging Station (LEE_37)

HWA validation was also conducted on the Cloghers watercourse at the Spring Water Lane gauging station (LEE_37). The flow record is derived from the water level record at the gauge. The input Q15 flow record was processed post-rating review by applying the new derived rating to the Q15 water level record for the period 2019-2021. However, uncertainty exists in the rating review carried out for Lisloose given the short record period and number of spot gaugings recorded.

A total of two AMAX years were extracted fromQ15 the continuous flow record from 2019-2020. The water year 2021 was removed from the analysis as it was incomplete. The AMAX events at Spring Water Lane were extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.15** shows the outputs of the HWA at Lisloose gauging station on the Balloonagh River.

The design hydrograph output following HWA appears to be follow the shape of the observed event hydrographs and mean of the observed events very well. While there are only two observed events to validate the hydrograph, the two observed events are very similar thus giving some certainty in the behaviour of the catchment at LEE_37.

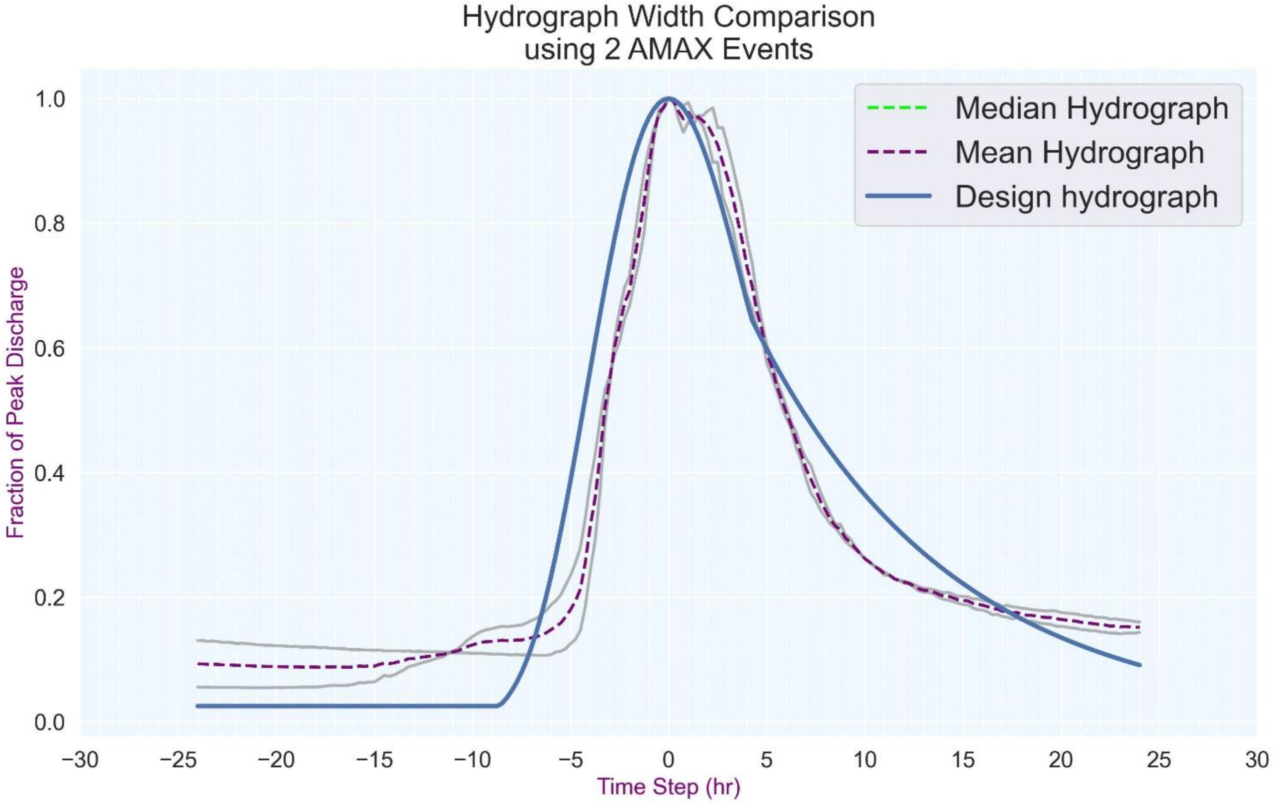


Figure 4.15 Spring Water Lane Hydrograph Shape Validation

4.5.2.5 Hydrograph Width Analysis of Big River at Oakpark Road 23047 Gauging Station (LEE_56)

HWA validation was also conducted on the Big River at the Oakpark Road gauging station (LEE_56). The flow record is derived from the water level record at the gauge. The input Q15 flow record was processed post-rating review by applying the new derived rating to the Q15 water level record for the period 2019-2021. However, uncertainty exists in the rating review carried out for Lisloose given the short record period and number of spot gaugings recorded.

A total of two AMAX years were extracted from the continuous flow record from 2019-2020. The water year 2021 was removed from the analysis as it was incomplete. The AMAX events at Oakpark Road were extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.16** shows the outputs of the HWA at Oakpark Road gauging station on the Big River.

The design hydrograph output following HWA appears to be follow the shape of the observed event hydrographs and mean of the observed events well. While there are only two observed events to validate the hydrograph, the two observed events are similar thus giving some certainty in the behaviour of the catchment at LEE_56.

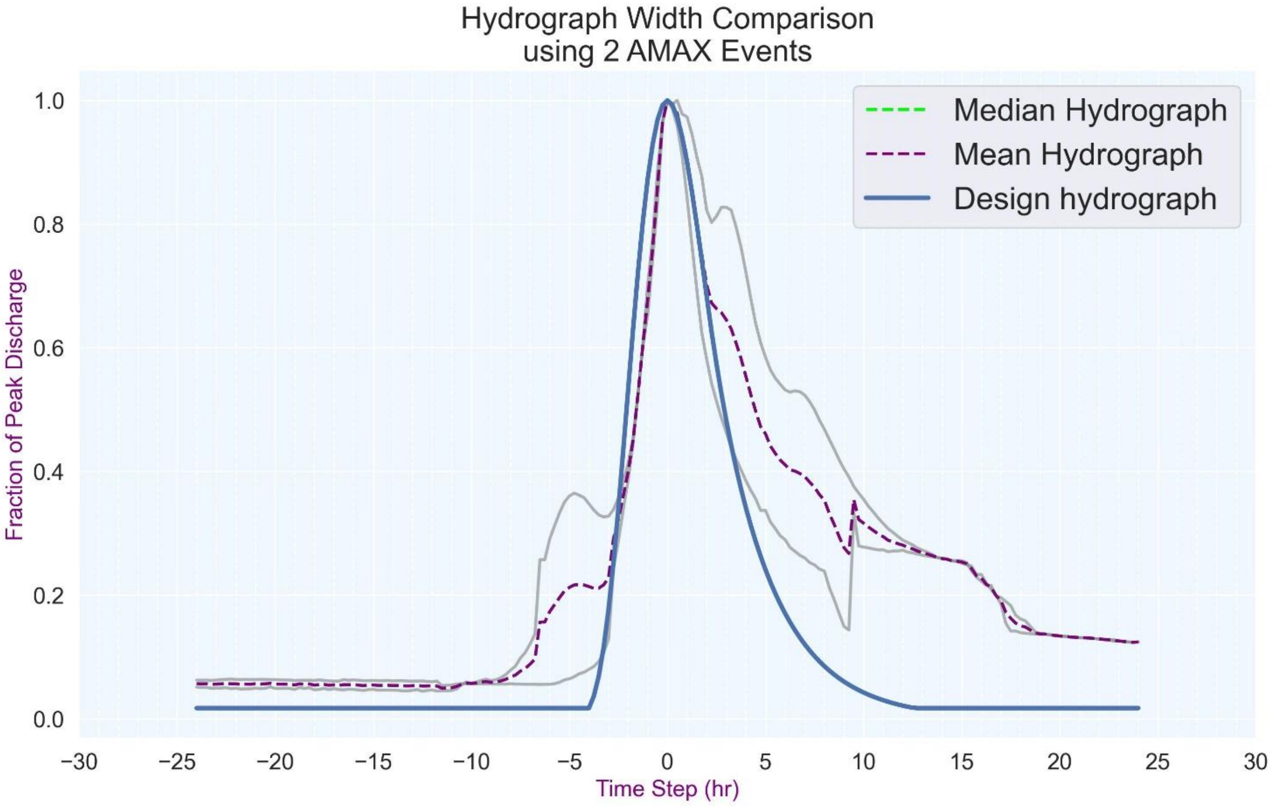


Figure 4.16: Oakpark Road Hydrograph Shape Validation

4.5.2.6 Hydrograph Width Analysis of Ballybeggan watercourse at Manor West 23049 Gauging Station (LEE_20)

HWA validation was also conducted on the Ballybeggan watercourse at the Oakpark Road gauging station (LEE_56). The flow record is derived from the water level record at the gauge. The input Q15 flow record was processed post-rating review by applying the new derived rating to the Q15 water level record for the period 2020-2021. However, uncertainty exists in the rating review carried out for Lisloose given the short record period and number of spot gaugings recorded.

A total of one AMAX year was extracted from the continuous flow record from 2020-2021. The water year 2021 was removed from the analysis as it was incomplete. The AMAX event at Oakpark Road was extracted, normalised, and superimposed with the design hydrograph generated by HWA. Mean and median observed hydrographs were also generated to aid comparison. **Figure 4.17** shows the outputs of the HWA at Oakpark Road gauging station on the Big River.

The design hydrograph output following HWA appears to be follow the shape of the observed event hydrographs well. While there is only one observed event to validate the hydrograph, the observed event and design hydrograph are similar in shape thus providing some certainty in the behaviour of the catchment at LEE_20.

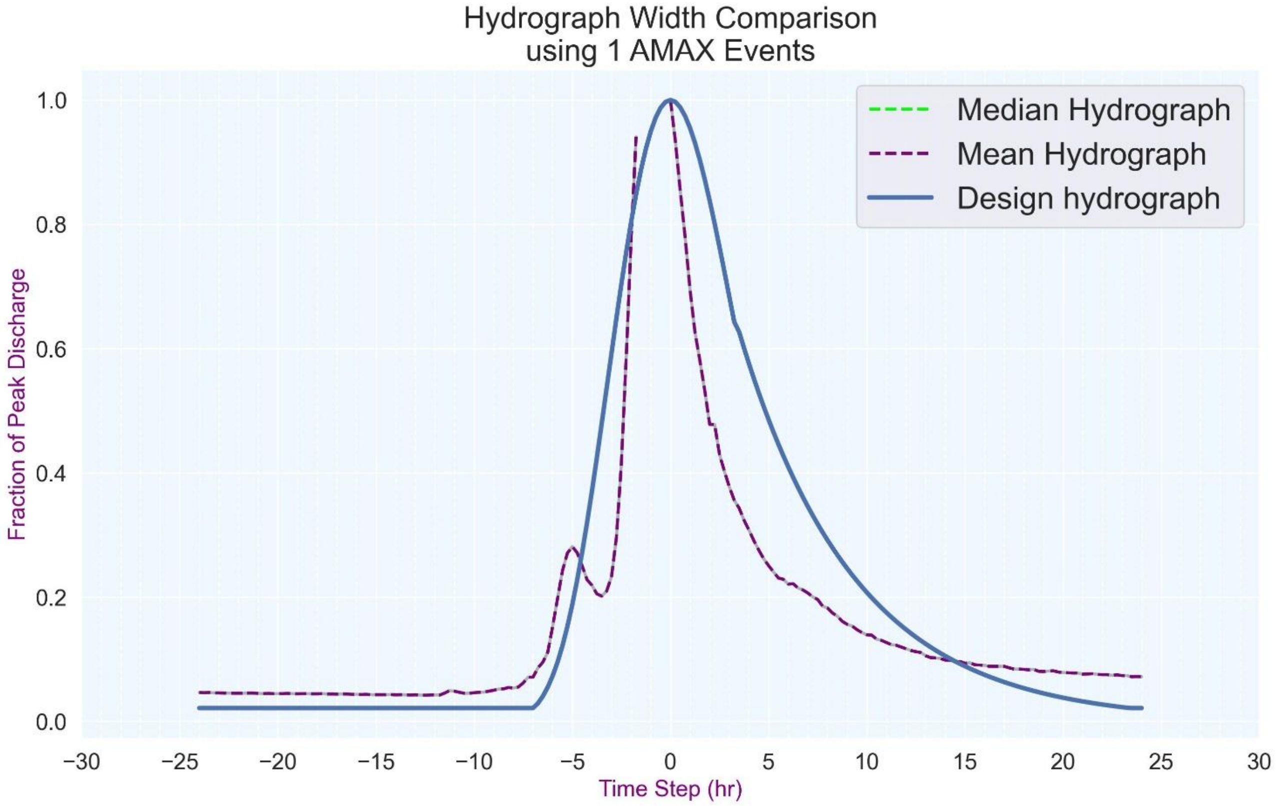


Figure 4.17: Manor West Hydrograph Shape Validation

4.5.3 Design Hydrographs at Ungauged Locations

The design hydrographs at ungauged locations have been generated using the FSU Shape Generator that derives hydrograph shape parameters based on PCDs and adjusted based on pivotal site adjustment, like the method used during Q_{med} estimation.

The update of the FSU database for the hydrograph shape library was conducted to allow further refinement of the design hydrographs such that the FSU Shape Generator tool produced acceptable answers.

The main focus of the design hydrograph development was on the inflows to the River Lee at the downstream extent of all major tributaries, as well as on the Pinure watercourse to the north-west of the study area. While many of the major tributaries of the River Lee are gauged, the record lengths as shown in Section 4.5.2 are insufficient and do not capture enough observed events to derive a robust hydrograph shape. Thus, for the majority of reaches, a hydrograph was generated from the FSU shape generator.

The results of the HWA on gauged reaches in the Tralee study area was taken into consideration when selecting suitable pivotal sites. In some cases where the HWA provided a consistent hydrograph shape over multiple observed events, manual adjustment of the shape parameters derived from the FSU shape generator was undertaken.

4.5.3.1 Design Hydrograph Pivotal Site Adjustment

Analysis of the Tralee hydrographs and the updated FSU pivotal site database allowed the selection of suitable pivotal sites. The pivotal sites selected for each reach is shown in **Table 4.15**.

Table 4.15: Pre- and Post-Pivotal Site Adjustment on Design Hydrograph Shape Parameters, Tralee

Watercourse	Pre-Pivotal Site Adjustment			Post-Pivotal Site Adjustment			Pivotal Sites used for Adjustment
	n	T_r	C	n	T_r	C	
Main Channel							
River Lee (LEE_02 Ballyseedy Gauging Station U/S extent)	8.06	44.58	21.92	**17.88**	**22.08**	**9.80**	**23012, 35002, 23045***
River Lee (LEE_39 Ballymullen Gauging Station Intermediary)	8.03	48.43	22.35	**14.14**	**21.80**	**10.60**	**23012*, 35002**
River Lee (LEE_94 D/S extent)	8.81	45.11	16.03	**12.73**	**28.83**	**9.99**	**23012*, 22006**
Tributaries							
Ballybeggan (LEE_20 Manor West Gauging Station and D/S extent)	10.57	31.44	8.40	**17.05**	**12.83**	**5.88**	**36021, 39001, 23049***
Cloghers (LEE_37 Spring Water Lane Gauging Station and D/S extent)	8.24	39.35	20.55	**19.85**	**9.38**	**5.94**	**16013, 23012, 23046***
Big River (LEE_56 Oakpark Road Gauging Station, Intermediary)	10.56	27.74	8.45	**18.71**	**13.38**	**6.55**	**36021, 1016, 23047***
Big River (LEE_64 D/S extent)	10.95	28.09	7.40	**21.30**	**13.65**	**4.71**	**36021, 32011**
Balloonagh Watercourse (LEE_69 Lisloose Gauging Station Intermediary)	10.55	26.75	8.47	**15.46**	**9.50**	**3.71**	**36021, 1016, 23044***
Balloonagh Watercourse (LEE_76 D/S extent)	11.51	26.61	6.20	**22.69**	**15.66**	**3.98**	**36021, 32011**
Ballyvelly (LEE_93 D/S extent)	10.07	27.83	10.02	**12.60**	**7.39**	**3.64**	**1016, 36021**
Pinure Watercourse (LEE_125 D/S extent)	8.08	47.34	21.98	**20.99**	**20.75**	**8.24**	**23012, 35002**

*Representative of the hydrographs developed with manually adjusted parameters to consider observed events on the watercourse.

4.5.4 Final Design Hydrograph Shapes

The final output semi-dimensional hydrographs that have been taken forward for flow estimation and input into the hydraulic model are presented in **Figure 4.18** to **Figure 4.20**.

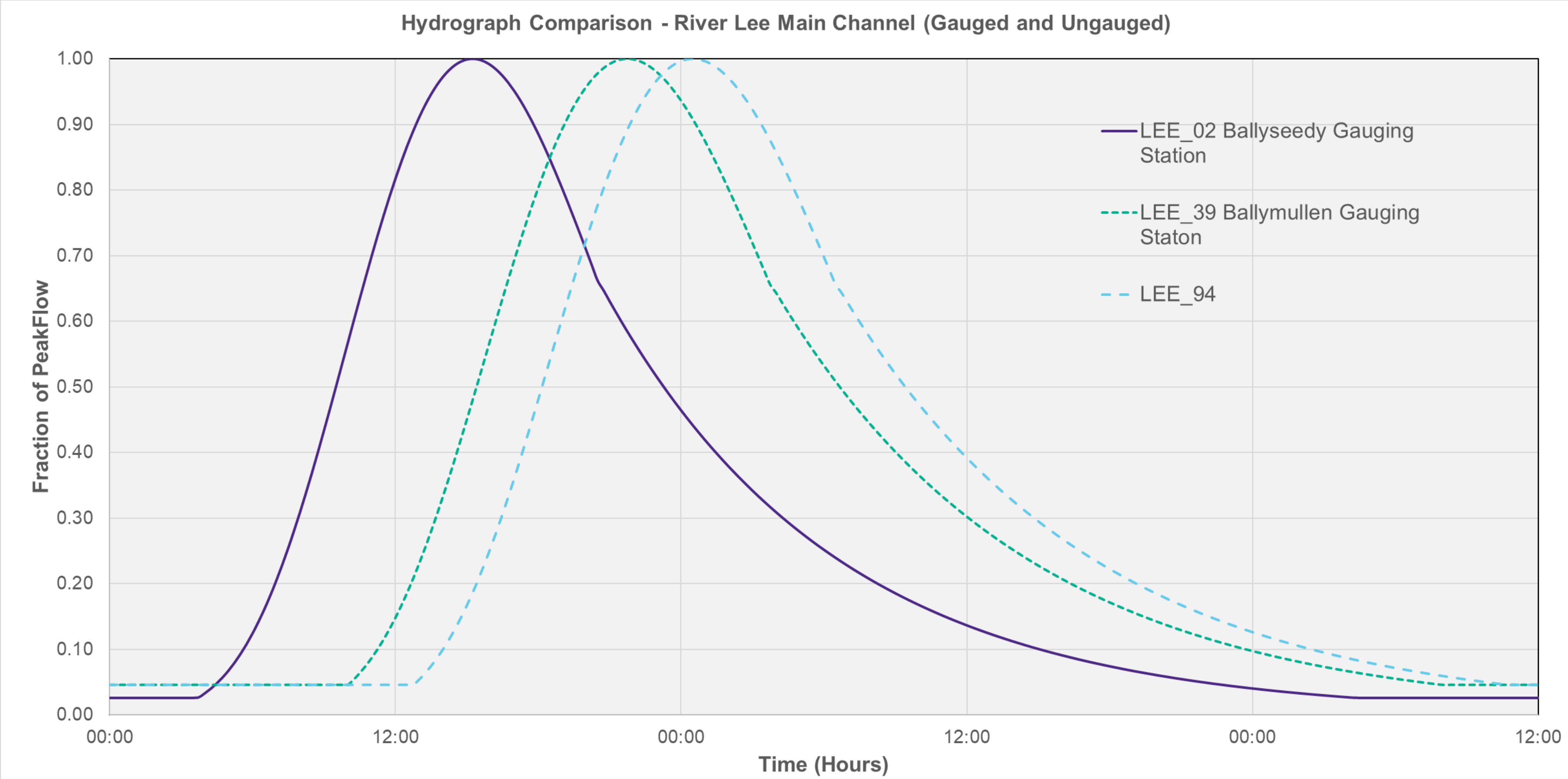


Figure 4.18: Design Hydrograph Shapes, River Lee – Main Channel, Tralee

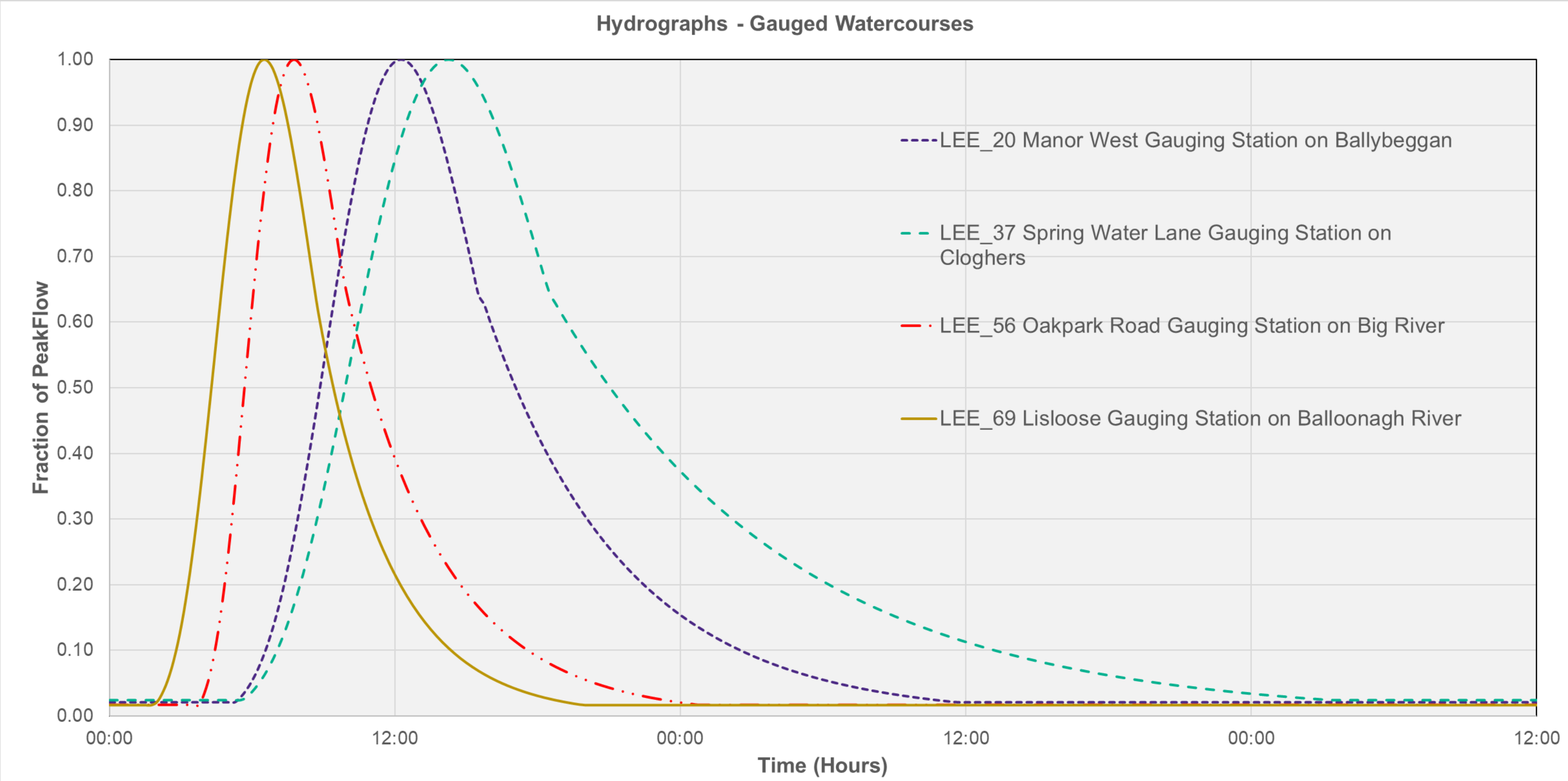


Figure 4.19: Design Hydrograph Shapes, Gauged Watercourses Tralee

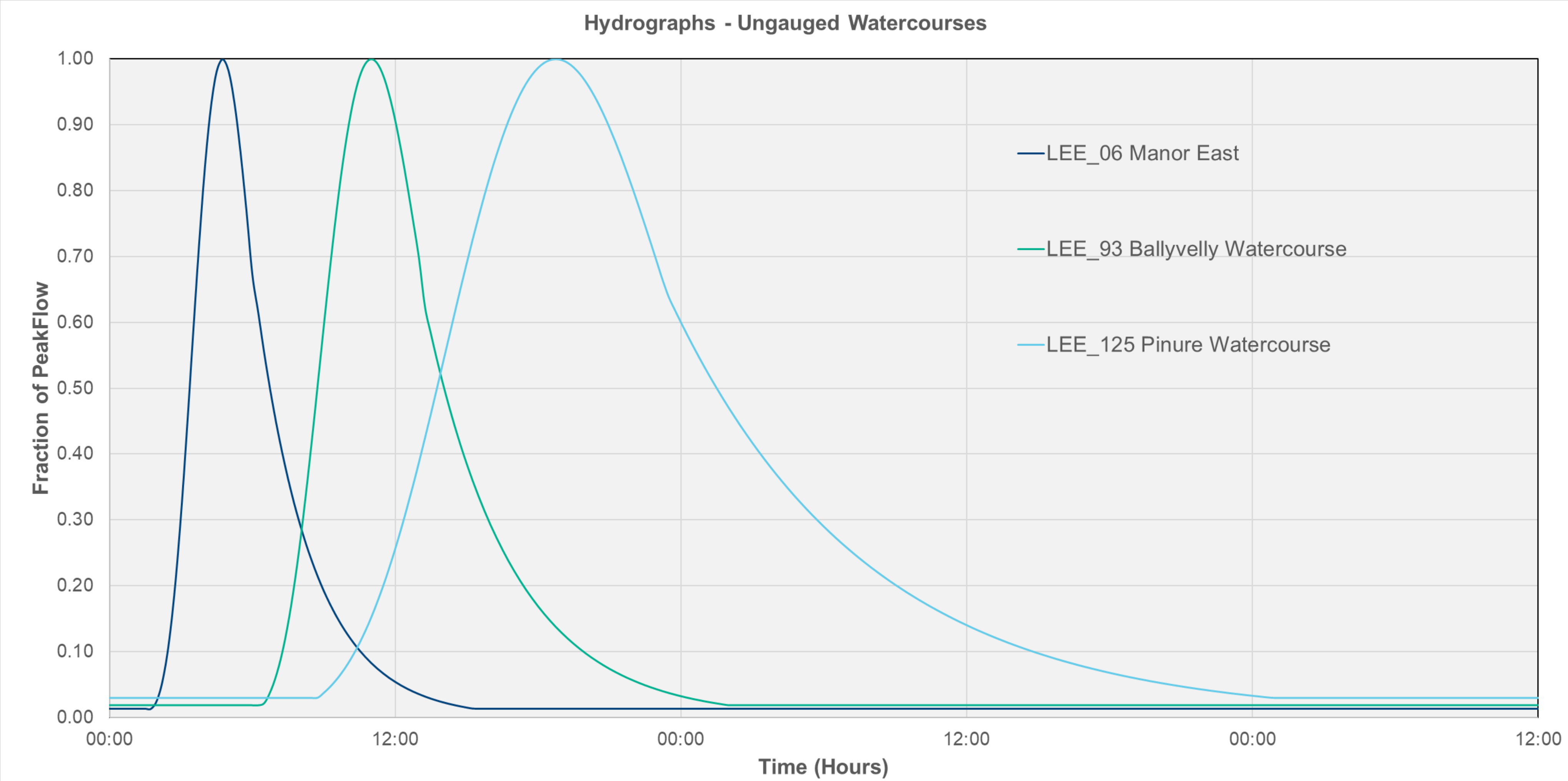


Figure 4.20: Design Hydrograph Shapes, Ungauged Watercourses Tralee

4.6 Design Flow Reconciliation and Hydraulic Model Inputs

Peak design flows have been estimated following the statistical methods set out in the FSU. Significant efforts were made to update the existing FSU datasets in terms of pivotal sites (and pivotal site adjustment), pooled analysis, growth curve development, and derivation of design hydrograph widths.

Pivotal site adjustments have been implemented to the peak design flow estimates and design hydrograph widths. The output design semi-dimensional hydrographs were then scaled to the statistical peak flow estimates across the range of return periods (50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP).

4.6.1 Lateral inflow generation

The preferred approach for the Tralee study area is to estimate the lateral inflow hydrograph by scaling the downstream HEP hydrograph to the lateral catchment area (between the subject HEP and the next HEP upstream).

$$Lateral\ Inflow = \frac{Lateral\ Catchment\ Area}{Downstream\ HEP\ Area} \times Downstream\ Check\ Flow$$

As an initial check before modelling, the sum of the inflow hydrographs is calculated at each of the intermediate and downstream HEPs to check the total inflow to the model at that point. This approach cannot account for the hydraulic effects on flows within the model such as flood peak travel time and attenuation but provides a check before modelling.

Where the initial sum of inflows for Q_{med} at a HEP is less than 10% greater than that of the FSU Q_{med} event at the same HEP, i.e., peak flow is overestimating compared to the FSU statistical method by 10%, this was deemed within acceptable limits. Where the initial sum of inflow was less that the FSU Q_{med} estimate, an uplift was applied. Uplifts were limited to 1.5 of the original inflow.

A complete check on whether appropriate lateral inflows have been estimated can only be made following hydraulic modelling of the inflows and as such the lateral inflows to the model may be amended following initial hydraulic modelling as part of the integrated approach to hydrological and hydraulic analysis.

5 COASTAL DESIGN WATER LEVELS

The basis of deriving design coastal water levels at Tralee is the coastal level dataset from Phase 1 of the Irish Coastal Wave and Water Level Study (ICWWS) 2018[33] which is an update of the original Irish Coastal Protection Strategy Study (ICPSS)[34] undertaken by RPS on behalf of the OPW.

The location of the relevant estimation point for Tralee is shown in **Figure 5.1**. The closest ICWWS estimation point (node no. SW37) is situated within Tralee Bay and located circa 3.3 km west from Blennerville Bridge. A numerical model was developed to facilitate the transfer of extreme still water level information available at estimation point SW37 to the Tralee frontage.

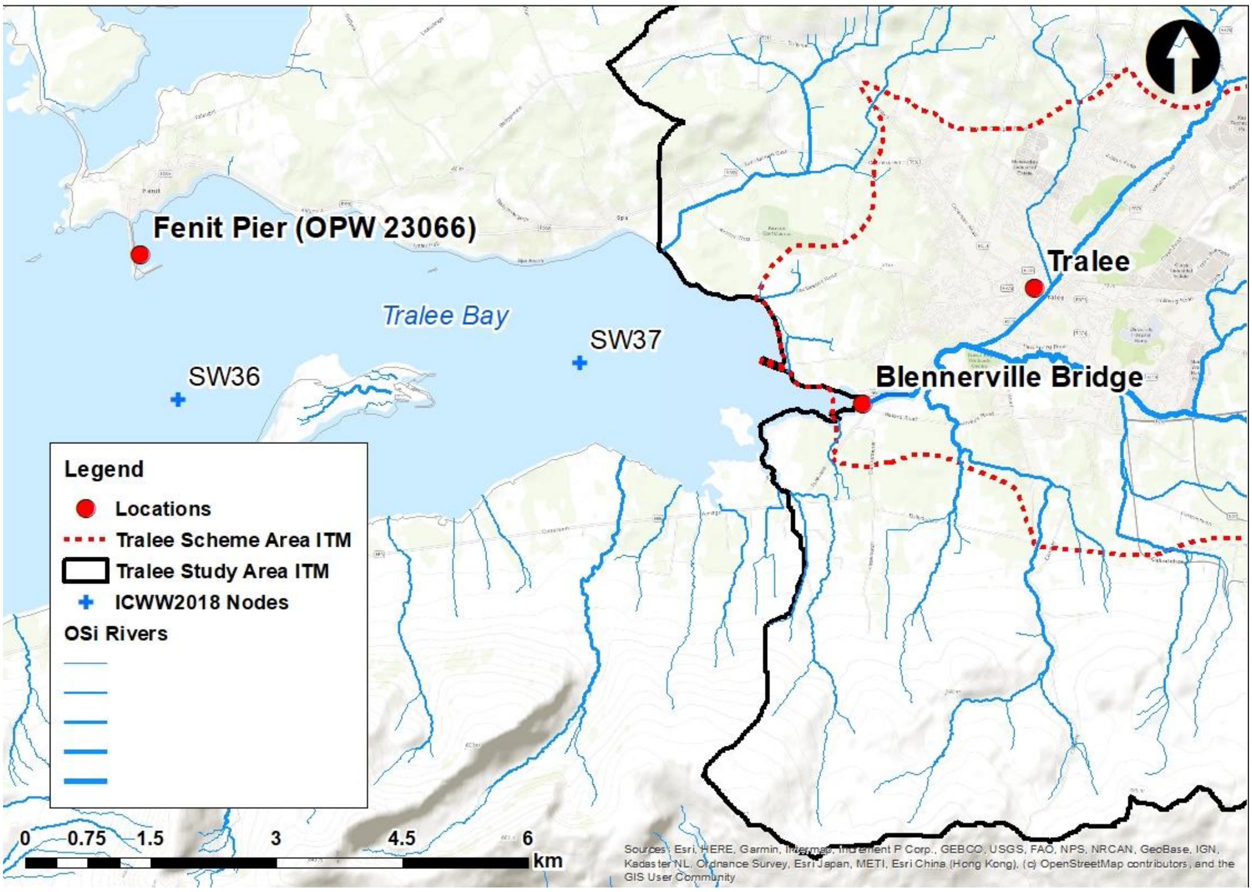


Figure 5.1: Location of Extreme Water Level Nodes

The levels taken from the ICWWS 2018 at estimation point (node no. SW_37) for the relevant AEPs are presented in **Figure 5.1**. The water levels are provided to Ordnance Datum (OD) Malin OSGM15 and Mean Sea Level (MSL).

[33] RPS, 2020. Irish Coastal Wave and Water Level Modelling Study 2018: Phase 1 – Extreme Water Levels. Available at: https://s3-eu-west-1.amazonaws.com/docs.floodinfo.opw/floodinfo_docs/ICWWS-2018/Phase-1/IBE1505_ICWWS_Ph1_Rp01_F02.pdf

[34] RPS 2013. Irish Coastal Protection Strategy Study: Phase 4 - South West Coast: Work Packages 2,3 & 4A - Technical Report. Available at: https://www.gov.ie/en/collection/572115-irish-coastal-protection-strategy-study-phase-4-south-west-coast/

Table 5.1: Extreme Coastal Water Levels - metres to OD Malin OSGM15 and Mean Sea Level (MSL)

ICWWS 2018 Peak Water Level								
AEP % (return period)	50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
SW_37 (OD Malin OSGM15)	3.07	3.18	3.25	3.32	3.40	3.45	3.50	3.62
SW_37 (MSL)	2.95	3.06	3.13	3.20	3.28	3.33	3.38	3.50

5.1 Development of Coastal Model

A detailed coastal model of Tralee Bay was developed, using the MIKE21 Flexible Mesh Flow Model. This included extensive bathymetric and topographic data available within the model domain, which was primarily obtained from INFOMAR, a joint project between the Geological Survey Ireland (GSI) and the Marine Institute, with data collected from 2008 to 2014. This was supplemented with LiDAR data from Bluesky (2018) and Admiralty Chart data as digitally supplied by C-Map of Norway to ensure coverage of the entire domain. All model data was converted to Mean Sea Level, as shown in **Figure 5.2**.

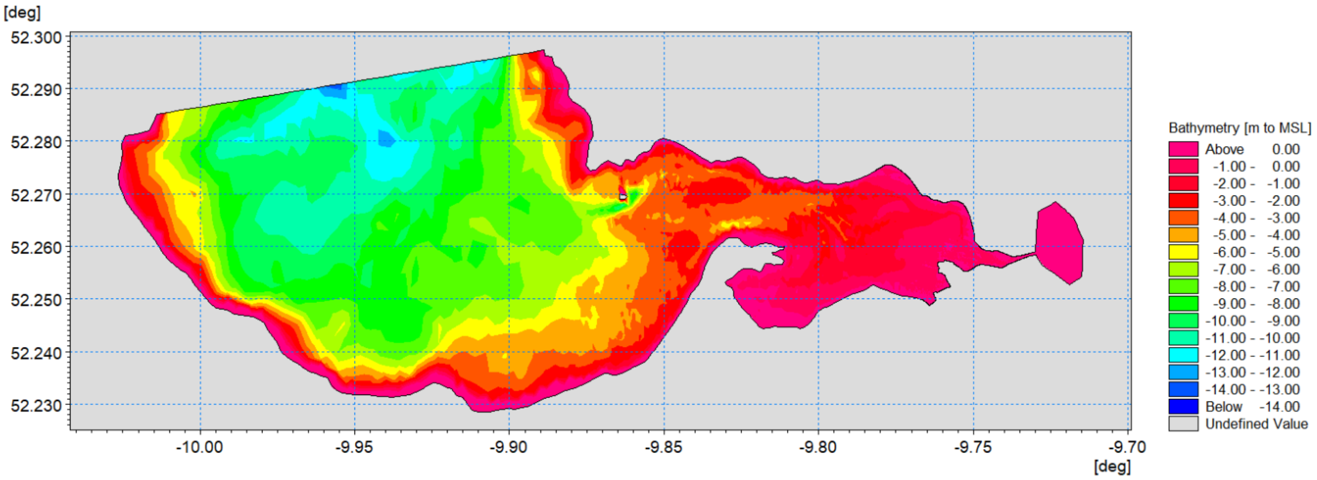


Figure 5.2: Tralee Coastal Model Extents and Bathymetry

The model utilises flexible mesh technology allowing the size of the computational cells to vary depending on user requirements. To adequately represent the variable bathymetry, the model mesh was generated and refined in regions of most importance to ensure satisfactory model performance. Thus, the model provides greater detail along the shoreline and over banks and coastal features when compared to other parts of the model domain. Cell sizes range from 200-300m at the boundary to circa 10m adjacent to the shoreline.

5.1.1 Derivation of Coastal Model Boundaries

The Tralee Bay coastal model has one open boundary to the north, and was driven using Flather conditions, whereby both the surface elevation and velocity components were defined at a series of nodes along the line boundary. Conditions for the boundary were extracted from the RPS Irish Seas Model (ISM), the extent of which is shown in **Figure 5.3**. The ISM was used to hindcast a series of historic storm events, as part of the ICWWS 2018.

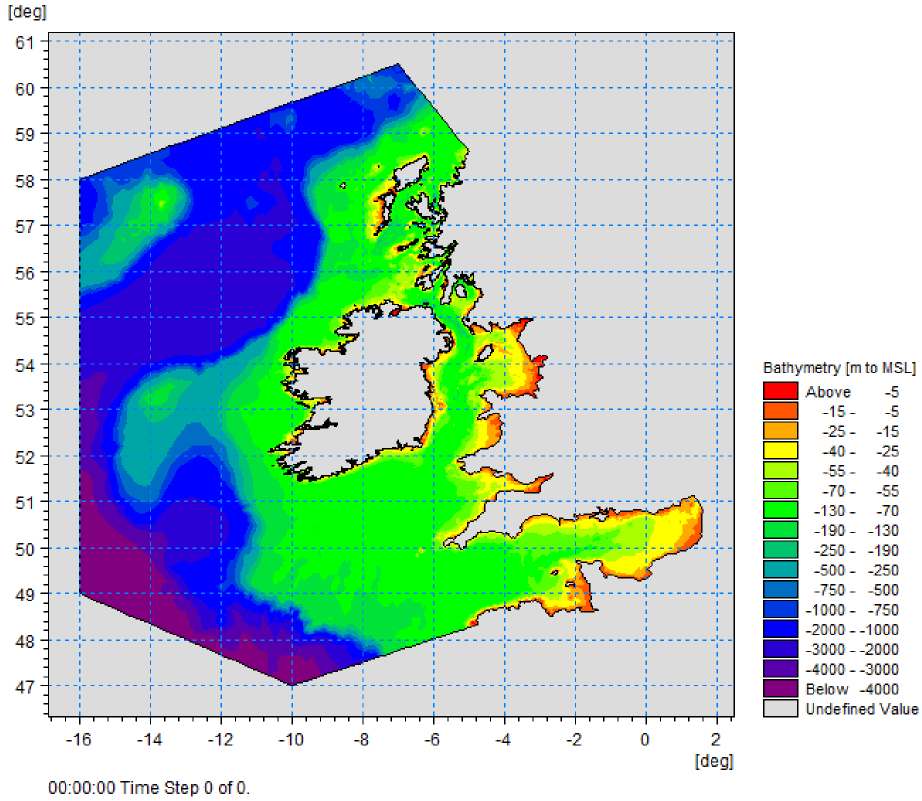


Figure 5.3: RPS Irish Seas Surge Model

RPS selected the January 2014 event as a representative extreme coastal water level event for the area in terms of meteorological influences, and which was one of the largest coastal water levels on record at the Fenit gauge. The corresponding simulation was then ran using the ISM with no meteorological effects. This allowed for a separation of the tidal and surge elements to be assessed and scaled appropriately at the Tralee Bay model boundary. The derived surge element was normalised and scaled, without adjusting the existing tidal element and when recombined, this created a series of coastal total water level boundaries which were used to drive the Tralee Bay coastal model and produce the required water levels at SW37 and the corresponding water levels in Tralee Bay for the range of required AEPs.

Figure 5.4 and **Figure 5.5** depict how the boundary surges at the 50% and 0.1% AEP events respectively have been scaled and used to adjust the total water level profiles.

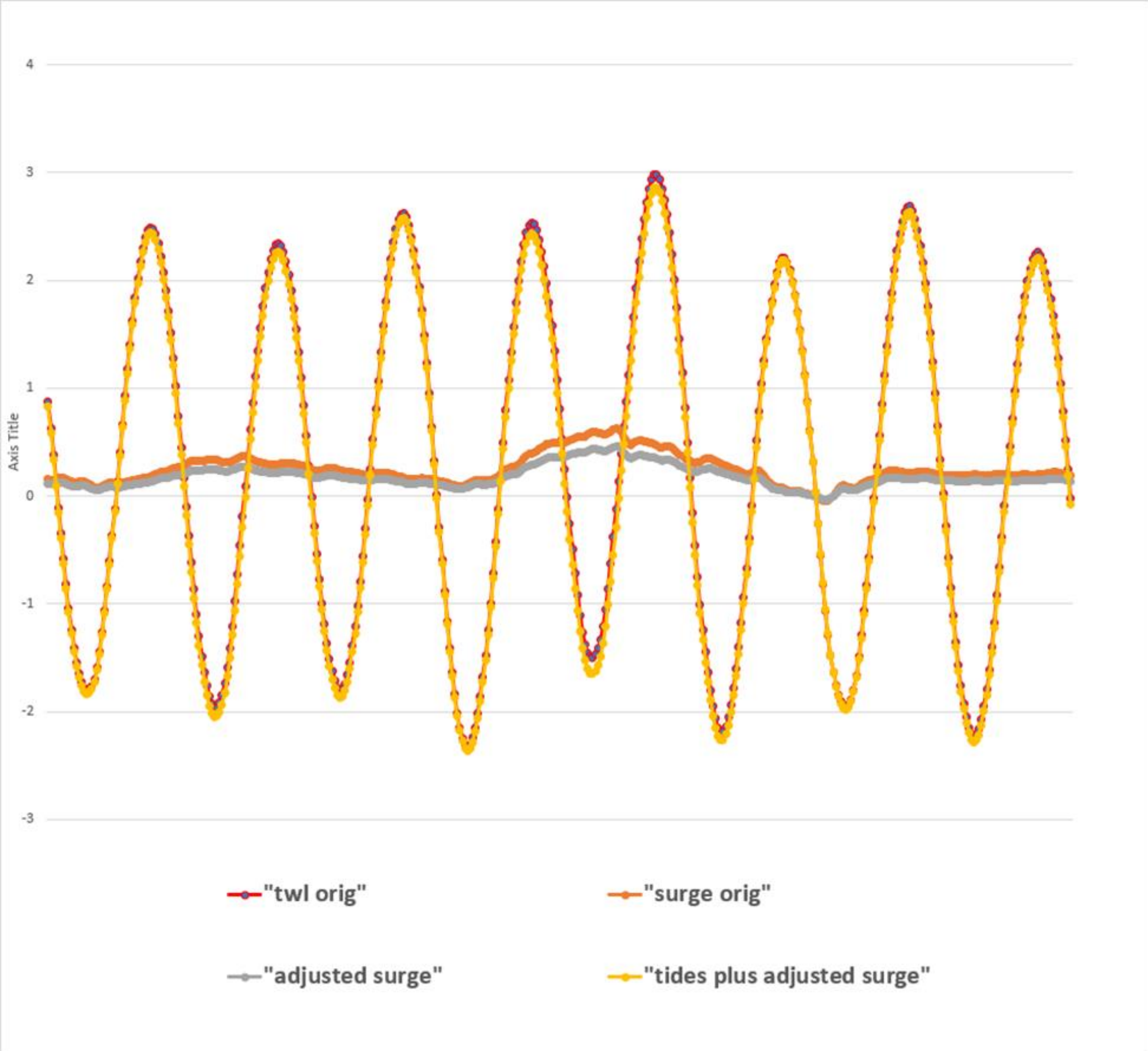


Figure 5.4: Development of Tralee Coastal Model Boundaries – 50% AEP

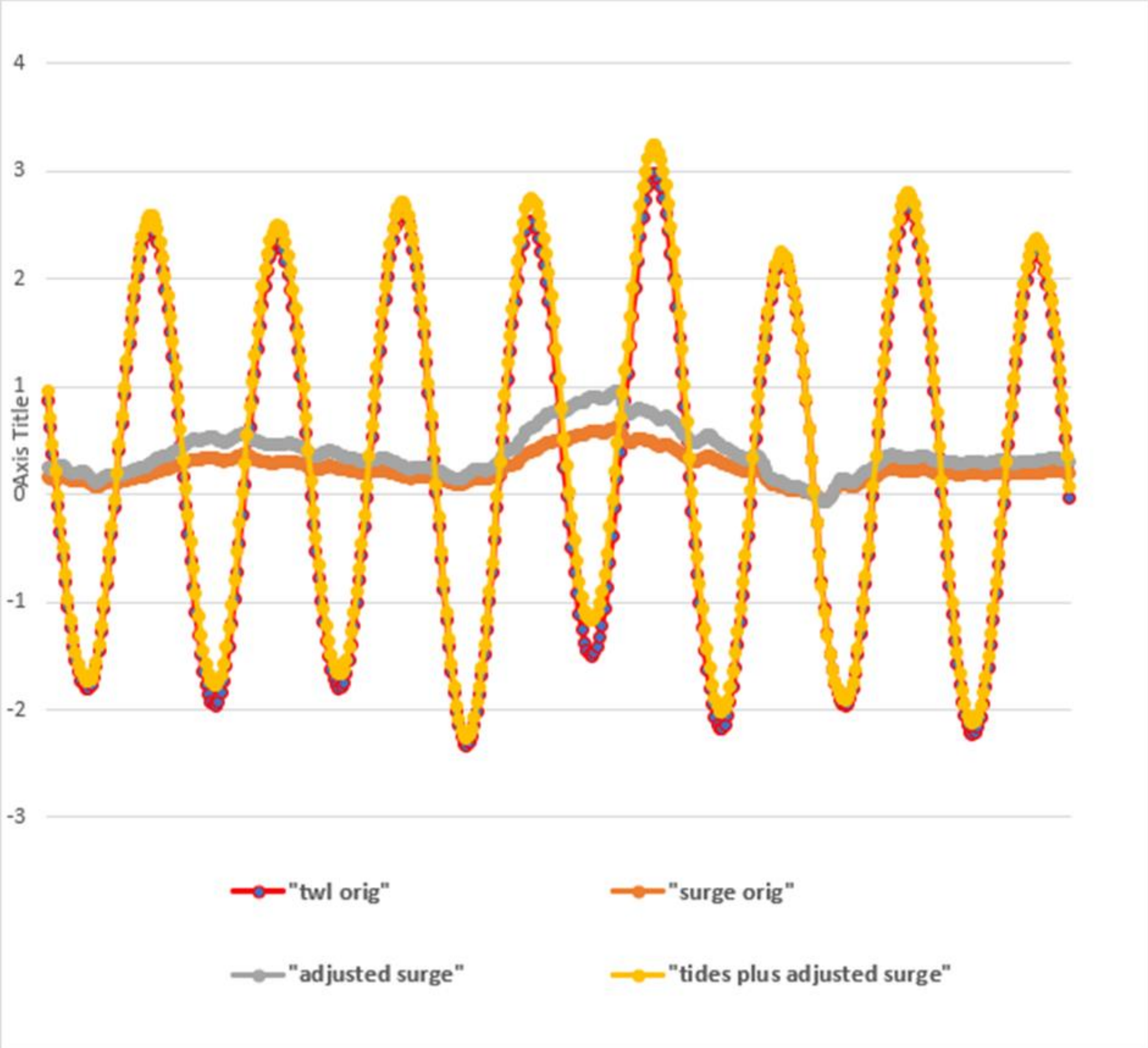


Figure 5.5: Development of Tralee Coastal Model Boundaries – 0.1% AEP

5.1.2 Wind Speeds

An assessment of appropriate wind speeds for use in the modelling was undertaken, with the dominant wind direction in the area from the westerly sectors. Wind speeds from 270° were applied to the model, with wind speeds determined with reference to those used in the ICWWS 2018 Phase 3[35] study (original source DHI MetOcean Data Portal at 52.5N, 11W). These wind speeds were adjusted and factored taking account of fetch length and duration, in order to produce a representative data set for the various AEPs, whilst maintaining an appropriate scale factor for the surge element of the water level. The resulting wind speeds for various AEP events are shown in **Table 5.2**.

[35] RPS, 2020. Irish Coastal Wave and Water Level Modelling Study 2018: Phase 3 - Tralee. Available at: https://s3-eu-west-1.amazonaws.com/docs.floodinfo.opw/floodinfo_docs/ICWWS-2018/Phase-3/IBE1505_ICWWS_Ph3_Rp003_Tralee_F03.pdf

Table 5.2: Wind Speeds Applied to Tralee Coastal Model for Various Events

% AEP event	Wind Speed (m/s)
50	14.125
20	15.195
10	16.005
5	16.805
2	17.87
1	18.68
0.5	19.48
0.1	21.35

5.1.3 Model Validation

Validation of the model was undertaken to ensure tidal flows and levels were representative of the area. Observed data from Fenit Pier (OPW 23066) was used for comparison with the modelled data, as shown in **Figure 5.6**, with both datasets correlating well. Astronomic tides and total water levels were compared.

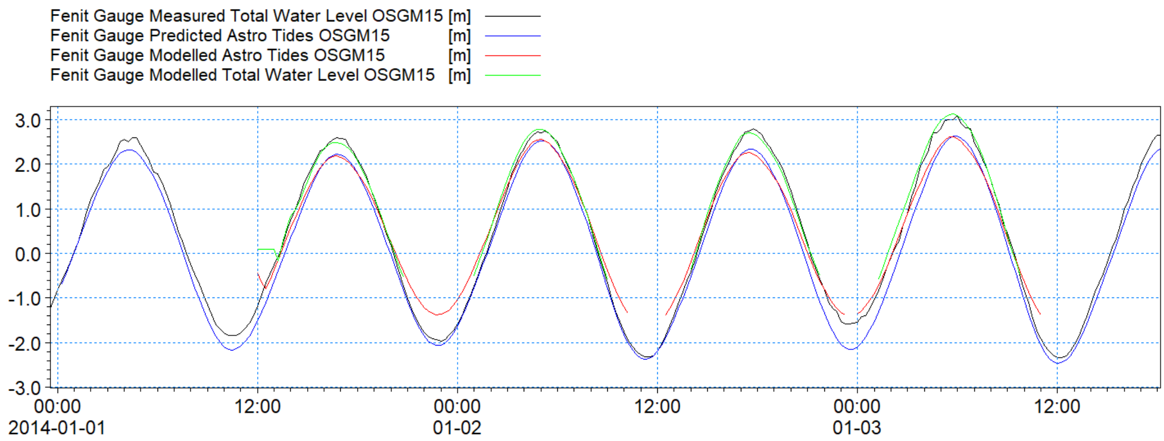


Figure 5.6: Comparison of Modelled data and Gauged Data at Fenit

5.2 Derivation of Downstream Fluvial Boundaries

Peak water levels were extracted from the MIKE21 coastal model result files and adjusted to OD Malin OSGM15 ready for incorporation into the ICM model at the relevant downstream fluvial boundaries and to facilitate coastal inundation modelling were relevant. The locations of the extraction points can be seen in **Figure 5.7**, with nodes 1-3 representing the Tralee frontage and node 4 representing ICWWS 2018 node SW37. **Table 5.3** shows the difference between the three nodes in comparison to the extreme levels estimated at SW37.

There is an uplift in these levels from SW37 to the Tralee frontage of between 30mm at P1 and 80-90mm at P2 and P3, due to wind and tidal effects and the hydrodynamic response of the estuary.

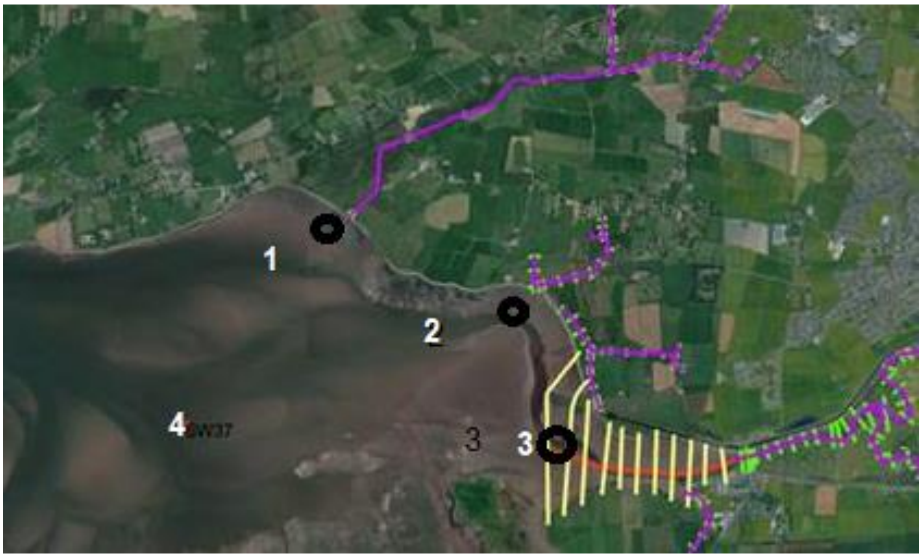


Figure 5.7: Derivation of ICM Model Boundaries

Table 5.3: Comparison of Estimated Extreme Water Levels (to MSL) at P1-3 along the Tralee Frontage with SW37

AEP (%)	P1	P2	P3	SW37	P1-SW37	P2-SW37	P3-SW37
50%	2.97	2.98	2.99	2.95	0.02	0.03	0.04
20%	3.07	3.1	3.1	3.06	0.01	0.04	0.04
10%	3.14	3.17	3.17	3.13	0.01	0.04	0.04
5%	3.21	3.24	3.24	3.2	0.01	0.04	0.04
2%	3.3	3.33	3.33	3.28	0.02	0.05	0.05
1%	3.35	3.39	3.39	3.33	0.02	0.06	0.06
0.5%	3.41	3.45	3.45	3.38	0.03	0.07	0.07
0.1%	3.53	3.58	3.59	3.5	0.03	0.08	0.09

An Extreme Value Analysis (EVA) was undertaken on the Blennerville tide gauge as a reality check on the estimated extreme water levels. This was undertaken by means of extracting Peaks over Threshold values on the digital gauge record from 2004 to present. Using a MSL to OD Malin conversion of +0.122 taken from the ICWWS 2018 Phase 4 report[36], the modelled levels at P3 were converted to 3.29m, 3.57m and 3.71m to OD Malin OSGM15 for the 10%, 0.5% and 0.1% AEPs respectively. This compares to estimates of 3.07m, 3.49m and 3.72m to OD Malin OSGM15 output from the EVA on the Blennerville gauge data, as shown in **Figure 5.8**. This equates to an estimated 80mm difference at the 0.5% AEP, well within the anticipated tolerance of 150mm. Note that as part of the ICPSS and subsequently the ICWWS 2018, a blanket seiching or local wind set-up value of 100mm was added to all AEPs at SW37. This is reflected in the higher estimations for the 50% AEP in comparison to the recorded gauge estimations.

[36] RPS, 2020. Irish Coastal Wave and Water Level Modelling Study 2018: Phase 4 - Review of Mean Sea Level and OD Malin Relationship for the Coast of Ireland. Available at: https://s3-eu-west-1.amazonaws.com/docs.floodinfo.opw/floodinfo_docs/ICWWS-2018/Phase-4/IBE1505_ICWWS_Ph4_Rp04_F02.pdf

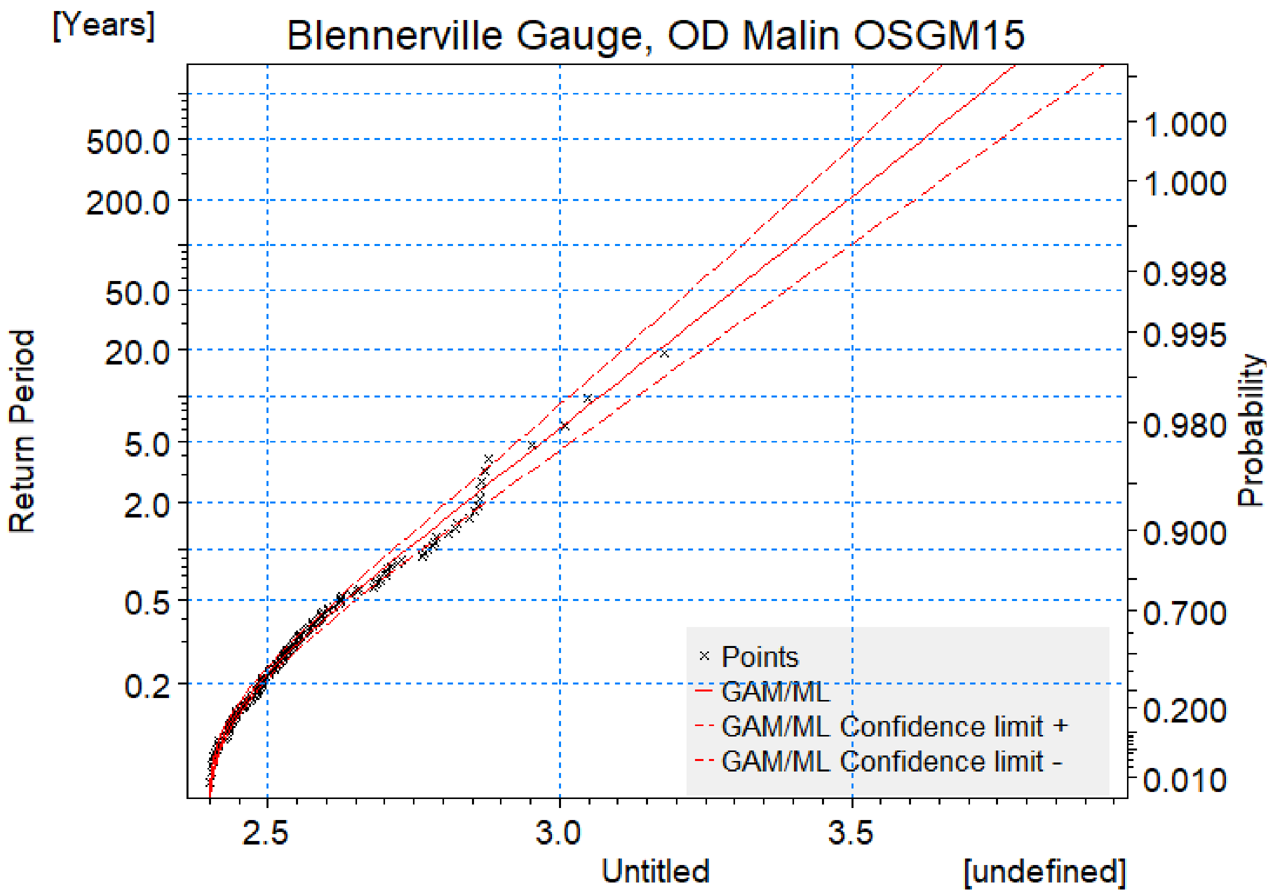


Figure 5.8: EVA on Blennerville Gauge (OD Malin OSGM15)

5.3 Allowance for Uncertainty

The sources of uncertainty in the derivation of the coastal water levels are complex and are affected by uncertainty in the observed calibration data, uncertainty in the model parameterisation, the statistical uncertainty, and the joint probability considerations which form the boundaries. The tolerance on the coastal water level boundaries is taken as ±150mm in line with the ICWWS 2018 Phase 1 report.

6 WAVE-OVERTOPPING ASSESSMENT

A wave overtopping assessment was completed for the Tralee coastline using the results of the detailed assessment of wave climate and corresponding extreme water levels, derived during Phase 3 of the ICWWS 2018[37]. This made use of geometric information on the various coastal structures at Tralee, and the EurOtop empirical models to derive overtopping rates and hence inform a subsequent assessment of associated flood risk.

6.1 Overtopping Locations

As per the ICWWS 2018, the Tralee frontage is separated into a series of Coastal Areas Potentially vulnerable to wave Overtopping (CAPOs), as shown in **Figure 6.1**. There are four CAPOs, including Lohercannon East, Blennerville, Lohercannon West and Cockleshell Beach at CAPOs A-D respectively.

Figure 6.1: Location of Tralee CAPOs

[37] RPS, 2020. Irish Coastal Wave and Water Level Modelling Study 2018: Phase 3 - Tralee. Available at: https://s3-eu-west-1.amazonaws.com/docs.floodinfo.opw/floodinfo_docs/ICWWS-2018/Phase-3/IBE1505_ICWWS_Ph3_Rp003_Tralee_F03.pdf

In order to understand if the risk of wave overtopping is significant along the Tralee frontage, a range of cross sections were chosen for each CAPO, at 100m intervals, or less where there were notable changes in the coastline definition or orientation. The locations are shown in **Figure 6.2**, whereby CAPOs A, C and D are described by a series of embankments with varying degrees of protection to the seaward side, and CAPO B features a combination of an embankment and a series of walls in differing degrees of repair.

Figure 6.2: Location of Wave Overtopping Assessment Cross-Sections at Tralee Frontage

6.2 EurOtop Empirical Calculations

The second edition of the EurOtop “Manual on Wave Overtopping of Sea Defences and Related Structures” published in 2018[38] has an author team comprising experts in the field of overtopping from several European countries. This “European Overtopping Manual” presents the latest techniques and approved methods for establishing overtopping hazards and flooding for an extensive range of structure types. Currently this approach is recognised within Europe as best practice on the analysis and/or prediction of wave overtopping for flood defences attacked by wave action.

Calculations were performed using the guidance provided in the EurOtop Second Edition 2018 manual to derive the rates (if any) of discharge over the crest of each profile along the frontage of Tralee. However, in almost all instances, the use of any of the EurOtop methods involves some degree of simplification of the true situation. Thus, as recommended in the EurOtop documentation, in situations where the importance of the assets being defended is high, and/or the uncertainties in using these methods are large, a final design solution may require use of site-specific physical model tests.

No online calculation tools were included in the release of EurOtop II and consequently RPS has used the formulas provided in the manual, to develop a series of Matlab scripts to establish overtopping predictions for a range of combined tidal levels and wave heights, of known joint AEP for each of the cross sections along the Tralee frontage. For the embankments at A, B, C and D, RPS used the design and assessment approach formulas for impermeable coastal embankments for breaking and non-breaking waves (Equations 5.12 and 5.13), using influence factors for roughness, obliqueness of the incident waves and crest walls. For the wall at CAPO B, RPS used the design and assessment approach vertical wall formulas, depending on influence of the foreshore and whether waves were impulsive or non-impulsive (Equations 7.2, 7.6, 7.9 and 7.10), which when relevant also took account of wave obliqueness.

Input parameters generally accounted for Spectral Significant Wave Height and Spectral Peak Wave Period and Mean Wave Direction, in combination with a joint probability water level, as taken from the detailed study for Tralee as part of Phase 3 of the ICWWS 2018 and shown in **Table 6.1**[39]. The Phase 3 study involved the use of a numerical model to simulate the local generation of wind waves. Relevant offshore wind speeds were combined with present day water levels using a joint probability analysis and the combinations used to provide the input to the wave model. Six combinations of present-day wave climate (heights, periods and directions) and water levels for AEPs of 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% were derived for the Tralee shoreline / defence line for the present day scenario and four future scenarios.

Structure geometry, composition and orientation formed the basis of the remainder of the input parameters for the empirical calculations, taking account of water depth at the toe of the structure, freeboard of the structure crest above the still water level, slope of the structure, roughness of the stone or vegetated seaward facing slope, and orientation to the incident wave.

On occasions where the defence crest level was lower than the full joint probability range of water levels provided by the ICWWS 2018, an additional wave and water level combination was inferred by means of interpolation, whereby the water level was generally considered to be 10mm below crest level. This provided overtopping discharges for the full potential joint probability spectrum relative to each cross-section.

Mean overtopping discharges at each combination of wave and water level data within each AEP were calculated in m^3/s/m and then converted to l/s/m. These combinations represent the mean overtopping discharges at the peak of the tide within a given storm event.

[38] J.W.van der Meer et al. 2018. Manual on wave overtopping of sea defences and related structures: An overtopping manual largely based on European research, but for worldwide application: Second edition. Available at: www.overtopping-manual.com/eurotop/downloads/

[39] RPS, 2020. Irish Coastal Wave and Water Level Modelling Study 2018: Phase 3 - Tralee. Available at: https://s3-eu-west-1.amazonaws.com/docs.floodinfo.opw/floodinfo_docs/ICWWS-2018/Phase-3/IBE1505_ICWWS_Ph3_Rp003_Tralee_F03.pdf

The following sub-sections outline details of each of the CAPOs under assessment, including the location and number of cross sections analysed for each and the structure type.

Table 6.1: ICWWS 2018 Joint Probability Wave and Water Level data- Tralee CAPOs Present Day

	CAPO A				CAPO B				CAPO C				CAPO D			
		Wind Wave Component				Wind Wave Component				Wind Wave Component				Wind Wave Component		
AEP	OD Malin OSGM15	Hm0 (m)	Tp (s)	MWD (°)	OD Malin OSGM15	Hm0 (m)	Tp (s)	MWD (°)	OD Malin OSGM15	Hm0 (m)	Tp (s)	MWD (°)	OD Malin OSGM15	Hm0 (m)	Tp (s)	MWD (°)
50%	2.65	0.63	2.67	255	2.65	0.62	2.52	277	2.65	0.76	3.44	267	2.65	0.91	3.45	265
	2.73	0.60	2.60	256	2.73	0.58	2.49	277	2.73	0.72	3.38	269	2.73	0.86	3.37	264
	2.82	0.54	3.00	257	2.82	0.54	2.46	277	2.82	0.67	3.24	270	2.82	0.79	3.21	265
	2.95	0.48	3.09	259	2.95	0.48	2.71	278	2.95	0.59	3.16	271	2.95	0.70	3.11	264
	3.03	0.45	3.07	259	3.03	0.46	2.77	278	3.03	0.56	3.12	272	3.03	0.66	3.09	265
	3.13	0.39	3.05	261	3.13	0.40	2.84	278	3.13	0.49	3.04	273	3.13	0.57	3.00	265
20%	2.73	0.68	2.74	255	2.73	0.68	2.62	277	2.73	0.82	3.54	267	2.73	0.97	3.56	265
	2.82	0.66	2.75	256	2.82	0.65	2.61	277	2.82	0.79	3.47	269	2.82	0.94	3.46	265
	2.95	0.60	3.10	258	2.95	0.59	2.57	277	2.95	0.73	3.38	270	2.95	0.86	3.36	265
	3.03	0.54	3.21	259	3.03	0.54	2.75	277	3.03	0.66	3.30	271	3.03	0.78	3.24	264
	3.13	0.49	3.12	260	3.13	0.50	2.96	278	3.13	0.61	3.18	272	3.13	0.71	3.14	265
	3.25	0.41	3.04	262	3.25	0.43	2.72	279	3.25	0.52	3.02	273	3.25	0.60	3.00	265
10%	2.82	0.72	2.78	255	2.82	0.72	2.68	277	2.82	0.86	3.67	267	2.82	1.02	3.68	265
	2.95	0.68	2.96	257	2.95	0.67	2.69	277	2.95	0.82	3.51	269	2.95	0.98	3.49	265
	3.03	0.63	3.30	258	3.03	0.63	2.69	277	3.03	0.77	3.43	270	3.03	0.91	3.41	264
	3.13	0.58	3.31	259	3.13	0.58	2.83	277	3.13	0.71	3.36	271	3.13	0.83	3.33	264
	3.25	0.50	3.13	261	3.25	0.51	2.91	278	3.25	0.62	3.15	272	3.25	0.72	3.12	265
	3.33	0.43	2.99	262	3.33	0.46	2.88	278	3.33	0.55	3.03	274	3.33	0.64	3.02	265
5%	2.82	0.76	2.83	254	2.82	0.78	2.73	276	2.82	0.92	3.75	266	2.82	1.06	3.81	266
	3.03	0.72	3.33	257	3.03	0.71	2.78	277	3.03	0.86	3.60	269	3.03	1.02	3.56	265
	3.13	0.67	3.35	258	3.13	0.67	2.81	277	3.13	0.81	3.48	270	3.13	0.96	3.46	264
	3.25	0.59	3.32	260	3.25	0.60	3.01	278	3.25	0.73	3.37	272	3.25	0.85	3.35	265
	3.33	0.53	3.08	261	3.33	0.55	2.98	278	3.33	0.66	3.18	273	3.33	0.76	3.16	265
	3.41	0.47	3.02	262	3.41	0.49	2.90	279	3.41	0.59	3.08	274	3.41	0.68	3.07	265
2%	2.95	0.82	2.94	255	2.95	0.84	2.83	276	2.95	0.97	3.79	266	2.95	1.12	3.86	266
	3.13	0.79	3.08	257	3.13	0.78	2.86	277	3.13	0.93	3.77	269	3.13	1.10	3.76	265
	3.25	0.72	3.40	259	3.25	0.72	3.13	277	3.25	0.87	3.58	271	3.25	1.02	3.54	265
	3.33	0.66	3.36	260	3.33	0.67	3.17	278	3.33	0.81	3.47	271	3.33	0.94	3.45	265
	3.41	0.60	3.28	261	3.41	0.61	3.06	278	3.41	0.74	3.37	272	3.41	0.85	3.36	265
	3.51	0.51	3.08	262	3.51	0.54	2.99	279	3.51	0.64	3.15	273	3.51	0.74	3.14	265
1%	2.95	0.85	2.96	254	2.95	0.89	2.85	276	2.95	1.02	3.63	264	2.95	1.16	3.90	266
	3.13	0.85	3.03	256	3.13	0.85	2.92	277	3.13	1.00	3.84	268	3.13	1.17	3.87	266
	3.33	0.76	3.44	259	3.33	0.76	3.19	277	3.33	0.92	3.70	271	3.33	1.07	3.65	265
	3.41	0.70	3.40	260	3.41	0.71	3.31	278	3.41	0.85	3.52	272	3.41	0.99	3.49	265
	3.51	0.61	3.32	261	3.51	0.63	3.12	278	3.51	0.75	3.40	273	3.51	0.87	3.39	265
	3.59	0.55	3.13	262	3.59	0.57	3.04	278	3.59	0.68	3.24	273	3.59	0.78	3.22	265
0.5%	2.95	0.88	2.97	253	2.95	0.94	2.85	276	2.95	1.05	3.16	263	2.95	1.18	3.90	266
	3.13	0.89	3.04	255	3.13	0.92	2.96	276	3.13	1.06	3.86	266	3.13	1.21	3.96	266
	3.33	0.85	3.68	258	3.33	0.84	3.07	277	3.33	1.01	3.85	270	3.33	1.18	3.84	265
	3.51	0.72	3.43	260	3.51	0.73	3.34	278	3.51	0.87	3.55	272	3.51	1.01	3.52	265
	3.59	0.65	3.37	261	3.59	0.67	3.27	278	3.59	0.80	3.45	273	3.59	0.92	3.44	265
	3.67	0.58	3.27	262	3.67	0.61	3.09	278	3.67	0.73	3.35	273	3.67	0.83	3.33	265
0.1%	3.03	0.95	2.96	253	3.03	1.04	2.90	276	3.03	1.14	3.19	262	3.03	1.25	3.86	266
	3.25	0.98	3.10	255	3.25	1.03	3.04	276	3.25	1.16	3.71	265	3.25	1.30	4.14	266
	3.41	0.97	3.15	257	3.41	0.98	3.09	277	3.41	1.14	4.06	268	3.41	1.31	4.11	266
	3.51	0.94	3.74	258	3.51	0.94	3.20	277	3.51	1.10	3.96	270	3.51	1.28	3.95	265
	3.59	0.89	3.75	259	3.59	0.89	3.55	277	3.59	1.06	3.89	271	3.59	1.23	3.87	265
	3.67	0.83	3.70	260	3.67	0.84	3.50	277	3.67	1.00	3.81	272	3.67	1.16	3.79	265

6.2.1 CAPO A

CAPO A is located at Lohercannon East, seaward of the Tralee Canal consisting of an embankment, with a degree of stone protection and assumed impermeability. Five cross sections were analysed along this section of coastline, as shown in **Figure 6.3**. A view of the embankment from the Blennerville Bridge is provided in **Figure 6.4**.

Figure 6.3: Cross Sections Assessed at CAPO A

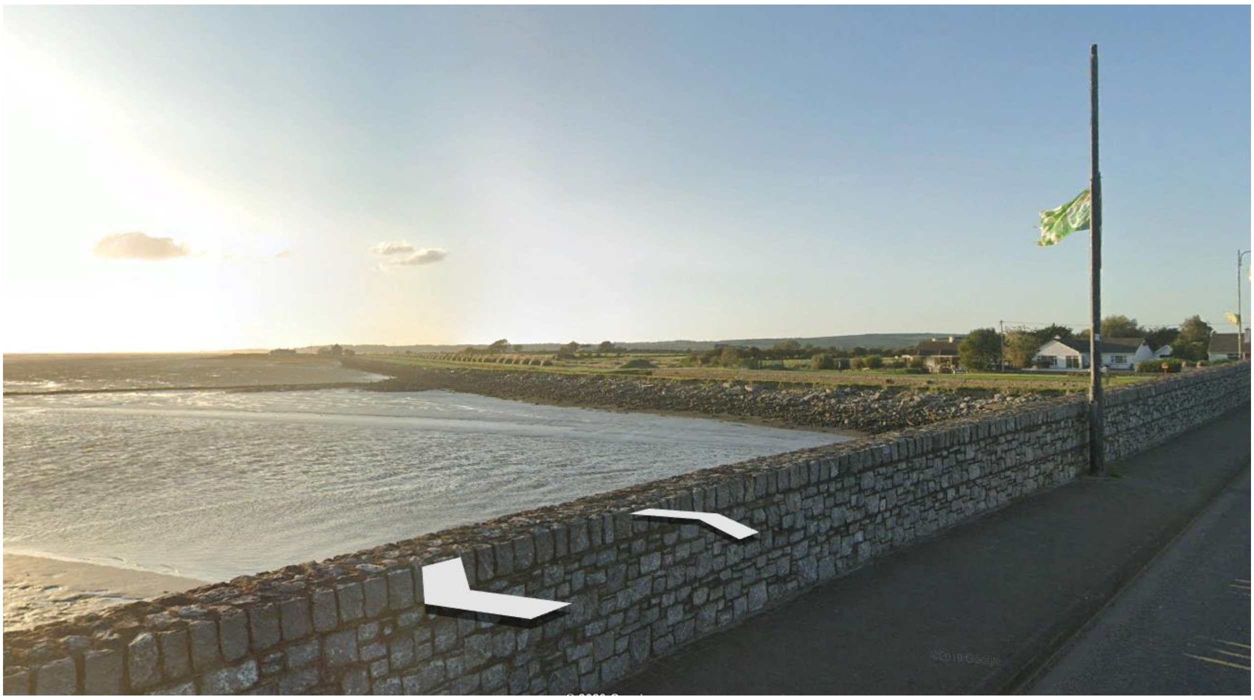


Figure 6.4: View of CAPO A from Blennerville Bridge (Google Streetview)

6.2.2 CAPO B

CAPO B is located at Blennerville, to the west of the bridge and consists of an embankment and vertical wall. Six cross sections were analysed along this section of coastline, as shown in **Figure 6.5**; a vertical wall between Blennerville Bridge and Cross Section 300S and an embankment to the west at 30W and 100W.

A view of a section of wall close to Cross Section 150 is shown in **Figure 6.6**, however sections of this wall are in a bad state of repair, as shown in **Figure 6.7**, thus two separate overtopping geometrical scenarios were considered for this CAPO in the calculations, with and without the wall above ground level.

Figure 6.8 shows the embankment at the west of CAPO B which consists of rubble frontage and is considered impermeable.

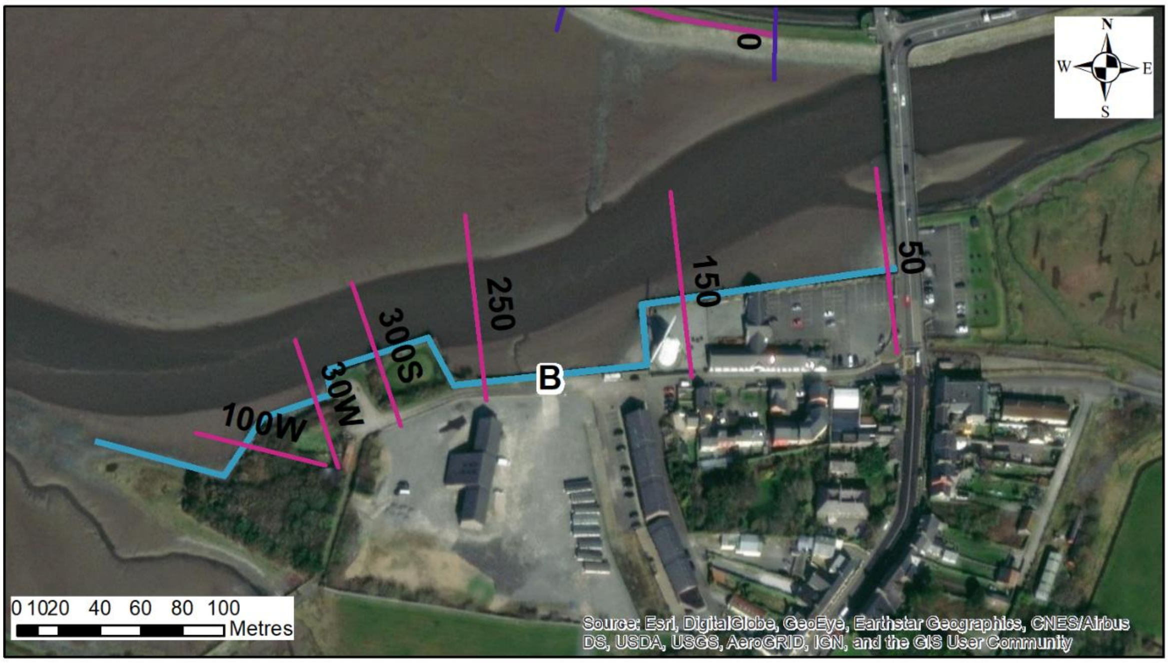


Figure 6.5: Cross Sections Assessed at CAPO B

Figure 6.6: Survey Imagery of Blennerville Wall nearby XS150

Figure 6.7: Survey Imagery Showing Damage to Blennerville Wall nearby XS250

Figure 6.8: Survey Imagery of Blennerville Embankment nearby XS100W

6.2.3 CAPO C

CAPO C is located at Lohercannon West, seaward of the Tralee Canal consisting of an embankment, with a degree of stone protection and assumed impermeability. Six cross sections were analysed along this section of coastline, as shown in **Figure 6.9**. A view of the embankment nearby Cross Section 300 is provided in **Figure 6.10**.

Figure 6.9: Cross Sections Assessed at CAPO C

Figure 6.10: Survey Imagery of Lohercannon Embankment nearby XS300

6.2.4 CAPO D

CAPO D is located at Cockleshell Beach, and consists of an embankment, with a degree of stone protection and assumed impermeability. A small wall is located at the northern end at Cross Section 800S. Nine cross sections were analysed along this section of coastline, as shown in **Figure 6.11**. A view of the wall at Cross Section 800S is shown in **Figure 6.12**, whilst **Figure 6.13**

Figure 6.13 shows a sample of the armoured defence in the vicinity of Cross Section 600. It is noted that the primary embankment is set back from the seaward edge at Cross Section 0S and is not the first line of defence.

Figure 6.11: Cross Sections Assessed at CAPO D

Figure 6.12: Survey Imagery of Wall at North End of Cockleshell Beach Embankment (XS800S)

Figure 6.13: Survey Imagery of Cockleshell Beach Embankment in vicinity of XS600

6.3 Results

Mean overtopping rates at storm peak were calculated for all four CAPOs at Tralee for each of the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP joint probability events for the present day, using combinations of water level and wave climate as produced in the ICWWS 2018 Phase 3.

Figure 6.14 to **Figure 6.17** show the range of mean overtopping discharges for the various wave and water level combinations for each return period at the location which generated the most significant rates for each CAPO.

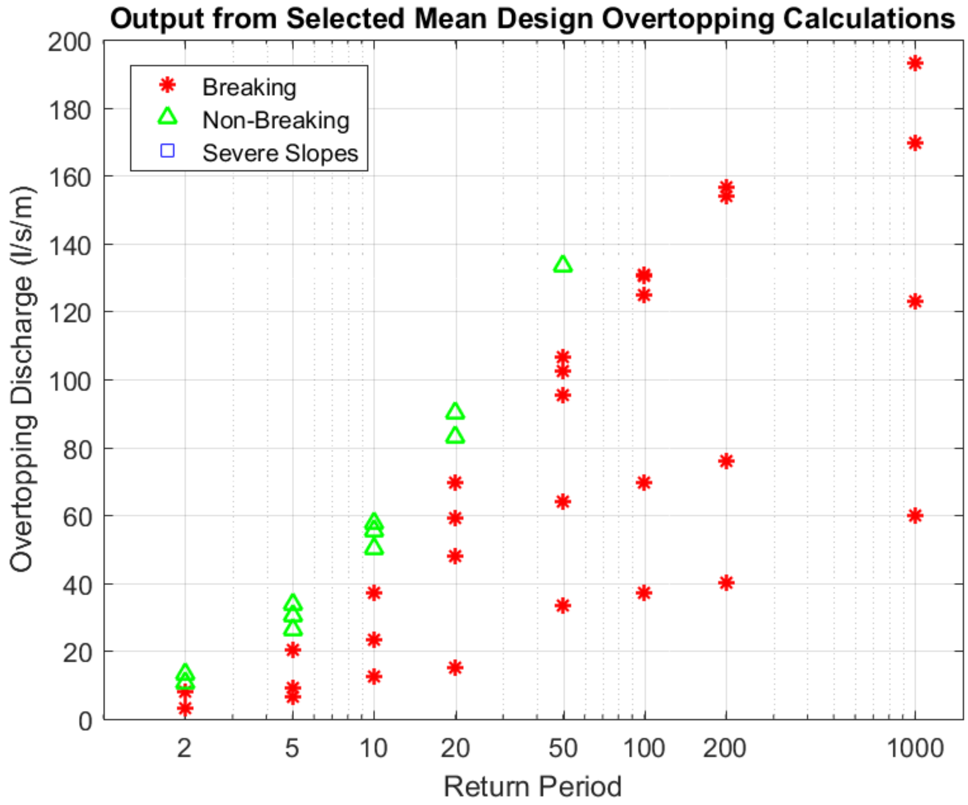


Figure 6.14: Largest Mean Overtopping Discharge at CAPO A – Ch400

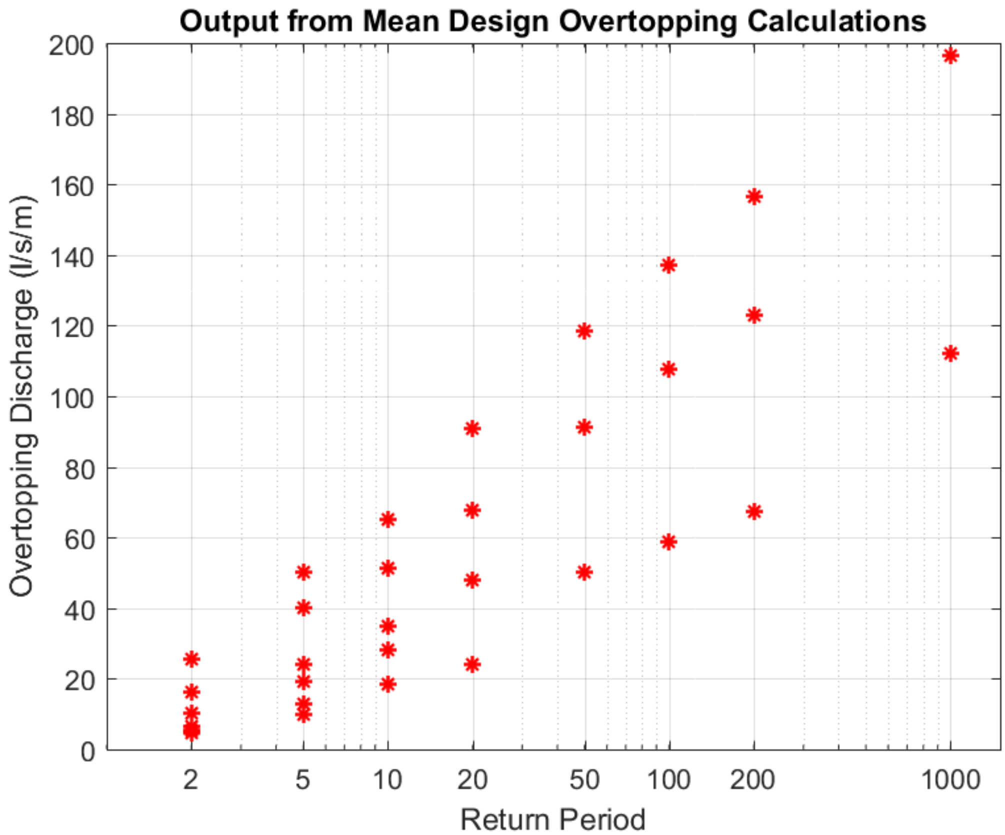


Figure 6.15: Largest Mean Overtopping Discharge at CAPO B – Ch300S

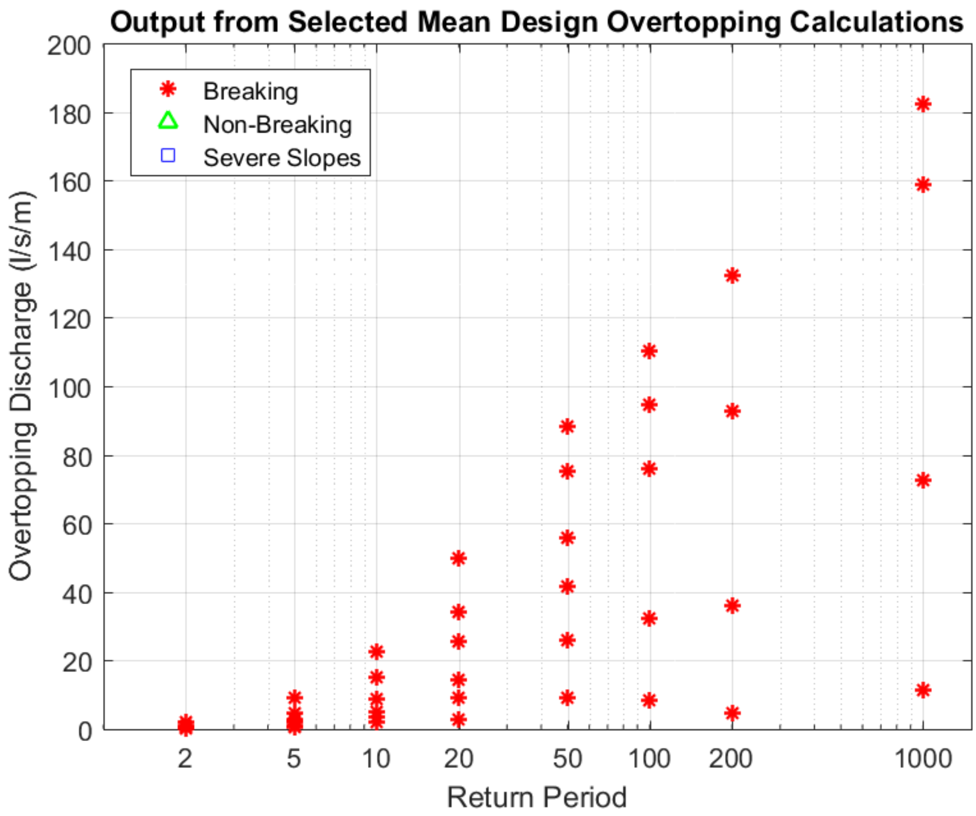


Figure 6.16: Largest Mean Overtopping Discharge at CAPO C – Ch0

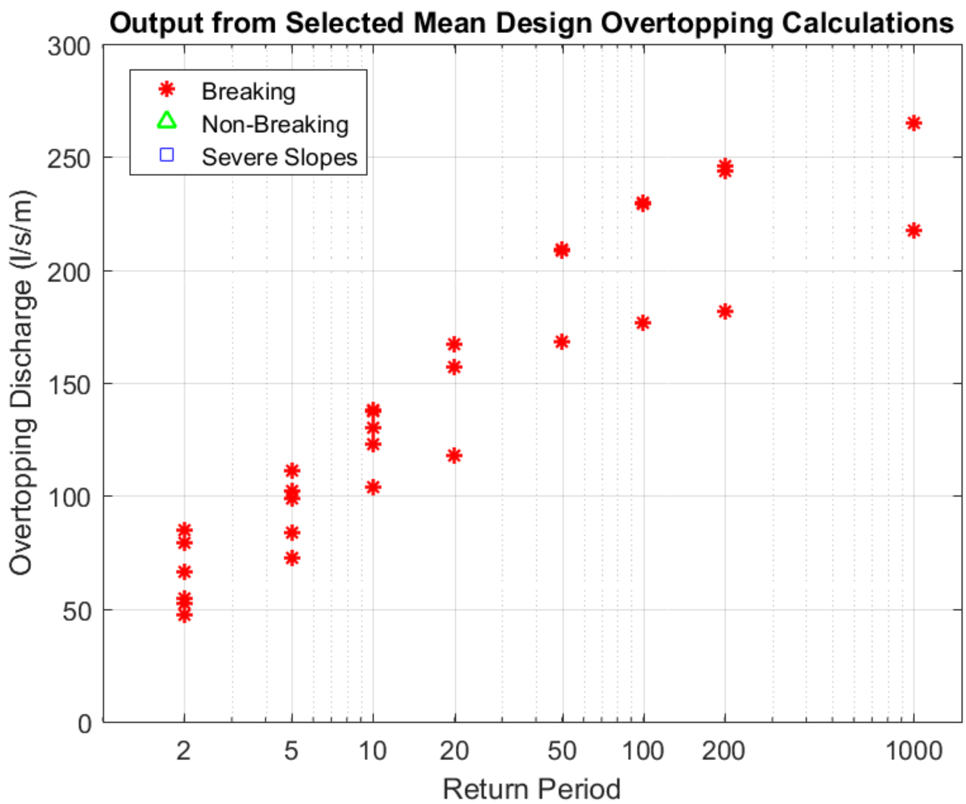


Figure 6.17: Largest Mean Overtopping Discharge at CAPO D – Ch0S

A summary of the results of the overtopping analysis are presented in **Table 6.2** to **Table 6.6** for CAPOs A to D. These tables provide the critical mean overtopping discharge estimated at the peak of the storm for a range of AEP joint probability events. These results are associated with a short time window around high water, with overtopping for the remainder of the tidal range reducing rapidly on either side of high water.

Where cells have been assigned a grey fill, this denotes the likely occurrence of still coastal water level inundation for that particular AEP event at each cross section. This means that even though wave overtopping may occur along the joint probability spectrum, when the water levels are lower and wave heights are larger, still water levels alone, with the absence of wave climate are in excess of the defined crest level for that particular AEP and will likely allow greater inundation volumes to propagate into the floodplain. This is based upon estimated still water levels at P2 and P3, as discussed under Section 5.2.

The EurOtop manual indicates acceptable limits for wave overtopping in relation to various receptors. Rates of overtopping below a threshold level of 0.03l/s/m are considered insignificant and are not considered to require further analysis.

Table 6.2: Critical Mean Overtopping Discharges at CAPO A

CAPO A	Mean Overtopping Discharge (l/s/m)				
AEP	Ch0	Ch100	Ch200	Ch300	Ch400
50%	38.101	63.685	29.490	38.869	13.261
20%	82.218	82.786	61.126	63.703	33.818
10%	86.678	103.644	76.650	80.662	57.675
5%	109.703	121.948	96.903	102.549	89.977
2%	138.297	133.758	122.304	131.111	133.424
1%	147.965	142.961	130.687	144.082	130.944
0.50%	169.243	170.867	149.428	166.764	156.444
0.10%	172.816	159.219	152.605	167.932	193.034

Table 6.3: Critical Mean Overtopping Discharges at CAPO B with Wall in Place

CAPO B	Mean Overtopping Discharge (l/s/m)					
AEP	Ch50	Ch150	Ch250	Ch300S	Ch30W	Ch100W
50%	0.161	0.003	0.011	25.638	0.000	0.000
20%	0.424	0.011	0.041	50.345	0.000	0.000
10%	0.925	0.030	0.118	64.915	0.001	0.001
5%	1.724	0.075	0.278	90.679	0.003	0.003
2%	3.991	0.255	0.876	118.403	0.021	0.016
1%	6.154	0.528	1.758	137.266	0.064	0.051
0.50%	8.581	1.071	3.274	156.407	0.172	0.120
0.10%	29.032	4.259	12.662	196.410	1.139	0.780

Table 6.4: Critical Mean Overtopping Discharges at CAPO B with Wall Removed

	Sensitivity - wall removed					
CAPO B	Mean Overtopping Discharge (l/s/m)					
AEP	Ch50	Ch150	Ch250	Ch300S	Ch30W	Ch100W
50%	58.052	24.888	0.795			
20%	77.781	50.271	4.802			
10%	96.677	64.837	13.439			
5%	115.664	90.463	30.177			
2%	140.582	118.165	61.694			
1%	156.113	137.014	83.409			
0.50%	170.620	156.143	111.673			
0.10%		196.122	166.478			

Table 6.5: Critical Mean Overtopping Discharges at CAPO C

CAPO C	Mean Overtopping Discharge (l/s/m)					
AEP	Ch0	Ch100	Ch200	Ch300	Ch400	Ch500
50%	1.892	76.701	0.378	0.715	1.226	0.745
20%	8.969	95.775	1.269	2.205	3.271	2.637
10%	22.646	115.249	3.119	5.033	6.912	6.104
5%	49.715	135.869	6.757	10.599	12.549	13.969
2%	88.052	156.457	15.973	24.243	27.073	33.551
1%	110.222	162.404	28.812	43.140	42.622	60.507
0.50%	132.327	156.450	50.095	71.065	61.989	97.122
0.10%	182.327	158.786	107.079	138.874	137.644	167.773

Table 6.6: Critical Mean Overtopping Discharges at CAPO D

CAPO D	Mean Overtopping Discharge (l/s/m)								
AEP	Ch0S	Ch100	Ch200	Ch300	Ch400	Ch500	Ch600	Ch700	Ch800S
50%	84.556	6.083	1.209	1.034	4.850	1.781	7.373	7.205	0.150
20%	111.104	13.211	2.908	2.515	9.628	4.250	15.764	13.226	0.421
10%	138.065	23.493	5.663	4.944	16.301	8.185	27.665	21.141	0.975
5%	167.216	36.230	9.578	8.441	25.167	13.597	42.477	30.863	1.604
2%	209.255	65.142	19.142	17.101	45.504	26.393	74.927	53.840	3.753
1%	229.850	92.344	28.437	25.437	63.017	39.714	104.948	70.999	5.538
0.50%	246.230	124.466	43.890	39.767	85.823	58.982	139.389	95.468	7.184
0.10%	265.102	229.275	79.384	84.305	154.946	121.225	251.768	160.456	11.556

The results of the overtopping assessment indicate that presently the embankment at CAPO A is predicted to be subject to mean overtopping discharges at the peak of a 0.1% AEP event approaching 200l/s/m. However, at this AEP, still water level inundation is anticipated to occur along the length of this CAPO. The highest mean overtopping discharge calculated, where still water level inundation is not estimated to occur is at cross section 400 during the 2% AEP event, however still water level inundation is anticipated to occur along much of the frontage of CAPO A at this AEP. No still water level inundation is estimated to occur at the 50% AEP for the cross sections analysed along CAPO A, with the highest mean discharges calculated to be 63.685 l/s/m at Cross Section 100 during this AEP.

As can be seen by comparison of **Table 6.3** and **Table 6.4** for CAPO B, when the vertical wall is set to ground level at the relevant cross sections, 50, 150 and 250, the rate of overtopping is significantly higher and may reach 196.122 l/s/m at Section 150 during the 0.1% AEP event. However, the removal of the wall, also brings the potential for much more significant inundation from still water levels, without the presence of waves. Significant overtopping and still water level inundation also has the potential to occur at Section 300S, with 0.1% AEP rates up to 196.410 l/s/m. Sections 30W and 100W at the embankment are subject to much less risk from overtopping, with the greatest estimated rates at 1.139 l/s/m at Section 30W during the 0.1% AEP event.

At CAPO C, Cross Section 100 has the potential for both still water level inundation and wave overtopping at all AEPs, due to the low level of the crest at this location. To the west of this however, still water level inundation is not anticipated to occur at any AEP, with overtopping rates greatest at Cross Section 500 with rates of 167.773 l/s/m anticipated for the 0.1% AEP.

Finally at CAPO D, still water level inundation is expected to occur for most AEP events at Section 0S, however, as mentioned previously the primary defence embankment is set back from the high-water mark and the crest level used in the calculations is not the last line of defence against still water level inundation. The remainder of the sections are generally unlikely to be subject to still water level inundation at any of the AEP events, with the highest estimated overtopping discharges at Ch600, with 251.768 l/s/m at the 0.1% AEP.

7 JOINT PROBABILITY ANALYSIS

Joint probability refers to the chance or likelihood of two or more conditions occurring at the same time. Joint probability analysis (JPA) is an important aspect of flood risk management as flood conditions are often caused by more than one environmental variable acting simultaneously.

Guidance was provided on how to conduct joint probability analysis from a DEFRA and EA joint publication in 2005; ‘Use of Joint Probability Methods in Flood Management: A Guide to Best Practice – R&D Technical Report FD2308/TR2 (2005)’ (referred to hereafter as FD2308). The technique outlined in this report follows that of the ‘desk study approach’ or the ‘simplified method’. which involves the calculation of a dependence measure – ρ – or otherwise called the Pearson correlation coefficient or bivariate correlation. The Pearson correlation coefficient is a measure of linear correlation between two datasets; therefore, one can carry out JPA for multiple pairs of variables.

As the FD2308 was developed in the mainland UK, dependence factors for pairs of variables have been developed for the UK context. Where JPA has been carried out in the UK, comparison may be possible with the analysis carried out in this report due to the shared driving forces behind dependencies. For example, from the FD2038 technical reports, the areas of highest dependency between costal water levels and river flows are found in the south-west orientated coastal estuaries. This high dependency between the two environmental variables is expected due to the direction of the prevailing weather systems across the UK. This dependency is likely mirrored in an Irish context as the Irish coastlines are affected by the same prevailing meteorological conditions.

In the case of Tralee, the Tralee Bay is orientated in a west/south-westerly direction and as such may be subject to high levels of dependency (in terms of the fluvial-coastal mechanism). The prevailing weather conditions from the Atlantic Ocean often generate coastal surges and intense rainfall events driving both fluvial flooding in the River Lee and its tributaries and coastal flooding of the lower River Lee in Tralee town.

The calculation of dependency measures in FD2308 was solely conducted on the coastal regions of the mainland UK i.e., England, Scotland, and Wales. Therefore, no proxy site exists to model joint probability in the Republic of Ireland with a different approach being required to calculate the dependency measure for a target site.

7.1 Calculation of Dependency Measure, Rho

A number of joint probability scenarios are required for Tralee FRS. Boundary combinations and the calculation of a dependency measure to input into the joint probability software is required for the following combinations:

1. Fluvial – Fluvial
2. Fluvial – Coastal
3. Fluvial – Pluvial

The statistic of Rho (ρ) is a dependency measure, otherwise called the Pearson correlation coefficient or bivariate correlation. The Pearson correlation coefficient is a measure of linear correlation between two datasets; therefore, one can carry out JPA for multiple pairs of variables. ρ was calculated for each combination to allow the assessment of dependency and hence joint probability for Tralee FRS.

An extreme value analysis (EVA) was conducted on both records in each combination following the extraction of a Peaks-Over-Threshold (POT) series. Once the POT series was extracted, the datasets were compared, and events matched based on the timing of events with a time window either side of the events. The application of a suitable time window was subjective yet an important part of the analysis to capture any true dependency between environmental variables.

A power transformation was applied to the datasets to transform the parametric datasets into a normal-like distribution. The data transformation techniques help stabilise variance in datasets and improves the validity of dependency measures, such as the Pearson correlation. It is important to note that this method of power transformation of data does not change the underlying data.

7.2 Fluvial – Fluvial Joint Probability

The fluvial-fluvial dependency measure and joint probability was considered based on the relationship between fluvial flows from Ballymullen gauging station on the River Lee and the inactive Tralee Clonalour gauging station on the Big River. Tralee Clonalour gauging station was not an ideal candidate to carry out the dependency calculation as the station has been inactive since 2012 and the rating curve produced for the station was highly uncertain at higher and out of bank flows. However, all other inactive and active gauging stations in the Tralee study area were either influenced by the tide or had record periods too short to determine any dependency with sufficient certainty.

The fluvial flows from Ballymullen and Tralee Clonalour gauging stations from the EVAs at each site were plotted and a Pearson correlation carried out to obtain the ρ value.

The scatter plot shows a strong positive correlation between POT events at Ballymullen and Tralee Clonalour gauging station. The Pearson's R test returned a ρ of 0.648, shown in **Figure 7.1**. This reflects the high likelihood of flood flows in the River Lee being coincident with flood flows in the Big River.

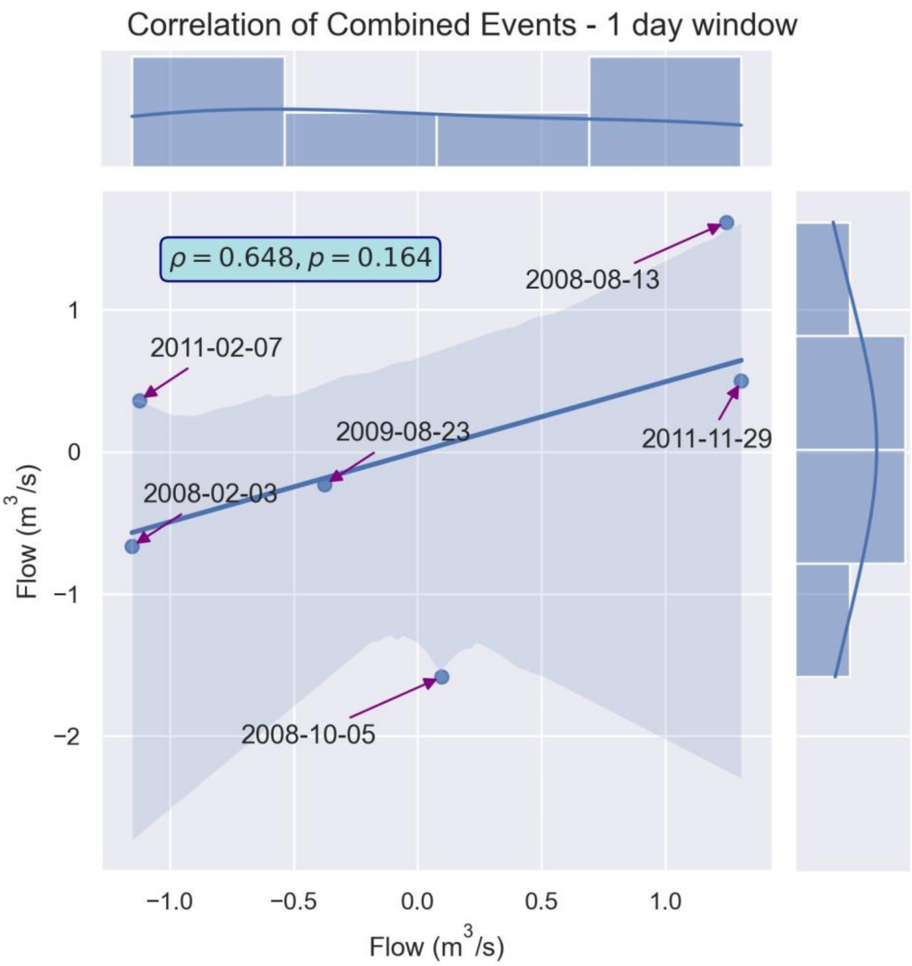


Figure 7.1: Calculation of Rho (ρ) using Pearson's R Method of Correlation Testing Ballymullen Flow with Tralee Clonalour Flow. Also included is the p-value. A p-value less than 0.01 shows statistical significance, however, the p-value is only reliable for large datasets, typically greater than 500

7.2.1 Fluvial-Fluvial Joint Probability using FD2308 Software

The dependency measure ρ was taken forward as 0.648. This was input into the FD2308 software alongside the peak flows for the River Lee at Ballymullen and for the Big River tributary at Tralee Clonalour. The output of the joint probability assessment is shown in **Figure 7.2**.

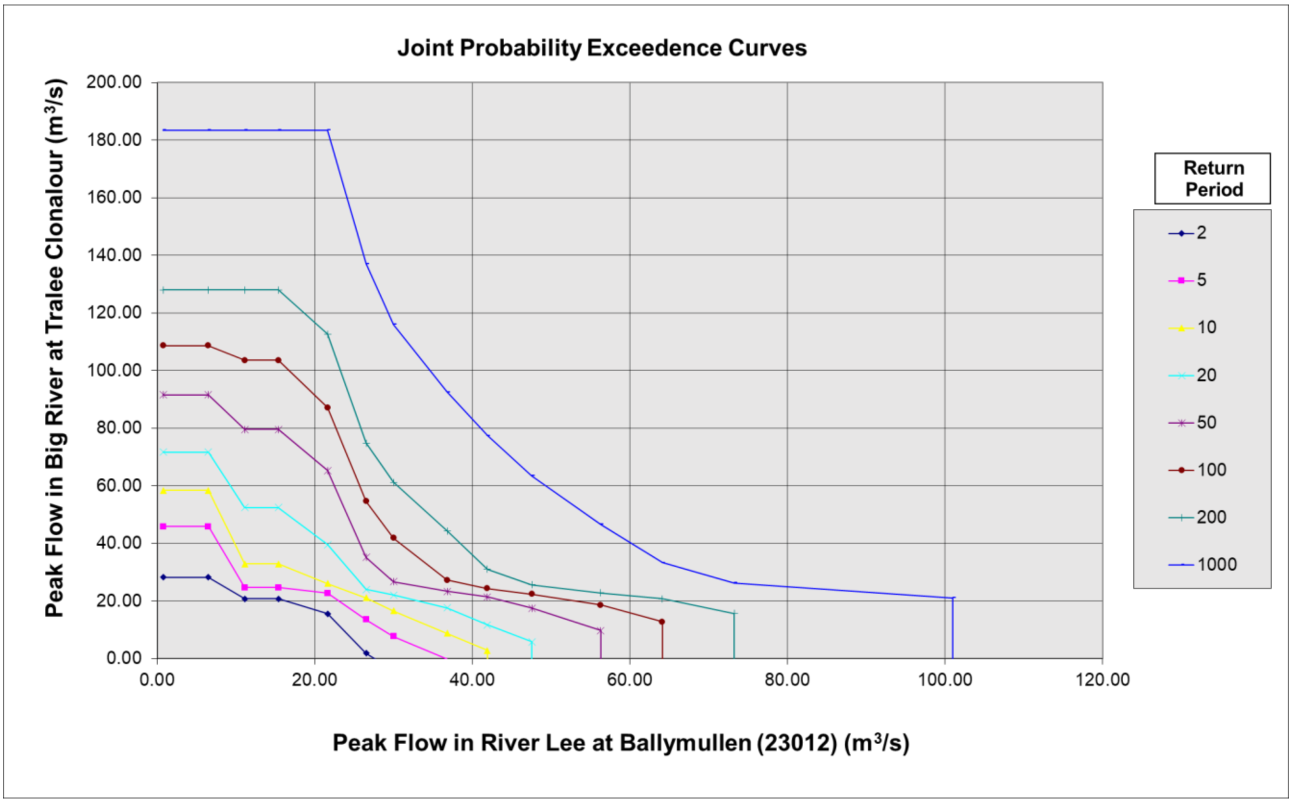


Figure 7.2: Joint Exceedance Curve Output for Ballymullen and Tralee Clonalour Gauging Station from FD2308 Software of Joint Probability

Table 7.1 shows the fluvial components from the River Lee and its tributaries (indicative of the Tralee Clonalour gauging station) of a Q100 or 1% AEP joint exceedance event for a number of points along the curve shown in **Figure 7.2**. Smaller events, where flows may fall below the QMED are considered invalid.

Table 7.1: Fluvial-Fluvial Joint Probability Combinations of a 1% AEP Joint Exceedance Event

Peak Flow in River Lee, at Ballymullen (m^3/s)	Peak Flow In Big River, at Tralee Clonalour (m^3/s)
0.77	108.63
6.48	108.63
11.11	103.56
15.38	103.56
21.64	87.08
26.51	54.64
30.00	41.87
36.80	27.27
41.91	24.37
47.56	22.40

Peak Flow in River Lee, at Ballymullen (m^3/s)	Peak Flow In Big River, at Tralee Clonalour (m^3/s)
56.27	18.63
64.10	12.74

7.3 Fluvial – Coastal Joint Probability

The fluvial-coastal dependency measure and joint probability was considered based on the relationship between fluvial flows from Ballymullen gauging station on the River Lee and the coastal station at Blennerville over a two-day window. In the case of Blennerville, node on. SW37 (located ≈ 3.3 km west of Blennerville) is more appropriate to extract extreme water levels from the ICWWS 2018 analysis due to its proximity to the station.

Blennerville coastal gauge is located at the mouth of the River Lee. Due to its close proximity to the River Lee, it is important to note a false dependency may be obtained, i.e., the fluvial mechanism driving flood events at Ballymullen gauging station may also drive peak events at Blennerville gauging station. For this reason, analysis was also carried out between flows at Ballymullen and Fenit Pier coastal gauge as a sense check.

Observed data from Fenit Pier (OPW 23066) was available from August 2010 to March 2020, while observed data from Blennerville (OPW 23062) was available from February 2004 to December 2022. The longer record present at Blennerville offers more opportunity to find correlated extreme events between it and Ballymullen gauging station, offering more certainty in the correlation analysis, assuming insignificant fluvial influence at Blennerville.

The fluvial flows from Ballymullen and coastal events from Fenit Pier and Blennerville from EVAs at each site were plotted and a Pearson correlation carried out to obtain the ρ value (**Figure 7.3**). A ρ value of 0.047 was obtained for the correlation relationship between Ballymullen and Fenit Pier, while a ρ value of 0.143 was obtained for the correlation relationship between Ballymullen and Blennerville. There is a weak positive correlation between Blennerville and Ballymullen, and more so when compared to Fenit Pier.

While there are multiple POT events coincident from both mechanisms in both analyses (22 coinciding events at Fenit Pier/Ballymullen and 28 at Blennerville/Ballymullen), the ρ value reflects the low likelihood of flood flow conditions in the River Lee being coincident with peak sea water levels. The ρ value of 0.143 obtained from the Blennerville/Ballymullen correlation is as a more conservative estimate compared to that obtained from Fenit Pier, albeit a very low correlation factor. Concern of fluvial influence, if any, at Blennerville is deemed redundant with such a low correlation factor and it is deemed suitable to proceed with Blennerville as the coastal proxy in the analysis.

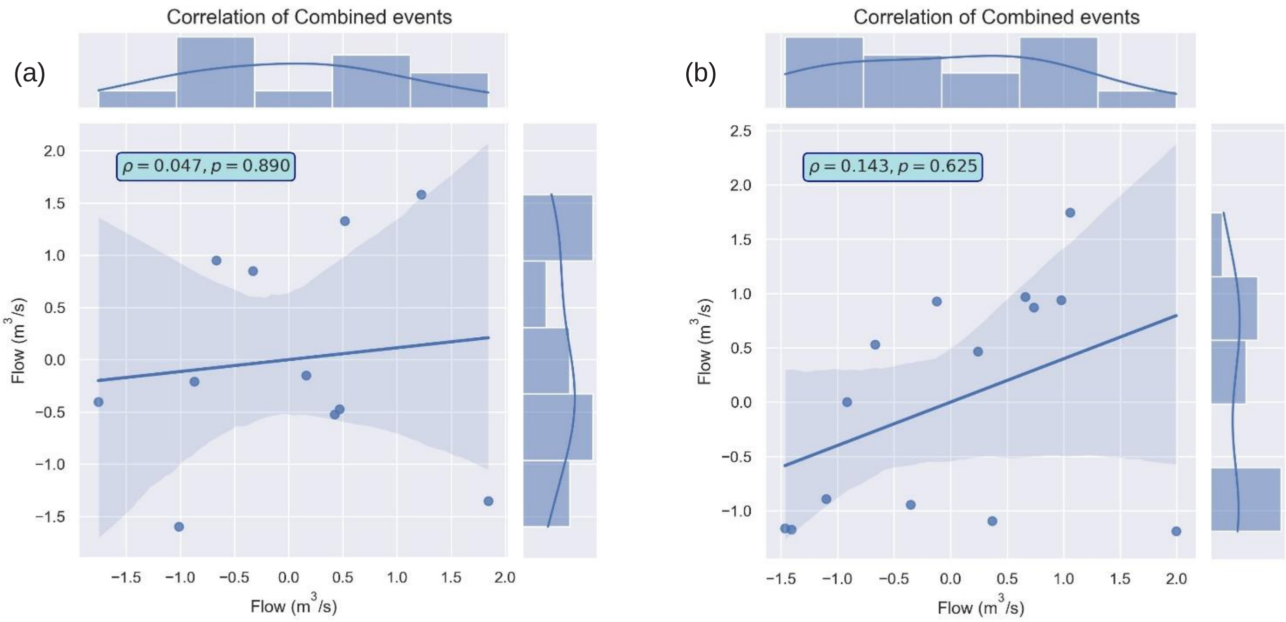


Figure 7.3: Calculation of Rho (ρ) using Pearson's R method of correlation testing Ballymullen Flow with (a) Fenit Pier Water Levels and (b) Blennerville Water Levels. Also included is the p-value. A p-value less than 0.01 shows statistical significance, however, the p-value is only reliable for large datasets, typically greater than 500

7.3.1 Fluvial-Coastal Joint Probability using FD2308 Software

The dependency measure ρ was taken forward as 0.143. This was input into the FD2308 software alongside the peak flows for the River Lee at Ballymullen and peak sea water levels for Blennerville, taken fat the ICWWS node SW37. The output of the joint probability assessment is shown in **Figure 7.4**.

Table 7.2 shows the fluvial components from the River Lee and the coastal component from Blennerville of a Q100 or 1% AEP joint exceedance event for a number of points along the curve shown in **Figure 7.4**. Smaller events, where flows may fall below the Q_{med} are considered invalid.

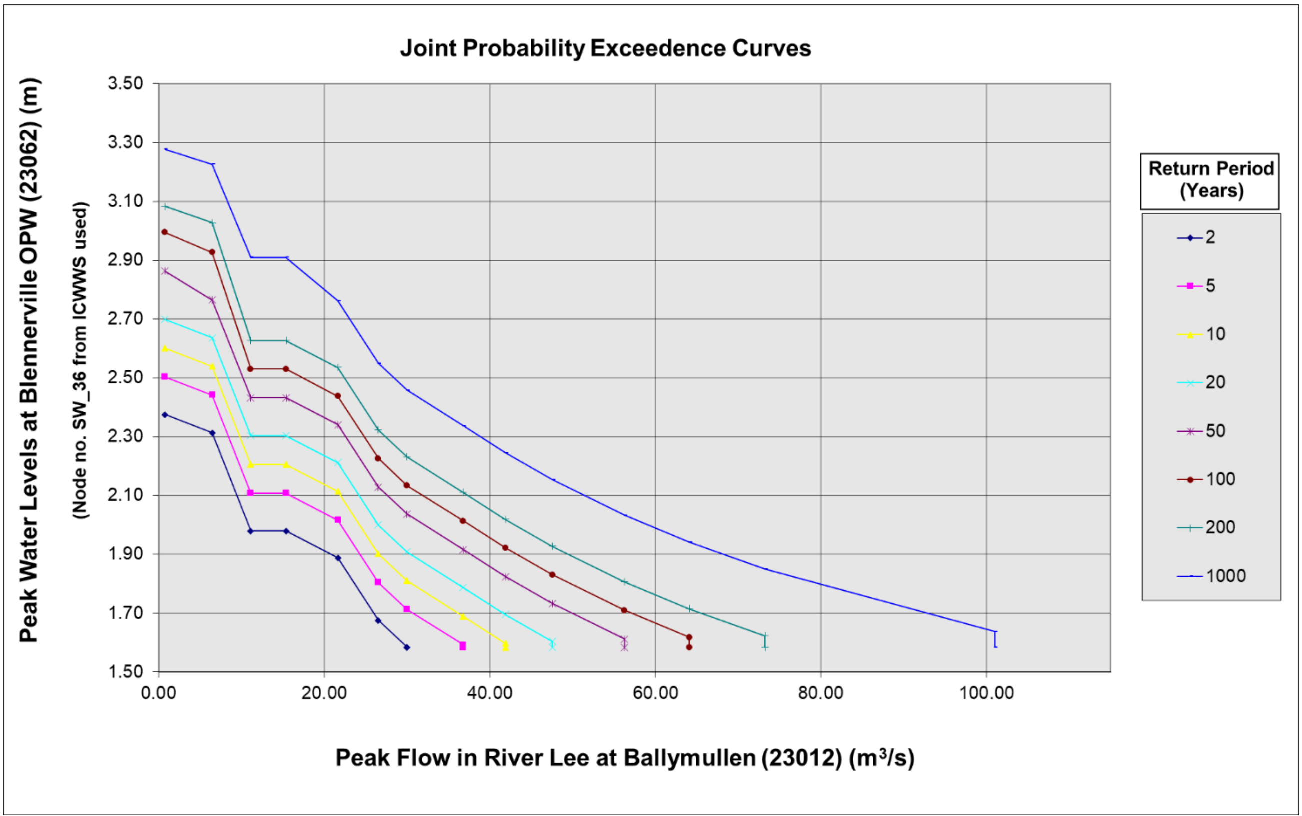


Figure 7.4: Joint Exceedance Curve Output for Ballymullen and Fenit Pier Station from FD2308 Software of Joint Probability

Table 7.2: Fluvial-Coastal Joint Probability Combinations of a 1% AEP Joint Exceedance Event

Peak Flow in River Lee, at Ballymullen (m^3/s)	Peak Sea Water Level at Blennerville (m)
0.77	3.11
6.48	3.03
11.11	2.53
15.38	2.53
21.64	2.44
26.51	2.23
30.00	2.13
36.80	2.01
41.91	1.92
47.56	1.83
56.27	1.71
64.10	1.62

7.4 Fluvial – Pluvial Joint Probability

The fluvial-pluvial dependency measure and joint probability was considered based on the relationship between fluvial flows from Ballymullen gauging station on the River Lee and the daily rainfall gauging station at Lisaboula. Lisaboula is the nearest rainfall gauge for which data is available from 1980 to present day and is located approximately 6km south-east of Tralee town centre (latitude: 52.229, longitude: -9.649).

The fluvial flows from Ballymullen and pluvial events from Lisaboula rainfall station from EVAs at each site were plotted and a Pearson correlation carried out to obtain the ρ value.

The scatter plot shows a positive correlation between POT events at Ballymullen gauging station and Lisaboula rainfall station, however, it is a weak to low correlation. The Pearson's R test returned a ρ of 0.214, shown in **Figure 7.5**. While there are multiple POT events coincident from both mechanisms, the ρ value reflects the low likelihood of flood flow conditions in the River Lee being coincident with peak daily rainfall events.

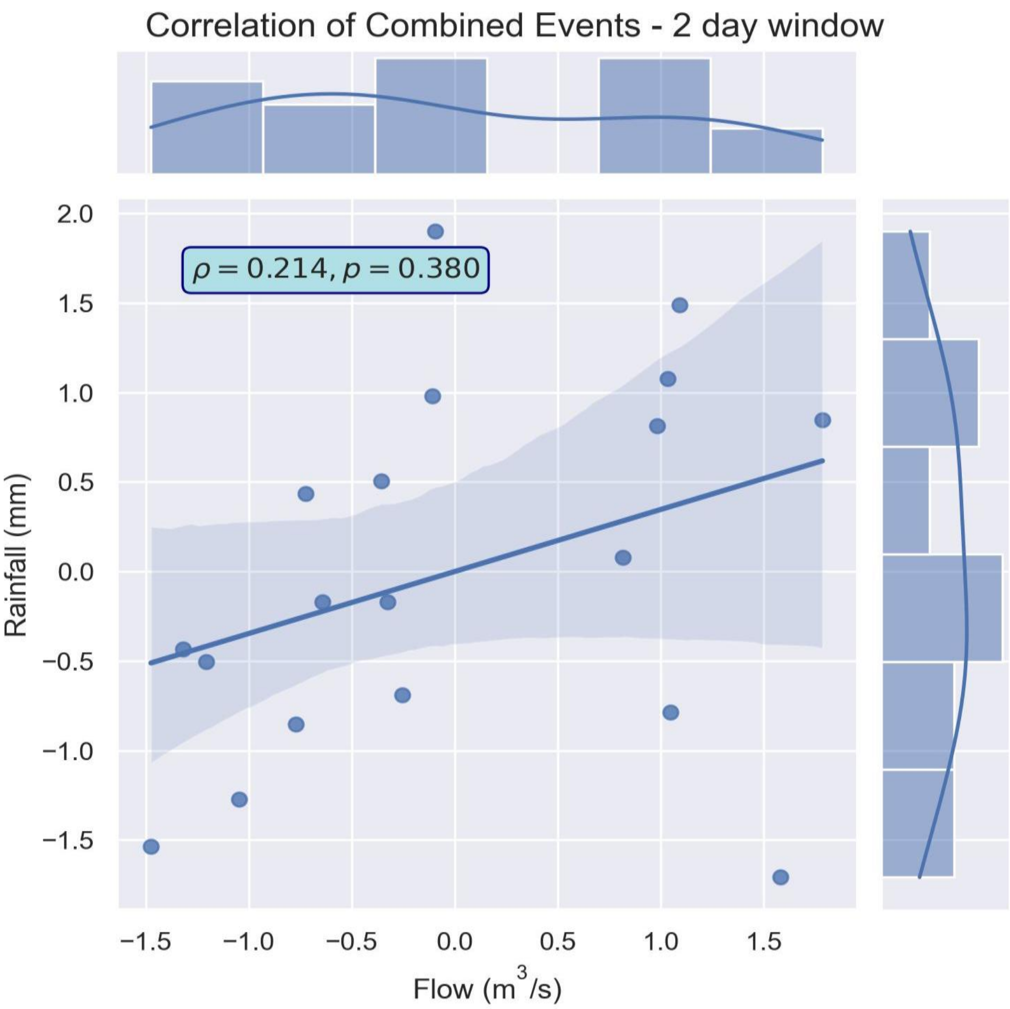


Figure 7.5: Calculation of Rho (ρ) using Pearson's R method of correlation testing Ballymullen Flow with Lisaboula Daily Rainfall. Also included is the p-value. A p-value less than 0.01 shows statistical significance, however, the p-value is only reliable for large datasets, typically greater than 500

7.4.1 Fluvial-Pluvial Joint Probability using FD2308 Software

The dependency measure ρ was taken forward as 0.214. This was input into the FD2308 software alongside the peak flows for the River Lee at Ballymullen and peak daily rainfall levels at Lisaboula. The output of the joint probability assessment is shown in **Figure 7.6**.

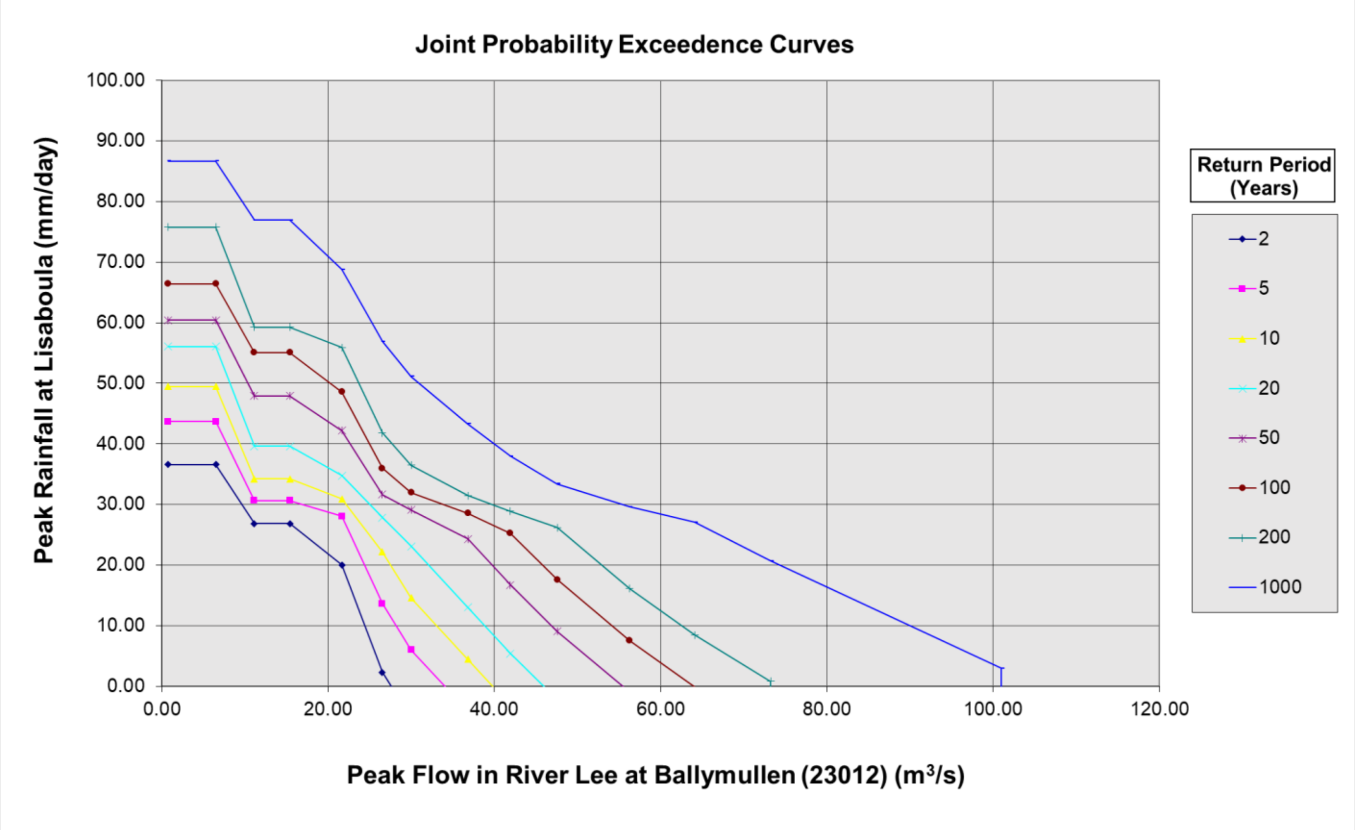


Figure 7.6: Joint Exceedance Curve Output for Ballymullen Gauging Station and Lisaboula Rainfall Station from FD2308 Software of Joint Probability

Table 7.3 shows the fluvial components from the River Lee and the pluvial component from Lisaboula of a Q100 or 1% AEP joint exceedance event for a number of points along the curve shown in **Figure 7.6**. Smaller events, where flows may fall below the Q_{med} are considered invalid.

Table 7.3 Fluvial-Coastal Joint Probability Combinations of a 1% AEP Joint Exceedance Event

Peak Flow in River Lee, at Ballymullen (m^3/s)	Peak Daily Rainfall at Lisaboula Rainfall Station (mm/day)
0.77	66.43
6.48	66.43
11.11	55.08
15.38	55.08
21.64	48.60
26.51	35.95
30.00	31.95
36.80	28.57
41.91	25.24

Peak Flow in River Lee, at Ballymullen (m^3/s)	Peak Daily Rainfall at Lisaboula Rainfall Station (mm/day)
47.56	17.61
56.27	7.53
64.10	-0.10

8 SENSITIVITY ANALYSIS

Hydrological analysis and design flow estimation are probabilistic assessments which originate from observed data. The long-term conditions which affect the observations, whether they are climatic or catchment-based, have been shown to varying degrees to be changing over time. The following factors which may affect the quality of both the analysed historic events and the estimation of the future design events are listed below:

- Hydrometric data record length and gaps
- Hydrometric data quality (classified in terms of the rating confidence)
- Standard error of flow estimation (catchment descriptor-based) techniques
- Future catchment changes, urbanisation, afforestation, sedimentation, etc.
- Climate change

The above list is not exhaustive but seeks to identify the main potential sources of uncertainty in the hydrological analysis. It is not possible to make a quantitative assessment of all the uncertainties as some of the factors are extremely complex. Nevertheless, it is important that an assessment is made such that the results can be taken forward and built upon in the hydraulic modelling phase. It is also important that the potential sources of uncertainty in the hydrological analysis and design flow estimation are flagged such that the integrated process of refining the hydrological inputs and achieving model calibration can be achieved more efficiently through a targeted approach. A qualitative assessment has been undertaken to assess the potential for uncertainty/sensitivity.

The assessment of uncertainty is used to derive a score for the uncertainty in the hydrological analysis for the river reaches in the Tralee FRS based on the approach adopted for the CFRAM Studies. The parameters affecting uncertainty are assessed out of a possible maximum score dependent on the likely impact of the source of uncertainty. Uncertainty scores out of a maximum of 20 were assigned to the main inflows to the model in the CFRAM Studies.

The CFRAM Study Guidance Note 22 derived a range of uncertainty factors associated with the FSU methodologies for deriving peak flows. The uncertainty associated with Q_{med} for ungauged catchments is 1.37 but for well-gauged catchments with long records can be as low as 1.06. It is appropriate therefore that the higher end of this range of error (1.37) is applied as a sensitivity factor to the Q_{med} flows for the Tralee study area for the ungauged HEPs where there is high uncertainty. The guidance note also suggests that for more extreme design flood events where the uncertainty is inherently higher a further factor should be applied based on the known uncertainties. The recommended uncertainty factors were adapted for CFRAM Study into recommendations to be applied to the 10% and 1% AEP design events as shown in **Table 8.1**.

Table 8.1: Example of Uncertainty Factors Derived Under the CFRAM Study

<table>
<tr><th rowspan="2">Event AEP%</th><th colspan="20">Summed Score</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th><th>6</th><th>7</th><th>8</th><th>9</th><th>10</th><th>11</th><th>12</th><th>13</th><th>14</th><th>15</th><th>16</th><th>17</th><th>18</th><th>19</th><th>20</th></tr>
<tr><td>10%</td><td colspan="4">1.06</td><td colspan="6">1.14</td><td colspan="5">1.21</td><td colspan="3">1.29</td><td colspan="2">1.37</td></tr>
<tr><td>1%</td><td colspan="6">Use 10% uncertainty factor x 1.1</td><td colspan="6">Use 10% uncertainty factor x 1.2</td><td colspan="6">Use 10% uncertainty factor x 1.3</td><td colspan="2">Use 10% factor x 1.5</td></tr>
</table>

Uncertainty factors have been derived based on the quality of the available hydrometric data, the uncertainty in the catchment data, and the application of the chosen design flow method. The scores for the various aspects of uncertainty for each HEP, the summed score, and uncertainty factors and flows to be modelled are shown in **Table 8.2**.

Table 8.2: Summary of Uncertainty Scores for Major Watercourses, Tralee FRS

Sensitivity Assessment										
Reach	HEP	Observed Data (Out of 10)	Catchment Data (Out of 5)	Ungauged Flow Estimates (Out of 5)	Comments	Summed Sensitivity Score	Q10 Sensitivity Factor [2]	Q10 Sensitivity Flow [2]	Q100 Sensitivity Factor [3]	Q100 Sensitivity Flow
River Lee	**LEE_02**	**3**	**1**	**1**	Ballyseedy GS @ LEE_02 - rating review resulted in a curve which fit the spot gaugings well. However, low number of spot gaugings increases uncertainty in ungauged flow estimates up and down stream of the GS. Catchment is well-defined under FSU	**5**	1.14	26.49	1.25	46.83
Manor West	**LEE_05**	**8**	**3**	**4**	Ungauged and high uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale. Catchment is well to moderately defined under FSU	**15**	1.29	0.36	1.55	0.65
Ballybeggan	**LEE_09**	**5**	**2**	**3**	Gauged downstream. Rating review produced a good fit, but only one spot gauging so highly uncertain. Uncertainty in pivotal site. Moderate/Low uncertainty in catchment data and moderate uncertainty in FSU methods at small catchment scale.	**10**	1.21	7.85	1.33	12.16
Undefined Trib 1	**LEE_11**	**8**	**3**	**4**	Ungauged and high uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale. Catchment is well to moderately defined under FSU,	**15**	1.29	0.76	1.55	1.28

Sensitivity Assessment										
Reach	**HEP**	**Observed Data (Out of 10)**	**Catchment Data (Out of 5)**	**Ungauged Flow Estimates (Out of 5)**	**Comments**	**Summed Sensitivity Score**	**Q10 Sensitivity Factor [2]**	**Q10 Sensitivity Flow [2]**	**Q100 Sensitivity Factor [3]**	**Q100 Sensitivity Flow**
Rathass	**LEE_14**	**8**	**3**	**4**	Ungauged and moderate uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale. Catchment is well to moderately defined under FSU	**15**	1.29	0.22	1.55	0.37
Ballinorig	**LEE_19**	**8**	**2**	**4**	Ungauged and moderate uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale. Catchment is well to moderately defined under FSU	**14**	1.21	5.19	1.45	8.76
Ballybeggan	**LEE_20**	**4**	**2**	**3**	Gauged @ Manor West GS @ LEE_20. Rating review produced a good fit, but only one spot gauging so highly uncertain. Uncertainty in pivotal site. Moderate/Low uncertainty in catchment data and moderate uncertainty in FSU methods at small catchment scale.	**9**	1.14	13.54	1.25	20.96
Cloghers	**LEE_33**	**5**	**2**	**3**	Gauged downstream. Rating review produced a good fit, but only one spot gauging so highly uncertain. Uncertainty in pivotal site. Moderate/Low uncertainty in catchment data and moderate uncertainty in FSU methods at small catchment scale.	**10**	1.21	11.16	1.33	17.89

Sensitivity Assessment										
Reach	**HEP**	**Observed Data (Out of 10)**	**Catchment Data (Out of 5)**	**Ungauged Flow Estimates (Out of 5)**	**Comments**	**Summed Sensitivity Score**	**Q10 Sensitivity Factor [2]**	**Q10 Sensitivity Flow [2]**	**Q100 Sensitivity Factor [3]**	**Q100 Sensitivity Flow**
Caherweesheen	**LEE_35**	**8**	**2**	**4**	Ungauged with moderate to low uncertainty in pivotal site. Moderate to low uncertainty in catchment data and FSU methods at small catchment scale.	**14**	1.21	9.02	1.45	15.79
Cloghers	**LEE_37**	**3**	**2**	**3**	Gauged @ Spring Water Lane @ LEE_37 downstream. Rating review produced a good fit, but lack of data means the rating is highly uncertain. Uncertainty in pivotal site. Moderate/Low uncertainty in catchment data and moderate uncertainty in FSU methods at small catchment scale.	**8**	1.14	15.11	1.25	24.24
River Lee	**LEE_39**	**2**	**1**	**1**	Ballymullen GS @ LEE_39 - rating review resulted in a curve which fit the spot gaugings well. High certainty at GS following rating curve. Catchment is well-defined under FSU	**4**	1.06	45.30	1.17	77.72
Big River	**LEE_56**	**4**	**2**	**3**	Oakpark Road GS @ LEE_56 - rating review resulted in a curve which fit the spot gaugings well, but low number of spot gaugings at higher flows as well as hydraulic features on the reach make the rating uncertain. Catchment description under FSU becomes more uncertain here as you enter the town of Tralee.	**9**	1.14	12.01	1.25	18.63

Sensitivity Assessment										
Reach	**HEP**	**Observed Data (Out of 10)**	**Catchment Data (Out of 5)**	**Ungauged Flow Estimates (Out of 5)**	**Comments**	**Summed Sensitivity Score**	**Q10 Sensitivity Factor [2]**	**Q10 Sensitivity Flow [2]**	**Q100 Sensitivity Factor [3]**	**Q100 Sensitivity Flow**
Laheran	**LEE_51**	**8**	**3**	**3**	Ungauged and high uncertainty in pivotal site. Low to moderate uncertainty in catchment data and FSU methods at small catchment scale.	**14**	1.21	5.11	1.45	8.65
OUTFALLS	**LEE_62**	**8**	**5**	**4**	Ungauged and high uncertainty in pivotal site. High uncertainty in catchment data and FSU methods at small catchment scale.	**17**	1.29	0.20	1.55	0.33
Listellick	**LEE_68**	**8**	**3**	**4**	Ungauged and high uncertainty in pivotal site. High uncertainty in catchment data and FSU methods at small catchment scale.	**15**	1.29	0.92	1.55	1.58
Culverted Section	**LEE_74**	**8**	**5**	**4**	Ungauged and high uncertainty in pivotal site. High uncertainty in catchment data and FSU methods at small catchment scale.	**17**	1.29	0.92	1.55	1.57
Balloonagh	**LEE_76**	**4**	**3**	**3**	Applicability of gauge data drops moving downstream. Catchment description under FSU becomes more uncertain here as you enter the town of Tralee.	**10**	1.21	8.19	1.33	12.83
Big River	**LEE_77**	**4**	**3**	**3**	Applicability of gauge data drops moving downstream. Catchment description under FSU becomes more uncertain here as you enter the town of Tralee.	**10**	1.21	24.90	1.33	38.63
Undefined Trib 2	**LEE_82**	**8**	**4**	**4**	Ungauged and high uncertainty in pivotal site. Moderate to high uncertainty in catchment data and FSU methods at small catchment scale.	**16**	1.29	3.39	1.55	6.30

Sensitivity Assessment										
Reach	**HEP**	**Observed Data (Out of 10)**	**Catchment Data (Out of 5)**	**Ungauged Flow Estimates (Out of 5)**	**Comments**	**Summed Sensitivity Score**	**Q10 Sensitivity Factor [2]**	**Q10 Sensitivity Flow [2]**	**Q100 Sensitivity Factor [3]**	**Q100 Sensitivity Flow**
Ballyard	**LEE_90**	**8**	**3**	**3**	Ungauged and high uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale.	**14**	1.21	8.24	1.45	15.32
Ballyvelly	**LEE_93**	**8**	**2**	**3**	Ungauged and high uncertainty in pivotal site. Moderate uncertainty in catchment data and FSU methods at small catchment scale.	**13**	1.21	24.01	1.45	44.61
River Lee	**LEE_94**	**5**	**2**	**3**	Applicability of gauge data drops moving downstream. Catchment well defined under FSU.	**10**	1.21	93.86	1.33	161.04
Undefined Trib 4	**LEE_99**	**8**	**5**	**4**	Ungauged and high uncertainty in pivotal site. High uncertainty in catchment data and FSU methods at small catchment scale.	**17**	1.29	2.11	1.55	3.92
Kerries East	**LEE_105**	**8**	**4**	**4**	Ungauged and high uncertainty in pivotal site. High to moderately high uncertainty in catchment data and FSU methods at small catchment scale.	**16**	1.29	1.07	1.55	2.02
Lohercannon	**LEE_111**	**8**	**5**	**4**	Ungauged and high uncertainty in pivotal site. High uncertainty in catchment data and FSU methods at small catchment scale.	**17**	1.29	1.65	1.55	3.11
Pinure	**LEE_125**	**8**	**3**	**4**	Ungauged and high uncertainty in pivotal site. Moderate to low uncertainty in catchment data and FSU methods at small catchment scale.	**15**	1.29	12.98	1.55	25.15

9 DESIGN RAINFALL

It may be necessary to drive portions of the hydraulic model with pluvial inputs. This will be the case where there is evidence of significant pluvial flooding from direct runoff or where the performance of the urban drainage sub-catchments needs to be modelled to determine the level of existing or residual flood risk. There may be no apparent risk from the drainage system presently, but this may need to be tested in the post scheme model to assess the level of residual flood risk and to design secondary mitigation (e.g., flap valves, pumps).

Where this is the case design rainfall inputs must be derived to use as boundary conditions within rainfall-driven hydraulic models/calculations. These are extracted from the FSU DDF model. Rainfall sums for a range of durations are extracted from the DDF model for the relevant sub-catchment or point which best represents the drainage sub-catchment in question.

9.1 Design Rainfall Sums

Design rainfall sums have been extracted from the FSU DDF model[40] for the suite of return periods. Rainfall sums have been extracted as an average across the Tralee FRS Study Area. Each point represents a 2 km grid, therefore the total area covered is 36 km^2. FSU design rainfall sums are presented in **Table 9.1**.

Table 9.1: FSU Rainfall Sums, Tralee FRS Study Area

Event Duration Hours	AEP (%) / Return Period (Year)							
	50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1%* (1000)
0.25	7.43	9.80	11.53	13.41	16.27	18.79	21.69	42.18
0.5	9.81	12.82	15.01	17.36	20.91	24.01	27.57	52.99
1	12.95	16.79	19.56	22.50	26.92	30.75	35.12	66.75
2	17.11	22.00	25.49	29.21	34.71	39.47	44.84	84.31
3	20.15	25.79	29.79	34.04	40.31	45.72	51.80	96.77
4	22.64	28.87	33.29	37.97	44.84	50.76	57.41	106.77
6	26.67	33.86	38.92	44.28	52.13	58.86	66.39	122.72
9	31.42	39.71	45.54	51.67	60.65	68.30	76.85	141.19
12	35.31	44.49	50.93	57.68	67.55	75.95	85.33	156.09
18	41.64	52.22	59.62	67.36	78.66	88.25	98.92	179.90
24	47.00	58.75	66.96	75.53	88.01	98.59	110.35	199.81

*The 0.1% AEP frequency event has had rainfall depths calculated by extrapolation from the depth values in the FSU DDF grid and the estimates provided for the various rainfall frequencies (50% - 0.5% AEP). The extrapolation was carried out using the least squares method to find the best fit of the data points between AEP.

[40] FSU Technical Research Report Volume II Rainfall Frequency

9.2 Design Rainfall Profiles

Design rainfall sums are disaggregated over the FSR summer and winter rainfall profiles[41] depending on an assessment of the likely seasonality of a drainage system flood event. Both summer and winter profiles will be tested due to the variation in flooding reports in terms of seasonality at Tralee. The FSR summer and winter rainfall profiles across which the rainfall sums are applied are shown in **Figure 9.1**.

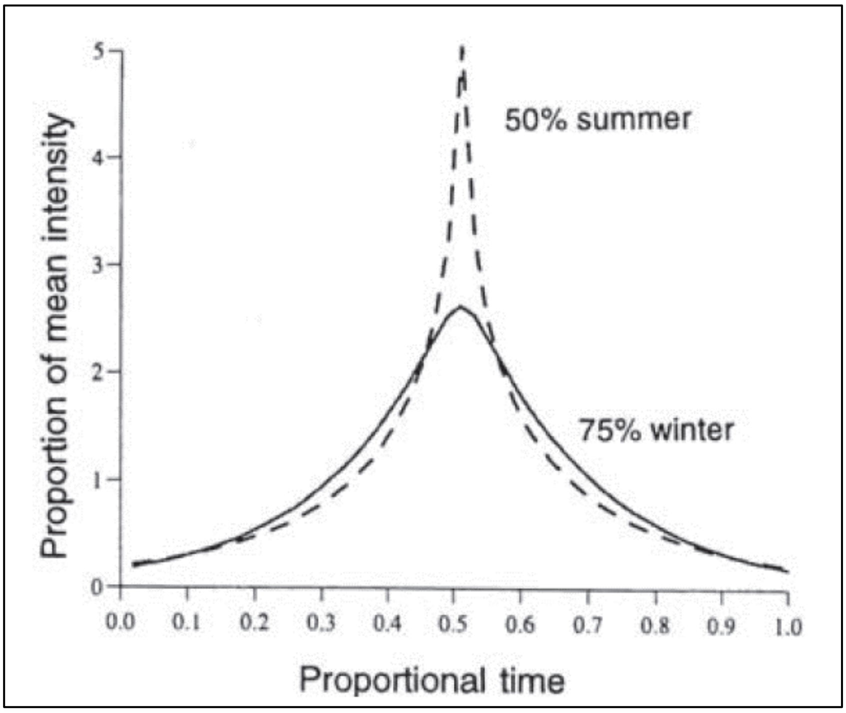


Figure 9.1: FSR Rainfall Profiles, extracted from FEH Volume 2, Chapter 4 (1999)

Where the pluvial analysis is used to test behind defence drainage, the critical event duration and season may need to be considered against the fluvial critical storm duration and seasonality where the critical mechanism is locked out of the drainage system due to high levels in the receiving river/watercourse. The range of event durations will be tested within the model to determine the critical event in relation to the sub-catchment / drainage system and the full range of current and future scenario design events run for that duration.

[41] See Flood Estimation Handbook Volume 2, Chapter 4 (Institute of Hydrology, 1999)

10 FUTURE SCENARIOS

There are a number of potential future changes which could impact the effectiveness of any flood management measures proposed. It is prudent that the potential impacts are identified and quantified such that effective planning and design for adaptation can be accommodated. The adjustments applied to the design flow estimates are outlined in 'Flood Risk Management, Climate Change Sectoral Adaptation Plan' (OPW, 2019)[42] and are reproduced in **Table 10.1**.

Table 10.1: Allowances for Future Scenarios for 100-Year Time Horizon Considering Urbanisation Growth Rates Estimated as Part of the Tralee FRS

Parameter	Mid-Range Future Scenario (MRFS)	High-End Future Scenario (HEFS)
Extreme Rainfall Depths	+ 20%	+ 30%
Peak Flood Flows	+ 20%	+ 30%
Mean Sea Level Rise	+ 500 mm	+ 1000 mm
Urbanisation	0.39% per annum	0.46% per annum
Forestation	Reduce time to peak by 1/6	Reduce time to peak by 1/3 + 10% on SPR (Standard Percentage Runoff)

10.1 Urbanisation

Rates of urbanisation were assessed considering CORINE Land Cover data from 2000 to 2018[43] as well as population growth from the Central Statistics Office[44] for the south-west region of Ireland.

10.1.1 CORINE Land-Use Change

The CORINE Land-Use Change datasets for 2000, 2006, 2012, and 2018 were analysed and trend in urbanisation in the Tralee FRS scheme area was estimated. A linear relationship was calculated in urban extent change between 2000 and 2018 and extrapolated to 2100. A compound annual growth rate urban extent factor was estimated to be 0.22% per year from the extrapolated data. This growth rate factor for urban extent in the Tralee FRS Scheme Area was determined by finding the rate of change in area between the relevant urban land use codes (**Table 10.2**).

Table 10.2: CORINE Land Use Cover and Corresponding Codes for Urban Extent Land Cover Found in the Tralee Scheme Area

CORINE Land Use Cover	CORINE Land Use Cover Codes
Continuous Urban Fabric	111
Discontinuous Urban Fabric	112
Industrial or Commercial Units	121

[42] Accessed at: https://www.gov.ie/en/publication/97984b-climate-change-and-sectoral-adaptation-plan/

[43] https://land.copernicus.eu/pan-european/corine-land-cover

[44] https://www.cso.ie/en/releasesandpublications/ep/p-rpp/regionalpopulationprojections2017-2036/

10.1.2 Population Growth

Population growth data for the south-west region of Ireland (where Tralee FRS is located as defined by the CSO) was obtained from the CSO the years 2011 to 2016, inclusive. Similar to the analysis carried out on the CORINE data, a linear trend in population growth for the region was estimated which was then used to extrapolate to the year 2100. A compound annual growth rate for population growth was estimated to be 0.54% per year for the present day.

The CSO have recently released regional projections of population growth to the year 2036 under three different scenarios: M1F2, M2F2, and M3F2. More information on what factors each scenario considers can be found here on the CSO website[45]. For each scenario, it is further split into two projections for both inflow and outflow models from the main population centre of Ireland, Dublin. The inflow model into Dublin results in a decreasing trend in population for the mid-west region of Ireland. In this analysis the outflow model was only considered anticipating an increase in population in the mid-west region.

M1F2 can be considered a high-end projection of population while M2F2 and M3F2 can be considered medium- and low-end projections, respectively. For this analysis M1F2 was considered for the HEFS and M2F2 for the MRFS Future Scenarios. A linear trend in population growth for the region was estimated between 2016 and 2036 which was then used to interpolate between 2016 and 2036 (**Figure 10.1**) and extrapolate to the year 2100.

From the projected population growth to 2100 for the mid-west region of Ireland, an annual compound growth factor was calculated for both the MRFS and HEFS. Compound annual growth rate factors of 0.69% and 0.56% for the HEFS and MRFS, respectively, were obtained.

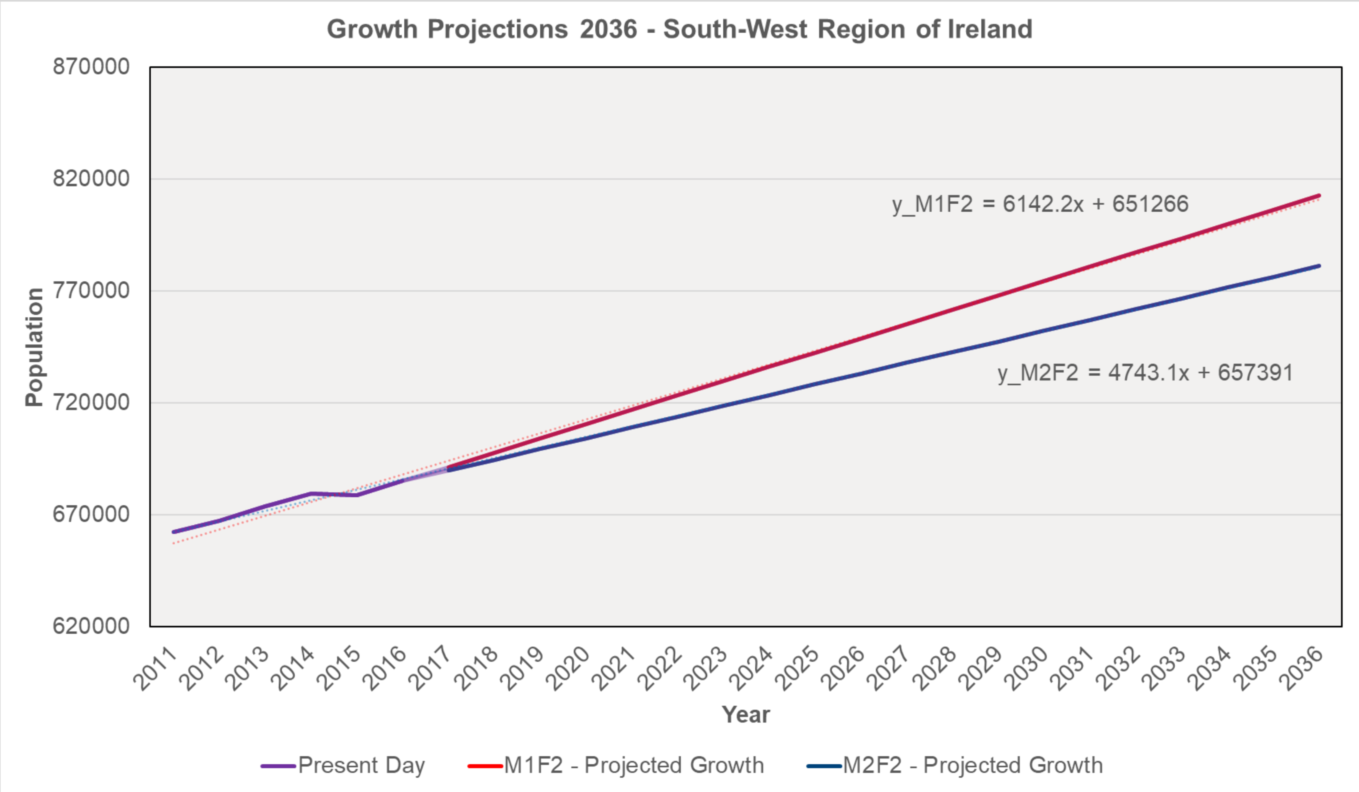


Figure 10.1: M1F2 and M2F2 (HEFS and MRFS, respectively) Population Growth Scenarios for South-West Region of Ireland (2017-2036)

[45] https://www.cso.ie/en/releasesandpublications/ep/p-rpp/regionalpopulationprojections2017-2036/

10.1.3 Final Urbanisation Factors

Assuming population growth in Ireland is directly proportional to urbanisation, an average was taken between the compound annual growth rate for urban extent and population growth. This resulted in urbanisation factors for the MRFS and HEFS of 0.39% and 0.46%, respectively (**Table 10.1**).

11 CONCLUSIONS

Tralee Town and scheme area covered as part of the Tralee FRS lies at the downstream extent of the River Lee where it discharges into Tralee Bay. The River Lee drains the Stack and Slieve Mish Mountains to the north and south of the catchment area, respectively. Multiple tributaries of the River Lee are also contained within the Tralee FRS scheme area, including the Big River, Balloonagh River, Lissardboola and Ballybeggan River. Flooding can be driven by extreme coastal events and high fluvial events acting in isolation or in tandem to cause significant damages to various parts of the scheme area.

Hydrometric gauging stations are available on the River Lee and the major tributaries in the scheme area. Many of the gauging stations located on the major tributaries around Tralee Town were recently installed between 2019 and 2021 following recommendation from the Shannon CFRAM Study. The recently installed stations offered vital data when carrying out the hydrological analysis contained in this report indicating the behaviour of fluvial behaviour in these catchments. However, short record periods at recently installed gauging stations also introduced levels of uncertainty into the analysis. Ballymullen gauging station on the River Lee at the downstream extent provided a long-term hydrometric record for the main source of fluvial risk to Tralee Town. Concerns exist surrounding the potential tidal influence captured at the station due to its proximity to the coastal boundary, however, this tidal fingerprint was determined negligible at high flood flows on the River Lee.

Rating reviews were conducted at six gauging stations to help increase the confidence in the design flow estimation across the Tralee FRS study area. The key outcomes of the rating reviews were:

- An initial rating was developed for the newly installed gauging stations at Lisloose (23044), Ballyseedy (23045), Spring Water Lane (23046), Oakpark Road (23047), and Manor West (23049) gauging stations which allowed an initial estimation of a Q_{med} flood flow on the respective watercourses. This provided some validation of the design flow estimate of Q_{med}.
  - Due to the short record period and thus low certainty in the Q_{med} estimate it was only used to validate the hydrological analysis.
  - Q_{med} estimates derived from FSU methodologies were used to form the basis of design flow estimation on the major tributaries in the Tralee FRS study area.
- The River Lee at Ballymullen (23022) gauging station was subject to the development of an updated rating curve that deviated from the current OPW curve, particularly at the high flows. This has led to a 44.9% increase in Q_{med} as well as an increase in the recorded peak flows across the AMAX record.
- The increase of Q_{med} on the River Lee at Ballymullen gauging station formed the basis of the design flow estimation on the River Lee including Q_{med} estimation and pivotal site adjustment.

Design flow estimation was conducted for the Tralee FRS study area including Q_{med} estimation, pivotal site adjustment, growth curve development, hydrograph width analysis and development for model input. The downstream boundary was derived from an updated analysis of the ICPSS (2018) Report. The hydrological analysis relies on the statistical methods for flood flow estimation set out in the FSU. Namely the standard FSU 7 variable equation for ungauged catchment but also the FSU 5 variable equation optimised for small Irish catchments. The ungauged estimates are anchored to the rating reviewed gauge data as recommended within the FSU. For catchments less than 25 km^2, two extra methods were used to estimate Q_{med}, namely the FEH (UK hydrological equations) and Rational Method (rainfall run-off estimation). RPS' preference is to retain the FSU estimations where suitable, with small, urbanised catchments being directly driven by rainfall in the hydraulic model where applicable.

Joint probability analysis was also conducted to test a number of dependency relationships; fluvial-fluvial, fluvial-coastal, and fluvial-pluvial. It was found that there was a high dependency between fluvial-fluvial events with Ballymullen gauging station being used to represent the River Lee and Tralee Clonalour on the Big River, an inactive station, being used to represent all major tributaries on the River Lee. This high dependency is expected due to the influence that water levels in the River Lee can have on the surrounding downstream reaches of its tributaries. A low to minimal dependency was found between fluvial-coastal events albeit many Peaks-Over-Threshold events from both mechanisms were found to coincide within a two-day window. Fenit Pier and Blennerville were both explored as potential coastal proxy stations. Blennerville coastal gauging station was anticipated to be too close to Ballymullen hydrometric gauging station thus a false dependency may have been determined, however, due to the physical channel and similar correlation factor found at Fenit Pier, it was determined suitable to use Blennerville as the proxy.

Finally, future change in catchment and subsequent flood flow behaviour were accounted for by applying factors to final design flow estimates. Factors included climate change allowances for mean sea-level rise, extreme rainfall depths, flood flow increase, urbanisation, and forestation.

There is a good level of certainty in the flood flow estimates for the River Lee and its major tributary reaches owing to their size and gauged nature, however further data collection on tributaries will offer significantly more certainty in the future.

11.1 Recommendations for Data Collection

Data collection remains vital in understanding flood frequency conditions and estimating design flow estimates at any site. The presence of gauged data lessens the margin of uncertainty that is associated within ungauged estimates.

The following recommendations for data collection should be considered:

- The Ballymullen gauging station on the River Lee should be moved upstream to remove the tidal signal at low flows.

The presence of gauging stations allows for the estimation of flows on all major watercourses in the Tralee study area. To fully understand the flood behaviour on the watercourses an increased data collection programme of spot gauging measurements should be the main focus of hydrometric activities in the coming years at Tralee. The spot gaugings should continue to be captured across the range of flow events, with a particular focus on the high flows, if possible, to help validate the preliminary rating developed as part of this project.