Carrickmines Shanganagh River
Flood Relief Scheme

Hydrology Report

Final Report

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Purpose

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1 Introduction

JBA Consulting and JBB Barry were commissioned by Dun Laoghaire Rathdown County Council (DLRCC) and the Office of Public Works (OPW) to develop a Flood Relief Scheme (FRS) for the Carrickmines – Shanganagh catchment in south County Dublin. Figure 1-2 shows the catchment location and watercourses considered in the scheme.

As part of the FRS development a hydraulic model with corresponding hydrological inputs is to be developed. This Hydrology Report aims to provide an overview of the available hydrological data for the area and the methods used for deriving flows to be used in the FRS hydraulic model and scheme design.

1.1.1 Catchment overview

The study area is topographically variable with the upper catchment areas located within the Dublin Mountains to the west with the topography flattening moving east and downstream towards the outflow at the Irish sea (Figure 1-1). The land use across the area also varies, a large portion is heavily urbanised especially at the downstream extent, in contrast the upland areas are dominated by rural land use, this division is clearly seen in the background image in Figure 1-2. The M50 motorway runs across the centre of the catchment area creating a hydraulic barrier which potentially impacts flow behaviour downstream of the motorway.

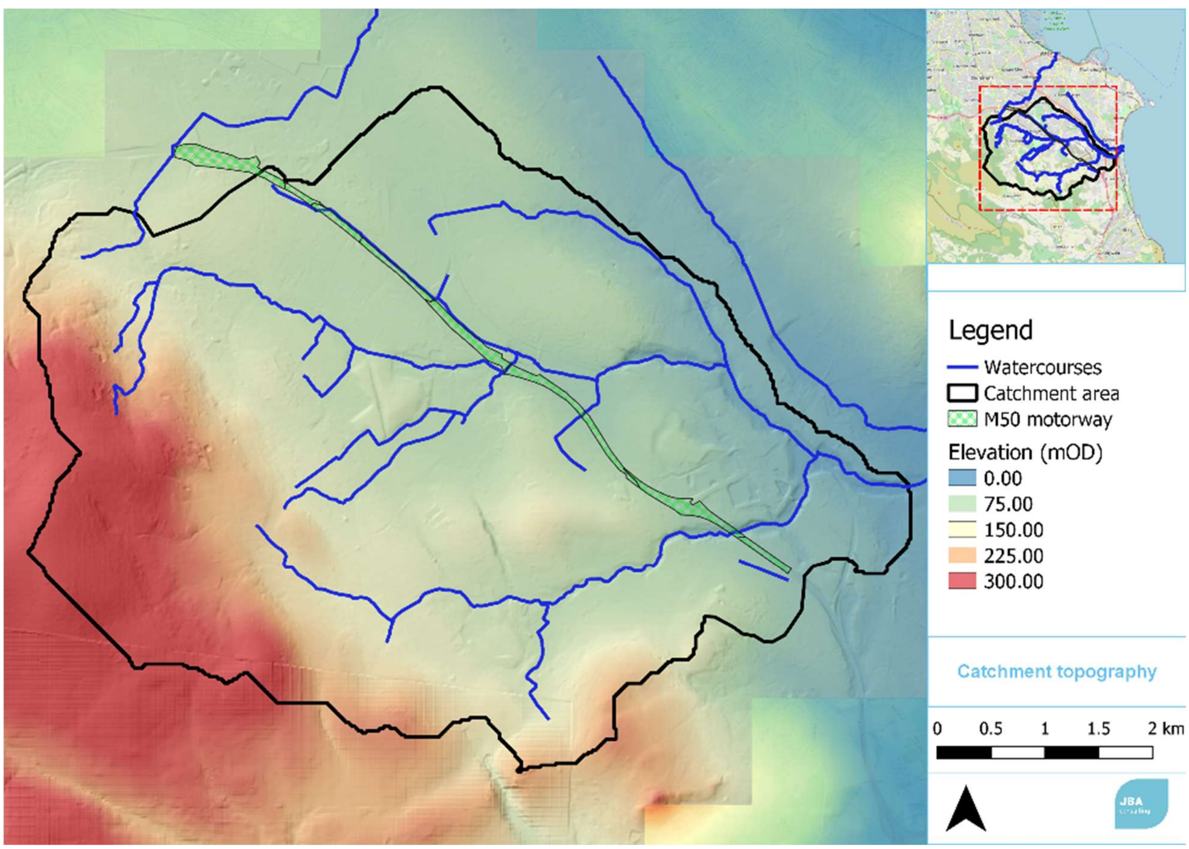


Figure 1-1: Catchment topography (base maps: ESRi Satellite and OSM standard)

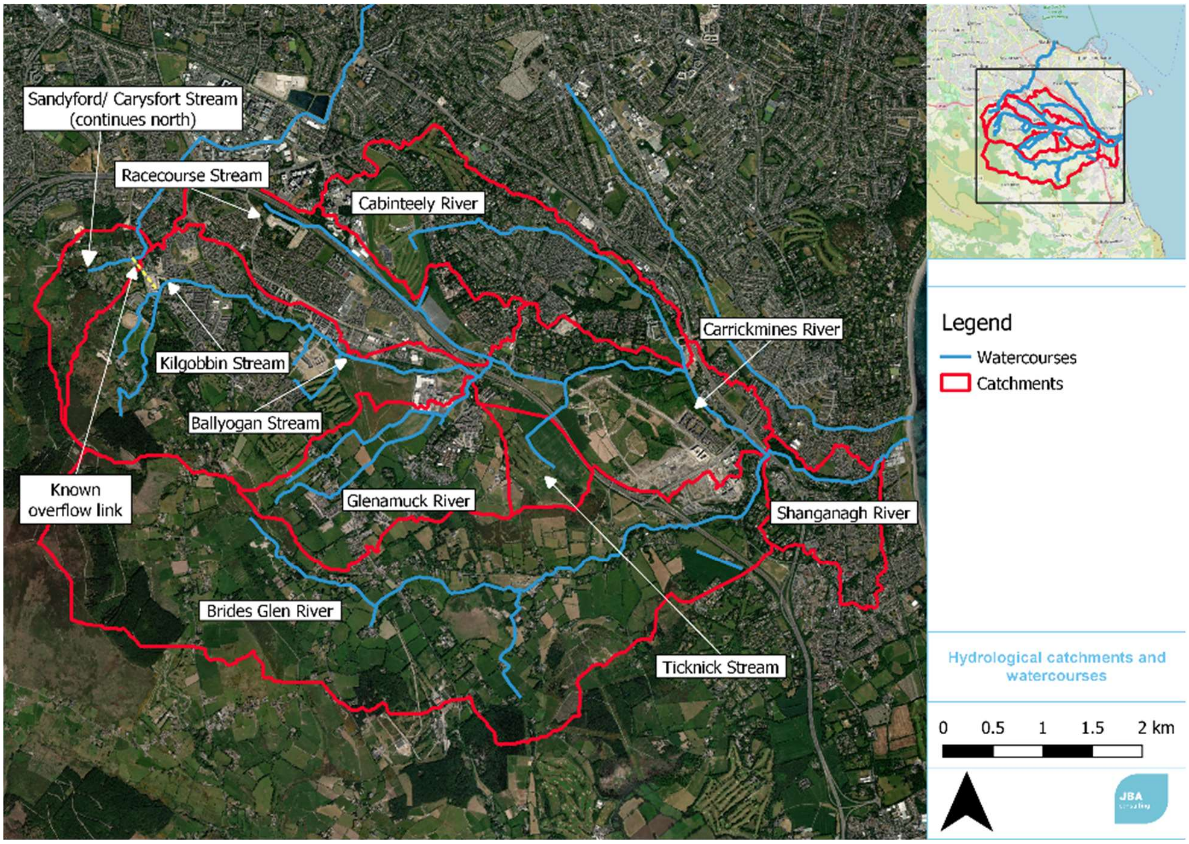


Figure 1-2: Hydrological catchments and watercourses (base maps: Bing Satellite and OSM Standard)

2 Review of available data

This section provides a summary of the available hydrological data for the study including flood history, hydrometric and meteorological gauges. A detailed review of the hydrology from the Eastern Catchment Flood Risk Assessment and Management study (ECFRAM), the most recent in-depth study for the area is reviewed separately in Section 3.

2.1 Hydrometric data

There is data for four gauging stations within the study area, three are active at time of writing and one is discontinued. Table 2-1 provides a summary of the gauge information and their locations highlighted in Figure 2-1. As part of this study a short-term flow monitoring period took place to allow greater understanding of the flow contributions and timings of smaller tributaries within the catchment. The findings of this monitoring period are discussed in Section 2.6. The Commons Road gauge is reviewed in detail in Section 2.3 while the comment on the other three gauges is made in this section.

Table 2-1: Hydrometric gauge summary

Gauge Name	Number	Operator	Watercourse	Type	FSU ranking	Data record	Notes
Commons Road	10021	EPA	Loughlinstown River	Level recorder	A1 (pre-2005)	1980-present	Significant catchment changes in 2005
Carrickmines	10022	EPA	Loughlinstown River	Level recorder	A1 (pre-2005)	1980–1999 2001-2005	Removed due to M50 motorway construction
Cherrywood	10048	OPW	Loughlinstown River (South)	Level recorder	NA	2019 - present	Recently installed
Brides Glen River	10050	OPW	St Brides stream	Level recorder	NA	2020-present	Recently installed

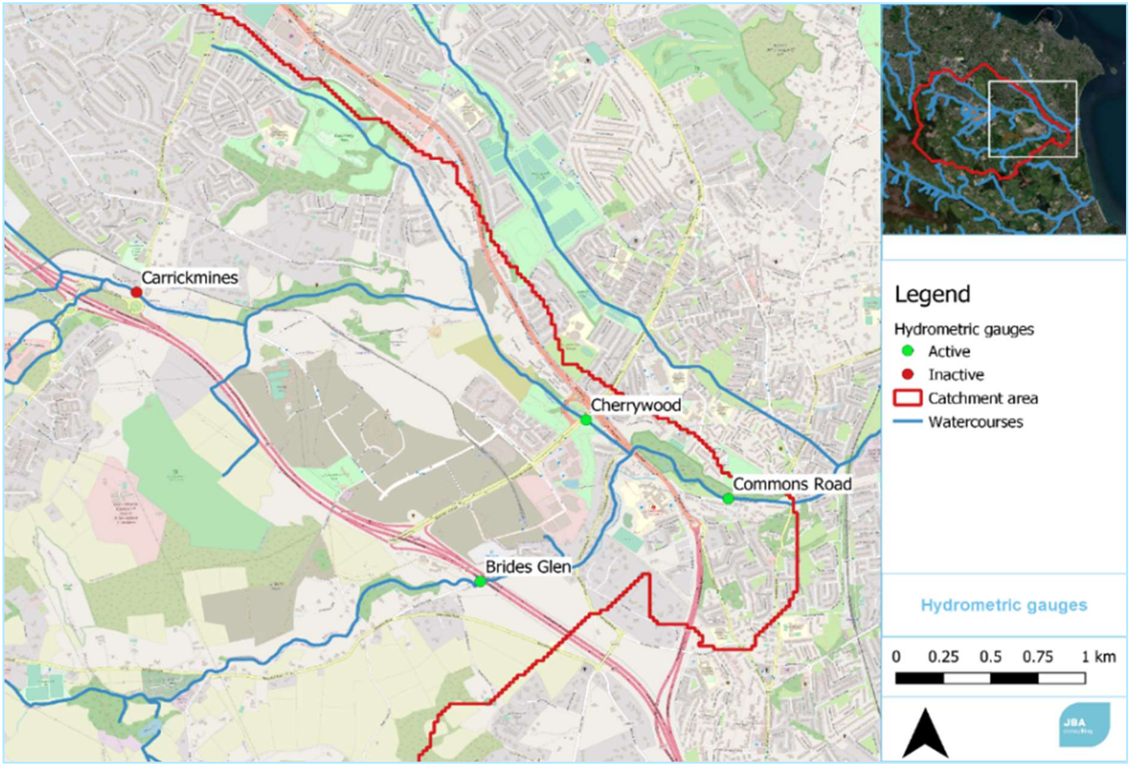


Figure 2-1: Hydrometric gauges (base maps: OSM Standard, ESRi Satellite)

2.1.1 Gauge 10022 Carrickmines (discontinued)

The Carrickmines gauge was active from 1980 until 2005 when it was removed during the construction of the M50 motorway and was not reinstalled. Figure 2-2 shows the gauge flow AMAX record for the active period. There is an upward trend in the flows overtime and a portion of the gauge record missing prior to the construction of the M50 motorway (1999 - 2001). The gauge received an FSU A1 ranking indicating the data recorded is of good quality. While the gauge is not active it may possibly be of use as a pivotal gauge for upstream catchments in the study area which would not be impacted by the development of the M50 further downstream in the catchment.

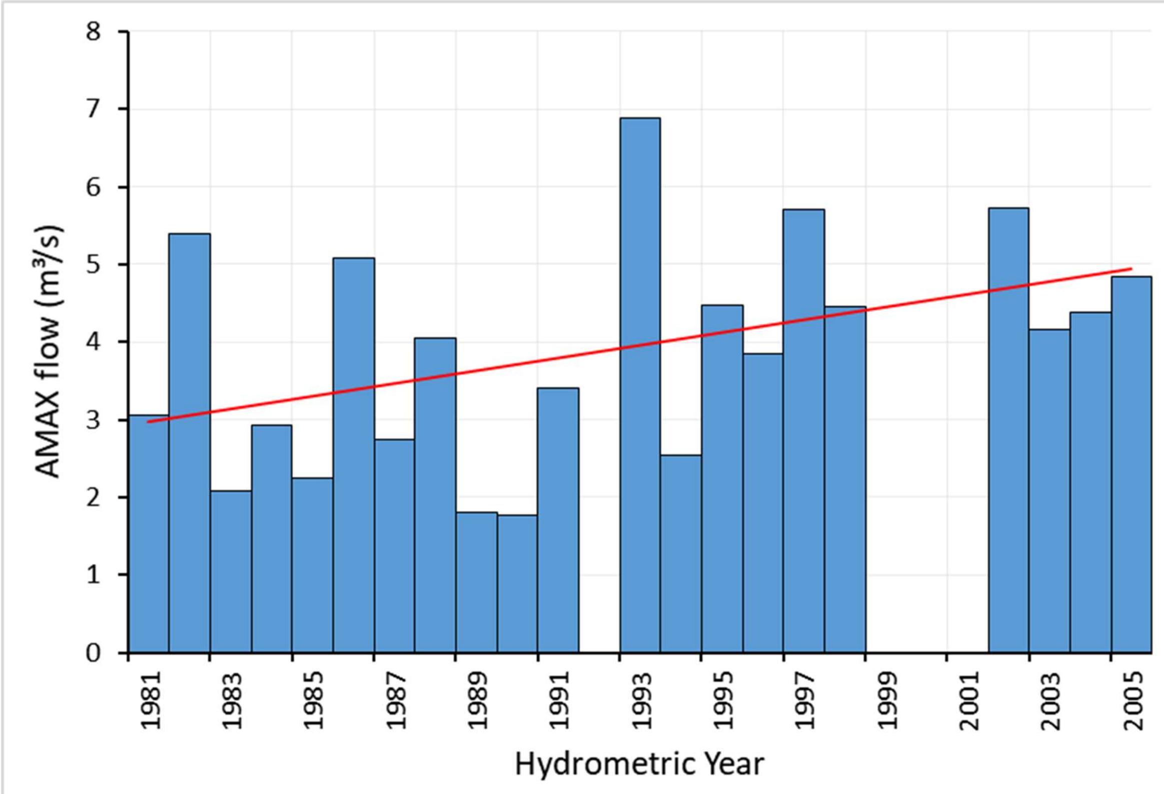


Figure 2-2: Flow AMAX series for gauge 10022 with trend line

2.1.2 Gauge 10048 Cherrywood

The Cherrywood gauge is located upstream of the junction between the Shanganagh River and the Brides Glen River. The level gauge has only been active since 2019 and therefore has a short data record, no check flows, and no established flow level relationship. This limits its use within the FRS for estimation of design flows. However, should another flood event occur the gauge will provide useful data in relation to the response of the watercourse.

A site visit was carried out in early 2021 and while examining the gauge location it was noted that there was no concrete control installed at the gauge location. Further to this evidence of bank erosion was observed on the channel sides and around the concrete block holding the gauge in place (refer to Figure 2-3). Continued erosion of the banks may impact the gauge reliability and stability in the channel. It is recommended that the channel condition be assessed in relation to erosion and measures be put in place around the gauge if necessary.

Figure 2-3: Gauge 10048 facing downstream and showing eroded left and right banks.

2.1.3 Gauge 10050 Brides Glen

The Brides Glen gauge was installed in December 2020 and therefore has the shortest available record water level and no derived flow level relationship. The gauge will be of benefit for future work within the catchment.

2.2 Review of descriptor data and river network

2.2.1 Catchment descriptors

A total of 19 Hydrological Estimation Points (HEPs) were derived for this study which are shown in Figure 2-4. A review of the key catchment characteristics was carried out for each HEP and corresponding catchment area. Characteristics reviewed include SAAR, URBEXT and S1085. The updated catchment descriptors were then used in the estimation of flows by traditional methods at each HEP for comparison against the routing model flows (refer to Section 4.3). Appendix A provides the updated catchment descriptors for all HEPs.

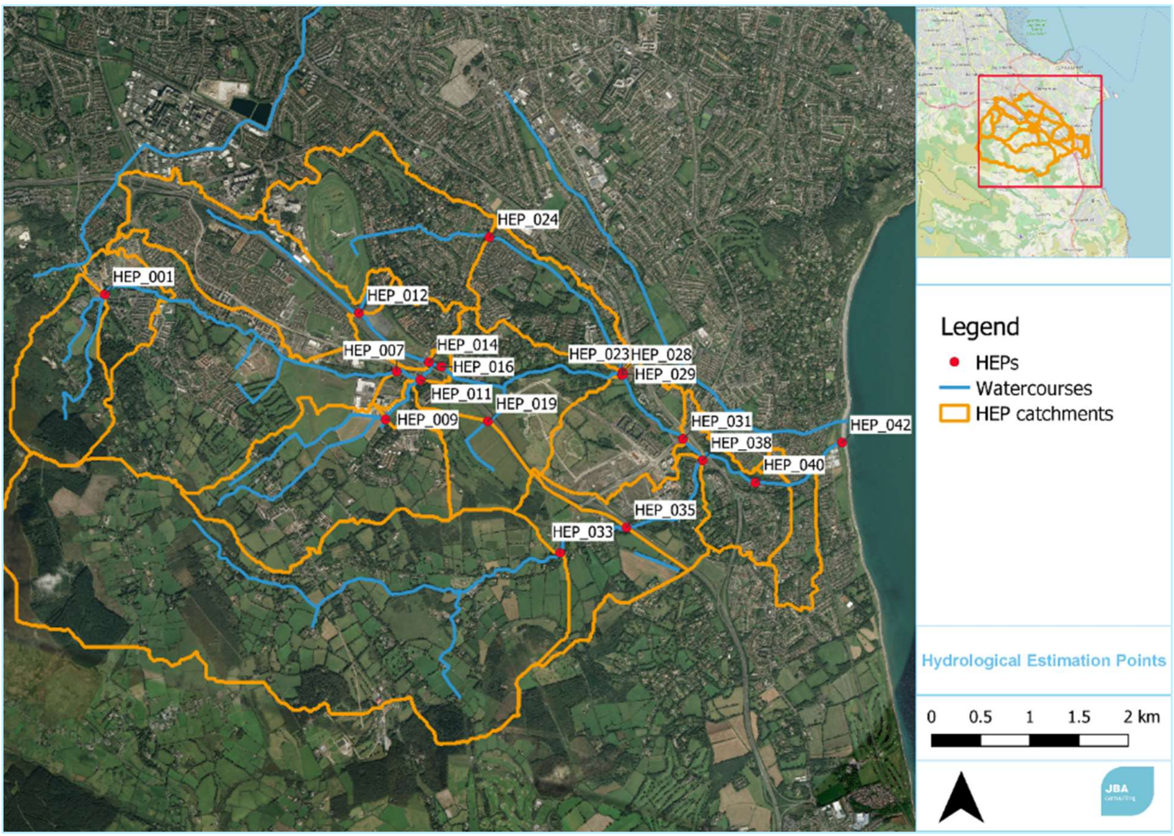


Figure 2-4: Hydrological Estimation Points (base maps: Bing Satellite and OSM Standard)

2.2.2 River network review

The EPA River network GIS layer was reviewed for the study area and compared with satellite imagery and background mapping (ESRi and Bing satellite imagery and OSM standard base map data). Review of the data found that there were locations within the catchment where the river network line was not following the correct line of the watercourses compared to the background data. Figure 2-5 shows the previous and updated EPA river network line with the corrected river alignment.

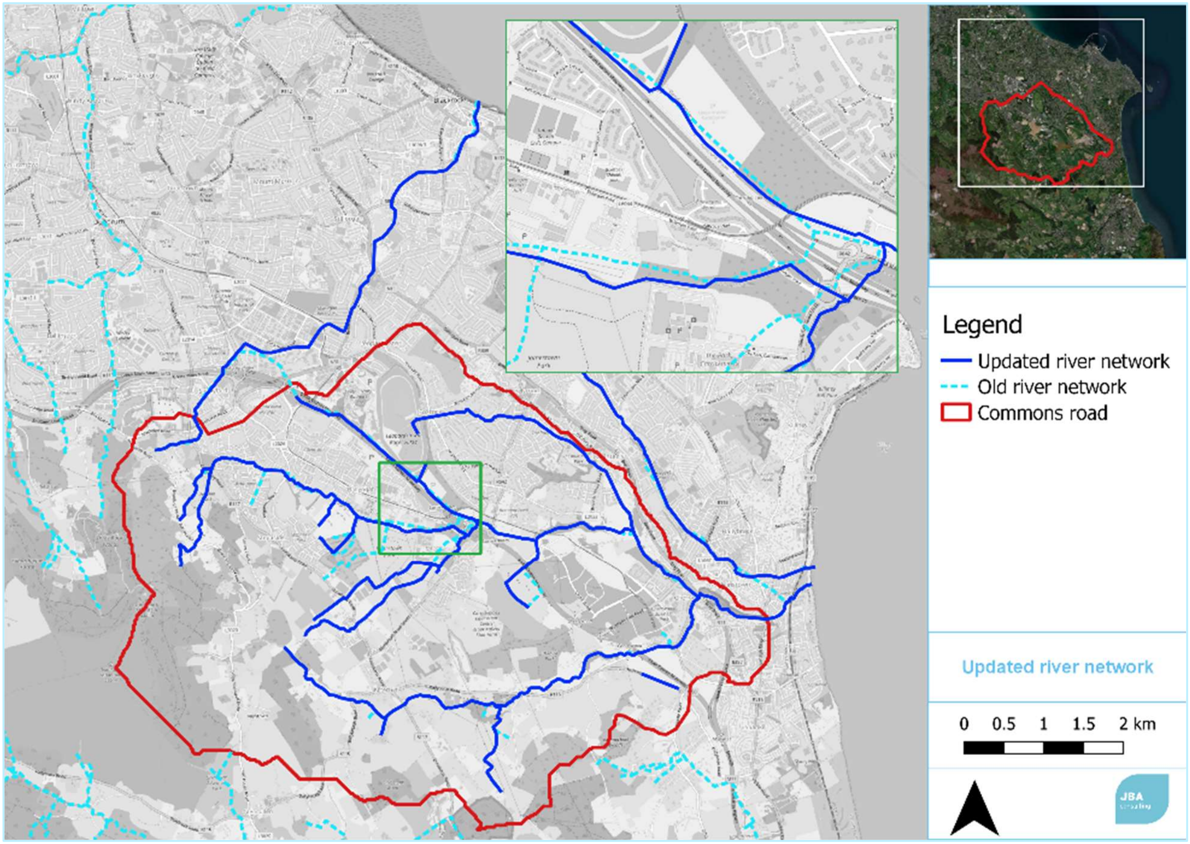


Figure 2-5: Updated river network (base maps: OSM Standard and ESRi Satellite)

2.3 Commons Road gauge (10021)

Commons Road Gauge (10021) is the only active gauge within the catchment which has been active for any length of time. It is considered the critical gauge for the catchment. Table 2-2 summarises the key events in the gauge's history.

Table 2-2: Summary of Gauge 10021 history

Date	Event
1980	Gauge Installation
21st January 1980	Start of the recording. Station Rating Quality Classification - A1 (1980-2004)
6th November 1982	Flood event in catchment
26th August 1986	Flood event in catchment
26th May 1993	Largest flood event prior to 2011 – (13.84m3/s). Was ranked 1 before 2011 flood event
1997	Flood event in catchment
27th November 2002	Flood event in catchment
200?-2005	South Eastern Motorway M50 extension was opened on 30th June 2005 (alteration of catchment hydrologically)
2005	Gauge reach significantly altered due to flood defences and embankments (Shanganagh River Management Scheme, 2005 DLRCC)
24th October 2011	Flood event – highest level recorded at gauge
21st March 2013	Flood event in catchment
2014	Gauge and rating curve are reviewed under the Eastern CFRAM study and a new updated rating curve is derived.
2nd August 2014	Flood event in catchment
12th – 14th November 2014	Flood event in catchment
14th March 2018	Flood event within catchment, over-road flooding R116, Brides Glen River, Kilternan

2.3.1 Gauge AMAX record

Table 2-3 examines the variation of AMAX level (and consequently flow) values when different portions of the gauge record are considered. From the table there is a noted difference in median AMAX level pre- and post-2005. This difference suggests changes in the catchment around 2005 altered the flow and catchment behaviour with increased levels and as a result an increased AMAX values being recorded.

As highlighted in Table 2-2 significant changes occurred in the catchment around 2005 including the building/completion of the M50 motorway and the construction of the Commons road flood defences downstream of the gauge location. These hydraulic changes in the system and recorded gauge data were also noted in the ECFRAM. The hydraulic changes were considered the reason for change in gauge record in this study. It was also noted in the ECFRAM study that the post 2005 period appeared to be wetter which may also contribute to the change (refer to Section 2.4 for meteorological analysis). Figure 2-6 which shows the variance in AMAX levels relative to the median value for the whole gauge record also supports this reasoning with a clear increase in AMAX levels starting from 2005 onwards relative to the whole record.

This difference in average value suggests that data pre 2005 no longer reflects the catchment condition and therefore should not be considered when deriving flows from the gauge. While a post 2005 record is shorter it is long enough to establish a Qmed value and provides a more accurate estimate than a 9-year record used in ECFRAM. The Factorial Standard Errors (FSE) for a Qmed estimate for a 16- and 9-year record are 1.097 and 1.153 respectively[1]. Figure 2-7 shows the AMAX series plot for the post 2005 record. There are no gaps in the record and no upward or downward trends observed which provides more confidence in the gauge and record.

Table 2-3: Summary of gauge AMAX data

	Full record (1980 – present)	Pre 2005 record (1980 – 2005)	Post 2005 ECFRAM record (2005 – 2013)	Updated post 2005 record (2005 – present)
Median level (m)	1.18	0.89	1.52	1.58
AMAX Qmed (m^3/s)	8.73	6.33	14.03	14.83

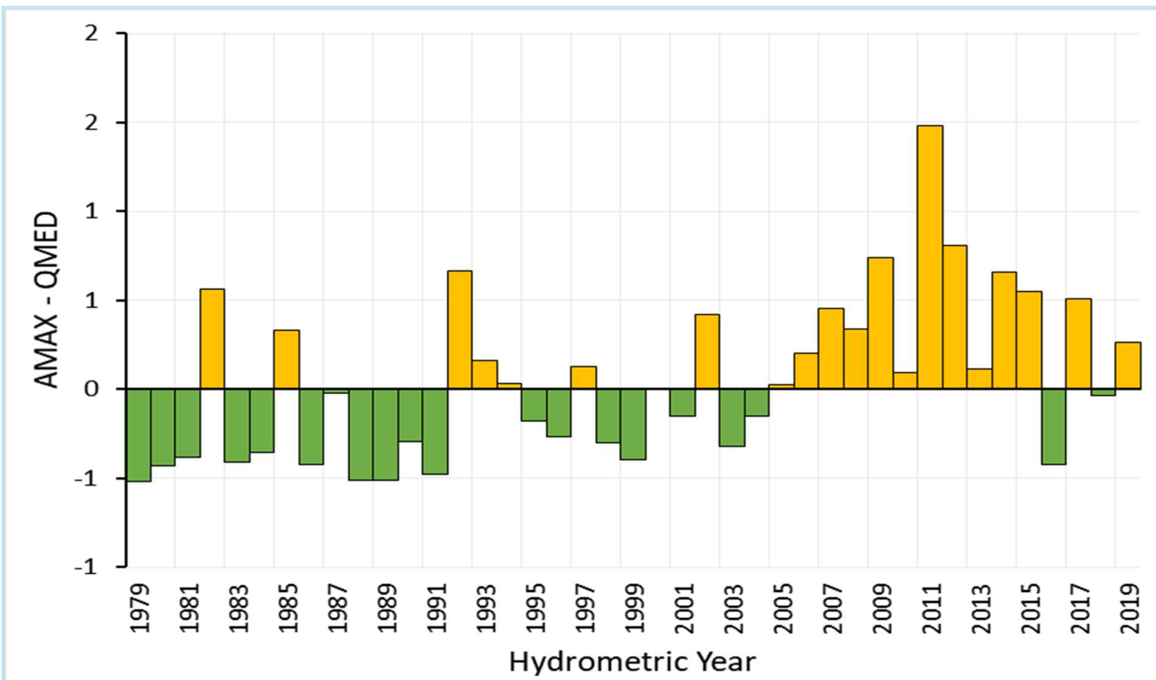


Figure 2-6: AMAX level variation relative to the median value for gauge 10021

[1] Table 12.3 of the Flood Estimation Handbook Volume 3 (pg. 92)

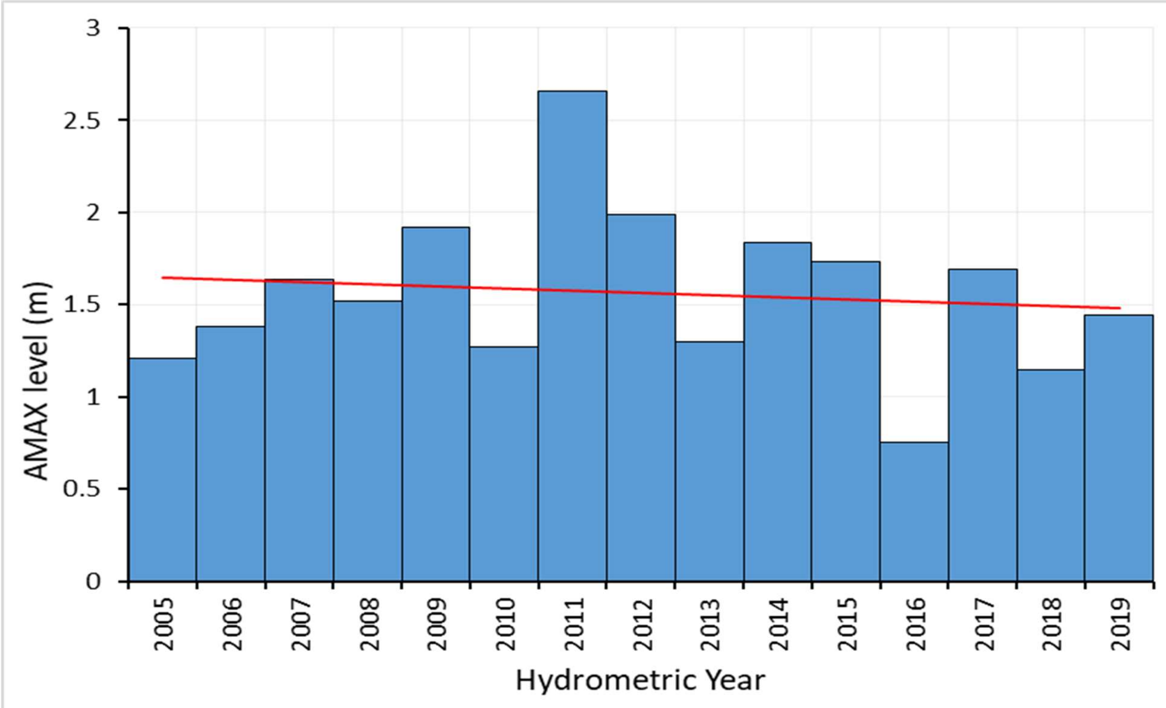


Figure 2-7: Gauge 10021 AMAX record 2005 – present

2.4 Gauge 10021 Commons Road Rating review

2.4.1 Previous rating curves

As highlighted in the gauge history (Table 2-2) the gauge has been active since 1980. Figure 2-8 shows the check flows recorded at the gauge pre- and post-2005 noted as a point of significant change in the catchment. Within the lifetime of the gauge two rating curves have been developed:

- EPA rating curve (1980 – 2005): This initial rating curve was developed over time using check flows measured at the gauge. It is valid up until 2005 when the movement of the gauge and significant alteration in the catchment resulted changes in the stage - discharge relationship.
- ECFRAM rating curve (2005 – present): The second rating curve was derived following a rating review of the gauge carried out under the ECFRAM programme. Check flows recorded post 2005 were used to validate a curve generated using a hydraulic model of the gauge reach. There were no high flow check gaugings recorded in the post 2005 period therefore the upper limbs of the curve are still to be validated by recorded data.

Table 2-4 and Table 2-5 show the two previous rating curve equations. The ECFRAM rating curve was adopted by the EPA and is currently used to report present day flows.

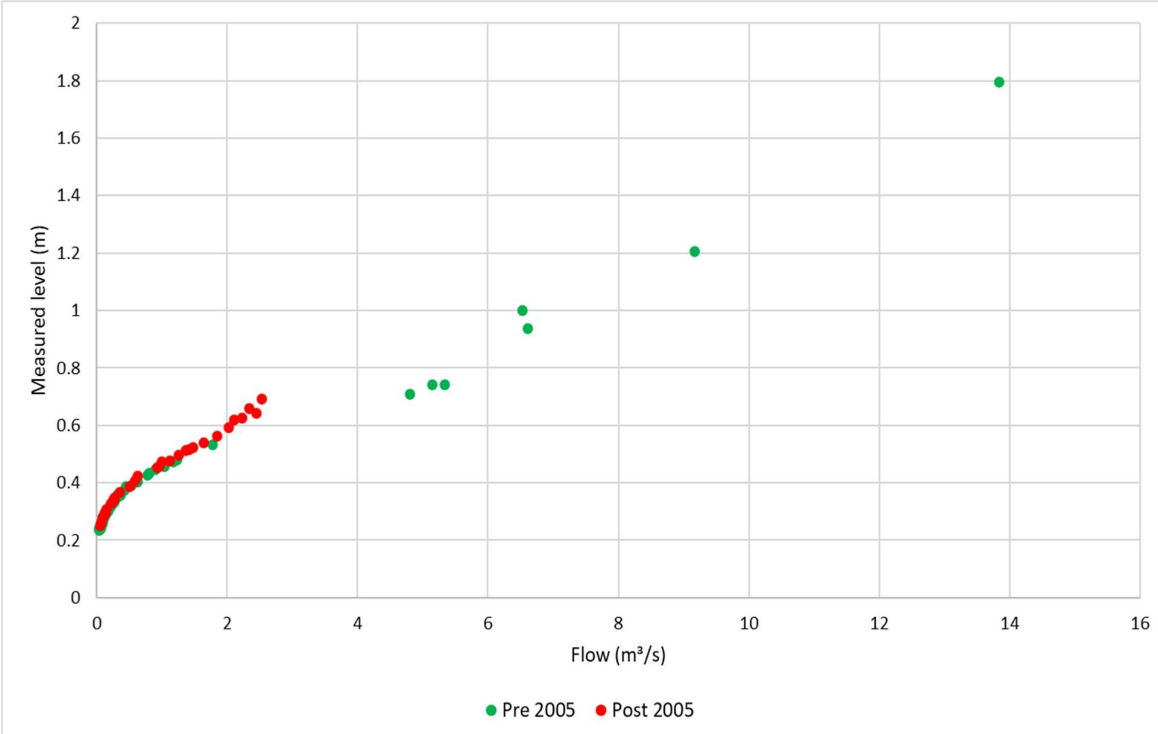


Figure 2-8: Check flows recorded during gauges active record

Table 2-4: EPA rating curve equation (1980 – 2005)

Limb no.	Min Stage (m)	Max Stage (m)	C	a	b
1	0.233	0.304	88.329	-0.100	3.901
2	0.304	0.703	22.221	-0.100	3.032
3	0.703	1.794	8.067	-0.100	1.029

Table 2-5: ECFRAM rating curve equation (2005 – present)

Limb no.	Min Stage (m)	Max Stage (m)	C	a	b
1	0.233	0.304	88.329	-0.100	3.901
2	0.304	0.360	22.221	-0.100	3.032
3	0.360	1.563	9.024	-0.214	1.653
4	1.563	2.073	0.01	3.726	4.376
5	2.073	2.547	5.029×10^{-9}	7.588	9.788

2.4.2 Updated rating curve analysis and testing

As part of this study a rating review of the gauge has been carried out to further test and refine the Q-h relationship at the gauge. Like the ECFRAM study the rating review was carried out using a hydraulic model. Updated survey information and additional check flow gauging were available for use in the analysis. Further to this additional sensitivity analysis was carried out using the model to identify key features or limitations that may influence the Q-h relationship at the gauge. From this testing the following was identified:

- The gauge is not influenced by tidal levels downstream,
- The gauge is not sensitive to variation in time to peak,
- The gauge is not overly sensitive to changes in channel roughness (seasonal vegetation variation,
- The Q-h relationship at the gauge is significantly impacted if the pedestrian bridge downstream is blocked due to the resulting backwater effect.

Refer to Appendix C for full details of the hydraulic model used and the testing carried out.

2.4.3 Examination of gauge Q-h behaviour

Following sensitivity testing within the model analysis of the modelled Q-h relationship at the gauge was carried out. The Q-h relationship at gauge 10021 changes with increased flow and with interaction of hydraulic features and flood plain activation. Figure 2-9 shows the modelled Q-h curve for the gauge with key hydraulic points and location of slope changes marking changes in the Q-h relationship. From the figure most changes in the Q-h relationship occur around points where banks are overtopped or levels exceed structures. The identification of key points aided in the derivation of the rating curve for the gauge.

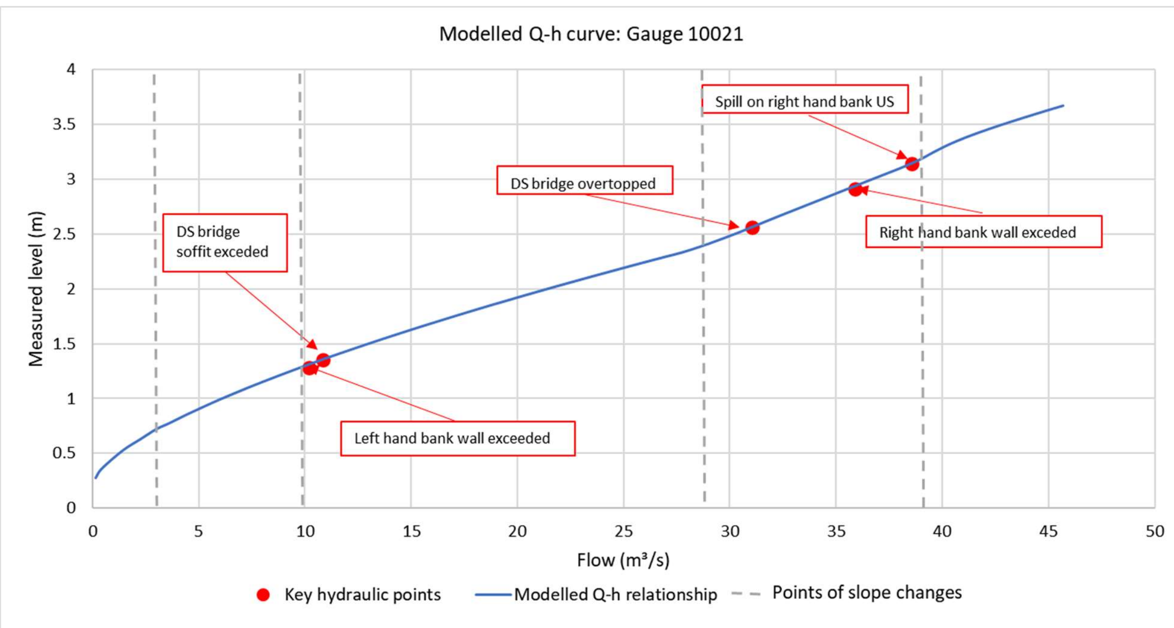


Figure 2-9: Modelled Q-h curve: Gauge 10021

2.4.4 Updated derived rating curve equation for Gauge 10021

The rising limb of the modelled Q-h curve generated by the 0.1% AEP ECFRAM flow was used to derive the updated rating equation. Limb breaks were placed initially at points of hydraulic change identified in Figure 2-9 and then refined to produce a suitable goodness of fit relative to the modelled data and check flow gaugings. The upper limit of reliability of the gauge rating has been set to the height of the right bank wall at the gauge location (2.91mOD). Above this level out of bank spill is occurring with flow bypassing the gauge and therefore not all flow will be accounted for in the gauge readings above this level. Table 2-6 shows the updated rating curve equations for the gauge and the curve is presented in Figure 2-10.

Table 2-6: Updated rating curve equation

Limb no.	Min Stage (m)	Max Stage (m)	C	a	b
1					
2					
3					
4					

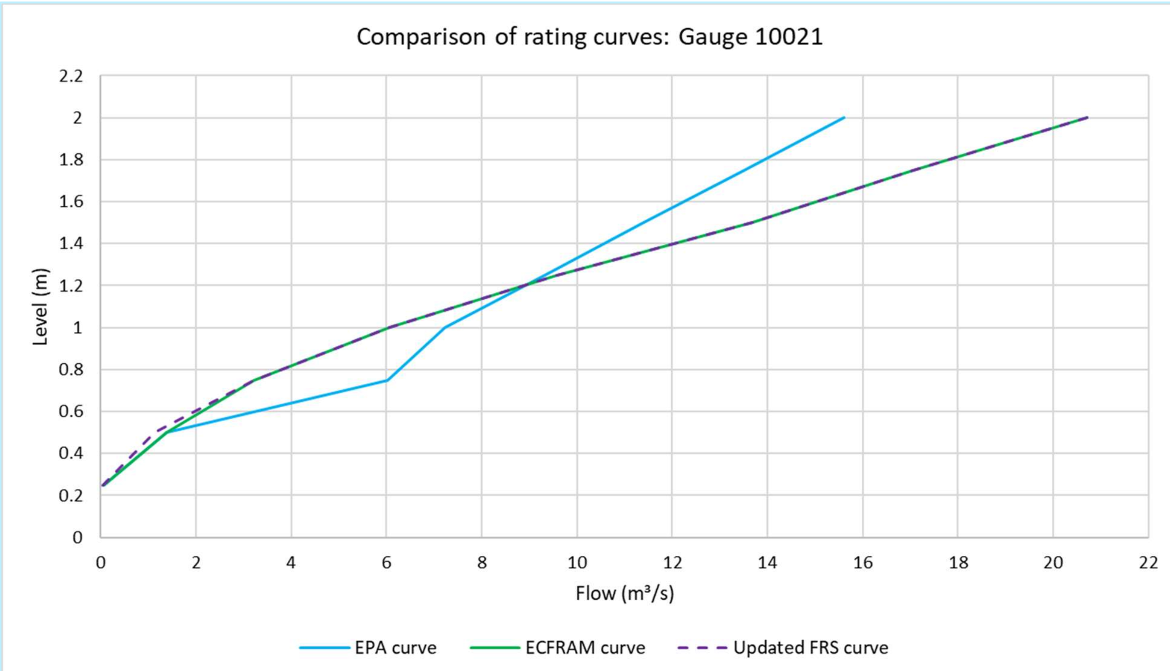


Figure 2-10: Comparison of rating curve: Gauge 10021

2.5 Meteorological Data Analysis

Rainfall records were obtained for all gauges within a distance of 20km of the catchment from Met Éireann. There are three hourly and 3 daily gauges in proximity to the study area. The only gauging station located within the catchment is Balleydmonduff House. Refer to Figure 2-11 for MET Eireann gauge locations and Table 2-7 for a summary of gauge information. It should be noted that the daily total is measured as from 9am to 9am of the next day.

To gain further understanding of the spatial variability of rainfall across the study area additional rain gauges recording at sub daily intervals were placed within and near the catchment. Figure 2-11 shows their locations and the data collected is discussed further in Section 2.6.

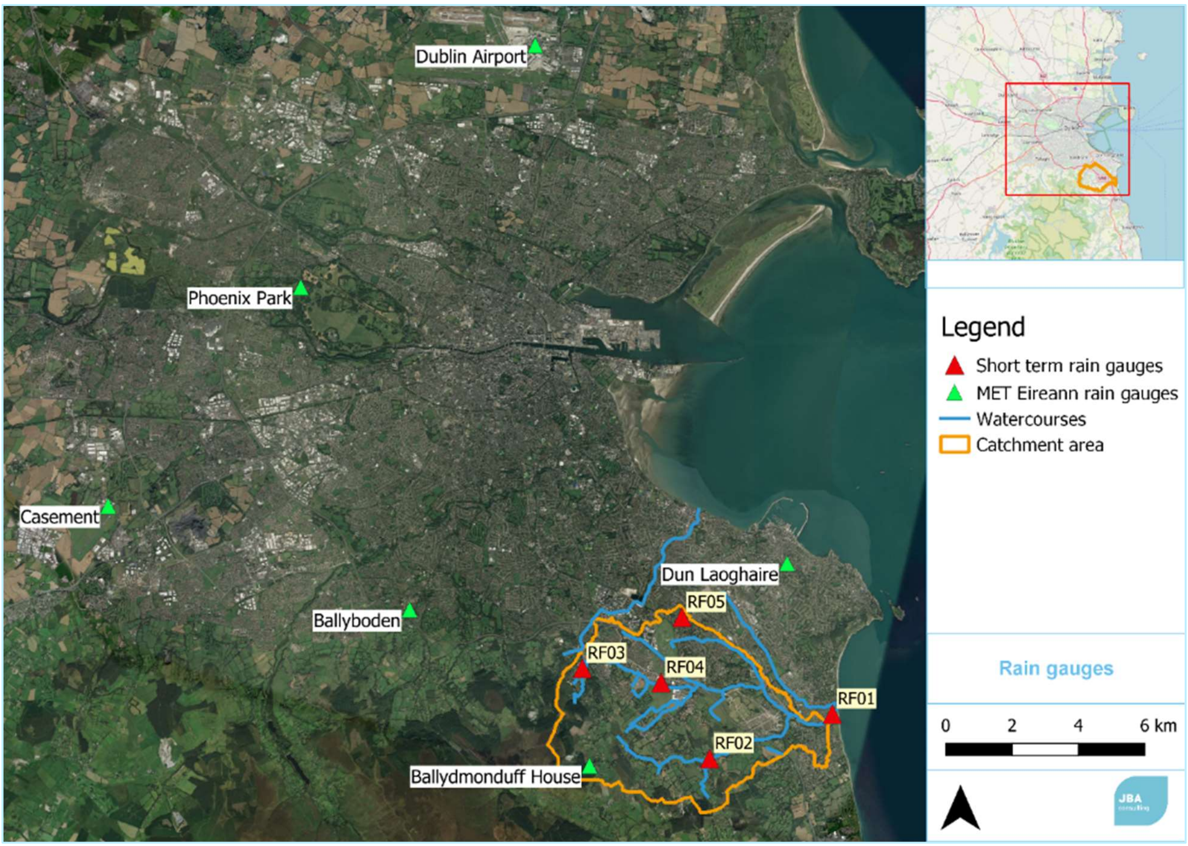


Figure 2-11: Rain gauges (base maps: Bing Satellite and OSM Standard)

Table 2-7: Summary of rainfall gauging stations information.

Gauge Name	Number	Operator	Type	Available record	Distance to centre of catchment (km)	SAAR value	Notes
Casement	3723	Met Éireann	Hourly	1991-2020	18.3	758.25	
Dublin Airport	532	Met Éireann	Hourly	1991-2020	20.5	769.25	
Phoenix Park	175	Met Éireann	Hourly	2004-2020	17.2	773.00	

Gauge Name	Number	Operator	Type	Available record	Distance to centre of catchment (km)	SAAR value	Notes
Ballyboden	6623	Met Éireann	Daily	1967-2020	8.7	873.00	Data for Oct 2011 is missing, gaps in record
Balleydmonduff House	3524	Met Éireann	Daily	1985-2020	3.2	1227.50	
Dun Laoghaire	9223	Met Éireann	Daily	1997-2020	5.5	769.75	
Fernhill House and Gardens	NA	-	2 min interval	Feb – Jun 2021	In catchment	1086	
DLRCC Ops Depot	NA	-	2 min interval	Feb – Jun 2021	In catchment	935	
Foxrock golf course	NA	-	2 min interval	Feb – Jun 2021	In catchment	861	
Kilternan Cemetery	NA	-	2 min interval	Feb – Jun 2021	In catchment	964	
Irish Water WWTP	NA	Met Éireann and short-term gauge	2 min interval (short term)	Feb – Jun 2021 (short term)	In catchment	794	Both short term gauge and MET Eireann gauge located here

2.5.1 Examination of wider meteorological data with reference to changes at Common's Road gauge

Figure 2-12 shows the total annual rainfall recorded at each of the permanent gauges listed in Table 2-7 for the same period of the Common's Road gauge has been active. From the data the Balleydmonduff House gauge records the highest annual rainfall depths due to its upland location in the Dublin Mountains. Table 2-8 shows the average total rainfall values for each gauge pre- and post-2005. The periods were chosen to examine whether there is any meteorological variation observed around the point at which the flow-stage relationship at Common's Road gauge changed.

The average total rainfall for the post-2005 period is higher for all but the Balleydmonduff House gauge. The trendlines for the total gauge records where the post-2005 total average is higher are also increasing over time (refer to Figure 2-13 for example). This indicates that the more recent record is for an increasingly wetter period relative to the whole recorded period. Whether this increase in rainfall is a result of a marked change in climate or just a period of increased wetness remains to be seen. The increase in rainfall may have contributed the observed change in the Commons Road gauge flow-stage relationship discussed in Section 2.3 however with so many other significant changes occurring in the catchment at that time (M50 motorway and changes at gauge reach) it is impossible to isolate the impact of each factor. All that can be gleaned from the available meteorological data is that the more recent record is wetter than previous years.

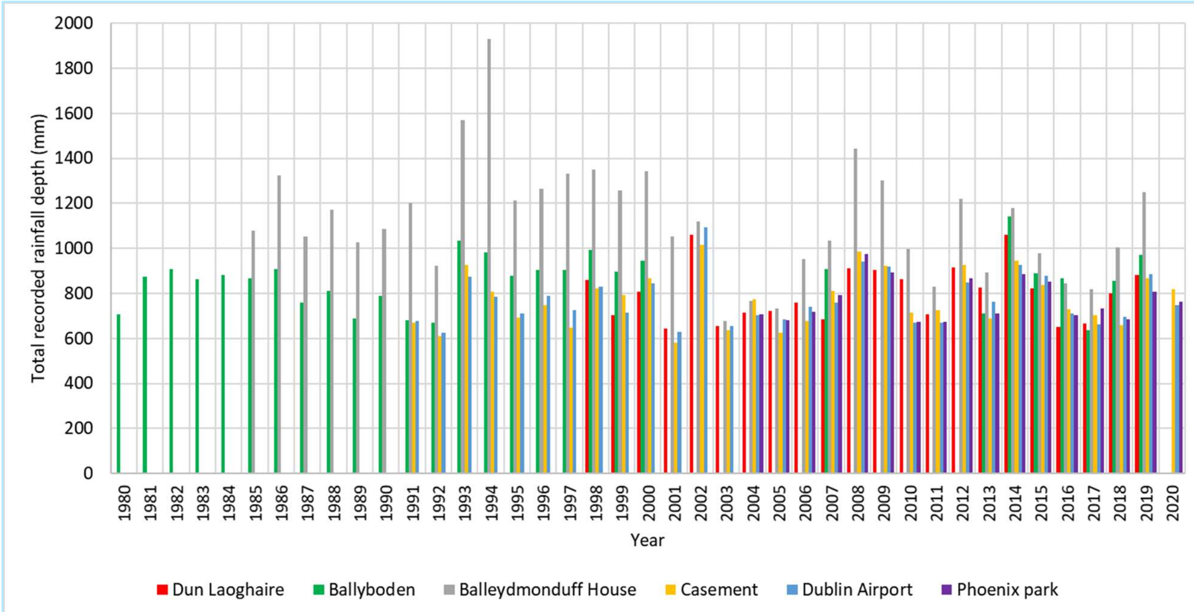


Figure 2-12: Recorded total annual rainfall depth: 1980 – 2020.

Table 2-8: Average total rainfall depth for meteorological gauges

Gauge	**Average total rainfall depth (mm)**		
	Total gauge record	Pre-2005 record	Post-2005 record
Dun Laoghaire	801.16	770.94	818.44
Ballyboden	801.78	782.26	872.56
Balleydmonduff House	1120.67	1165.35	1053.65
Casement	774.3	747.60	801.00
Dublin Airport	772.30	756.05	788.56
Phoenix Park	771.64	NA	782.12

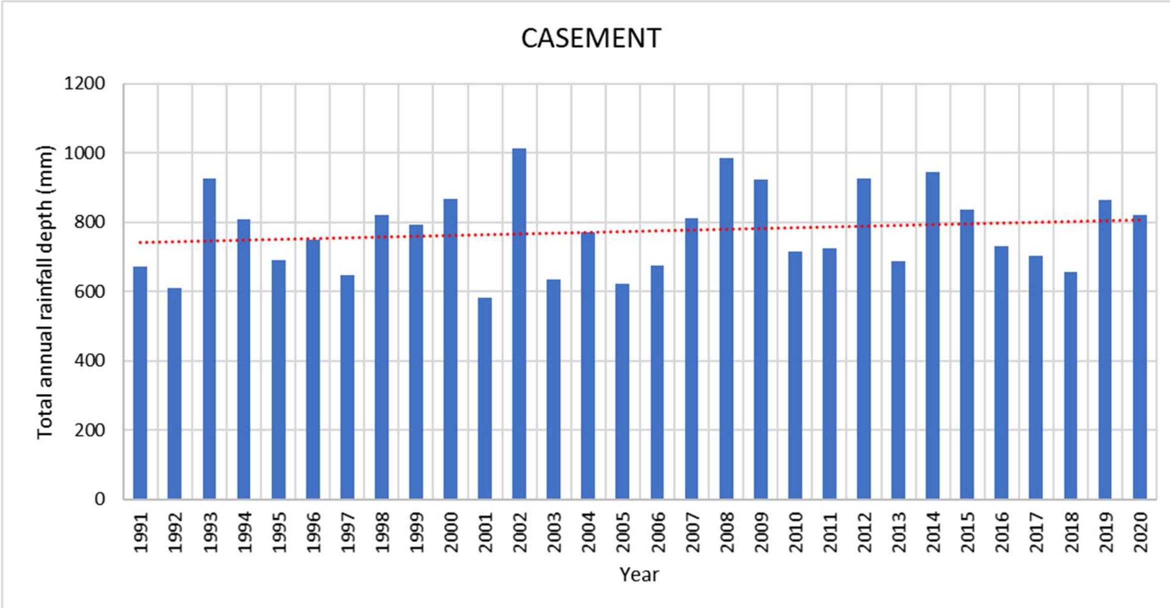


Figure 2-13: Casement rain gauge record – total annual rainfall depth

2.6 Short term flow monitoring study

2.6.1 Monitoring locations and equipment

Additional flow monitors and rain gauges were placed in the catchment for 13 weeks between February and May 2021. The aim of the monitoring was to understand different flow responses of watercourses within the catchment. Figure 2-14 and Figure 2-15 show the locations of the flow and rain gauges respectively while Table 2-9 and Table 2-10 provide details of the equipment used.

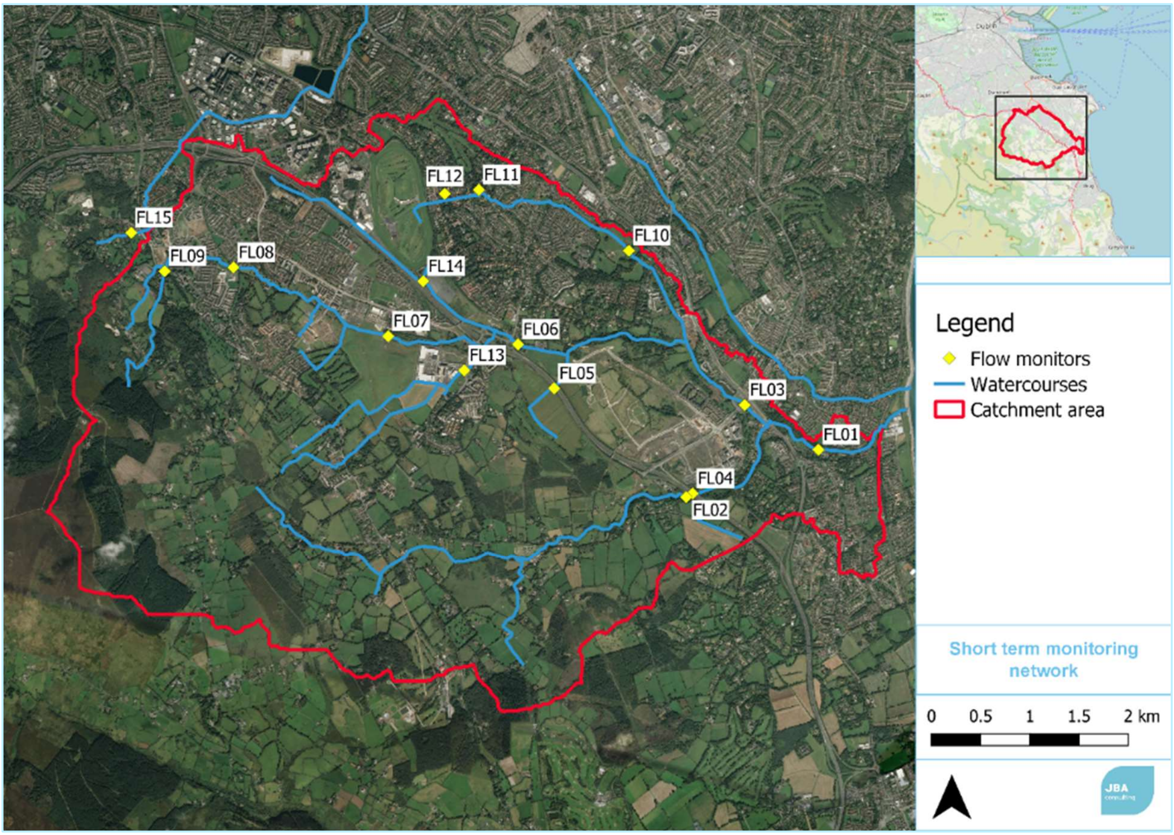


Figure 2-14: Short term flow monitoring network (base maps: Bing Satellite and OSM Standard)

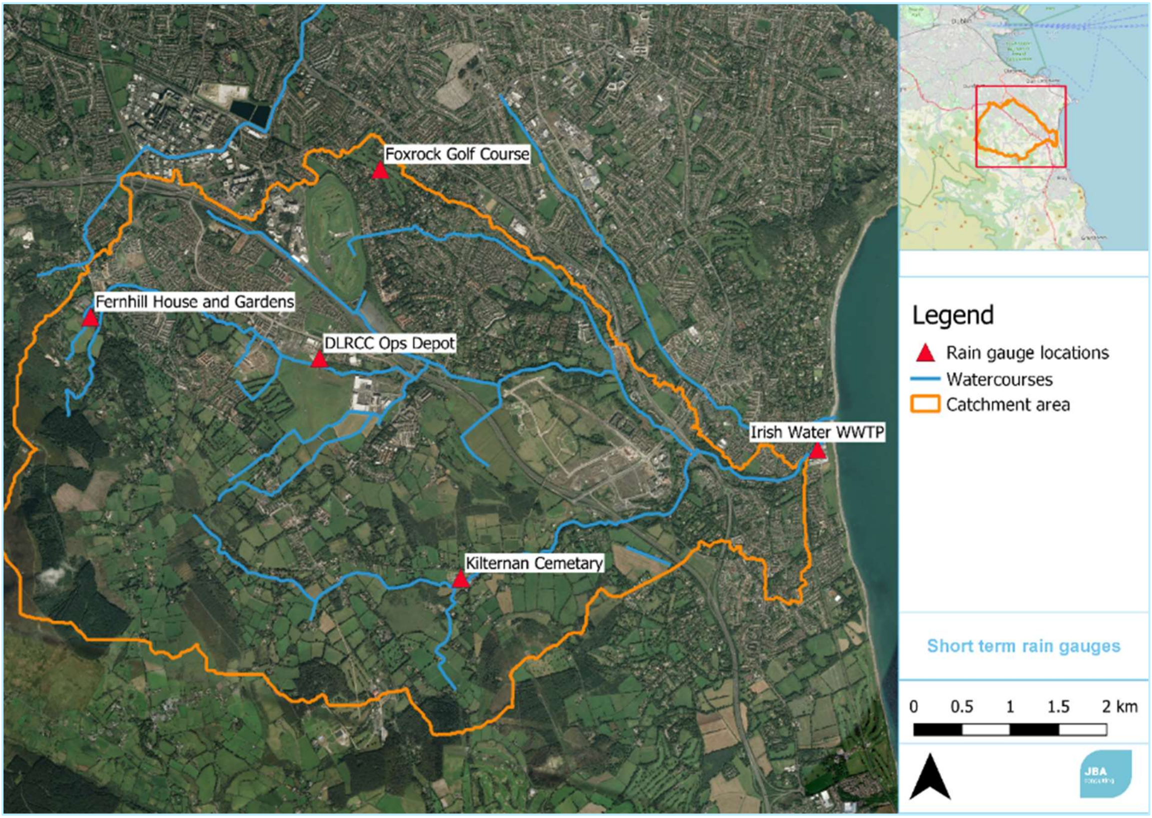


Figure 2-15: Short term rain gauges (base maps: Bing Satellite and OSM Standard)

Table 2-9: Flow recorder details

Flow recorder details	
Type	Area/velocity loggers
Data recorded	Depth and velocity reported at 2-minute intervals (post processed to report flow)
Quoted margin of error/accuracy	Depth: cut range for accurate recordings typically below 10mm – 15mm. Velocity: values less than 0.20m/s are of lesser accuracy and values are generally subject to greater uncertainty when the depth of flow above the bottom of the probe is below 75mm

Table 2-10: Rain gauge details

Rain gauge details	
Type	ARG100 tipping bucket rain gauge with remote sensor data logger
Data recorded	Rainfall depth, 0.2mm tipping bucket
Quoted margin of error/accuracy	+/- 0.2mm (tip sensitivity)

2.6.2 Recorded event data

During the monitoring period four rainfall events occurred which triggered sizeable responses in the watercourses. Using the recorded data observations about the system behaviour and response could be made, the event data was also used in the hydrological routing model. The following four events were used in analysis:

- 03/05/2021 (10-hour double peak rainfall event)
- 08/05/2021 (7-hour single late peak rainfall event)
- 20/05/2021 (10-hour single front peak rainfall event)
- 21/05/2021 (15-hour low intensity double peak rainfall event)

Figure 2-16 shows the daily recorded rainfall for the entire monitoring period and the size of the key events relative to other rainfall that was recorded. No significant storm events occurred during the monitoring period. The average sum total recorded depth for the 08/05/2021 event for the 5 rainfall gauges is 18.08mm, for comparison the DDF estimated depth for a 50% AEP 6-hour storm is 23.3mm using the centre point of the catchment.

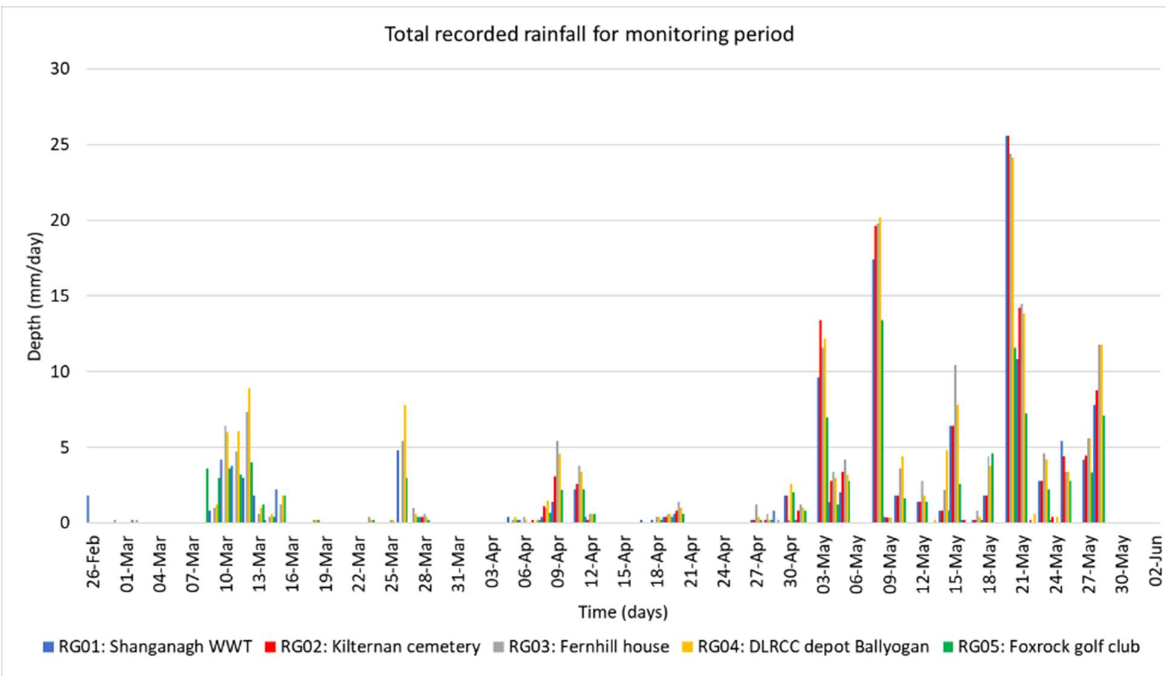


Figure 2-16: Daily rainfall recorded over the monitoring period

Figure 2-17 compares the recorded gauge data at Commons Road gauge (10021) with the monitored flows from gauge FM001 at the same location for the 08/05/2021 event. Comparing the data enables a sensibility checking of the monitoring data. From the figure the two gauges show the same hydrograph shape for the event and the flows are within the same order of magnitude. There is an approximate 0.50m^3/s difference in peak flow between the monitor and gauge and a time offset (approximately 20minutes). It is important to note the following points when comparing the permanent and short-term gauge data:

- The short-term gauge monitor was placed on the channel bed which differs to the location of the permanent gauge zero.

- The short-term gauge data was recorded at 2-minute intervals while the permanent gauge data is recorded at 15-minute intervals.

- The flows for the permanent gauge are derived using the ECFRAM rating curve equations (refer to Section 2.4.1) while the short-term flow data recorded velocity and depth (and hence flow) directly.

Overall, there is good consistency between the permanent and short-term gauge data which provides confidence in the data recorded at the flow gauges during the monitoring period.

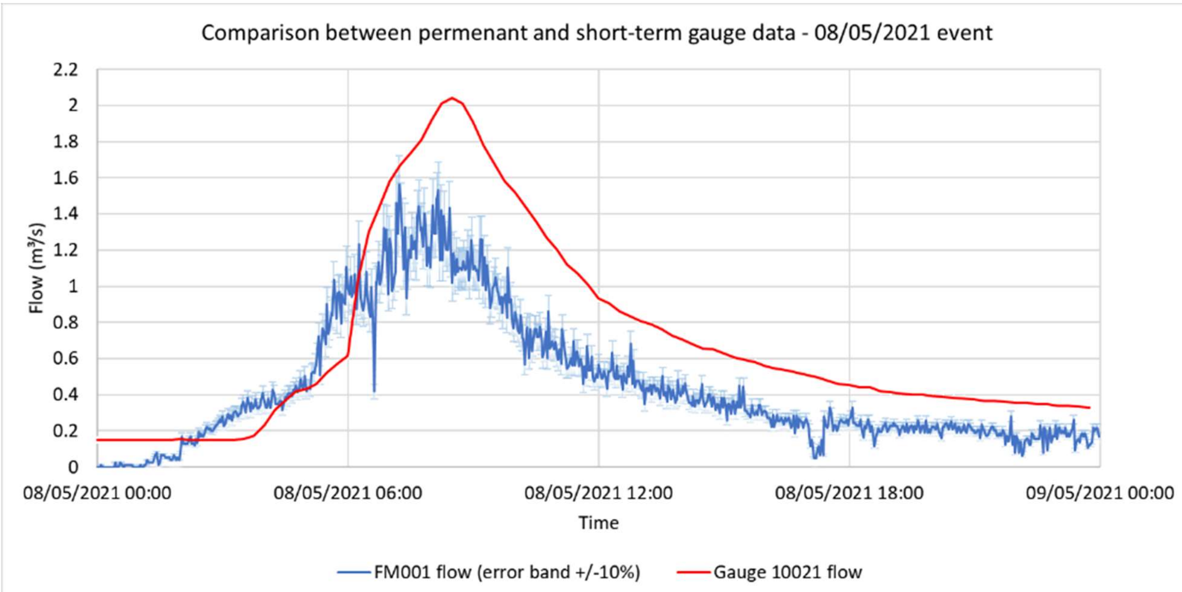


Figure 2-17: Comparison between permanent and short-term gauge data – 08/05/2021 event

2.6.3 Rainfall data

Figure 2-18 compares the recorded hyetographs for each of the four events, each showing a differ pattern. The following observations are made:

- Overall, across the events gauges at higher elevations (RG02, RG03 and RG04) record the greatest depths compared to the low-lying gauges (RG01 and RG05). The variation of recorded rainfall depths for the monitoring period events is similar to the variation in SAAR values for the 5 gauges (refer to Table 2-7 and peak depths presented in Figure 2-18).

- The rainfall pattern and timing are consistent between gauges indicating that the distance across the catchment is not large enough that large temporal variations can occur during a storm event. This is seen in Figure 2-18 where the peak rainfall occurs at the within the same hour for all 5 gauges.

- The consistent response within the gauges shows that a single storm impacts the entire catchment area at any given time.

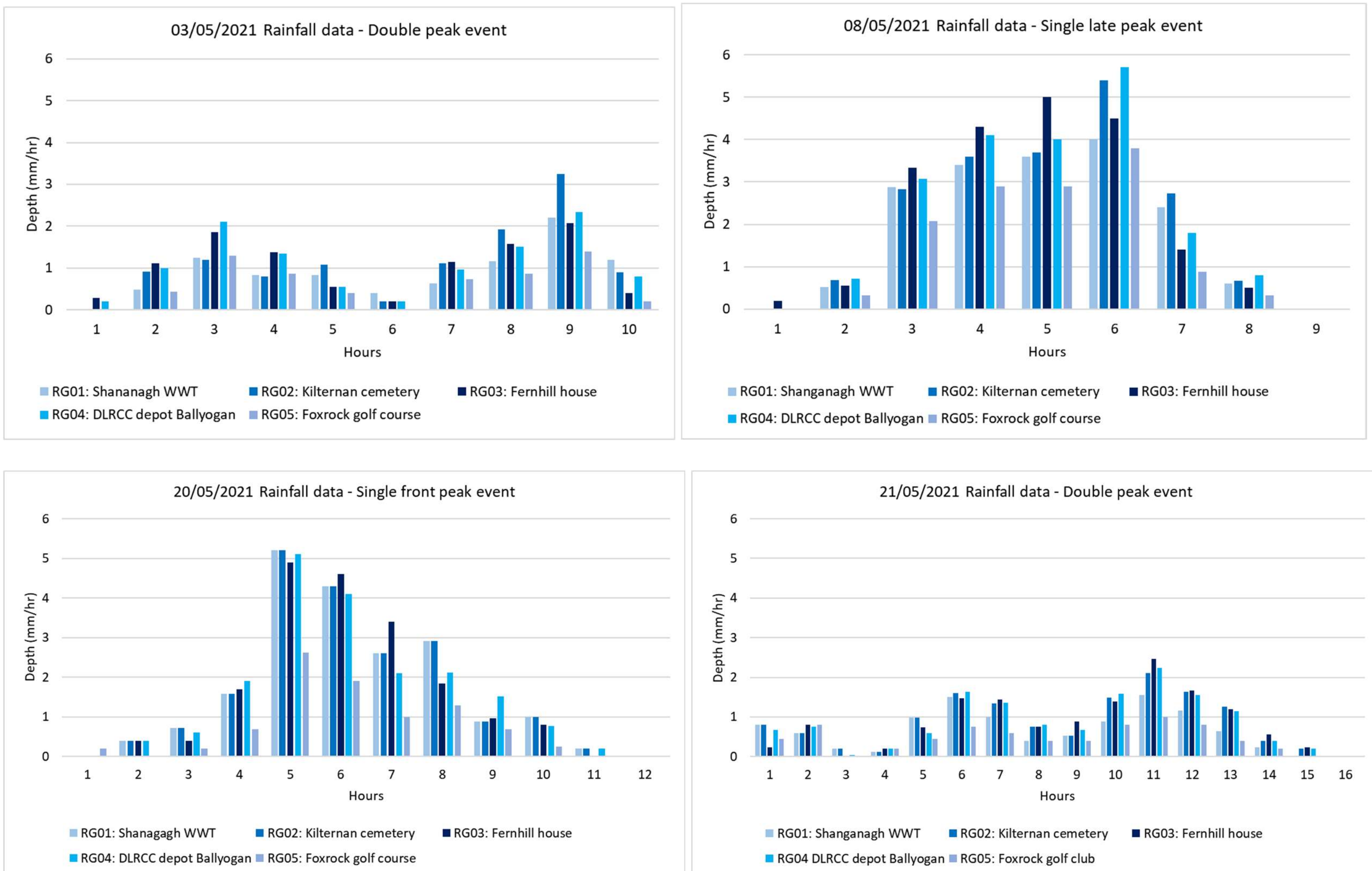


Figure 2-18: Hyetographs for recorded storm events

2.6.4 Flow data

Flow data was recorded along all the key watercourse and at some critical stormwater outfalls. From reviewing the data, the following general observations can be made:

- The recorded flow hydrograph response in the catchment is dictated by the observed rainfall pattern in all events. While stormwater and phasing of tributaries is present and does impact the response recorded at the downstream extent the dominant driver of response is rainfall. This is seen in the event hydrographs where the general hydrograph pattern is largely reflective of the observed rainfall pattern.

- Gauges in urban and storm water driven catchments have flashier and sharper responses compared to gauges in more rural areas (refer to Figure 2-19 and Figure 2-20 for comparison of responses for the 08/05/2021 event). This is reflective of the different land uses, permeability, and runoff behaviour in the catchment.

- Time between peak rainfall and peak flow at the downstream extent of the monitoring network (FM001 Commons Road gauge) is approximately 3-4 hours.

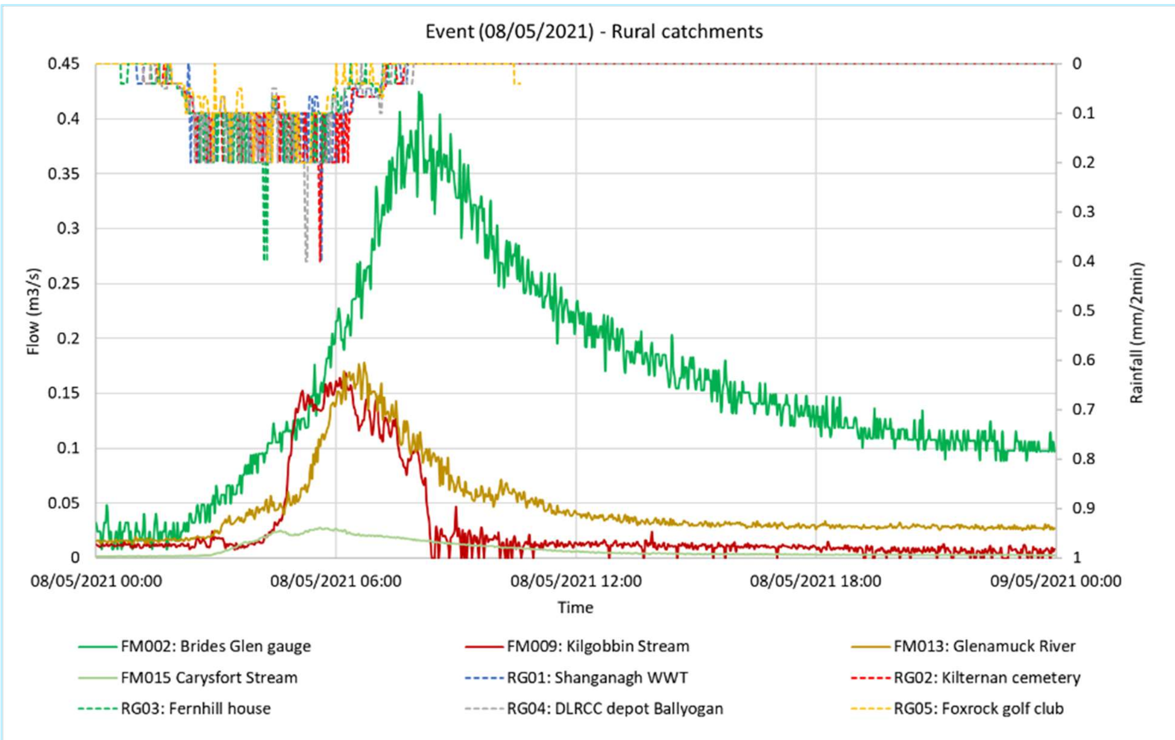


Figure 2-19: Event (08/05/2021) – Rural catchment responses

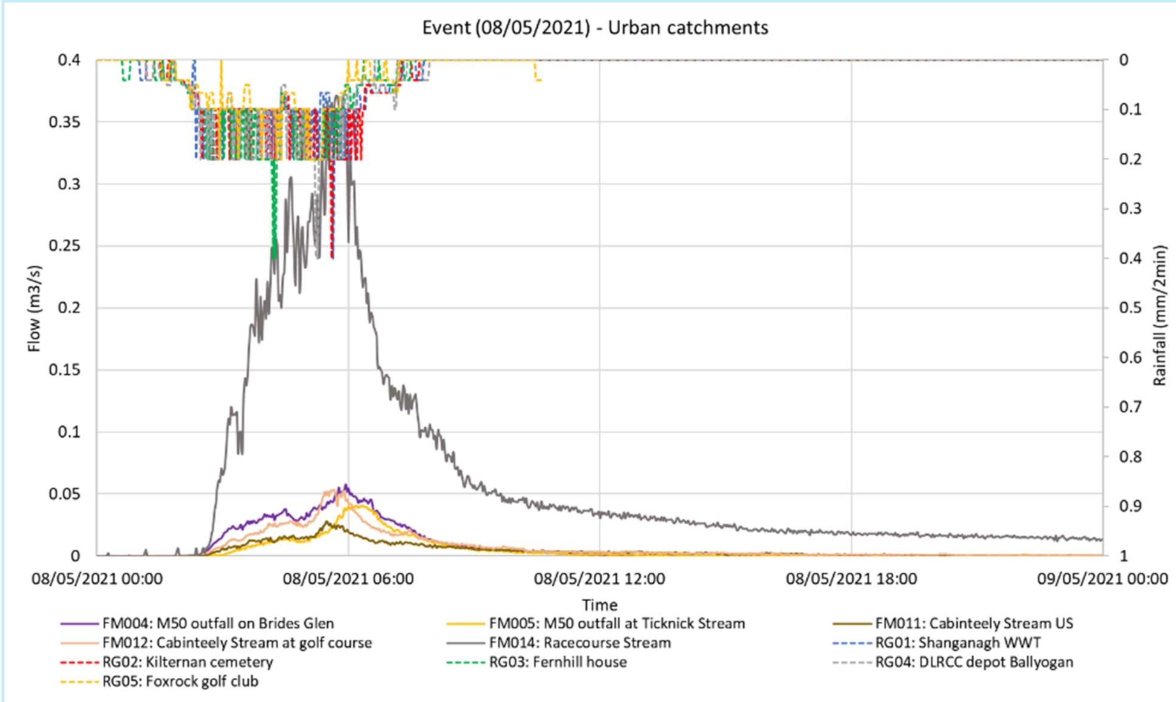


Figure 2-20: Event (08/05/2021) – Urban catchment responses

Examining the main watercourse (Carrickmines/Shanganagh River) the variation and sequencing of the flows can be seen in more detail. Figure 2-21 through to Figure 2-23 show the flow responses along the watercourse for all four events (note: the 20/05 and 21/05 events have been plotted on the same figure). The response changes moving from up- to downstream as the watercourse increases in size and the catchment urban land cover changes. The sharp flow response is dampened downstream as additional tributaries and increased flow smooth the response. This is clearly seen in the 20/05/2021 event where three peaks in response to rainfall are recorded in the upstream gauges (e.g., FM007) but only two are seen in the downstream data (e.g., FM006).

In all the events there is a prominent concave curve on the rising limb of the hydrograph at the downstream gauges (FM003 and FM001). This is likely the initial fast response of the local stormwater network entering the system in combination with flows from the smaller tributaries. This initial 'bump' highlights that while the upstream tributaries all peak around the same time their flow contributions pass through the system before the main watercourse peaks. The peak flow at gauge FM001 (Common's Road) occurs approximately 3/4hours after peak rainfall, review of the data highlights that the peak level and flow at this location is driven by the Brides Glen River which peaks at a later stage therefore contributing flow into the system at the peak and in the falling limb (refer to Figure 2-24).

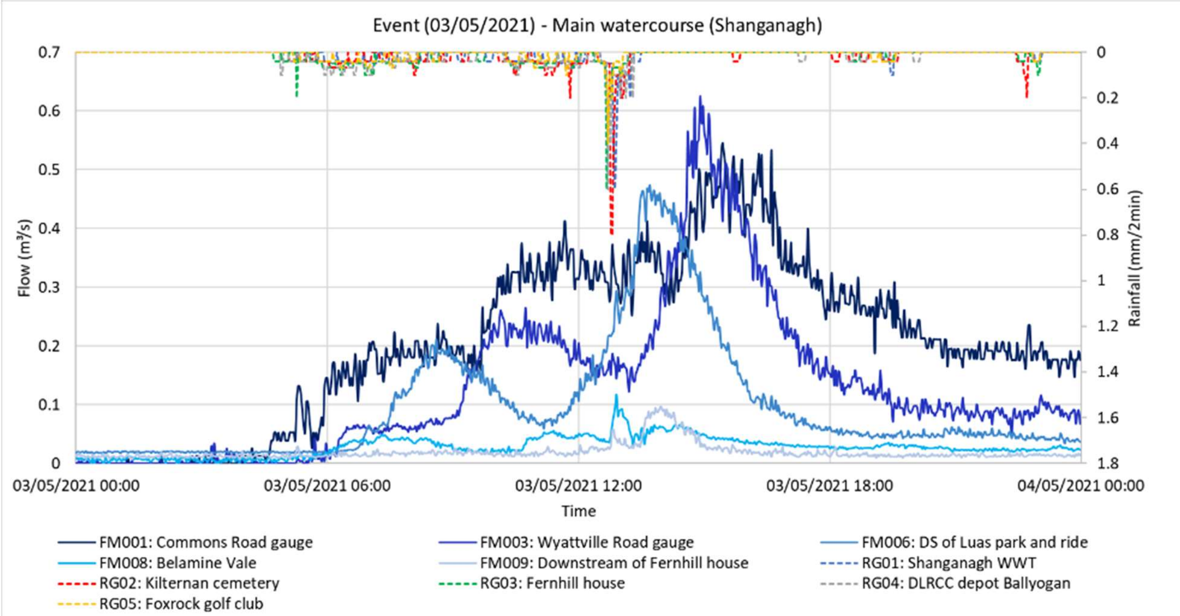


Figure 2-21: Event 03/05/2021 – main watercourse (Shanganagh)

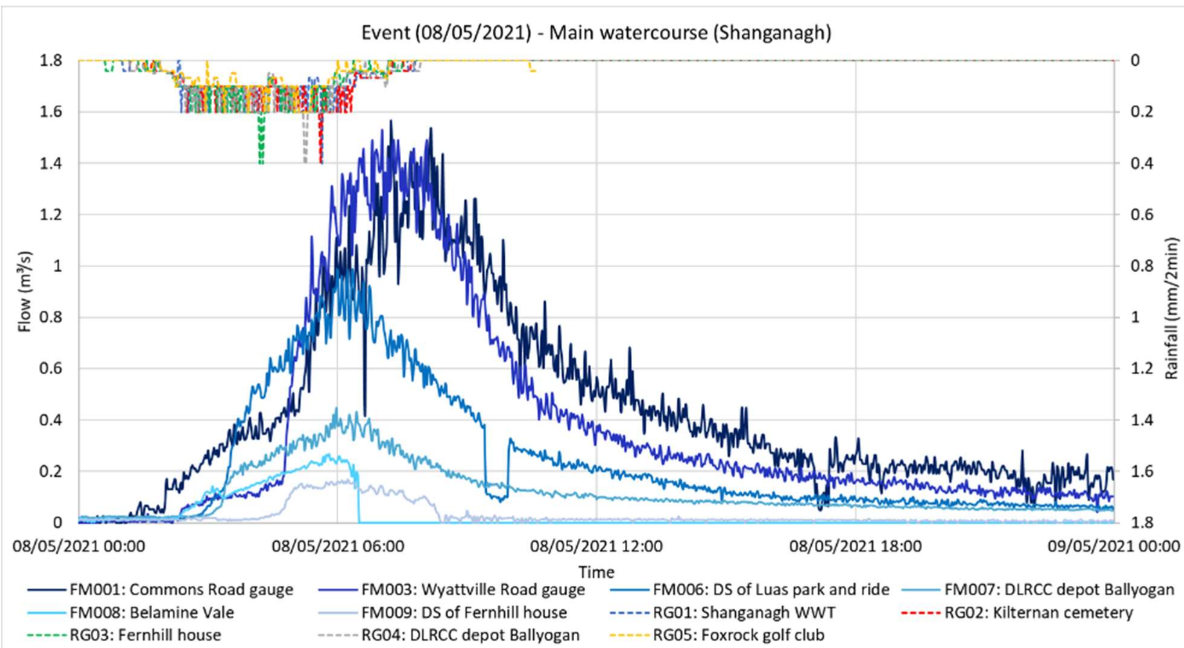


Figure 2-22: Event 08/05/2021 – main watercourse (Shanganagh)

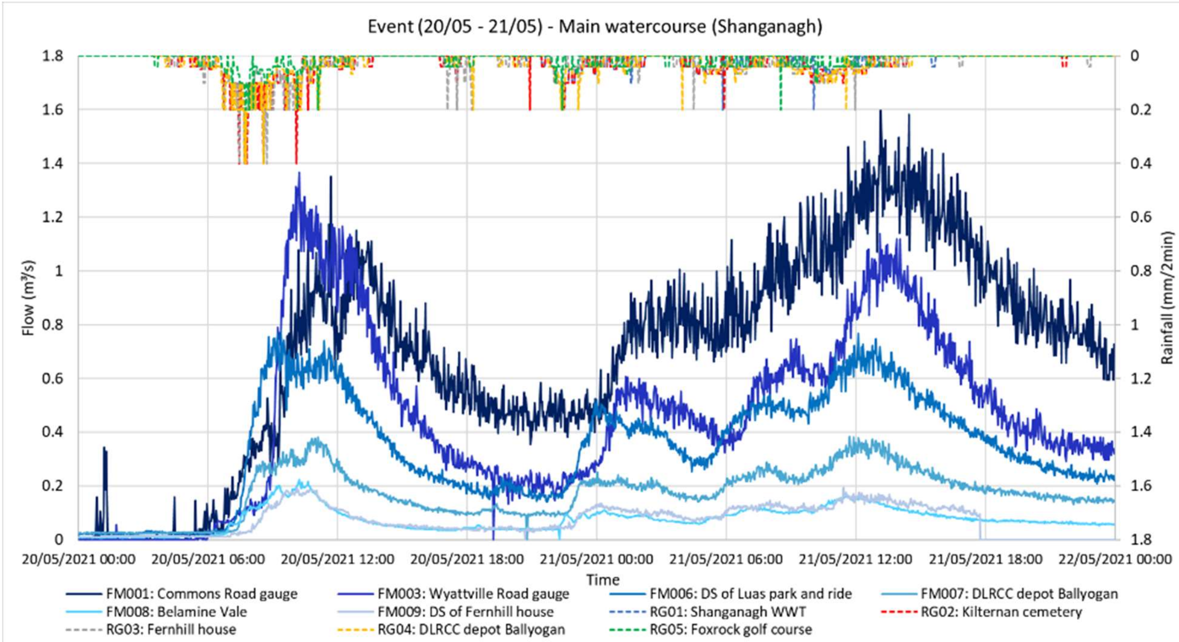


Figure 2-23: Event 20/05/2021 – 21/05/2021- main watercourse (Shanganagh)

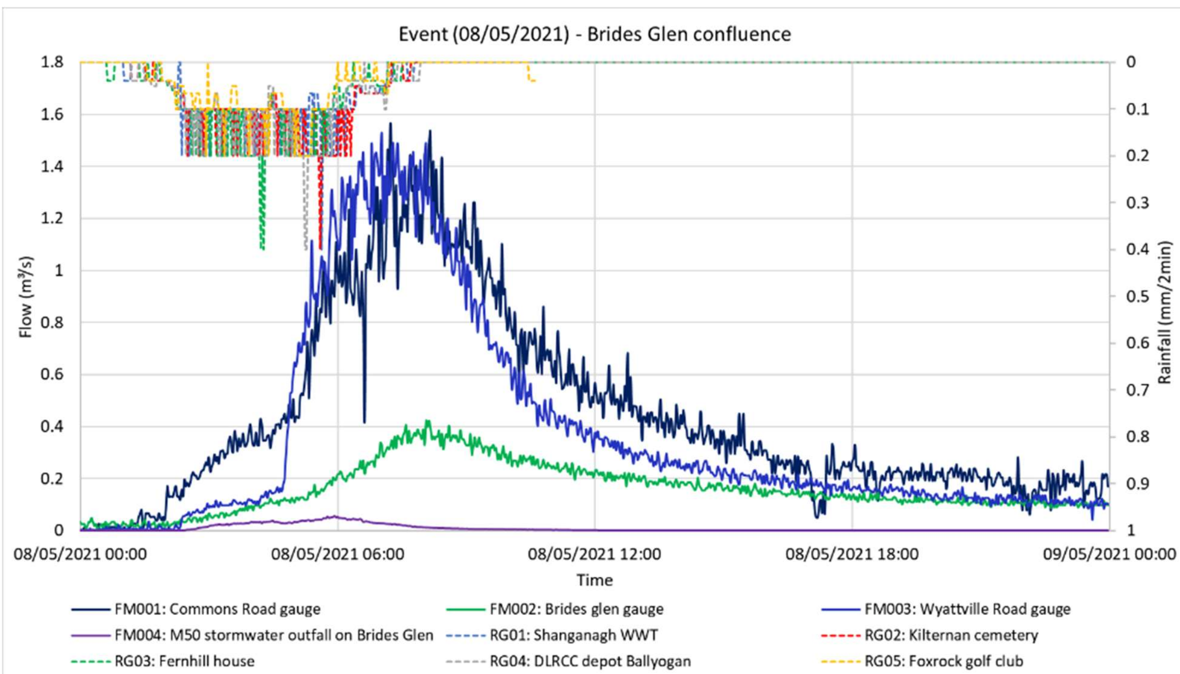


Figure 2-24: Event 08/05/2021 – Brides Glen confluence

Comparing the hydrographs recorded during the flow monitoring period at FM001 with recorded event data from the permanent Commons Road gauge (10021) in Figure 2-25 the hydrograph shape in the small and large events is largely consistent. This shows that there is no significant variation in system response between large and small magnitude events at the downstream extent of the catchment.

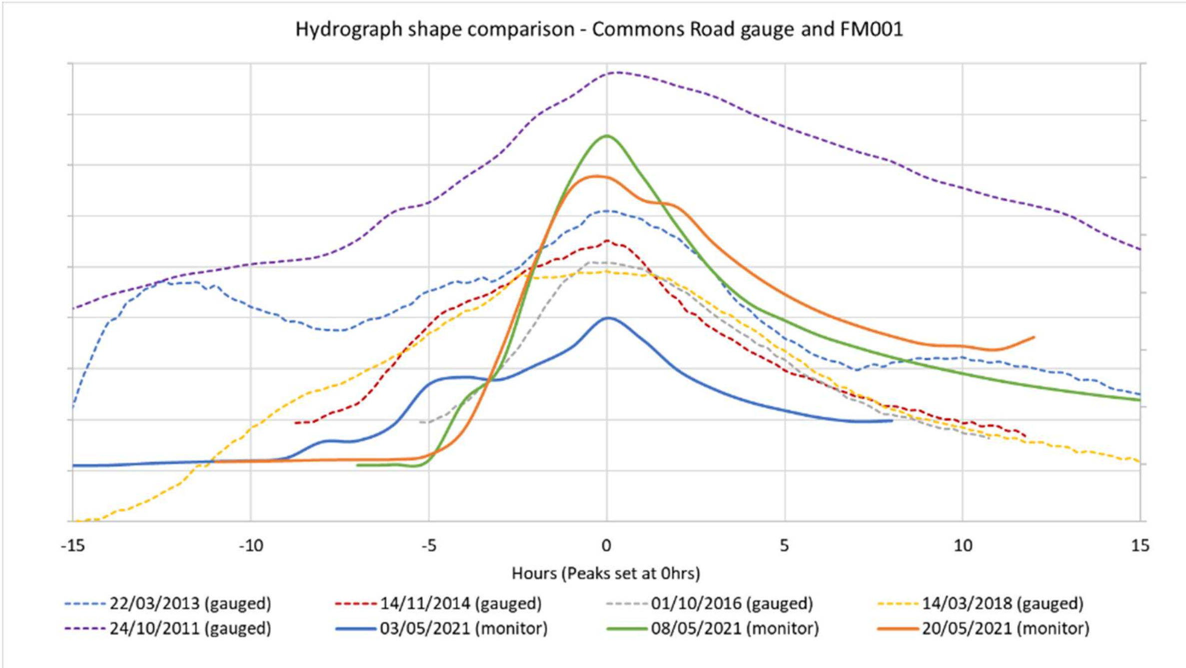


Figure 2-25: Hydrograph shape comparison – Commons road gauge and FM001

In summary the short-term monitoring has provided a greater understanding of system behaviour and the relationship between the tributaries and stormwater contributions. It has also highlighted the differing behaviours of catchments in relation to land use and how the rainfall pattern is the dominant driver of the observed fluvial response. The understanding and insight gained has been used to inform the hydrological routing model, refer to Section 4.4 for further discussion on how the flow monitoring data has been used in relation to the hydrological routing model.

2.7 Groundwater data

Figure 2-26 shows the bedrock underlying the catchment. The catchment bedrock is largely igneous in nature consisting of various granites. The bedrock for a large proportion of the catchment is defined as poorly productive aquifers which is expected given its nature. There is a portion of bedrock at the downstream extent of the area (area underlain by slates) which is deemed a locally important aquifer suggesting more potential for groundwater interaction.

Review of the Geological Survey of Ireland (GSI) groundwater data sets show that groundwater vulnerability (a measure of the potential risk for groundwater contamination and/or flooding) of the catchment is variable. Bedrock is exposed in the upland mountainous areas while the vulnerability is defined as high – moderate for the remainder of the catchment. This indicates an estimated depth to bed rock in the catchment between 3-10m. The groundwater permeability of the subsurface (consisting of granitic tills) is classified as moderate indicating it is reasonably well draining. Overall groundwater flooding is not considered to be a significant risk based on available information.

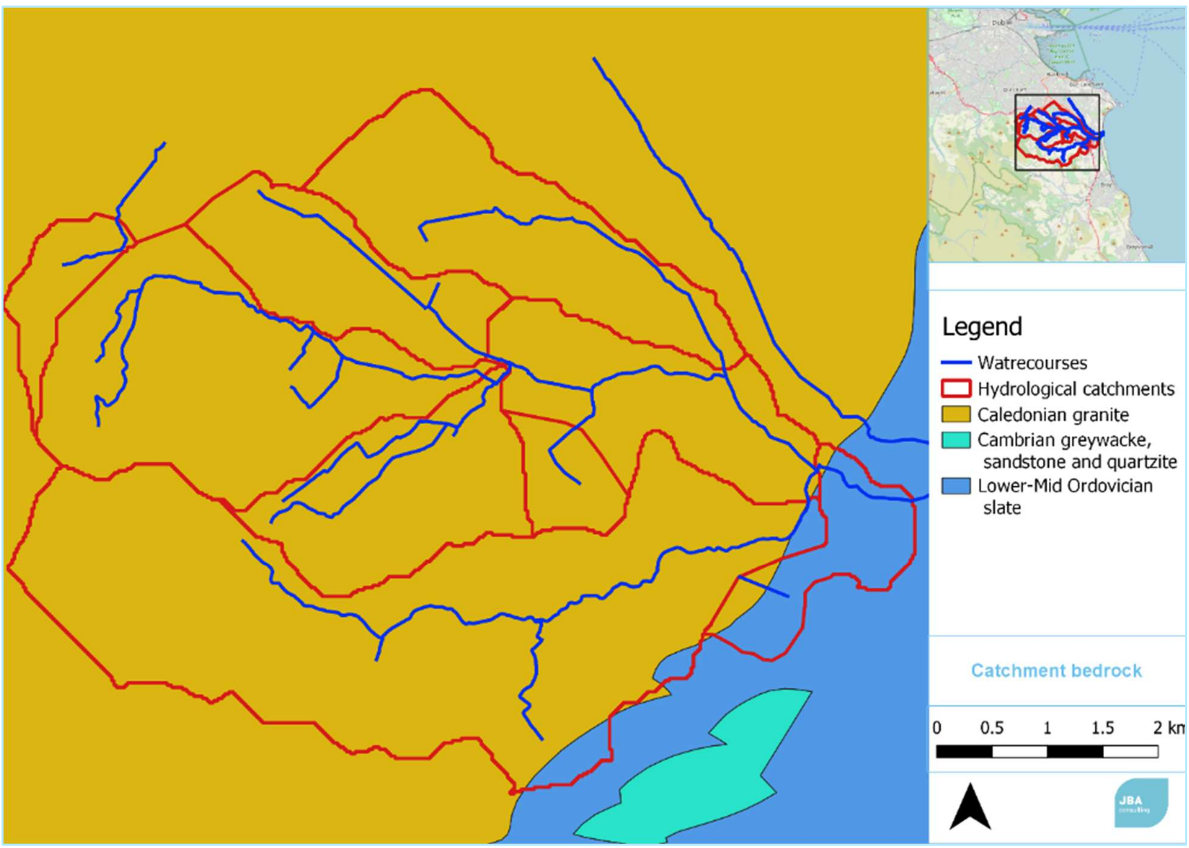


Figure 2-26: Catchment bedrock (base map: OSM standard)

2.8 Urban drainage

Figure 2-27 shows the stormwater network for the study area. As expected, the network is most dense in the area where there is the most urban land use. The general design standard for stormwater systems is up to the 10% - 3% AEP pluvial storm event for a calculated critical duration period. Stormwater outfalls do contribute to the overall flow within the fluvial system such as outfalls from the M50 motorway. Short term flow gauge data was collected at key outfall locations of the M50 to the main watercourse as well as other areas along the watercourses where there are potentially significant flows from stormwater systems. This data has been used to further refine the timing and volume of flow contributions into the main watercourse (refer to Section 4.4 for further analysis on urban area flows from the flow monitoring).

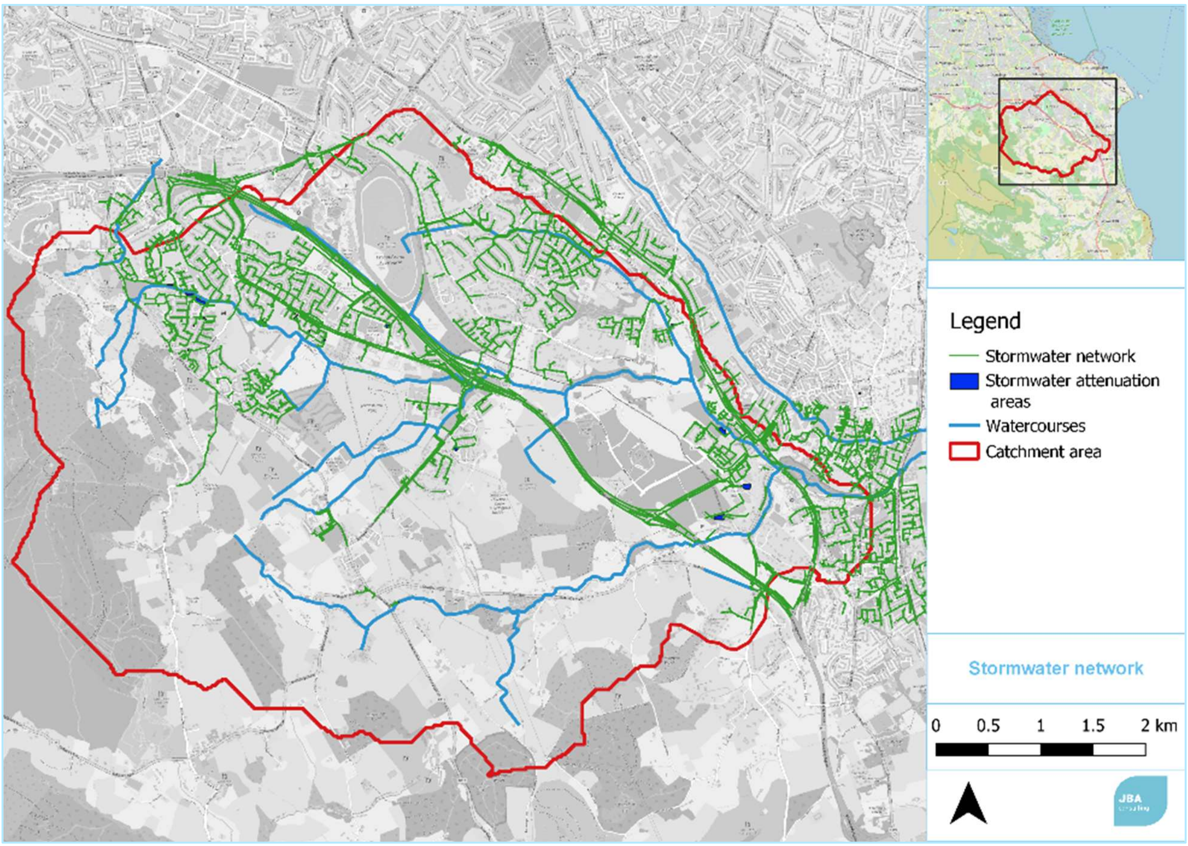


Figure 2-27: Stormwater network (base map: OSM standard)

2.9 Flood History

A summary of all the recorded flood events that have occurred in the catchment are provided in this section. Table 2-11 summarises the peak levels and flows recorded at Gauge 10021 for each event, the relevant growth curves have been applied to estimate flows for different periods. The estimated exceedance values for each event are also provided in Table 2-11 using the Gringorten extreme probability plotting method.

- **November 1982**
  The second highest flow on record was recorded at the Commons Road hydrometric station on 6th November 1982, measuring 13.30m^3/s, while at the Carrickmines hydrometric station, the fourth highest flow on record was recorded, measuring 5.40m^3/s. Flooding was caused when the Shanganagh River overflowed. At Cabinteely, debris was washed downstream and this, in conjunction with vegetation which had been growing in the stream, blocked up a culvert. Some minor road flooding occurred at Commons Road due to overflowing of the Shanganagh and flooding also occurred at Pottery Road/Johnstown Road in Cabinteely.
- **August 1986**
  The historical review indicated that the flow at the Carrickmines hydrometric station reached its fifth highest level on record (5.30m^3/s), while the flow rate downstream at the Commons Road gauge reached its fourth highest level on record (11.40m^3/s). No information is available for flood extents and damage caused due to this flooding.
- **May 1993**
  The historical review indicated that the highest flows on record were recorded at both the Carrickmines hydrometric station (approximately 6.70m^3/s) and the Commons Road hydrometric station (approximately 14.50m^3/s) on 26th May 1993, according to hydrographs produced in the EPA report. Photos at Commons Road and Carrickmines indicate flooding of low lying areas, including roads, adjacent to the river. There were no further details of extents or damage available for this event.
- **December 1997**
  The historical review indicated that the flow at the Carrickmines hydrometric station, measuring approximately 5.60m^3/s, reached its third highest level on record, according to hydrographs produced in the EPA report. The flow rate downstream at the Commons Road hydrometric station (approximately 9.80m^3/s) was approximately the sixth highest on record indicating heavy rainfall in only part of the catchment. There were no details of extents or damage available for this event.
- **November 2002**
  The historical review indicated that the Shanganagh River overflowed onto Commons Road and completely flooded the road. Sandbags, pumps, etc were used but flooding of nearby properties still occurred. From hydrographs at the Commons Road hydrometric station, the third highest flow on record occurred on 27th November 2002, measuring approximately 12.20m^3/s. Further upstream, at the Carrickmines hydrometric station, the second highest flow on record occurred on the same date according to the same source, measuring approximately 5.70m^3/s. No reports were found for this flood event detailing extents or return periods.
- **October 2011**
  Noted as the worst flood event to occur in the catchment in recent years widespread fluvial and pluvial flooding occurred across the area because of heavy rainfall falling on a saturated catchment and stormwater systems exceeding capacity due to the rain. The event caused major traffic disruption with many roads including portions of the M50 and M11 flooding within the study area. The highest water level to date was recorded at gauge 10021 during the event (2.66m) and resulted in the collapse of walls and flooding of several properties and roads.
- **March 2013**
  Heavy rain and sleet resulted in widespread pluvial and fluvial flooding in the catchment and wider Dublin area. The N11 and M50 roads were reported as being badly impacted and flooding of properties along commons road was also reported.

- **August 2014**
  Heavy rainfall relating to an orange weather warning falling in the early hours of the 2nd August resulted in fluvial and pluvial flooding to occur over the wider Dublin area. Road closures were reported in Ballybrack, north of the catchment where fluvial flows from the Deansgrange River were restricted by a road bridge[2]. It is noted that none of the 'hot spots' that flooded in 2011 within the study area were impacted in this event[3]. Reported flooding for this event in the wider area were mostly related to insufficient capacity in stormwater systems and pluvial flooding rather than fluvial flooding[4]. There are no reports of any areas within the study area being impacted in this event.
- **November 2014**
  Heavy rainfall occurring over the 12th – 14th November resulted in a pluvial flooding in several locations across Ireland. Flooding was reported in the Sandyford area with several properties impacted and roads becoming impassable (refer to Figure 2-28).
- **March 2018**
  Flooding reported in the upper catchment areas of the Brides Glen river during this event (road closures reported in Kilternan). No other flooding reported in the study area for this event.

Figure 2-28: Image of flooding on Enniskerry Road, Sandyford following the November 2014 flood event (source: https://www.irishtimes.com/news/environment/new-weather-alert-issued-as-flooding-hits-country-1.2000779)

[2] Report on flooding within the functional area of the committee, meeting minutes from 'Meeting of Dún Laoghaire Area Committee, Monday 1st September 2014, 5.00pm.'

[3] Report on flooding within the functional area of the committee, meeting minutes from 'Meeting of Dún Laoghaire Area Committee, Monday 1st September 2014, 5.00pm.'

[4] Report on flooding within the functional area of the committee, meeting minutes from 'Meeting of Dún Laoghaire Area Committee, Monday 1st September 2014, 5.00pm.'

Table 2-11: Summary of peak levels and flows at Gauge 10021 for historic flood events in the catchment

Event	Peak level at Gauge 10021 (m)	Peak flow at Gauge 10021 (m^3/s)	Estimated return period (Gringorten ranking)
November 1982	1.75	13.51	Between 10-20% AEP
August 1986	1.52	11.57	Between 20-25% AEP
May 1993	1.85	14.35	Between 10-20% AEP
December 1997	1.31	9.82	Between 20-25% AEP
November 2002	1.60	12.24	Between 10-20% AEP
October 2011	2.66	39.23	Between 1-2% AEP
March 2013	1.99	20.24	Between 4-5% AEP
August 2014	1.07	6.98	Between 20-25% AEP
November 2014	1.84	18.30	Between 5-10% AEP
March 2018	1.69	16.24	Approximately 10% AEP

2.9.1 October 2011 event

October 2011 is noted as being the most severe flood event to occur in the catchment in recent history. Heavy rainfall over the catchment on the 24th and 25th October (estimated as a 1 in 80 year rainfall event for the wider Dublin area[5]) resulted in widespread flooding as rivers overtopped and burst their banks and storm water system capacities were exceeded.

The rainfall event started around 10am on 24th with a peak at 5-6pm and finished at 8-9pm with 10 hrs duration and with total rainfall at that day varying from 62.1mm at Balleydmonduff House gauge and 83.2mm at Casement gauge. Figure 2-29 shows the recorded rainfall profiles for the three hourly gauges compared to the recorded levels at Commons Road gauge, it is noted that a smaller event occurred on the 23rd October indicating the catchment would be saturated when the main event on the 24th occurred. Table 2-12 shows the recorded rainfall depths at the closest rain gauges to the catchment, hourly and daily for the 2011 event.

[5] Commons Road, Shankill Preliminary Assessment of Flooding Incident 24th October 2011, RPS

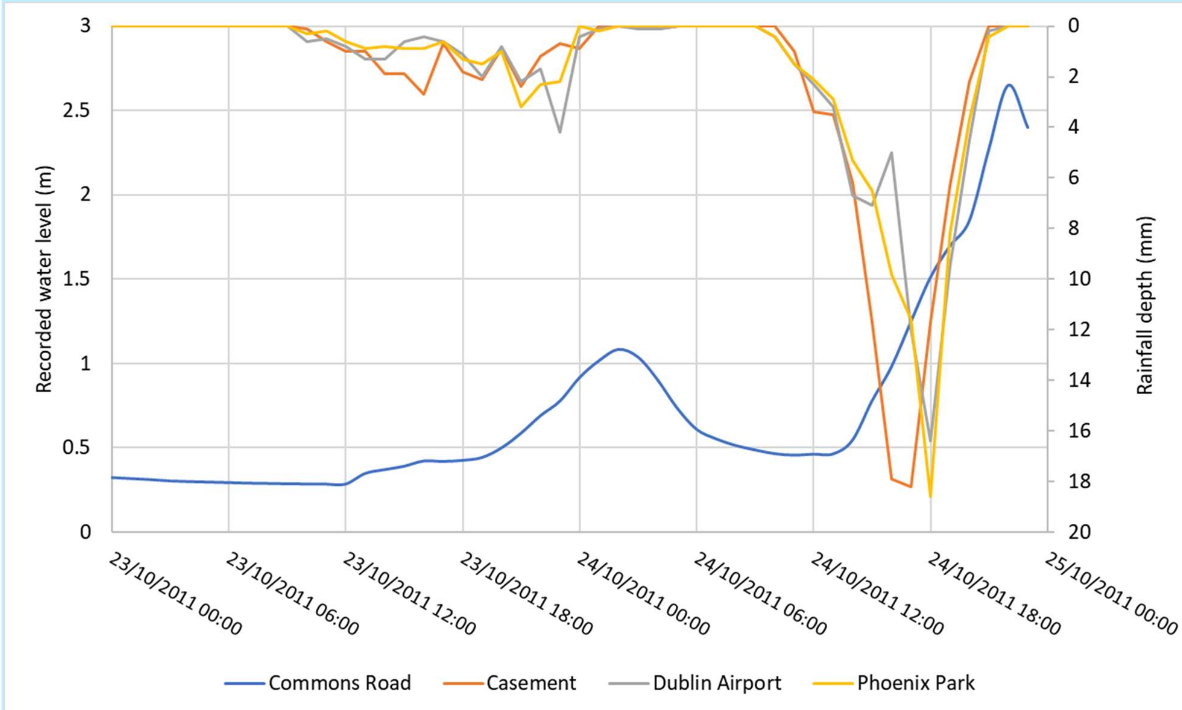


Figure 2-29: Hourly rainfall records, 23rd-24th of October 2011

Table 2-12: Recorded rainfall depths at closest rain gauges to study area

Gauge	Recorded rainfall depth (mm)	
	23rd October 2011	24th October 2011 (11:00-20:00)
Casement	20.0	83.2
Dublin Airport	18.8	69.2
Phoenix Park	16.9	71.5
Ballyboden	N/A	N/A
Balleydmonduff House	27.4	62.1
Dun Laoghaire	30.8	75.6

The event occurred when the catchment was already saturated from a period of wet weather further adding to the amount of runoff in the area (refer to Table 2-13); from Figure 2-29 it is also noted that a smaller event occurred on the 23rd October further indicating that the catchment would have been saturated. The Shanganagh River, the largest watercourse in the catchment overtopped its banks causing flooding in a range of locations (of note Kilgobbin Road, the N11 at Loughlinstown and Commons Road)[6]. As previously discussed, the highest level ever recorded at the Commons Road gauge was recorded during this event

[6] Commons Road, Shankill Preliminary Assessment of Flooding Incident 24th October 2011, RPS

(refer to Table 2-14) and the gauge was reported as being drowned during the peak of the event[7]. A wall downstream of the gauge on the left-hand side of Commons Road was undercut and collapsed during the event. A tree was also reported becoming stuck at the upstream face of a pedestrian bridge impacting conveyance through the bridge during the event (Figure 2-30). Increased water levels within the river resulted in out of bank flow within the Commons Road area resulting in the flooding of 9 properties downstream (Figure 2-31).

Flood defences along Commons Road built in 2005/6 were put in place with a Standard of Protection (SoP) up to the 2% AEP event were also overtopped during the event.

Table 2-13: Soil Moisture Deficit (SMD) values for available gauges

Date	**Casement gauge**			**Dublin airport gauge**		
	Well drained	Moderately drained	Poorly drained	Well drained	Moderately drained	Poorly drained
20/10/2011	2.0	2.0	-3.8	3.0	3.0	-3.4
21/10/2011	3.3	3.3	-2.3	4.1	4.1	-2.0
22/10/2011	0.0	-0.0	-5.4	3.1	3.1	-3.0
23/10/2011	0.0	-10.0	-10.0	0.0	-10.0	-10.0

Table 2-14: Summary of peak gauge data recorded at Gauge 10021 during the 2011 event

Maximum level recorded (m)	**Estimated flow m3/s (EPA rating curve)**	**Estimated flow m3/s (ECFRAM rating curve)**	**Notes**
2.66	21.22	39.23	Level is greater than upper limit of EPA rating curve and event is within period where it is no longer suitable

[7] Commons Road, Shankill Preliminary Assessment of Flooding Incident 24th October 2011, RPS

Figure 2-30: Left: Collapsed wall along Commons Road, Right: Tree stuck at upstream face of pedestrian access bridge (Source: Commons Road, Shankill Preliminary Assessment of Flooding Incident 24th October 2011, RPS)

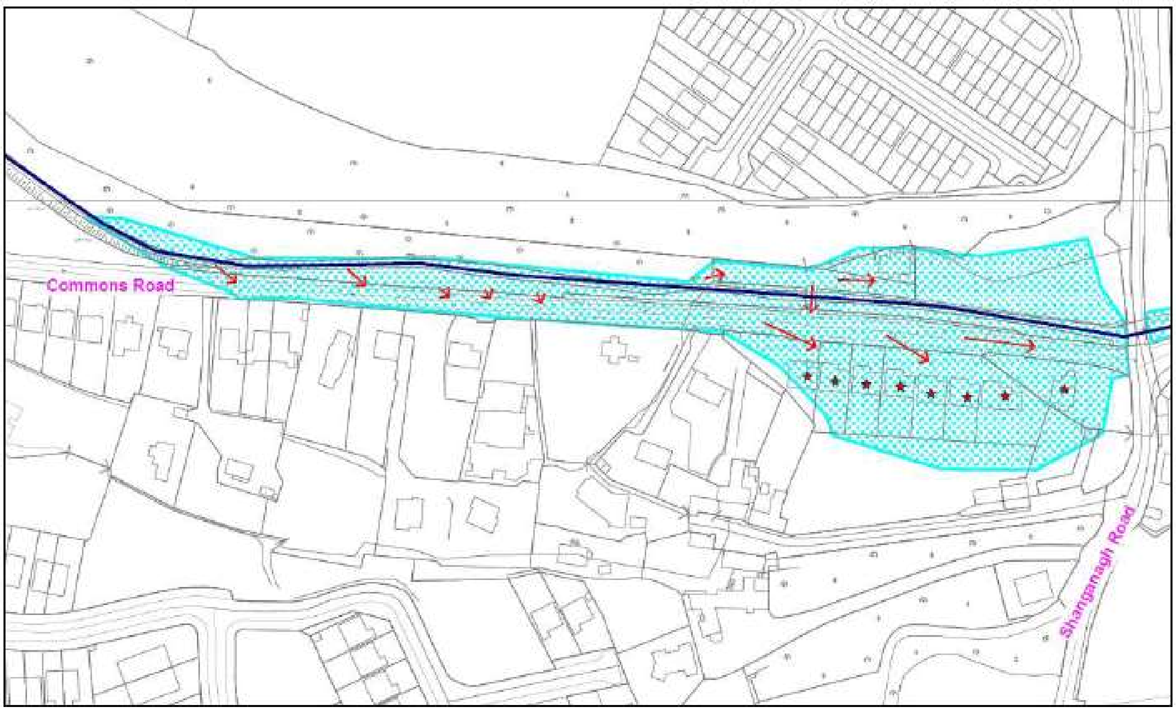


Figure 2-31: Properties impacted and recorded flood extents along Commons Road for the 2011 event (Source: Commons Road, Shankill Preliminary Assessment of Flooding Incident 24th October 2011, RPS)

3 Review of ECFRAM Hydrology

As the Eastern Catchment Flood Risk Assessment and Management (ECFRAM) study is the most recent and detailed hydrologic and hydraulic study to have been carried out in the area their hydrological methodology has been reviewed. This Section outlines the approaches taken in the ECFRAM study and comments on their applicability for use in the FRS study. Figure 3-1 shows the location of the Hydrological Estimation Points (HEPs) derived for the ECFRAM study for the Carrickmines/Shanganagh River and its modelled tributaries. Six HEPs were used:

- 2 located at the upstream extents of watercourses,
- 3 check flow HEPs located at gauge locations (former location of gauge 10022, at gauge 10021 and a check flow downstream of a confluence),
- 1 located at the model downstream boundary.

The HEPs are reasonably spaced along the watercourse and at key locations where hydraulic flows and flow estimates should be checked.

Figure 3-1: ECFRAM HEP locations

3.1 Index Flood Flow Estimation (Qmed)

3.1.1 Gauged Index Flood Flow

Commons Road (10021)

Within the ECFRAM study only the post-2005 data records have been used for any analysis on gauge 10021. This decision was justified in the ECFRAM Hydrology Report as due to the construction of M50 in 2005 the catchment flood run-off characteristics are considered to have changed and record before 2005 was deemed no longer representative.

To assess the gauge and derive a Qmed value a 'Catchment run-off model" was developed using MIKE 11 NAM & Urban runoff models. However, the Qmed value extracted from the gauge data was considered more suitable than the NAM model outputs. To provide greater confidence in the gauged Qmed value despite the short AMAX series available post 2005 (9 years at time of ECFRAM study) Peaks Over Threshold (POT) analysis was undertaken. This analysis resulted in a gauged Qmed value of 14.20m^3/s which was used within the study. This gauged Qmed value was used to derive a pivotal adjustment factor for the study area which was applied to all other ungauged HEPs in the Loughlinstown model and study area. The pivotal adjustment factor from the gauge was 1.96.

3.1.2 Ungauged Index Flood Flow

To calculate the Qmed for all ungauged Hydrological Estimation Points (HEPs) the FSU 7-variable ungauged catchment descriptor equation was used. Where available, gauge records or catchment run-off models were applied to adjust / improve the design flow estimations at these ungauged locations (NAM models and adjustment factors). The pivotal adjustment factor derived from gauge 10021 was applied to all Qmed estimates for the study area (factor of 1.96), there is no mention of any additional adjustment to the ungauged Qmed estimates from run-off models. Table 3-1 shows the final Qmed estimates for the study area.

Table 3-1: ECFRAM Qmed values (Source: ECFRAM Hydrology Report)

Node ID_CFRAMS	AREA (km^2)	Q_{med} (m^3/s)	Preferred Estimation Methodology
10_1211_1_RPS	2.36	2.03	FSU PCD Adjusted
10022_RPS Gauging Station (A1 pre 2005)	11.53	8.50	FSU PCD Adjusted
10_1518_4_RPS	9.90	3.51	FSU PCD Adjusted
10_1220_3_RPS	12.13	4.45	FSU PCD Adjusted
10021_RPS Gauging Station (A1 pre 2005)	30.65	14.20	Observed Q_{med}
10_1570_2_RPS	31.96	14.41	FSU PCD Adjusted

3.1.3 Review of ECFRAM Qmed estimation methods

The following points are noted in relation to the estimation of Qmed values in the ECFRAM study:

- For the gauged Qmed at Commons Road only data post 2005 was used. ECFRAM states that due to the building of the M50 motorway the catchment response has been altered and any data prior to this change is no longer representative. Review of the available gauge data highlights there is a noted change at this point in the record. Not only was the M50 constructed but work was also carried out around the gauge location in relation to the Commons Road flood relief scheme around this time which are also considered to have impacted the gauge record. These changes in the catchment and data support the decision to only use the post-2005 data. For the FRS scheme the same approach has been used.
- Since the ECFRAM study an additional 6 years of data have been recorded for the gauge. While the methods in the ECFRAM study are valid the additional available data means that the values need to be revised and updated for the FRS study. The additional data provides greater confidence in the Qmed value which has been updated considering the new information (refer to Section 2.3). This updated gauged Qmed impacts the pivotal adjustment factor derived from the gauge and those flows where it is applied.
- The FSU 7 variable equation was used to estimate Qmed for all ungauged HEP catchments. This method was designed for use on catchments greater than 25km^2, of the ECFRAM HEP catchments considered in this area only two are of suitable catchment size for this method. The use of an estimation method outside its recommended bounds introduces uncertainty in the estimation. There is no mention of other methods being considered for Qmed estimations for these smaller catchments.
- While the locations of the ECFRAM HEPs are suitable for a large-scale flood risk study there are a number of key watercourses not explicitly represented in the ECFRAM hydrology or study (e.g. the Cabinteely stream). The flow contributions are included in lumped FSU estimates and therefore the division of flow across tributaries and the impact of timings of peaks is not considered in the lumped HEP approach used.

3.2 Growth Curves Generation

Following a review of the ECFRAM Hydrology Report the growth curve approach taken is summarised as follows:

- Growth curves (GCs) were derived for all HEPs within the ECFRAM study area, to manage the number of GCs several general GC groups were derived based on catchment area.
- The GC shapes were reviewed visually, and the General logistic (GLO) distribution was found to be the best fit. No statistical analysis was carried out to assess the fit if the distribution.
- The Commons Road gauge (10021) catchment was in GC group 2 (catchments between 5 and 100km2). The recommended GC for catchments in this group was to apply the site-specific GC derived. It is mentioned that weighting between single site and pooled GCs was carried out but there is no mention of the weighting ratios applied.
- Further analysis and testing were carried out to cross check the GC values and the corresponding flood frequency curves (FFC) for gauged catchments.
- Review of the curves for the Commons Road gauge showed that the FFC generated by the pooling group was steeper than the gauge FFC. To ensure a conservative approach the steeper FFC curve was adopted for the gauge.

3.2.1 Review of ECFRAM growth curve generation

The methods used to generate the GCs in the ECFRAM study are sound however it is noted that there is a lack of information within the Hydrology Report needed to replicate the GCs (e.g., no statistical descriptors, no list of pooling group member or the weighting factors if used). Therefore, as there is no way to replicate and check the data used to generate the curve its reliability cannot be guaranteed and so it is recommended that the curves are not used in the used in the FRS study.

3.2.1.1 Further investigation into growth curve generation

As the ECFRAM GC cannot be replicated further research into potential GCs to use has been carried out. Table 3-2 shows the summary statistics for a range of GCs derived for the catchment and Figure 3-2 compares those GCs. There is a large amount of statistical variability between the growth curves with no clear indication of which is the most suitable. It is also noted that there is no statistical consistency within the study area when other estimation points are considered (refer to Table 3-2 and Figure 3-3). Further to this each curve has different limitations associated with it:

- As previously highlighted, there is no way to replicate the ECFRAM GC,
- The single site AMAX curve is limited by the lack of high flow recordings at the gauge,
- The FSU portal pooling groups include unsuitable gauges (not ranked A1, A2, or B) and only include AMAX data up to 2004 (if gauge 10021 included in pooling group it does not include changes to gauge behaviour discussed in Section 2.3.1),
- Rainfall growth curve based on Met Eireann DDF dataset which is derived from a separate depth duration frequency model and so any assumptions and uncertainties in the dataset are then carried through to the generated GC,
- FSR regional GC is derived for the whole of Ireland and is not catchment specific.

Of the available GCs the DDF rainfall growth curves have been used in the FRS HEP flow estimations. The statistical variability of the other GC options highlights the lack of certainty as to whether they are representative. The DDF curve values have been estimated for each HEP and the flows refined and checked against the generated flows for the gauge 10021 AMAX growth curve where appropriate to assess suitability. From Figure 3-2 which focuses on the Commons Road gauge HEP there is good agreement between the gauge AMAX GC and the rainfall GC for the lower events up to the 1 in 30-year event for which the AMAX GC estimations for the gauge are considered reliable. This provides confidence that the rainfall curve will provide a reasonable estimate of flows. As the DDF provides data for higher return periods from modelled outputs it also provides more confidence for the larger events compared to the AMAX GC.

Table 3-2: Statistical summary of a range of available growth curves

	L-skew	L-CV	L-kurt	Heterogeneity (H)	Best fit distribution
ECFRAM	NA	NA	NA	NA	GLO
Single Site (Post 2005)	0.145	0.178	-0.009	NA	GLO
FSU (Euclidean group)	0.174	0.216	0.157	5.618	EV1
FSU (Geographical group)	0.259	0.213	0.201	26.135	EV1
Rainfall growth curves (DDF)	NA	NA	NA	NA	NA
FSR regional growth curve for Ireland	NA	NA	NA	NA	NA
FSU (Euclidean for HEP upstream of gauge)	0.218	0.238	0.197	32.965	GLO

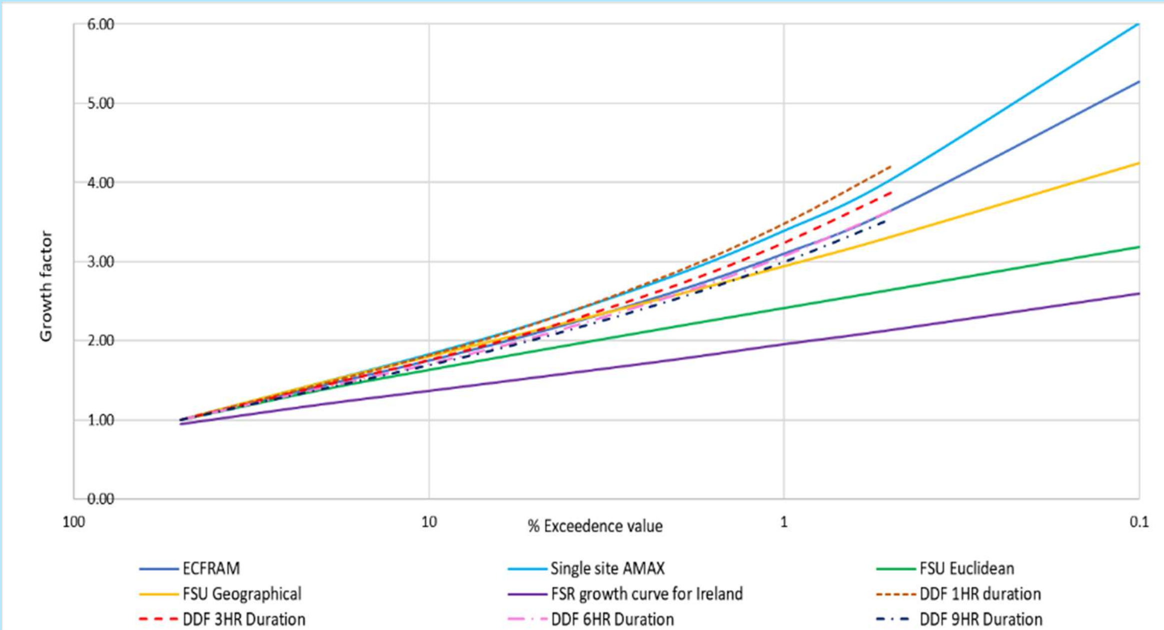


Figure 3-2: Derived growth curves for Gauge 10021

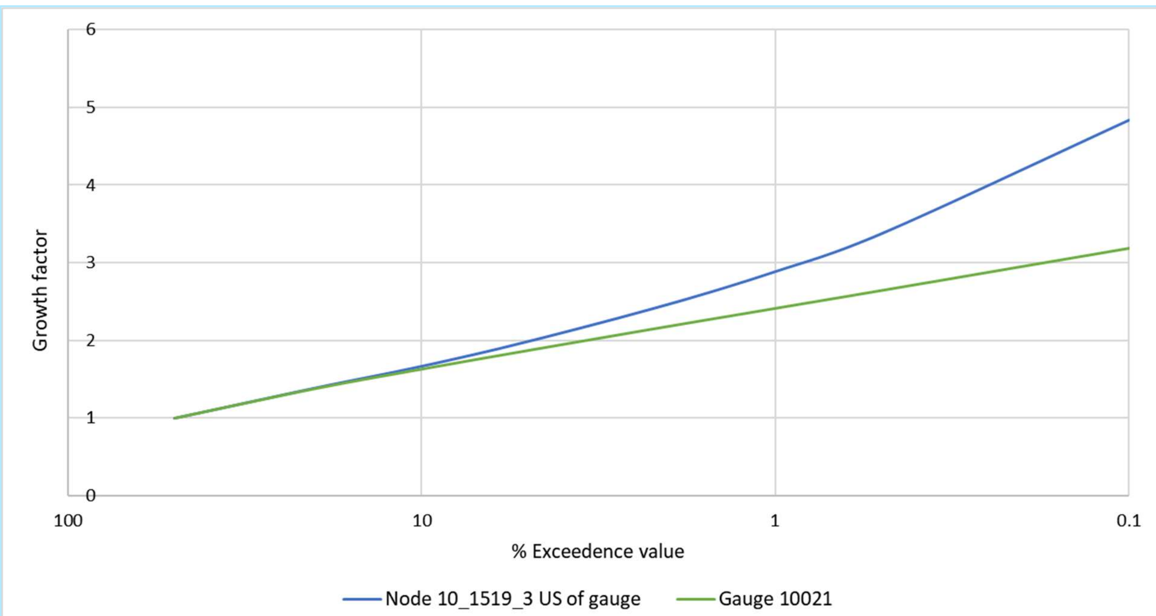


Figure 3-3: Comparison of growth curves derived for gauge 10021 and a node located upstream

3.3 Comparison of ECFRAM flows and updated FRS HEP flows

Following a review of the ECFRAM flow estimation and growth curve methodology a comparison between the flows estimated in this FRS study and those from ECFRAM has been carried out. Several of the FRS HEPs have been placed at the same location as the ECFRAM HEPs to allow for a like for like comparison. The following differences between the two estimations are summarised:

- The Commons Road gauge has been reviewed and the additional years of data since the release of ECFRAM taken into consideration (refer to Section 2.3.1). The gauged Qmed has since increased which changes the adjustment factor.

- A full review of catchment descriptors has been carried out with updated and more detailed datasets used (refer to Section 2.2.1 and Appendix A). The FRS HEPs therefore differ to the ECFRAM HEPs in terms of baseline data.

- The FSU method has not been applied to all HEPs as in ECFRAM. Each HEP was reviewed and the appropriate method applied. Other methods including FSR RR, FSU small catchments and IH124 were used in assessment.

- The Commons Road gauge (10021) pivotal adjustment factor was not applied to all HEPs as in ECFRAM. Review of the HEPs showed in some cases the differences between descriptors highlighted that the Commons Road gauge was not a suitable donor gauge and to apply the adjustment could potentially result in misrepresentation of catchment flows.

- MET Eireann DDF growth curves were applied to all HEPs instead of pooled growth curves as in ECFRAM. Refer to Section 3.2.1 for more detail and justification for this.

Refer to Appendix B for comparison between FRS traditional flow estimations at ECFRAM HEP locations and Appendix B for all FRS HEP design flows. Overall, the new estimated flows are generally consistent with the ECFRAM flows despite updates in catchment descriptors and methods used. The consistency of the estimations provides confidence that the flows are representative. The updating of the catchment descriptors and methods applied across the catchment for the FRS ensures that the most appropriate methods are being used in each instance. Therefore, the FRS HEP estimates will be used as the key check flows for the model derived flows.

3.4 Design Flow Hydrographs

From the ECFRAM Hydrology Report semi-dimensionless flow hydrograph shapes were derived from past events using the FSU Hydrograph Width Analysis (HWA) software for use in the design events. For the few locations with very small catchment area, or where a pivotal site to derive shape was not possible to select, the FSSR 16 Unit Hydrograph method was used to generate hydrograph shape. These methods provide a single smooth hydrograph shape for inflows and check flows into the model. Figure 3-4 shows examples of the final design inflow hydrograph shapes for the Carrickmines/Shanganagh watercourse used in the ECFRAM study.

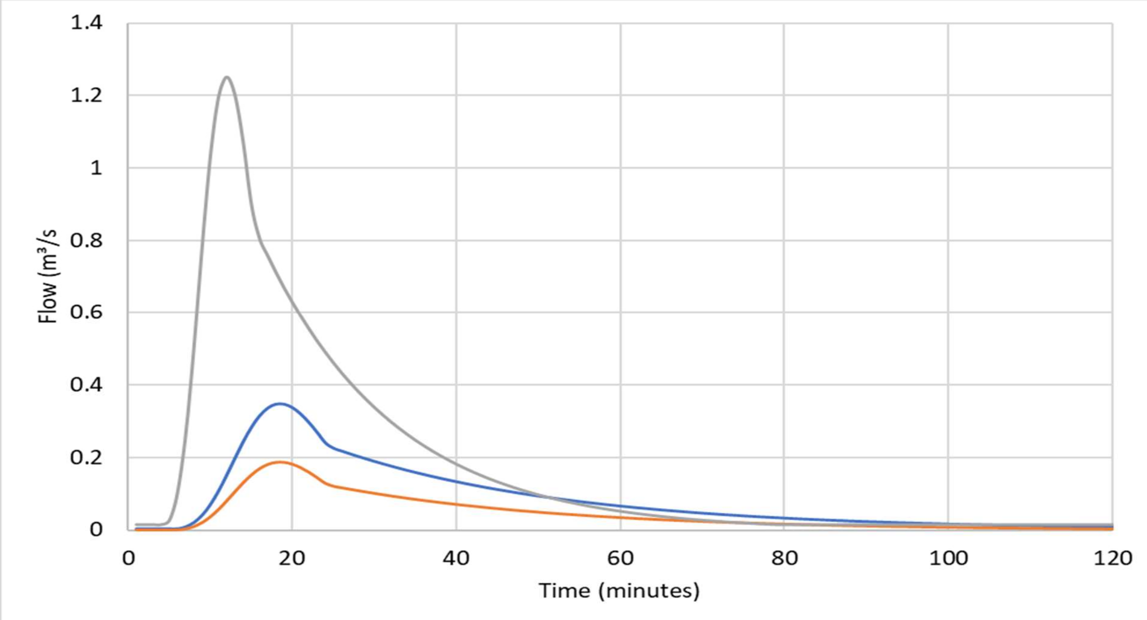


Figure 3-4: Example design flow hydrograph shapes used in the ECFRAM study

3.4.1 Review of ECFRAM flow design flow hydrograph generation

While the approach taken in the ECFRAM study is suitable for a wider scale study the lumped estimation and application approach used in ECFRAM means that the timing of different sub-catchment flows, and volumes is not fully captured. As a result, the resolution of phasing of peak flows is lost within the design hydrograph shape. This proves problematic within the study area as the sub-catchments are variable and their responses flow hydrographs and response times to a rainfall event will differ which in turn impacts the overall understanding of the system response and where the critical flows are coming from. A more detailed and refined approach to developing design hydrographs factoring in these considerations was required for the development of the FRS. A rainfall routing model was developed for generating flow hydrographs using DDF rainfall data in this study and is discussed further in Section 4.2.

3.5 Summary of ECFRAM review

From the review of the ECFRAM hydrological approach the following key observations are noted:

- The FSU method has been used to derive Qmed values for the study and a pivotal adjustment factor based on the Commons Road gauge has been applied. Since the completion of the ECFRAM data the gauge record has been extended. Review of the updated gauge data not only changes the gauge Qmed but also any pivotal adjustment factor that could be applied to other Qmed estimates.
- It is noted that the FSU 7-variable equation has been used to generate Qmed at all ungauged locations. This means in some cases the catchments considered are not within the recommended limits of the method, there is not discussion or consideration of other Qmed estimation methods for smaller catchments on the ECFRAM Hydrology Report.
- While a pooling group and growth curve was derived for the Commons Road gauge catchment there is no way to replicate the curve and therefore there is uncertainty associated with it. With no way of replicating the data or having any knowledge of the pooling group the GCs used in ECFRAM are not recommended for use in the FRS.
- FSU and FSR methods were used in ECFRAM to obtain inflow hydrographs. These methods generate averaged hydrographs which do not reflect variation in inflows and phasing of flows which considering the variability in the sub catchments within the study area is an important aspect to

consider. While suitable for a wide scale flood mapping study greater understanding of flow phasing is needed for the development of an FRS.

4 Design Flow Estimation – Rainfall Runoff Modelling

4.1 Introduction

From reviewing the available hydrological data for the catchment, it was found that the methods used in the ECFRAM study were not considered suitable for use in deriving the FRS flow estimations as there was need for more detail in relation to sub-catchment response times, peak phasing, and key areas of flow contribution. These key factors allow a better understanding of the catchment wide flood response and allow a more refined approach in developing the FRS.

Therefore, new hydrological flows were developed for the FRS using a rainfall routing model approach. The advantages of this approach included:

- The variability of sub-catchment response within the study area can be better understood in terms of flow volume and peak phasing through the system,

- The rainfall growth curve can be cross checked against gauge AMAX data for the Commons Road gauge (10021) allowing greater confidence and certainty for higher flow events compared to the FSU method curves,

- The rainfall routing model was cross-checked and further refined by using data from the flow monitoring programme for the tributaries in the system and Commons Road gauge data. The flow monitoring programme allowed understanding of peak phasing across the catchment and helped ensure the runoff model is representing flow proportionally for given catchment areas and provided validation data for the hydraulic model.

- The rain gauge network within the system was used for refining the rainfall inputs into the model ensuring that spatial variability is captured, something that cannot be explicitly done in the traditional estimation approaches.

- Development of a routing model allowed for improved the integration of the hydrological and hydraulic models for the study allowing for greater efficiency.

Hydrological estimation was combined with the hydraulic model using the rainfall-runoff routing capabilities within the InfoWorks-ICM software package. A simplified 1D hydraulic model was developed alongside the integrated 1D/2D model combining the existing ECFRAM and GDSDS models in InfoWorks ICM to test a range of runoff and routing models. The use of the hydraulically simplified model allows for more efficient run time and great flexibility in the development of the preferred rainfall runoff approach with rainfall applied spatially and routed using sub-catchments representing the varying land uses across the catchment. The following sections provide detail on how the hydrology was applied into the model and design flows generated.

4.2 Application of Hydrology – Rainfall Runoff Approach

4.2.1 Rainfall

A single design storm temporal pattern was applied across the catchment with a hyetograph shape generated using the FSR estimation for both summer and winter profiles for all design events. The hyetograph shapes were scaled to match the design storm volume based on the DDF total rainfall depths calculated for each return period. This allows spatially varying rainfall depths to be applied across the catchment through the Met Eireann DDF 2km gridded dataset. Summer and Winter profiles were calculated and compared to identify the critical storm conditions in the catchment.

Due to the complexity of the catchment and the varying responses across the different tributaries, the model does not have one critical duration. Different durations for the HEPs along different watercourses were identified in the FSU hydrology method assessment, and the routing model has also generated similar critical durations. The main area of interest with regard to flood alleviation is the area surrounding Commons Road and Brides Glen, which have critical storm durations of 3hours and 9-hours respectively for the summer storm when considering the peak flow. As part of the development of flood alleviation options a range of storm durations will be considered to ensure that any proposed options are not sensitive to storm duration. This can be particularly important when considering the volume of potential storage solutions for example. It is noted that the critical duration has been shown to vary throughout other catchment in the study area, for example the Racecourse Stream has a critical duration of 1 hour, which is driven by the response from the urban area and M50 runoff in this catchment, and therefore alternative storm durations may be required should alleviation measures be considered in these areas. This will be further assessed as part for the fully linked 1D-2D model as this may provide additional flow attenuation as a result of the detail representation of all structure and floodplain storage that are not explicitly modelled in the 1D routing model.

4.2.2 Runoff

Runoff in the Carrickmines FRS model was simulated using 1D sub-catchment within the ICM software. The runoff characteristics vary significantly across the study area, from the rural upper catchment to the urbanised areas in the middle and lower catchment. ICM allows this variation in land-uses to be included in the model using different runoff surfaces for impervious areas such as roads and roofs as well as permeable areas such as gardens, parks and the rural upper catchment.

The existing GDSDS stormwater model included a simplified representation of the watercourses with the catchment and sub-catchments that covered the full area. The GDSDS sub catchments were checked to identify any major changes to the model (see section 4.2.2.1). The rural areas of the model were updated with new sub-catchments to take account of the changes in topography and underlying soil and geology (refer to Figure 4-1). The sub-catchment approach also allowed runoff from key areas such as the M50 motorway and more recent developments to be represent directly within the model.

The previous inflows used in the ECFRAM model were replaced by the inputs from the catchments.

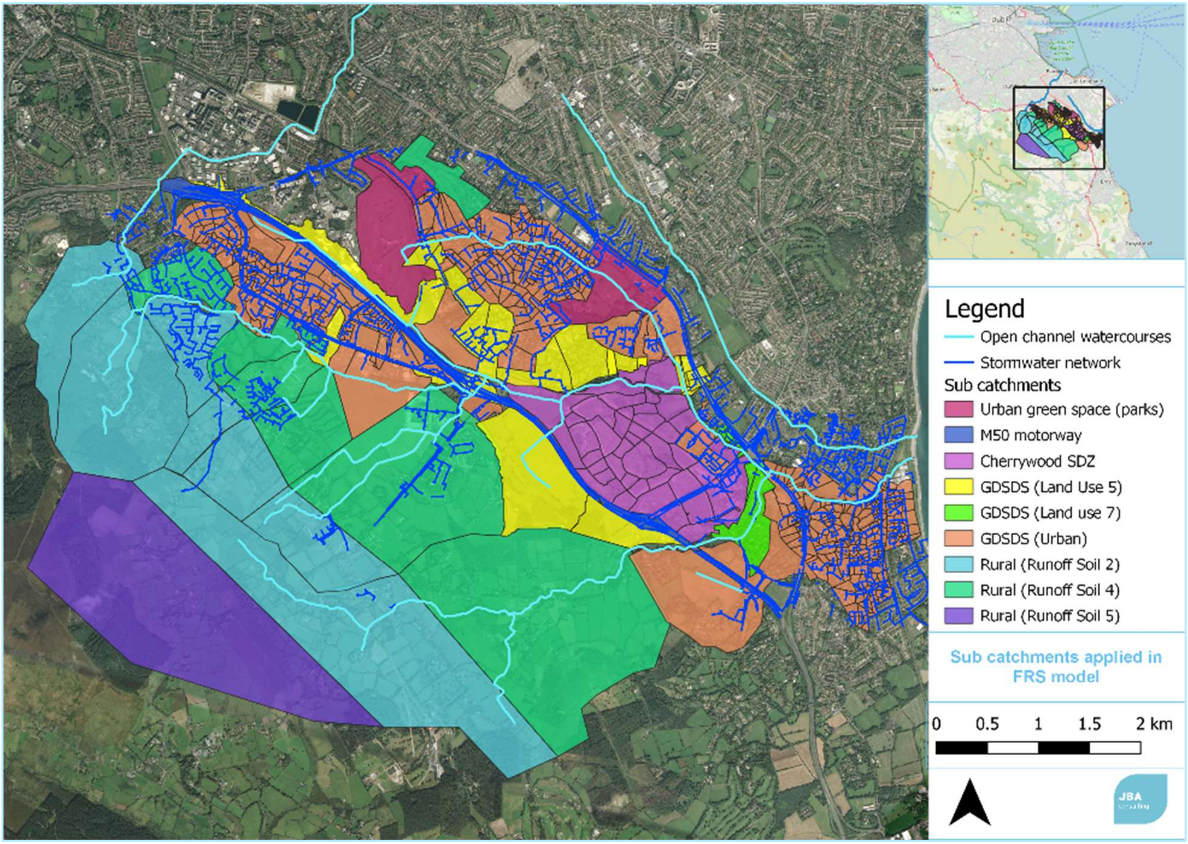


Figure 4-1: Sub catchments applied in FRS model (base map source: Bing Satellite, OSM standard)

4.2.2.1 Land use

InfoWorks ICM uses Land use IDs to differentiate the different land use areas within the model. Each Land use ID is made up of up to 10 different runoff surfaces, which can be used to define different characteristics. Each sub-catchment within the model was allocated the most suitable Land use ID based on available land use data from the CORINE 2018 land use dataset.

The runoff surfaces are used to define the both the runoff volume and runoff routing model for rainfall applied to that surface. ICM includes a range of models volume and routing model and the recommended models are described in the following paragraphs.

Urban areas

The existing GDSDS stormwater model included six different Land use IDs and 6 different runoff surfaces as shown in the Table 4-1 and Table 4-2. Each of these run-off surfaces has a runoff coefficient associated with it. The urban areas retained from the GDSDS model use Land Use ID 1 for Urban areas, which splits the sub catchments into runoff surface areas 6 and 7 which both represent impervious areas that are positively drained. Comparison between the previous model surfaces and up-to-date mapping was carried out to identify areas where development had taken place since the previous studies. The catchments were updated to reflect these changes. Within this process, it was assumed that all developed areas since the ECFRAM study are constructed with SUDS features which maintain discharge from the sites at the greenfield runoff rate.

A summary table of the land use id’s and run off surfaces used within the GDSDS model which has also been used as a basis for the urban areas of the Carrickmines FRS model is provided in Table 4-1 and Table 4-2.

Table 4-1: Land use IDs

Land Use ID	Connectivity	Description	Run-off Surface 1	Run-off Surface 2	Run-off Surface 3
DEFAULT	100%		1		
1	100%	URBAN	6	6	7
4	100%	RURAL 3	6	6	24
5	100%	RURAL 4	6	6	25
6	100%	RURAL 5	6	6	24
7	100%	RURAL 6	6	6	26
M50	100%	M50 Runoff	610		

Table 4-2: Runoff surface IDs

Runoff Surface ID	Description	Surface Type	Runoff volume Type	Initial Loss Value (m)	Fixed Run-off Co-efficient	NewUK Depth (PF) (m)	Routing Model
1		Impervious	Fixed	0.000071	1		Wallingford
6		Impervious	Fixed	0.000071	0.75		Wallingford
7		Impervious	Fixed	0.000071	0.70		Wallingford
24	new PR fast	Pervious	NewUK	0.0009		0.15	Wallingford
25	new PR slow	Pervious	NewUK	0.0009		0.25	Wallingford
26	new PR slow PF250	Pervious	NewUK	0.0009		0.3	Wallingford
610	M50	Impervious	Fixed	0	0.9		Wallingford

It is understood that runoff from the M50 is unattenuated through the model, and therefore a specific landuse for the highway, “M50” was included. The runoff surface has a runoff type set to fixed, a runoff coefficient of 0.9, a volume routing model of Wallingford.

Rural areas

For the sub-catchments in the upper catchment, e.g., those with and areas greater than 1ha and also not greenspace within the urban areas, the New UK Wallingford routing model is not considered to be suitable. Therefore, a Large Contributing Area routing model was used. It provided an improved representation of flow routing by using two equal linear reservoirs in series, whose routing coefficient depends on rainfall intensity, contributing area and surface slope as in the Wallingford model. The runoff volume type was set as fixed, as it involved the least uncertainty, although other alternative methods were considered. The information provided in the catchment descriptors included the WRAP classes, and therefore three separate runoff surfaces were made; one for WRAP class 2, one for WRAP class 4 and one for WRAP class 5. The fixed runoff coefficient was set to the SPR of the soil class. Large catchments, such as the Brides Glen, were split as they overlapped several WRAP classes, and were set to drain to each other.

Table 4-3: Rural Landuse ID

Land Use ID	Connectivity	Description	Run-off Surface 1	Run-off Surface 2	Run-off Surface 3
Upper Catchment_Fixed_2	100%	Rural Large Catchment	263	263	
Upper Catchment_Fixed_5	100%	Rural Large Catchment	267	267	
Upper Catchment_Fixed_4	100%	Rural Large Catchment	265	265	
5 (CAB Update)	100%	RURAL 4	6	6	301

Table 4-4: Rural runoff surface IDs

Runoff Surface ID	Description	Surface Type	Runoff volume Type	Initial Loss Value (m)	Fixed Run-off Co-efficient	NewUK Depth (PF) (m)	Routing Model
263	New PR slow – Soil 2	Pervious	Fixed	0.0009	0.3		Wallingford
265	New PR slow – Soil 4	Pervious	Fixed	0.0009	0.47		Wallingford
267	New PR slow – Soil 5	Pervious	Fixed	0.0009	0.53		Wallingford
301	Fixed runoff of 0.4 based on the Cabinteely report	Pervious	NewUK	0.0009	0.4		Wallingford

For the rural catchments the previous GDSDS runoff catchments were replaced to improve the understanding of response of the different watercourses within the study area. The existing Rural Land use IDs were retained however as these catchments are predominantly permeable additional Land use ID and runoff surfaces will be included in the model.

4.2.2.2 Runoff volume model type

The run-off volume model determines how much of the rainfall applied runs off the catchment into the drainage system after accounting for initial losses. A runoff volume model is chosen in InfoWorks for the sub-catchments. There are two types of model;

- Total catchment models – Applied to all surface types in a sub catchment,
- Individual models – can be applied to one surface type of a sub catchment.

The run-off models chosen for the Carrickmines FRS are Fixed percentage run-off models (individual Model) and New UK Variable PR Model (Total catchment)

Fixed percentage runoff model

Runoff losses after initial losses can be defined as fixed independent of antecedent conditions. This type of representation is only advised for use with impervious areas, or pervious areas where runoff does not vary significantly with antecedent conditions. The model defines a fixed percentage of the net rainfall, which becomes runoff. Different coefficients were used for different areas of the catchment particularly and typical values used within InfoWorks for fix run-off coefficients are provided in Table 4-5.

Table 4-5: Typical fixed percentage run off values (Infoworks ICM)

Description	Coefficient
High quality paved road with gullies	1.0
High quality paved roads with gullies	9.0
Medium quality paved roads	0.85
Poor quality paved roads	0.8
High density housing	0.55
Medium Density Housing	0.45
Low Density Housing	0.35
Open Areas	0.25

New UK Variable PR Model

This is a UK model which represents the condition of the catchment changing throughout the model simulation for a pervious surface. This is usually used when it is important to take account of the change in catchment wetness during long storms. The derived model is of the form:

$$PR = IF * PIMP + (100 - IF * PIMP) * \frac{NAPI}{PF} \quad (1)$$

where

IF - effective impervious area factor

PF - moisture depth parameter (mm)

NAPI - API30 derived from net rainfall after subtraction of running depression storage

The NewUK Depth parameter (PF), is one of the factors which determines the volume of runoff from pervious surfaces defined in the runoff model.

4.2.2.3 Routing model type

Runoff routing models determine how quickly the rainfall enters the drainage system from the catchment. Two routing models were used in the Carrickmines FRS model; the Wallingford Routing Model and the Large Contributing Area Routing Model.

Wallingford Routing Model

The Wallingford model is applicable to typical urban catchments. It uses a regression equation to predict the runoff coefficient depending on the density of development, the soil type and the antecedent wetness of each sub catchment. The model predicts the total runoff from all surfaces in the sub-catchment, both pervious and impervious surfaces. Runoff losses are assumed to be constant throughout a rainfall event and are defined by the relationship:

PR = 0.829 PIMP + 25.0 SOIL + UCWI -20.7 where,

PR = percentage run-off

PIMP = percentage impermeability

UCWI = Urban Catchment Wetness Index

SOIL = Soil Index (based on winter rain acceptance parameter (WRAP). Typical Soil classes used within Infoworks ICM are identified in Table 4-6;

Table 4-6: Soil classes

Soil Class	WRAP	Runoff	SOIL
1	Very High	Very Low	0.15
2	High	Low	0.3
3	Moderate	Moderate	0.4
4	Low	High	0.45
5	Very Low	Very High	0.5

Large Contributing Area routing model

The Wallingford model was designed to be used with catchments with an area of less than 1ha and therefore the Large Contributing Area Routing model was developed.

To give an accurate match of flow characteristics, the modified runoff routing model contains two elements that delay and attenuate the peak discharge from the single pipes. The two elements are:

- Routing coefficient multiplier
- Runoff time shift

These are applied to the standard (DLR/Wallingford model) routing coefficient and output, respectively.

4.2.2.4 Cherrywood SDZ Planning Scheme

The Cherrywood SDZ is currently under construction. The development covers 360ha and is to include 8700 residential units as well as office spaces, schools, and parks. The storm water for this development is to be managed by an on-site drainage network which will discharge into five storm water attenuation ponds which feed into the surrounding watercourses and network (refer to Figure 4-2 for pond locations). The overall aim of the development is to discharge at a rate of 1l/s per hectare to preserve the current discharge rate from the land.

There are 8 development areas planned for cherrywood across the site. Each development area included a breakdown of the proposed landuses in that area, and the area of these. These areas were schematised and added to the model, with 4 unique landuse IDs: "Cherrywood Residential". "Cherrywood Mixed Use", "Cherrywood Green Infrastructure" and "Cherrywood Greenspace". All of these sub catchments were then drained to an inflow point at the location of the ponds. A limiting discharge was then applied to these points in order to set the discharge rate to either 4.82ls/ha at the existing pond 4 (as specified in the Pond 4 capacity assessment), or to 1l/s/ha for the remaining ponds.

The flow contribution from this development is represented in the model via point inflows for each of the five attenuation ponds which will feed into the model. The proposed discharge rates for the ponds are based on values derived for a previous general assessment of stormwater on the site carried out by JBA. As the work is ongoing and design still in progress the final discharge values may be subject to change but the values used will provide an appropriate representation of the future discharge rate for the development. The pond discharge rates estimated for the 1% AEP event have been used for all design events, this provides a conservative approach to flow contribution from the site, and it is noted that the discharge rate would only vary slightly for lesser events. The allowed discharge rates for the development have been limited to 1l/s/ha (greenfield runoff rate).

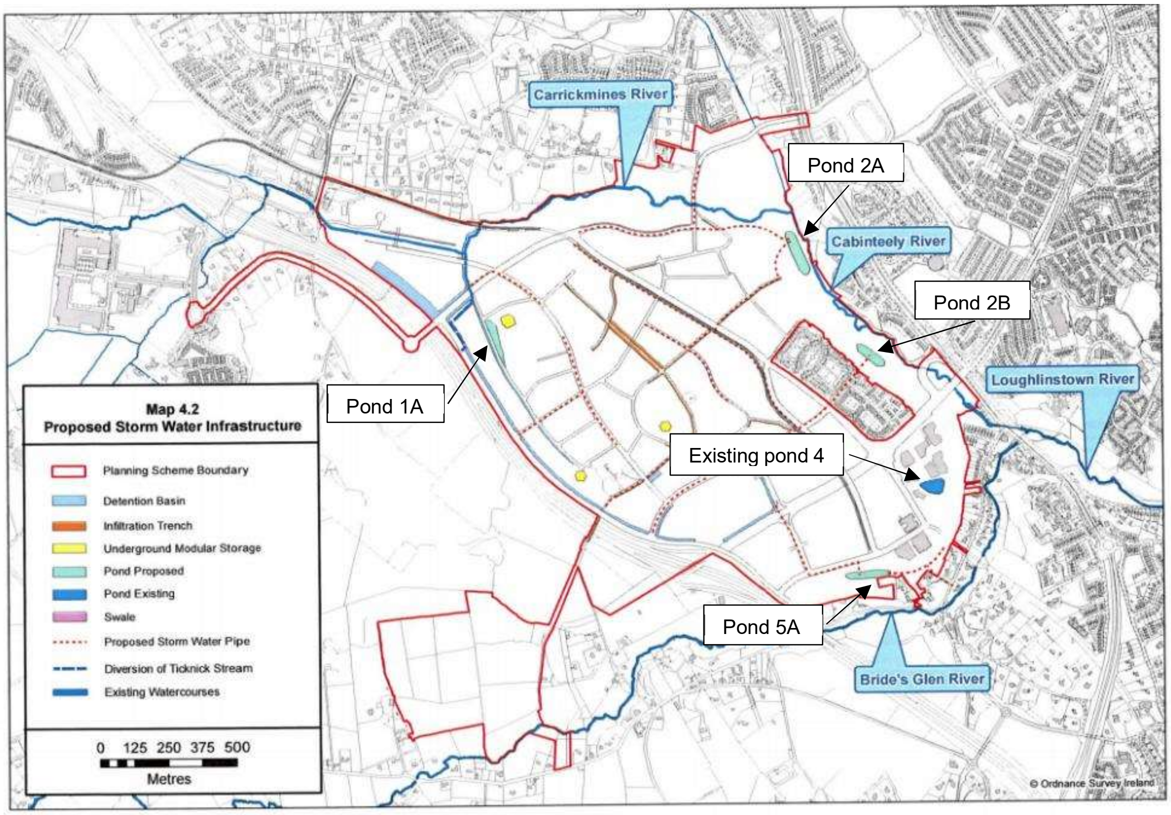


Figure 4-2: Proposed Storm water infrastructure with attenuation ponds labelled (source: www.dlrcc.ie (Cherrywood SDZ planning scheme))

4.3 Reporting Network - Cross checking of flows at HEPs

To ensure the flows output from the rainfall routing model were appropriate flow estimates using FSR hydrological estimation methods were carried out for each HEP identified in Section 3. A variety of Qmed estimation methods were used depending on catchment characteristics for the HEPs. The DDF rainfall growth curves were used for all HEPs with the appropriate storm duration curve applied to each catchment. Table 4-7 shows a sample of flow comparisons for the Commons Road gauge (10021) HEP. Flow comparisons for all HEP locations can be found in Appendix B. From the Table the routing model predicts lower flows compared to the FSR and FSU methods that have been discussed above although the flows are within the same order of magnitude. Overall the flows appear to be in reasonable agreement.

Table 4-7: Comparison between FSR and routing model flow estimations at Commons Road gauge

	50% AEP		**1% AEP**		**0.1% AEP**	
	Traditional method	**Routing Model**	**Traditional method**	**Routing Model**	**Traditional method**	**Routing Model**
HEP_040 (Commons Road gauge)	14.82 (Gauged Qmed)	14.37	47.94	35.9	78.40	39.93*

*1,000 year rainfall uplifts to be reviewed and confirmed.

4.4 Reporting Network – Cross checking of routing model with short term monitoring

Section 2 outlines the short-term flow monitoring study that was conducted in order to understand flow responses across the catchment. During the 13-week period, 4 notable rainfall events occurred, however it should be noted that none of which were considered to be significant flood events and, in all cases, the recorded flow at the Commons Road gauge was well below the gauged Qmed value. The following figures show comparisons of timings of flows within the catchment at key locations, flow monitors at Commons Road Gauge (FM01), Upstream Carrickmines river (FM06), Racecourse Stream (FM14) and the upstream end of Brides Glen (FM02). These gauges have been selected as they represent a range of catchments from predominantly rural catchments at FM06 and FM02 to a more heavily urbanised catchment at FM14 and the permanent gauge at Commons Road. The rural catchments show a good fit between the timing of the flows and show an elongated hydrograph compared with the flashier response shown in FM14. This is particularly the case in the 8th May event, which is most similar to the design hyetographs. It is noted that the predicted flows are greater than those recorded during the flow survey, however there is a good fit between the timing of the hydrographs. The increased modelled flow is due limited representation of the dry antecedent conditions in the fixed runoff model, however as the model is being developed as a flood model to assess the impacts of high order events it is not optimised for lower flow events (such as those recorded on the 8th and 20th May 2021). The comparison of the modelled flows with the permanent gauge record, particularly for the Qmed, which has the greatest level of confidence, shows that the model provides an improved representation of peak flood flows.

A full review of the temporary gauge information considering travel times between gauges in both the short-term survey and design events will be included in the hydraulic model report.

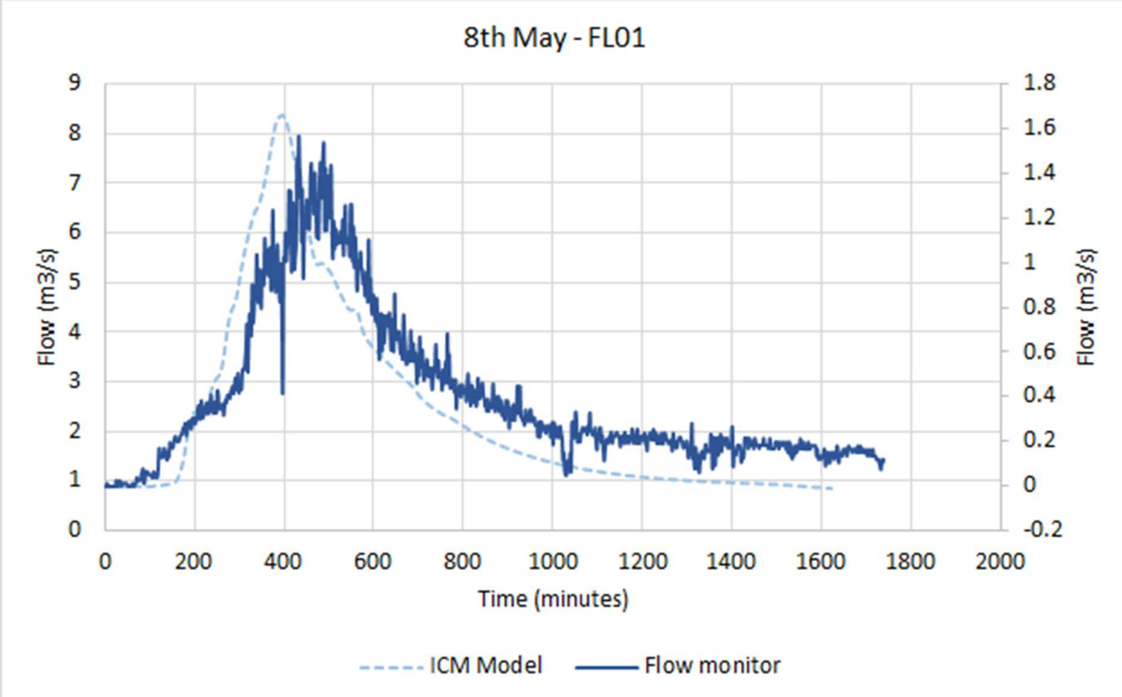


Figure 4-3: Flow comparison FL001 – 08/05/2021 event

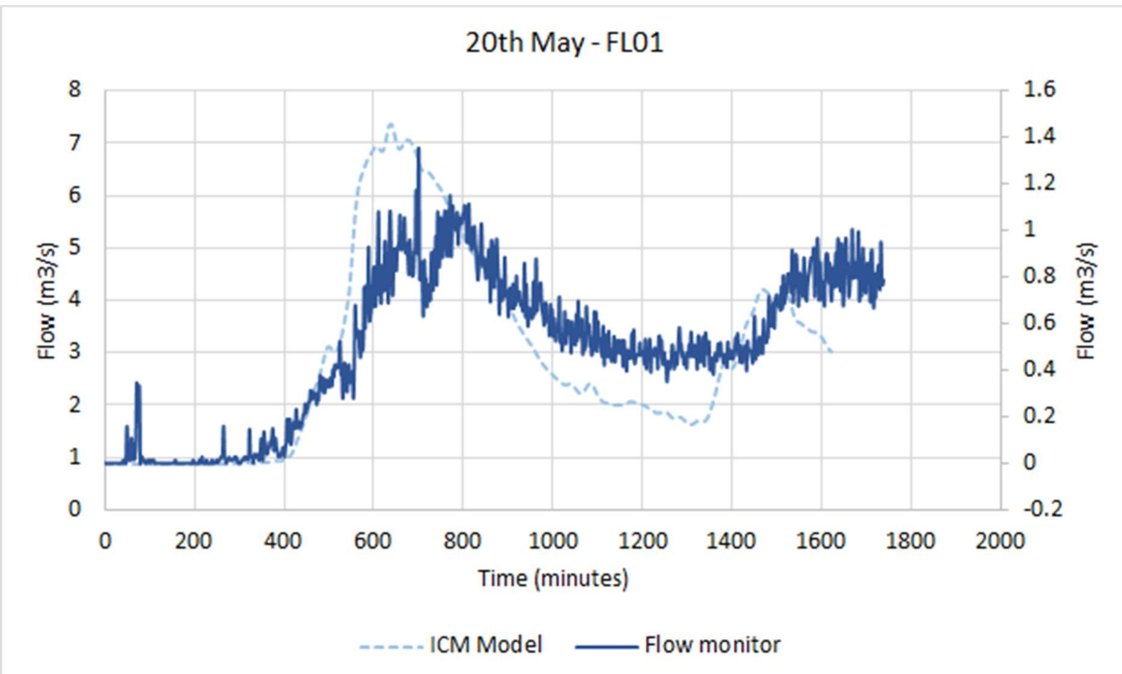


Figure 4-4: Flow comparison FL001 – 20/05/2021 event

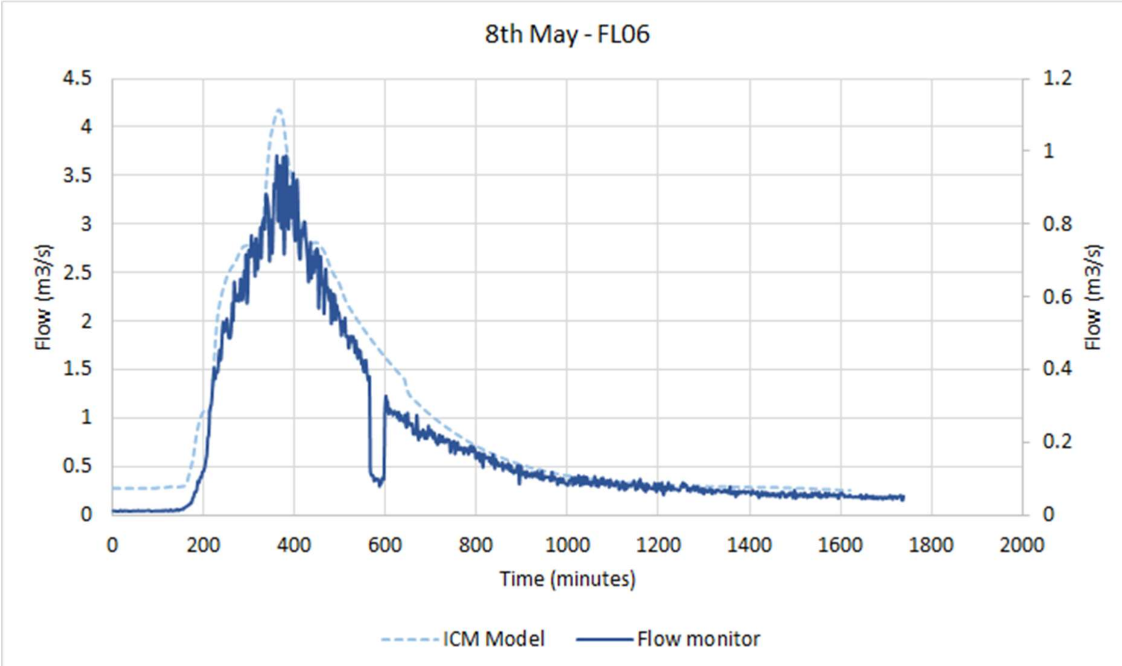


Figure 4-5: Flow comparison FL006 – 08/05/2021 event

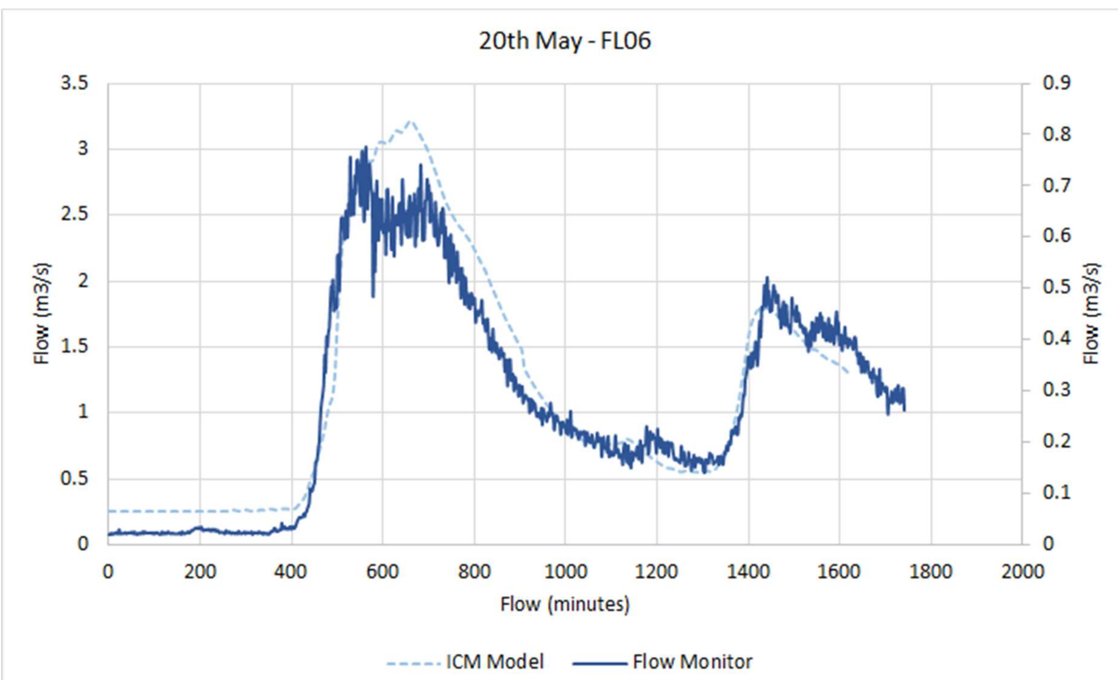


Figure 4-6: Flow comparison FL006 – 20/05/2021 event

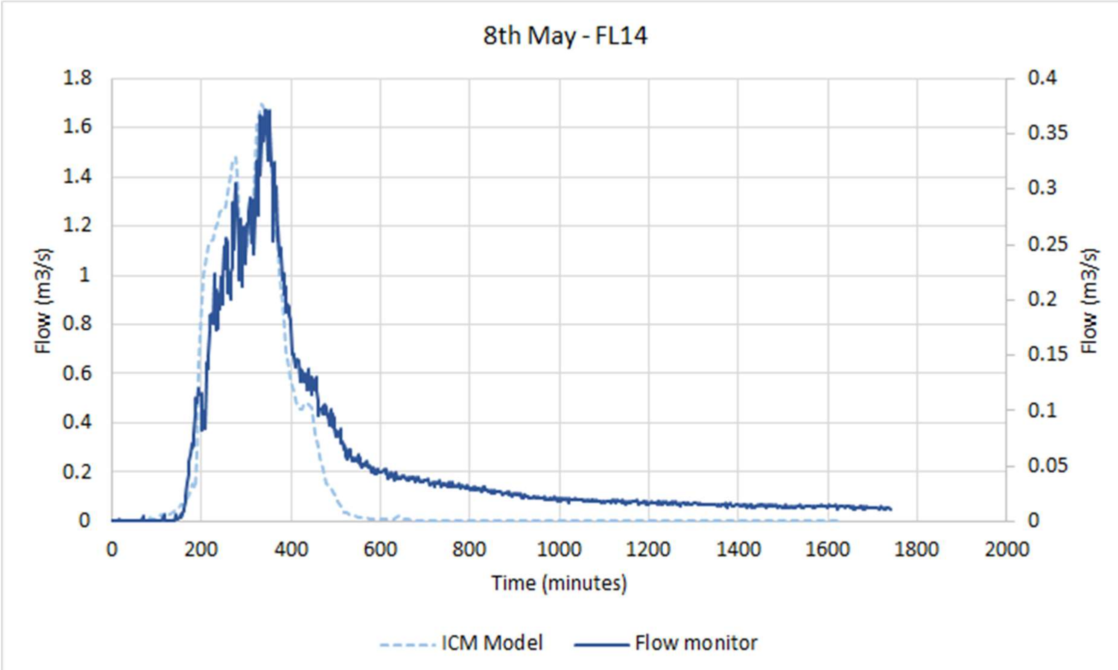


Figure 4-7: Flow comparison FL014 – 08/05/2021 event

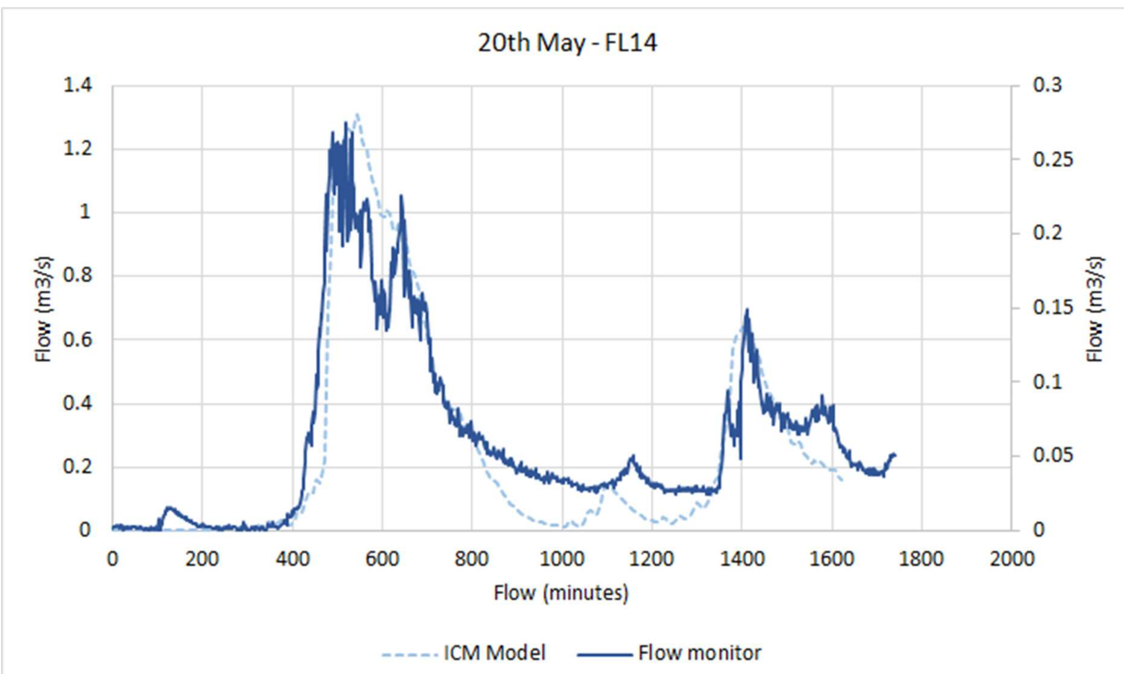
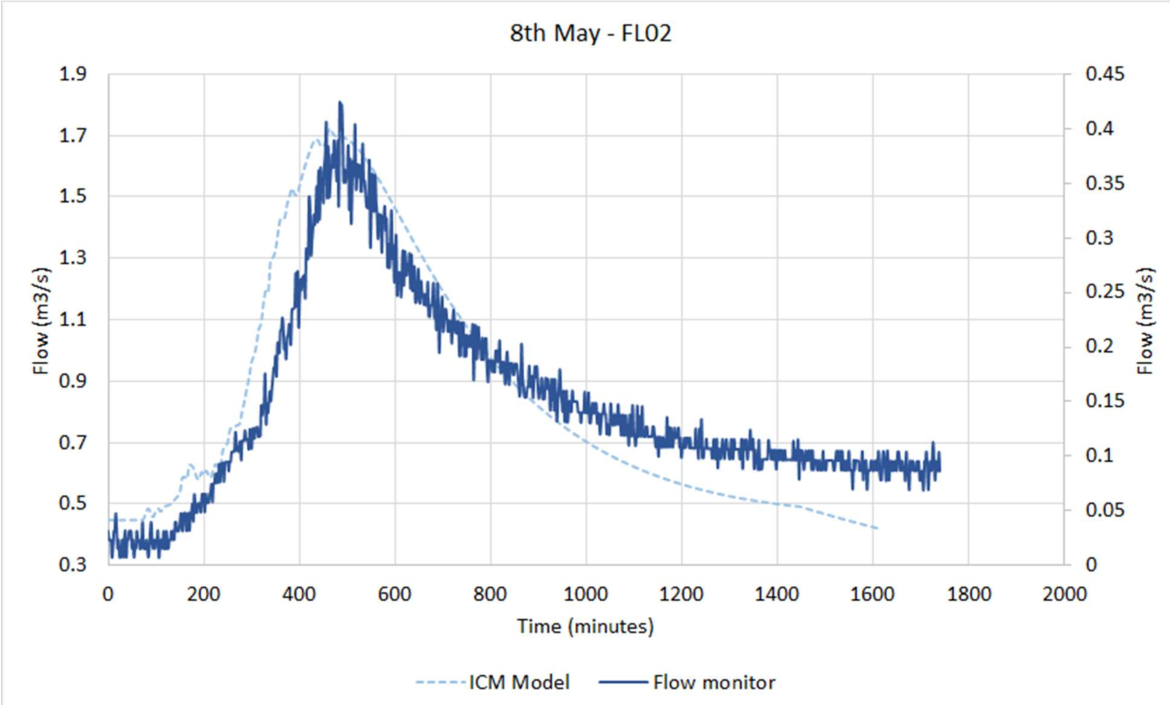
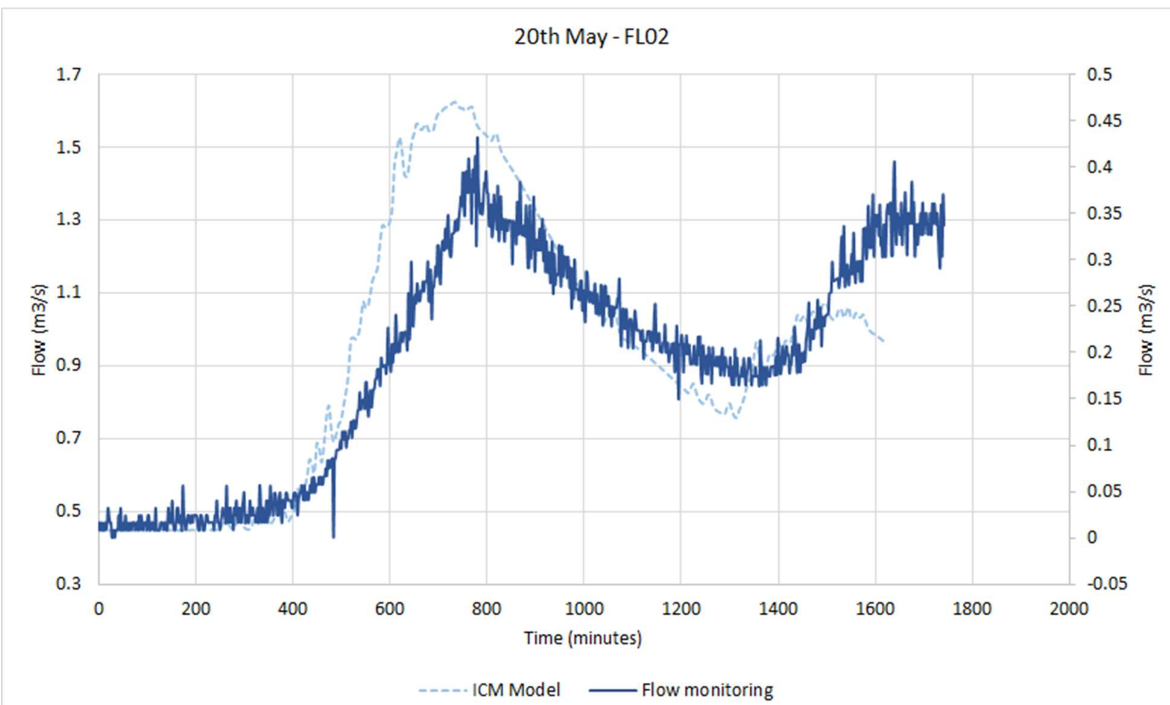


Figure 4-8: Flow comparison FL014 – 20/05/2021 event





4.5 Downstream boundary – Tidal levels

The downstream extent of the hydraulic model and study area is the Irish Sea. To ensure that tidal influences are appropriately represented a head-time boundary will be used at the downstream extent. The levels for this boundary have been sourced from the Irish Coastal Wave and Water Level Modelling Study 2018 (ICWWS) level data which is the most detailed and up to date source of tidal information for the area. Level data will be extracted from node SE4 which is the closest output location from the ICWWS to the study area and downstream boundary (refer to Figure 4-9).

Figure 4-9: Location of ICWWS tidal level node (base maps: Bing Satellite and OSM Standard)

The ECFRAM study sourced their tidal boundary data at the same node location from the Irish Coastal Protection Strategy Study (ICPSS), which was the most up to date tidal data available at the time of the study. Table 4-8 compares the reported peak tidal levels for node SE4 from both studies. It is noted that there is a minimum of 0.31m increase in peak level for in the ICWWS compared to the previous study. This increase will have an impact on flood extents modelled at the downstream modelled extent compared to ECFRAM.

Table 4-8: Comparison between ICPSS and ICWWS peak tide levels for node SE4

	50% AEP	**5% AEP**	**0.5% AEP**	**0.1% AEP**
ICPSS	2.23mOD	2.55mOD	2.88mOD	3.11mOD
ICWWS	2.60mOD	2.90mOD	3.21mOD	3.42mOD
Difference	+0.37m	+0.35m	+0.33m	+0.31m

4.6 Joint Probability

Pluvial – fluvial event joint probability is considered within the rainfall-runoff routing approach due to the nature of the method and therefore does not need to be explicitly analysed. In relation to fluvial – fluvial joint probability as the study area is relatively small (approximately 30km^2) it is assumed that the entire area is impacted by a single storm event of a single magnitude at any one time. Using the rainfall routing model, the timing of peak flows in response to a single storm event has been analysed to understand the impact of multiple watercourses of varying size responding to an event. It was found that the critical storm for the various sub catchments ranged from one to six hours.

The downstream extent of the hydraulic model and study area is the Irish Sea. Therefore fluvial-tidal joint probability was also considered. Table 4-9 shows the fluvial – tidal combinations considered in assessment and for sensitivity testing. Tide level data will be sourced from the ICWWS (refer to Section 4.5). The results of the joint probability and tidal sensitivity testing are discussed in the corresponding Hydraulics Report for this study.

Table 4-9: Proposed fluvial – tidal joint probability combinations

Fluvial %AEP event	Tidal % AEP event
All design runs	50% AEP
10% AEP	10% AEP
1% AEP	10% AEP
10% AEP	1% AEP
10% AEP	0.5% AEP

4.7 Climate Change

The relevant climate change factors were applied to the final estimated peak flows as outlined in the tender brief for the project for the Medium Range and High-End Forecast Scenario (MRFS and HEFS, refer to Table 4-10). Changes in forestation have been undertaken through adjustments to the time to peak (Tp) parameter for appropriate sub-catchments. Increased urbanisation was also considered and reflect the proposed development plan for the study area. In reference to future increases in urbanisation it is assumed that all new developments are designed with SUDS features which limit discharge from sites at the greenfield rate, therefore no explicit testing of urban land use variation will be caried out. The results of the climate change testing are discussed in the Hydraulics Report for this study.

Table 4-10: Climate change factors

	MRFS	HEFS
Extreme Rainfall	+20%	+30%
Flood flows	+20%	+30%
Mean Sea level Rise	+500mm	+1000mm
Forestation	-1/6 Tp	-1/3 Tp +10% SPR

5 APPENDIX A – HEP Catchment descriptors

HEP ID	AREA (km^2)	MSL (km)	S1085 (m/km)	DRAIND (km/km^2)	URBEXT	BFI Soil	SAAR	FARL	ARTDRAIN2
001	1.37	1.62	107.72	1.825	0.00	0.65	1086.00	1.00	0.00
007	5.58	5.02	38.80	1.279	0.32	0.64	1008.50	1.00	0.00
009	2.65	2.17	30.59	1.620	0.32	0.64	986.00	1.00	0.00
011	8.47	5.29	28.83	1.405	0.34	0.64	1002.88	1.00	0.00
012	1.99	1.91	10.73	1.110	0.93	0.62	937.50	1.00	0.00
014	2.78	2.81	11.64	1.110	0.95	0.64	937.50	1.00	0.00
016	8.55	5.79	33.33	2.214	0.42	0.61	979.78	1.00	0.00
019	0.76	0.73	33.47	0.954	0.00	0.61	945.00	1.00	0.00
023	13.58	7.68	23.47	1.350	0.48	0.61	969.08	1.00	0.00
024	2.40	1.68	12.12	0.698	0.67	0.52	874.67	1.00	0.00
028	3.63	3.78	15.84	1.042	0.70	0.55	863.50	1.00	0.00
029	17.21	7.69	22.74	1.280	0.53	0.61	945.13	1.00	0.00
031	18.58	8.69	23.43	1.100	0.55	0.64	939.11	1.00	0.00
033	10.03	6.15	41.03	0.769	0.03	0.64	1072.82	0.99	0.00
035	11.20	7.04	36.03	0.768	0.02	0.64	1049.54	0.99	0.00
038	11.99	8.30	32.91	0.823	0.05	0.65	1037.00	0.99	0.00

040	31.20	8.93	23.64	1.081	0.37	0.65	977.42	0.99	0.00
042	32.05	9.36	21.85	1.090	0.39	0.66	977.42	0.99	0.00
043	2.00	2.35	66.54	1.620	0.20	0.65	1086.00	1.00	0.00

6 APPENDIX B – HEP traditional estimates compared to ECFRAM flows

HEP ID	Estimation method	50% AEP (m^3/s)		1% AEP (m^3/s)		0.1% AEP (m^3/s)	
		ECFRAM	FRS	ECFRAM	FRS	ECFRAM	FRS
HEP_016 (ECFRAM HEP RPS_10022)	ECFRAM: FSU FRS: FSU	8.50	6.80	24.88	24.37	45.10	36.86
HEP_040 (ECFRAM HEP RPS_10021)	ECFRAM: FSU FRS: FSU	14.20	14.82	44.05	47.94	74.92	78.40
HEP_042 (ECFRM HEP 10_1570_2_RPS	ECFRAM: FSU FRS: FSU	14.41	15.04	41.58	48.65	67.63	79.57
HEP_038 (ECFRAM HEP 10_1220_3_RPS)	ECFRAM: FSU FRS: FSR RR	4.45	6.82	14.80	18.81	26.87	28.30
HEP_035 (ECFRAM HEP 10_1518_4_RPS)	ECFRAM: FSU FRS: FSR RR	3.51	6.27	11.89	17.30	21.55	26.02
HEP_043 (ECFRAM HEP 10_1211_1_RPS)	ECFRAM: FSU FRS: FSU SC	2.03	1.13	6.86	4.12	12.43	6.81

7 APPENDIX C – HEP Flow estimations

HEP ID	Traditional estimation method	50% AEP		20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.1% AEP	
		T	R	T	R	T	R	T	R	T	R	T	R	T	R	T	R
001	FSR RR	0.89	0.69	1.30	1.06	1.61	1.35	1.98	1.68	2.55	2.21	3.09	2.69	3.74	3.30	4.82	
007	FSU (ADJ)	3.44	4.02	5.00	6.15	6.23	7.87	7.63	9.98	9.86	10.45	11.93	12.53	14.43	20.69	19.45	
009*	FSU SC	1.74		2.52		3.12		3.80		4.90		5.91		7.12		9.67	
011	FSU (ADJ)	5.05	4.35	7.35	6.43	9.15	8.08	11.20	9.76	14.47	12.38	17.52	14.30	21.81	16.36	28.81	
012	FSU (UNADJ)	1.19	3.07	1.74	4.41	2.18	5.26	2.68	6.37	3.47	8.01	4.22	9.48	5.10	11.04	6.98	
014	FSU (UNADJ)	2.22	5.26	3.26	8.09	4.07	10.26	5.00	13.04	6.48	18.26	8.88	22.54	9.53	27.44	13.04	
016	FSU (ADJ)	6.80	5.38	10.03	8.16	12.51	10.34	15.43	13.11	20.05	18.16	24.37	22.66	29.55	27.40	36.86	
019	FSR RR	0.56	3.40	0.79	5.39	0.97	7.01	1.17	8.90	1.48	12.01	1.77	14.93	2.11	18.53	2.72	
023	FSR RR	8.62	6.74	12.58	11.52	15.60	14.36	18.90	17.76	24.25	22.66	29.15	26.27	34.95	29.54	47.28	
024	FSU SC	1.85	1.91	2.69	2.58	3.36	2.89	4.10	3.33	5.32	3.75	6.44	3.90	7.77	3.95	10.73	
028	FSU SC	2.59	3.20	3.77	4.46	4.70	2.26	5.75	6.12	7.44	7.39	9.01	8.18	10.88	8.88	15.02	
031	FSU (ADJ)	10.61	12.24	15.22	17.44	18.87	19.98	22.87	21.06	29.34	25.64	35.27	25.64	42.29	28.19	57.59	
033	FSR RR	5.98	2.19	8.13	2.95	9.70	3.52	11.46	4.15	14.14	5.13	16.50	6.02	19.25	7.03	24.82	
035	FSR RR	6.27	2.27	8.53	3.06	10.18	3.66	12.02	4.33	14.82	5.36	17.30	6.31	20.19	7.29	26.02	

038	FSR RR	6.82	2.40	9.27	3.28	11.07	3.93	13.07	4.65	16.12	5.79	18.81	6.85	21.96		28.30	
040	GAUGED	14.82	14.37	21.20	20.858	25.94	24.66	31.46	26.22	40.07	35.91	47.94	35.91	57.29	39.926	78.40	
042*	FSU (ADJ)	15.04		21.51		26.32		31.93		40.67		48.65		58.15		79.57	
043	FSU SC	1.58	1.11	2.34	1.67	2.93	2.11	3.62	2.6	4.73	3.39	5.76	4.11	7.01	5.00	9.53	
Abbreviations																	
T	Traditional estimation method	**R**	Rainfall routing model			**FSU SC**	Flood Studies Update Small Catchments method			**FSU (ADJ)**	Flood Studies Update method with Adjustment factor			**FSR RR**	Flood Studies Report Rainfall Runoff Method		

- In the routing model used to determine flows, the model did not extent as far so this is not included.

8 APPENDIX D – Rating review model file note

Introduction

This Appendix details the construction of a 1D only Flood Modeller hydraulic model of a portion of the Shanganagh River. The model has been developed to carry out a rating review of an existing gauge (10021) located along the reach. Figure 8-1 shows the modelled reach, gauge location and surveyed cross sections.

Flood Modelled v4.5 was used to build the model.

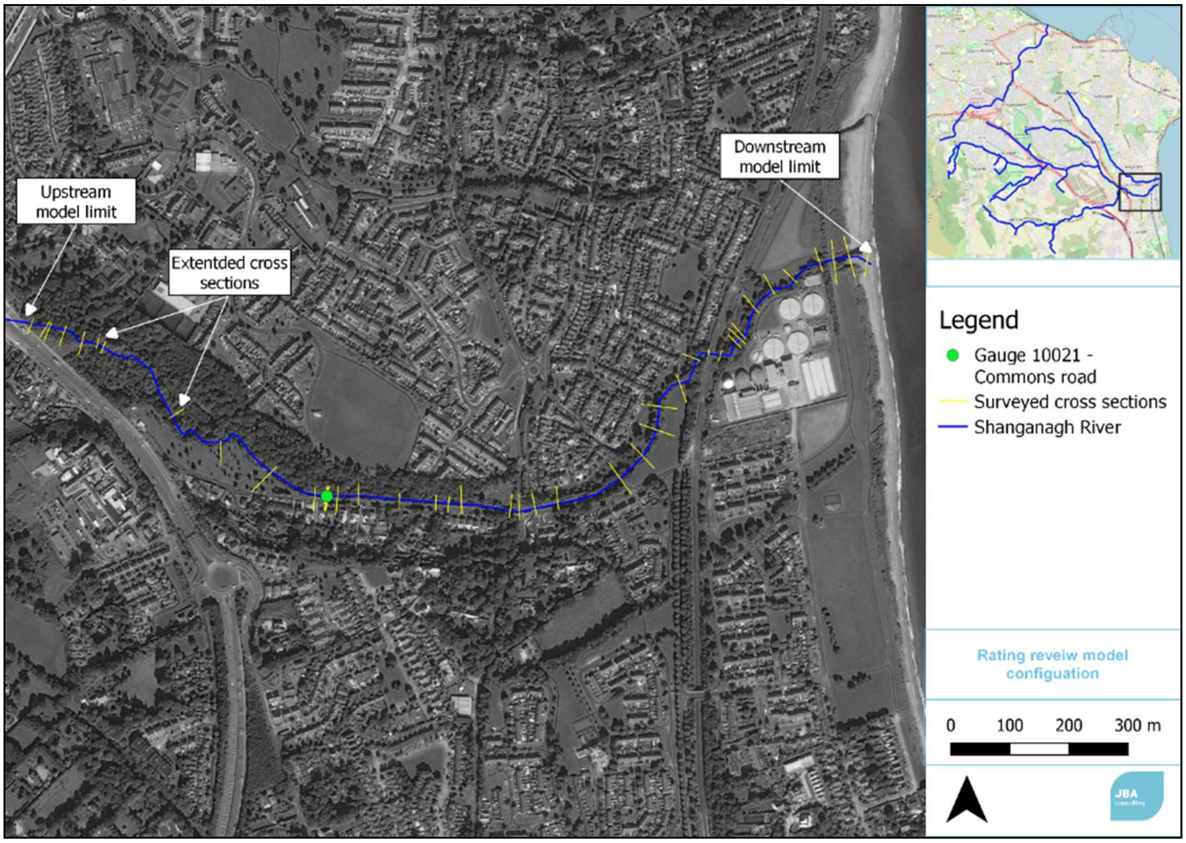


Figure 8-1: Rating review model configuration

Data used in model development

Survey data

The following river cross section survey data was reviewed and used to build the model. Review of top of bank levels and cross section extents was carried out by reviewing the cross-section levels, survey drawings and photographs. Walls have been retained in the cross sections to ensure the full capacity of the current channel was assessed in the model for the rating review. The following survey data was used.

Eastern CFRAM survey data (Murphy's Survey 2012): Cross section data of the Shanganagh river was collected as part of the ECFRAM study to develop a hydraulic model. The survey data includes survey of structures such as the gauge weir and bridges/culverts along the modelled reach. These surveys cross sections have been used to build the model. Figure 8-1 shows the cross-section locations.

Extended cross sections

Figure 8-1 shows three cross sections which were cut short in the original survey data due to inaccessibility and rough vegetation. To ensure cross section width consistency these cross sections were extended using OSi 2m resolution DTM data. The cross-section extensions were created using the JBA ISIS-TUFLOW in house plug-in. The extensions were checked to ensure that there were no issues where the survey and DTM data overlapped. Figure 8-2 and Figure 8-3 provide a comparison between the original and one of the extended cross sections.

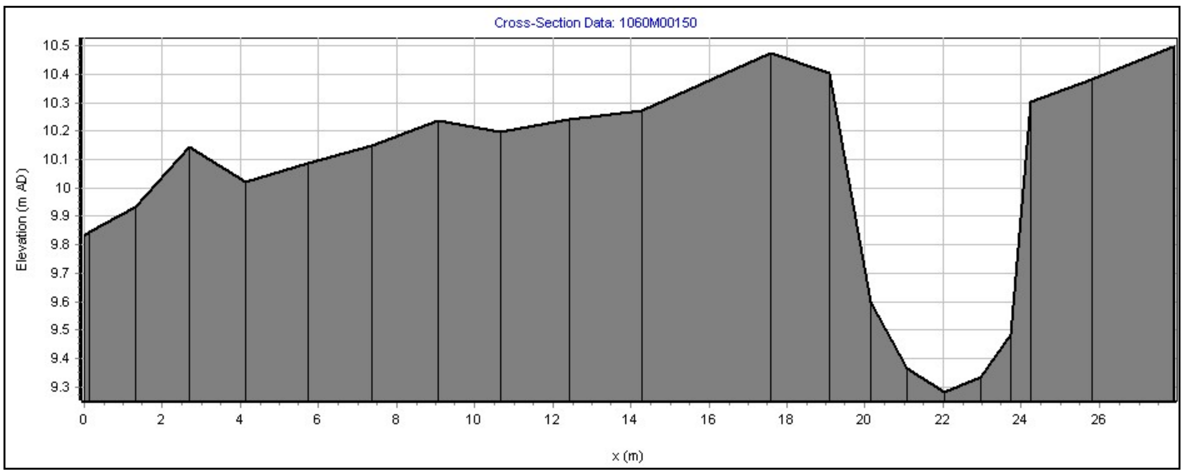


Figure 8-2: Original survey for cross section 1060M00150

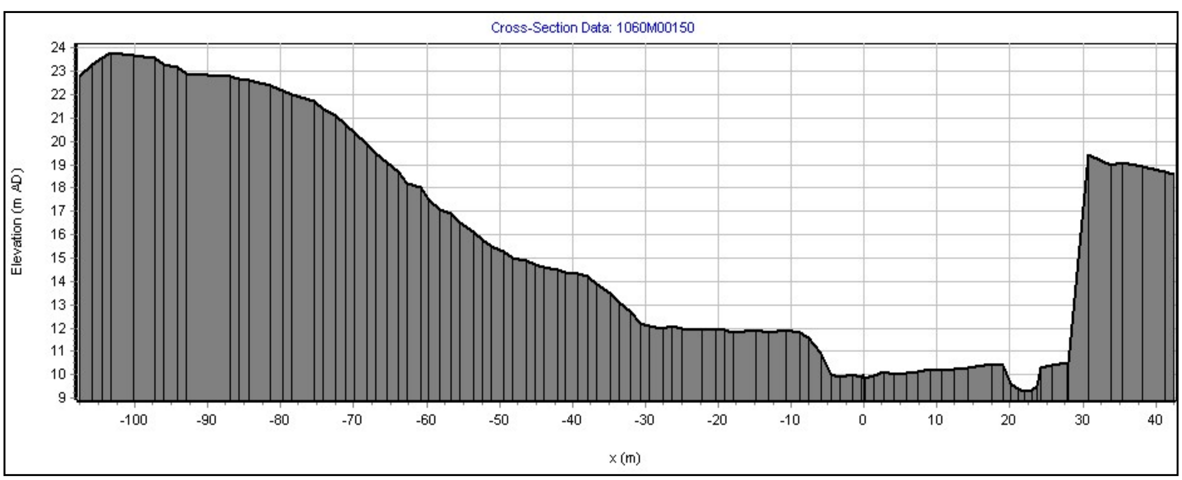


Figure 8-3: Extended cross section 1060M00150

Model build

Hydrological boundaries

The model has 1 inflow (flow-time boundary) and one outflow (head-time boundary). There is no derived hydrology for the model inflow as a range of flows from low to high are used to develop the gauge rating curve. The outflow is a head-time boundary set at a constant level of 2.60mOD which is the 50% AEP tide level for the area according to the Irish Coastal Wave and Water level Study 2018 (ICWWS).

Channel roughness

Channel roughness was represented using Manning's N values as described by Chow 1959 for varying channel conditions. Survey photographs were used to assess the channel condition for each cross section. Different roughness values were applied for channel bed, channel sides and floodplain to represent the variation. Changes in roughness have been marked using panel markers. Table 8-1 shows examples of the channel and the associated roughness value given.

Table 8-1: Examples of different Manning's N values applied

Channel description and Manning's N value from Chow 1959	Example channel
Straight channel with more stones and weeds. **Manning's applied: 0.040 on channel bed (upper range), increased roughness (0.055/0.060) applied to channel sides to account for vegetation.**	 1060M00150
Clean winding channel with some pools and shoals. **Manning's applied: 0.040 on channel bed, roughness of 0.020 applied to right hand bank wall.**	 1060M00080
Channel bottom consisting of gravels, cobbles and few boulders. **Manning's applied: 0.050 on channel bed (upper range)**	

	1060M00005

Representation of structures

8 structures are included in the model:

- 7 bridges (some represented as culverts depending on the length-width ratio of the channel and structure)

- 1 weir (located at gauge 10021 location)

Spills are included on structures where overtopping may occur. Some of the larger structures do not include spills as overtopping of these structures is highly unlikely. The weir coefficient used over the structures has been adjusted to reflect the material of the structures. Details of each structure are recorded within the model units.

A spill unit has been used to represent the gauge weir. The cross section of the top of the weir has been used to ensure that the weir is representative of the channel. A weir coefficient of 1.70 has been used to represent flow over the weir.

Out of bank spill

Flood defence walls run along a portion of the right-bank of the watercourse. In higher flow events out of bank spill will occur upstream of the wall and run along a flood plain flow path and bypass the gauge. To represent this and make sure that there is not an over estimation of flow within the channel a spill unit set to the bank height upstream of the wall has been put in the model. When the water level reaches above bank height the spill is activated and a portion of the flow is taken out of the channel. The flow entering the spill is added back to the channel at the downstream extent where the addition of flow will not impact the readings at the gauge (refer to downstream sensitivity is test discussion).

Model performance and health

The double precision version of FM v4.5 is used to run the model. It runs at a two second timestep and takes approximately 5 minutes to complete a 26hour run. Figure 8-4 shows the model health output plot. The following outputs have been reviewed to assess overall model health:

- Non-convergence: A period of non-convergence occurs at the downstream boundary when a flow of approximately 30m3/s passes out through downstream portion of the model. Review of the times series shows this non convergence is related to the HT boundary (DS cross section 1060M0000). The nonconvergence is limited to a short period and does not impact outputs at the gauge location (refer to downstream sensitivity test discussion).

Overall the model is considered to be fit for the purpose of assessing a gauge Q-h relationship.

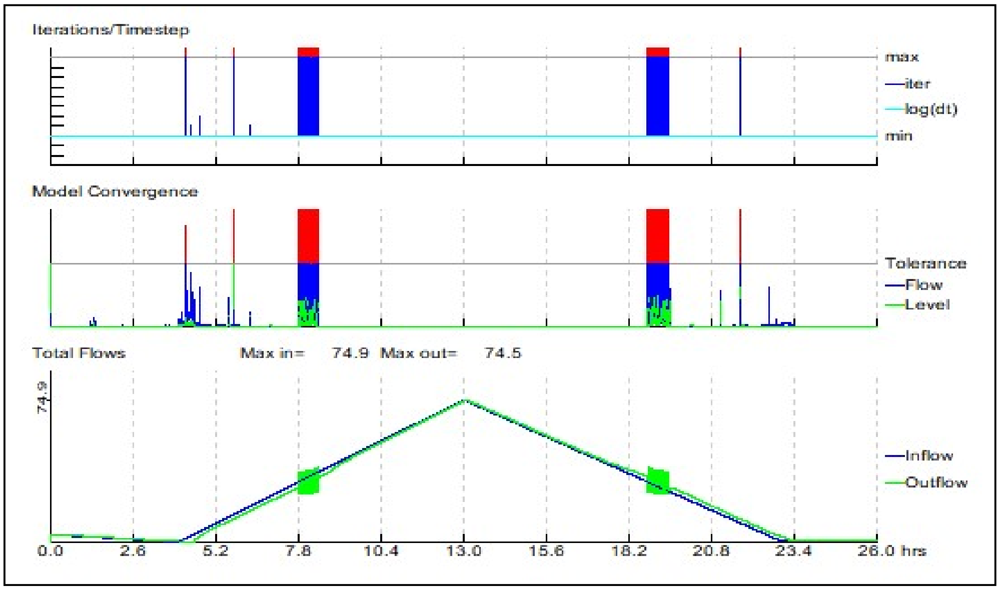


Figure 8-4: Model health output

Model results and sensitivity testing

To ensure the model is fit for purpose and to assess the gauge and potential impacts on the stage discharge relationship (Q-h relationship) derived sensitivity tests have been carried out and are described in this section.

Comparison with ECFRAM

A rating review assessment was carried out for gauge 10021 as part of the ECFRAM project. For their rating review model the entire 1D-2D model was used and the 0.1% AEP event run through the model to extract the full range of the Q-h relationship at the gauge. While the current rating review model is a 1D only model it uses the same survey information as the ECFRAM model and therefore the modelled Q-h relationship should be similar (changes in roughness values, will result in slight differences). To check this is the case the 0.1% AEP ECFRAM flow was run through the model and the resulting Q-h relationship compared against the ECFRAM Q-h curve derived for the gauge.

Figure 8-5 and Figure 8-6 compare the ECFRAM curve, modelled results, and check gaugings. Overall, the modelled Q-h curve produces a similar curve shape to the ECFRAM curve which is to be expected given the data used is the same. Examining the curves at lower flow values also shows the modelled Q-h curve better matches the plotting of the check gaugings with the concave curve shape being replicated by the model at low flows. This sensibility check has highlighted that the Q-h relationship at the gauge is consistent in both studies providing confidence in the modelled outputs.

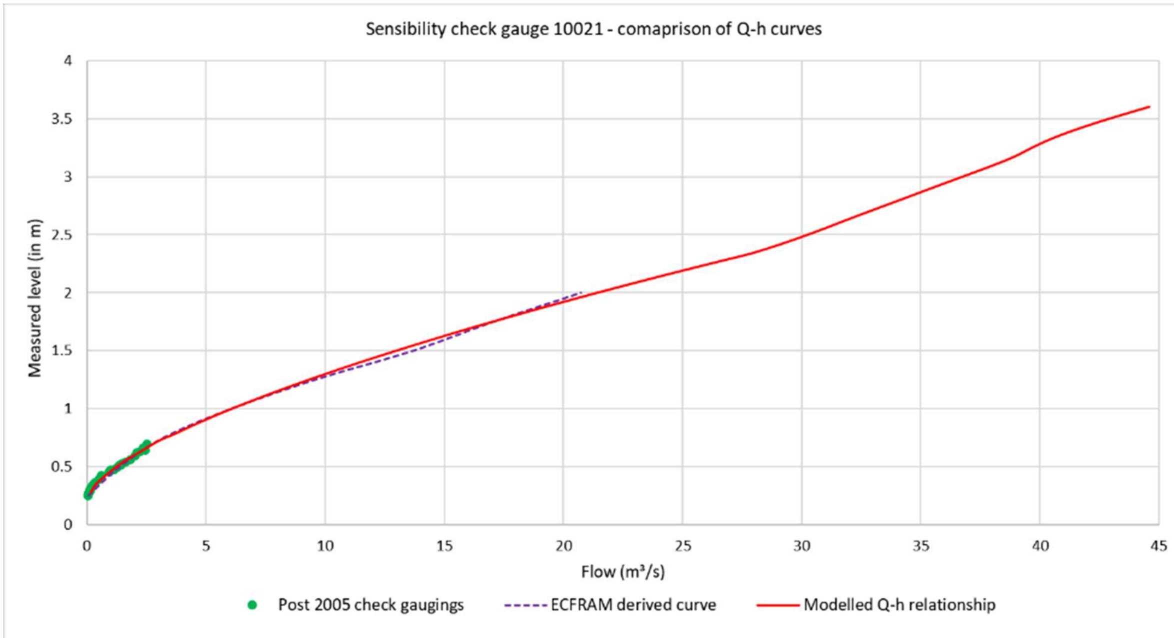


Figure 8-5: Gauge 10021 - comparison of Q-h curves

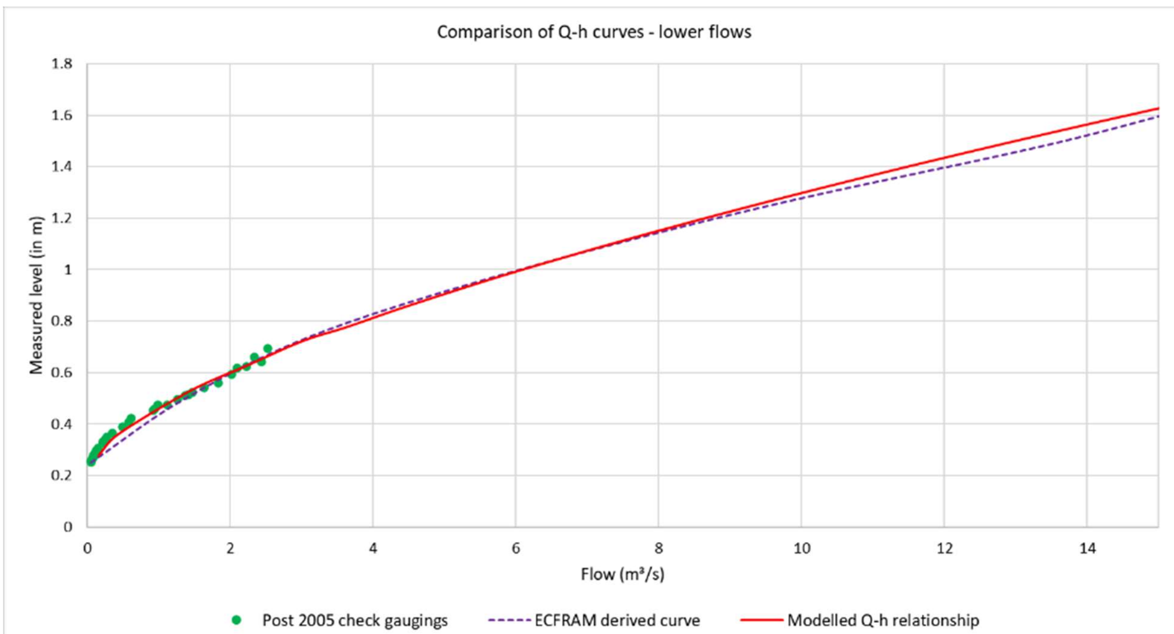


Figure 8-6: Gauge 10021 - comparison of Q-h curves at low flows

Sensitivity test: Downstream boundary sensitivity

A constant HT boundary is applied to the model representing the outflow of the model into the Irish sea. To test whether the gauge Q-h relationship is sensitive to changes in level resulting by the tide the following scenarios were run:

- A constant fluvial inflow of 4m3/s with the downstream HT level set at the 50% AEP value from the ICWWS 2018 (2.60mOD).

- A constant fluvial inflow of 4m3/s with the downstream HT level set at the 1% AEP value from the ICWWS 2018 (3.12mOD).

Figure 8-7 compares the maximum reported water level for the two scenarios. From the long section the area impacted by changes at the downstream boundary is restricted to the lower portion of the watercourse. The channel slope is such that the gauge location is not impacted by the HT downstream. The gauge is therefore not sensitive to tidal influences.

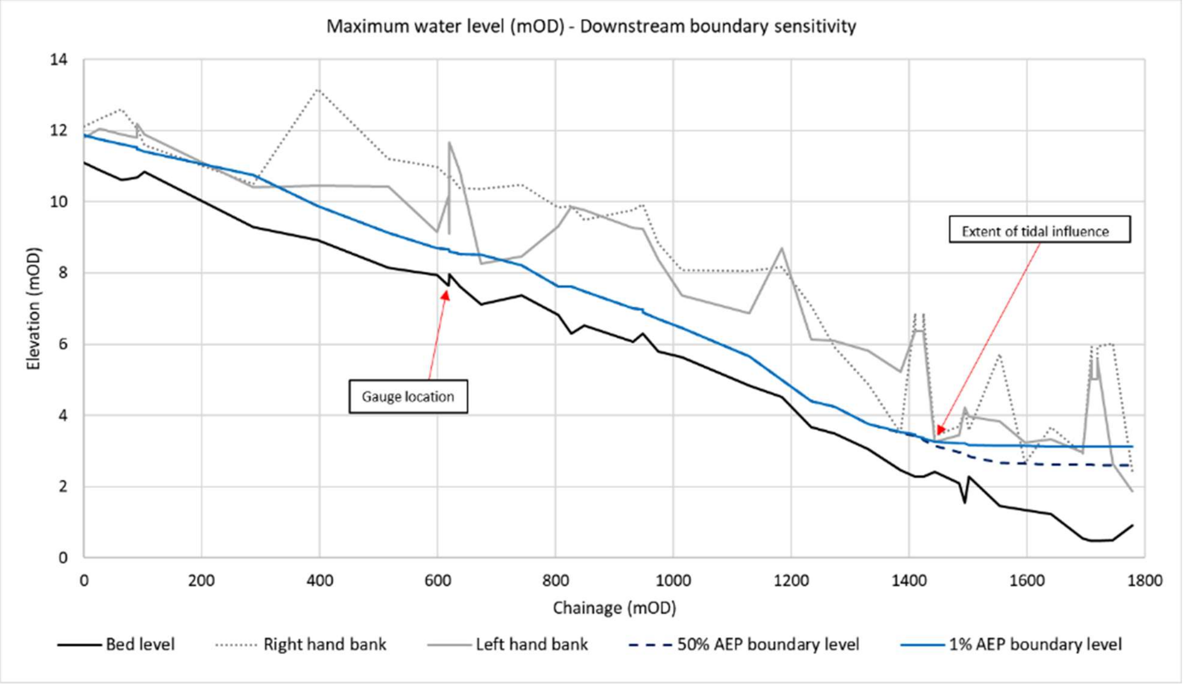


Figure 8-7: Maximum water level (mOD) - Downstream boundary sensitivity

Sensitivity test: Changes to channel roughness

Channel roughness can vary considerably over the year due to seasonality and maintenance. Changes in vegetation coverage can impact channel conveyance and efficiency and therefore the Q-h relationship at the gauge location. To assess this sensitivity tests have been carried out where the Manning's roughness values have been increased/decreased to the maximum/minimum values for the reach and vegetation types according to Chow 1959. Values which are already at the maximum range have been increased by 20% to allow robust testing of roughness.

Figure 8-8 Figure 8-9 compare the generated curves. The variation of the relationship between the model outputs increases with increased flow - The reduced Manning's and normal Q-h curve are largely identical until a flow of approximately 27m3/s is reached. The increased Manning's Q-h relationship shows a slightly more inefficient channel (higher level for a given flow) for the lower flows and this decrease in efficiency grows larger at higher flows. Figure 8-9 compares the curves at low flows against the check gaugings. A proportion of the measurements match well with the Q-h curves particularly with the lower and normal Manning's results. However, there are a number of values at the very low flows (below 1m3/s) that show a slightly more inefficient channel than the modelled results. This difference may be due to measurement uncertainty at low flows, slight differences in roughness within the model. Given that the difference occurs at the lowest of flows it is not considered an issue. In summary the testing of the Manning's N values shows that the gauge Q-h relationship is not overly sensitive to roughness.

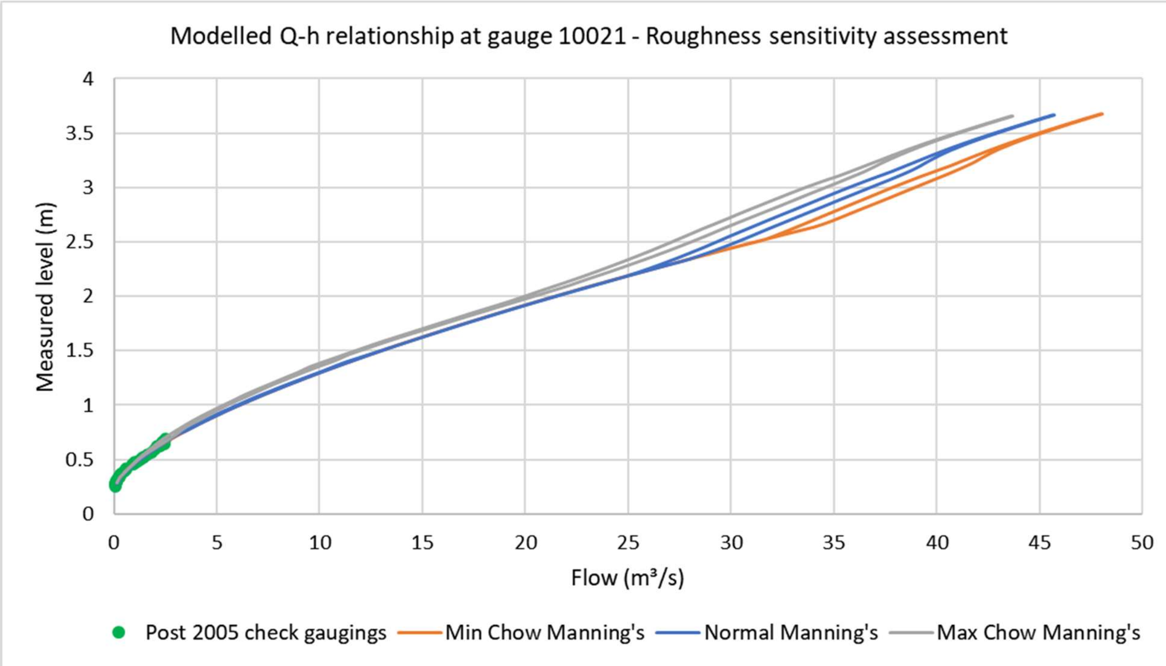


Figure 8-8: Modelled Q-h relationship at gauge 10021 - Roughness sensitivity assessment

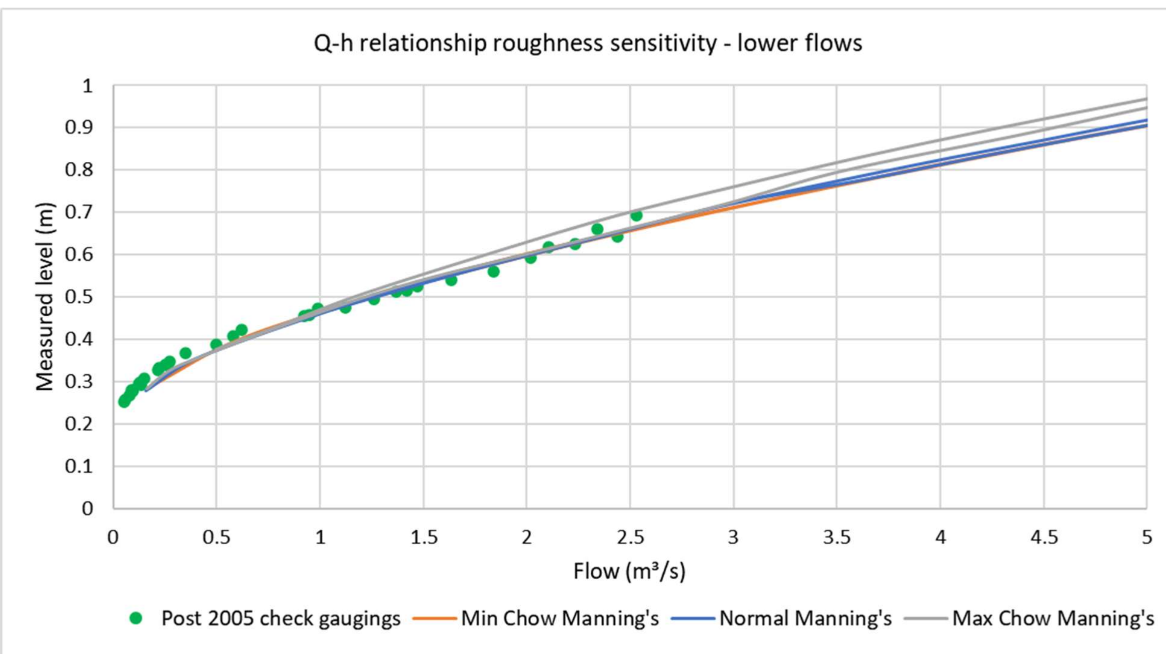


Figure 8-9: Q-h relationship roughness sensitivity - lower flows

Sensitivity test: Variation of storm duration

Storms have varying duration and intensity. To assess whether the Q-h relationship at the gauge is impacted by varying durations and intensity the model the following scenarios have been run and results compared:

- 0.1% AEP ECFRAM flow with the peak flow occurring at 6 hours (short duration storm),

- 0.1% AEP ECFRAM flow with peak flow occurring at 9 hours (time to peak recorded at gauge in ECFRAM model),
- 0.1% AEP ECFRAM flow with peak flow occurring at 12 hours (time to peak recorded at gauge during 2011 flood event).

Figure 8-10 compares the modelled Q-h curves at the gauge location. There is little to no variation in the Q-h relationship within any of the runs - the same flow is recorded for a given stage. To further test this sensitivity a comparison of the level recorded for a given flow was compared against the level reported when the flow was constant in the model. Table 8-2 compares the flows and levels reported. There is very little variation in recorded level for the given flow between the different scenarios. Overall, the testing shows the gauge is not sensitive to timing of peak flow.

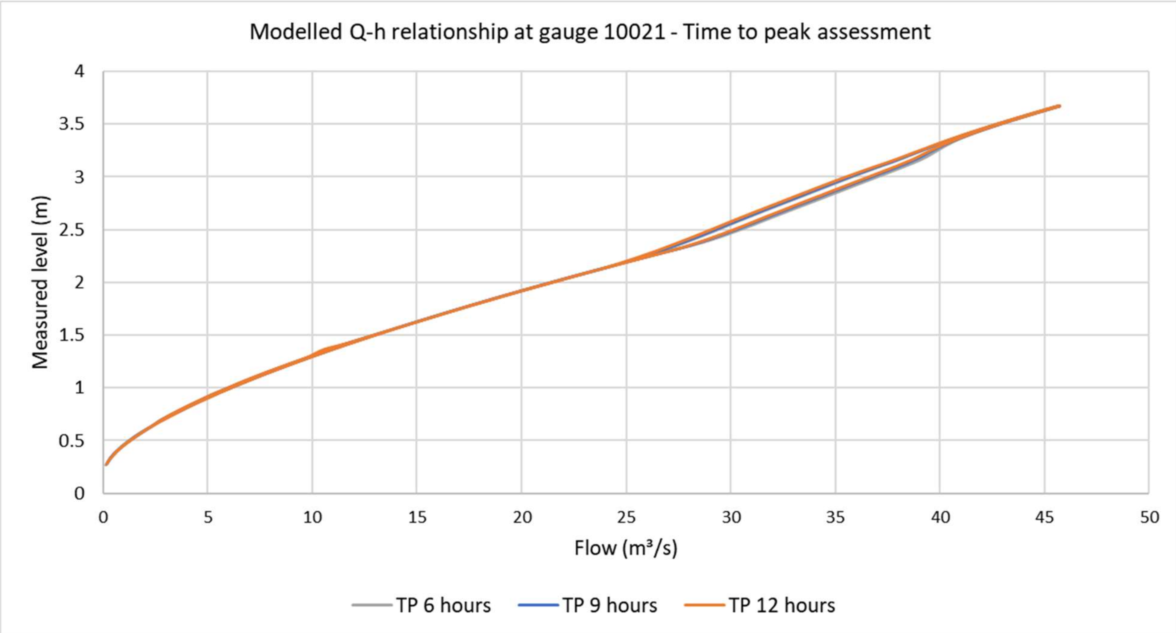


Figure 8-10: Modelled Q-h relationship at gauge 10021 - Time to peak assessment

Table 8-2: Modelled level and flow for tested time to peaks

Time to peak (TP)	Level (m above gauge datum)	Flow (m3/s)
Constant flow (no TP)	3.33	40.46
6-hour TP	3.32	40.38
9-hour TP	3.33	40.52
12-hour TP	3.33	40.47

Sensitivity test: Blockage of downstream structures

A pedestrian access bridge lies approximately 200m downstream of the gauge location. In the 2011 flood event a tree got caught on the upstream face of this bridge. While it did not greatly impact the conveyance of the structure is highlights the potential for blockage to occur. A constriction at the bridge will impact the water level at the gauge. To test the impact potential blockage may have on the Q-h relationship at the gauge the following scenarios were tested:

- 0.1% AEP ECFRAM flow hydrograph applied with a 33% blockage applied to the bridge,

- 0.1% AEP ECFRAM flow hydrograph applied with a 66% blockage applied to the bridge.

Figure 8-11 compares the modelled Q-h relationship at the gauge with the various levels of blockage. The importance of the backwater effect from the bridge becomes increasingly important with increased blockage. The modelled Q-h when a 33% blockage is applied is identical to the no blockage relationship up to flow of 20m3/s then impact begins to take effect. The impact of the bridge blockage on the gauge relationship occurs at around 10m3/s.

This test highlights that the gauge is impacted by changes downstream at the bridge and any record of blockage should be taken into account when estimating flows during storm events (increased uncertainty in flow values).

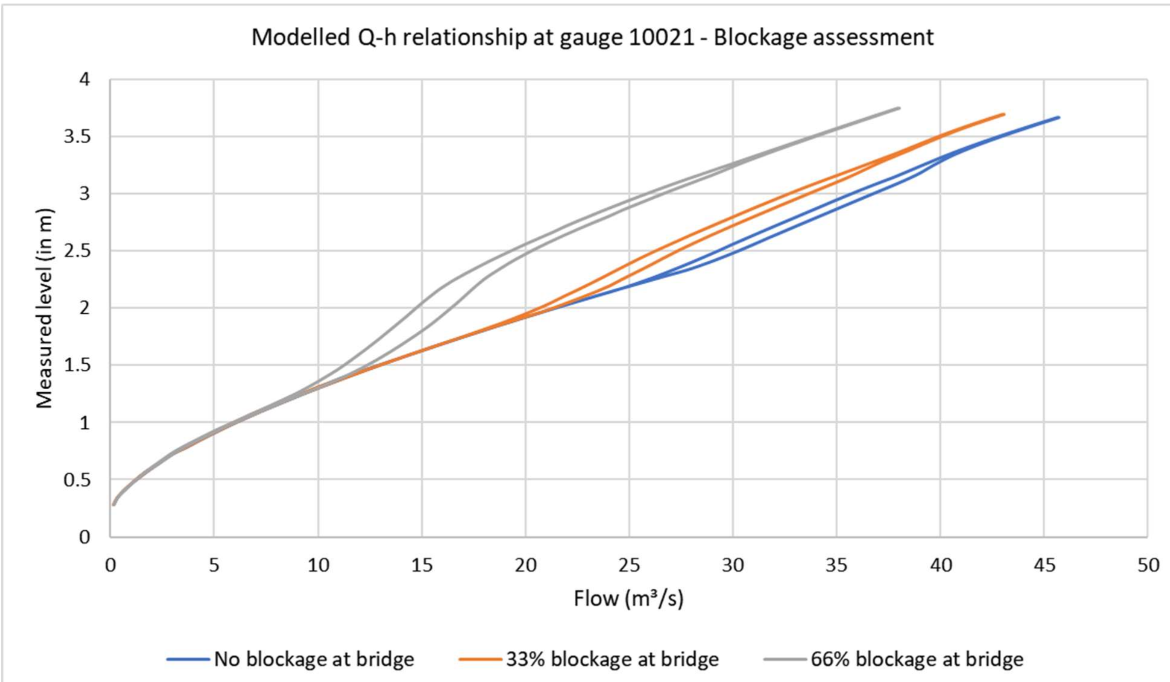


Figure 8-11: Modelled Q-h relationship at gauge 10021 - Blockage assessment