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Hydrology Report

Dodder FAS Phase 3

M02136-02_DG01 | April 2022

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1 INTRODUCTION

1.1 Terms of Reference

This Hydrology Report was commissioned by Dublin County Council as a deliverable of the River Dodder Flood Alleviation Scheme Phase 3. The report intends to summarise outputs and decision making made as part of the hydrological assessment for the project.

1.2 Statement of Authority

This report and assessment has been prepared and reviewed by qualified professionals with appropriate experience in the fields of flood risk, drainage, wastewater, and hydraulic modelling studies. The key staff members involved in this project are as follows:

- Stephen Neill *BEng (Hons) MIEI* - Senior Engineer and Modeller specialising in engineering hydrology, flood modelling, and flood risk assessment.
- Kyle Somerville *BEng (Hons) CEng* – Director and Chartered Engineer specializing in the fields of flood risk assessment, flood modelling, drainage and surface water management design for public and private sectors.

1.3 Purpose

The objective of this hydrology report is to provide detail on work undertaken to characterise flood hydrology, which will be utilised in hydraulic modelling to inform and assess the River Dodder Flood Alleviation Scheme (FAS) Phase 3. The report builds on an approach to the project agreed with the project client and stakeholders via a previous Hydrological and Hydraulic Method Statement[1].

This report will conduct the works outlined in the agreed hydrology method statement, refer to M02136-02_MS01, and solidify methodologies selected for use based on analysis of available data.

This report records the outcome of:

- a background review of information, including the previous River Dodder CFRAM Study,
- hydrological flood analysis and design flow estimation, and
- project risks associated and sensitivity testing to be undertaken.

Design flows determined by this assessment will be taken forward as inputs for the hydraulic modelling.

[1] M02136-02_MS01_Dodder FRS - Proposed Method Statement_Rev04 – Issued 24/06/2020

2 STUDY AREA

2.1 Study Location

The River Dodder catchment is located within the eastern district CFRAM study area, unit of management HA09. It stretches from the River Liffey Estuary at Ringsend in Dublin City, west as far as Tallaght and southwest as far as Kippure Mountain, draining a catchment of approximately 120km².

The current project covers the section of the River Dodder from Clonskeagh Road Bridge to Orwell Road Bridge including flood defence works on the Little Dargle Stream at Braemor Road-Woodside Drive south-eastern junction in the Dublin City Council (DCC) and Dun Laoghaire Rathdown County Council (DLRCC) areas.

The extent of the River Dodder subject to assessment is between the Orwell Road Bridge (upstream limit) and the Clonskeagh Road Bridge (downstream limit), refer to Figure 2-1. The brief includes review of the Little Dargle Stream at Braemor Road-Woodside Drive south-eastern junction to assess implementation of defences at that location.

The River Dodder and associated tributaries were modelled previously as part of CFRAM pilot study (published final 2008).

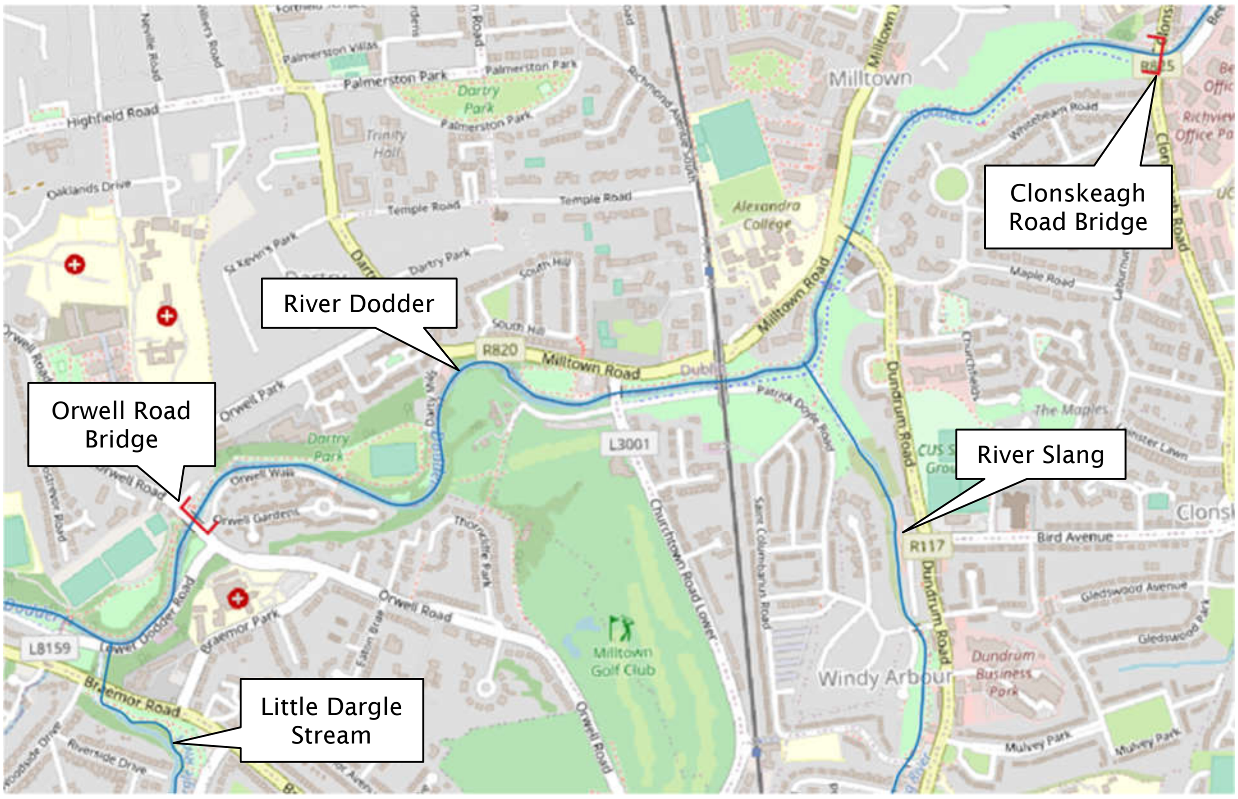

Figure 2-1 FAS Phase 3 Study Location

The River Dodder is primarily open channel for the extent of the study area with structures sited solely to provide crossing access. The Little Dargle and River Slang are categorised as urban drainage networks with a mix of open and culverted reaches.

2.2 Proposed Hydraulic Model Extent

While not subject to detailed discussion in this report; it has been pertinent to identify at the outset the intended hydraulic model extent to define the limits of the hydrological analysis.

The model will necessarily extend beyond the river reaches to be evaluated as part of the Flood Alleviation Scheme (FAS), to permit surety that the downstream model boundary condition is robust, and to ensure that the upstream model extent is sited sufficiently upstream / upgradient to capture any backwater effects predicted as a result of the implementation of options, refer to Figure 2-2.

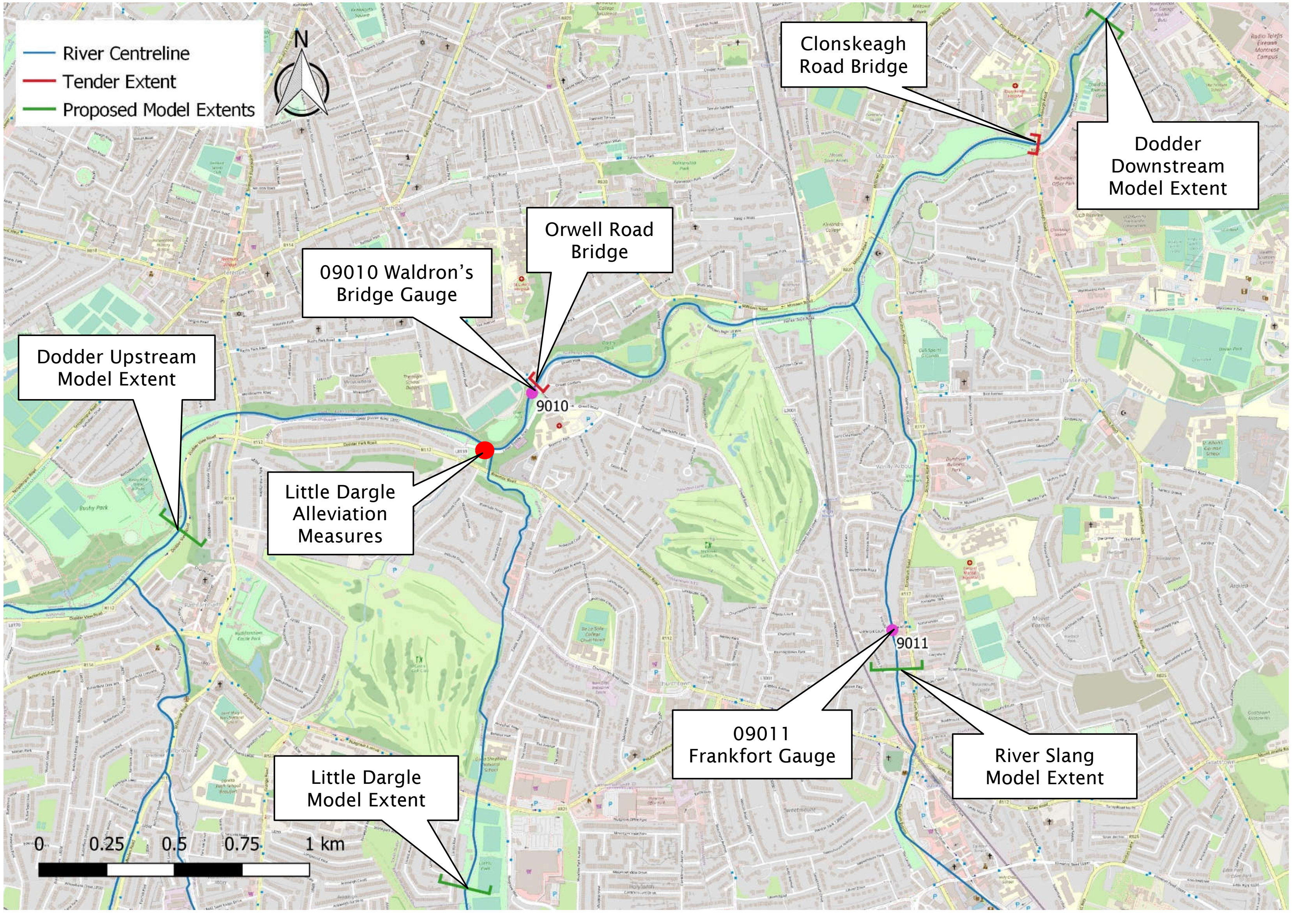

Figure 2-2 Proposed Hydraulic Modelling Extent

2.3 Location of Proposed Alleviation Measures

Proposed alleviation measures for review in this study were produced as part of the CFRAM study and include the use of hard defences, in the form of walls and embankments, to retain flows within the river corridor. A further measure was requested in the study scope to investigate the use of Scully's Field for attenuation purposes, refer to Figure 2-3 for locations of all defences.

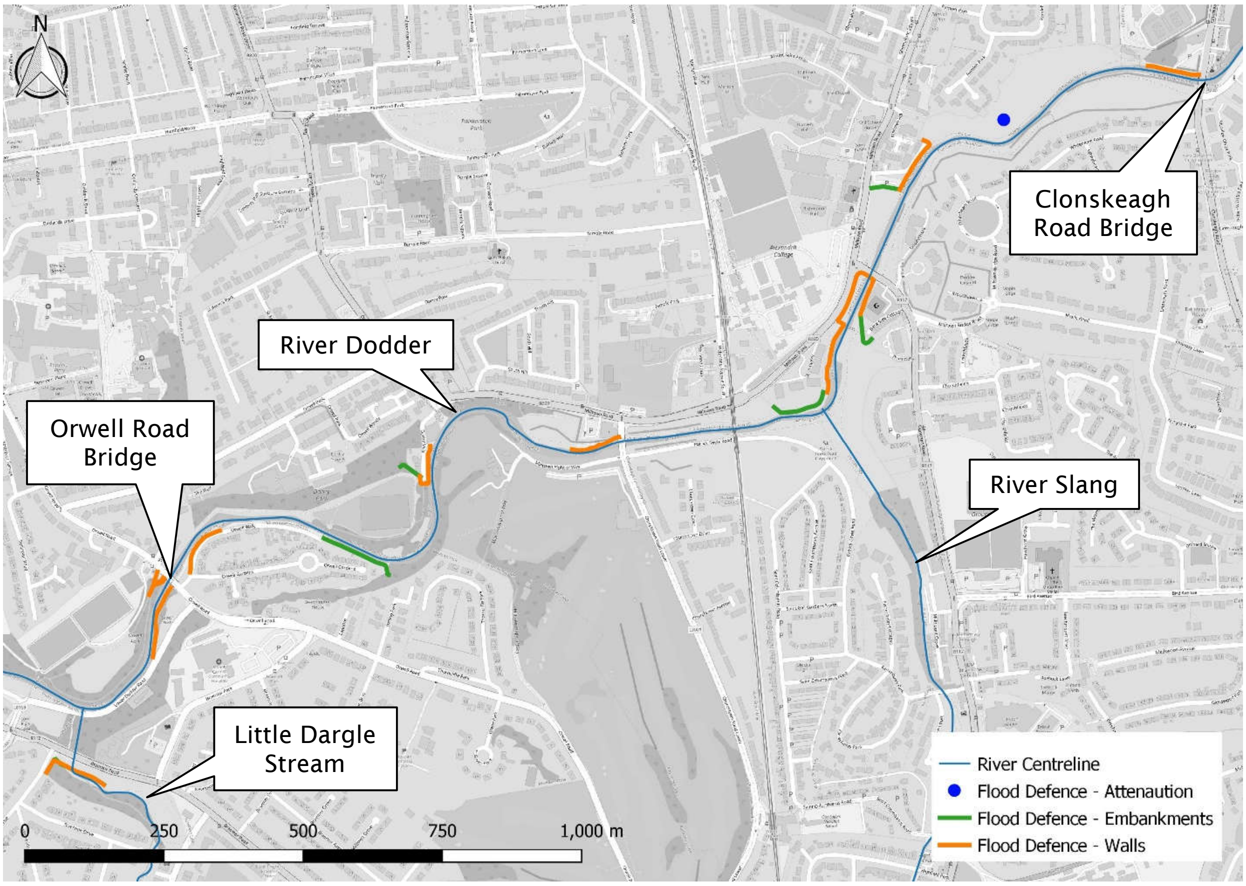

Figure 2-3 Locations of Proposed Defences for Review

The locations of the defences have been subject to a high-level review at this stage to ensure that the hydrology is applied to model best replicates in channel flows respective to defence locations. The design team discounted the use of attenuation as an alleviation option due to unsuitable topography in the attenuation location coupled with the location of the area within the study extent, i.e. downstream of the proposed options.

3 REVIEW OF AVAILABLE DATA

3.1 River Dodder CFRAM Study Hydrological Analysis

The River Dodder is located in Eastern Hydraulic Area 09 (HA09) and was studied between 2007-2010 in the River Dodder Catchment Flood Risk and Management Study (CFRAM). The modelling included the entirety of the River Dodder and its five major tributaries (Owendoher, Whitechurch, Dundrum/Slang, Little Dargle and Tallaght Stream).

The hydrological analysis was based on the standard statistical approach to estimating river flows using FSR and FEH methodologies but also incorporating the development of a wide range of rainfall run-off models using MIKE NAM.

When the River Dodder CFRAM study was conducted the use of Flood Studies Update (FSU) methodology was in its infancy and FSU data had not been provided for use by the OPW in the study. Therefore, assessment of hydrology at specific intervals along watercourses, specifically the use of Hydrological Estimation Points (HEPs), was not conducted. Considering this the potential for extraction of data for comparison is limited. The following sections outline works conducted at time of CFRAM and results captured where appropriate to this study.

3.1.1 Rating Curve Analysis

Data for this analysis was provided by the EPA and entailed water level and flow data. Rating curves were also provided by the EPA with curves adopted by the study to their maximum rating level based on verified data. Review of extrapolated data was conducted above the maximum rating with project rating curves calculated for use in the CFRAM study.

The rating curve review was conducted via 1D modelling where a stretch of watercourse was represented at the gauge locations ensuring any feature affecting the flow regime was represented, for example control weirs. The 1D models were calibrated to the EPA verified flow value with project rating curves calculated beyond that level. The EPA and project rating curves were then used to assign discharge values to the 12 highest recorded water levels at each station and results compared.

At both Waldron's bridge and Frankfort, the project rating curve provided lower discharge values, than those calculated by the EPA, indicating less flow passing through these channels.

3.1.2 Rainfall-Runoff Modelling

Rainfall-Runoff modelling was conducted with NAM modelling used for the rural areas and separate Urban modelling to reflect differing contributing area types. These models were combined to create a singular model for use in the CFRAM study. Review of reporting informs that NAM modelling was conducted for areas pertinent to this study, i.e. catchments draining to Waldron's Bridge and Frankfort gauges.

Captured rainfall data coupled with gauged data was used to calibrate the models. Results of this process informed that the NAM modelling was not accurately predicting catchment response so an Urban model was created for each of the gauged catchments. The urban modelling runoff is calculated as flow in an open channel with losses accounted for. The volume of runoff is dictated by catchment size and losses while the hydrograph is informed by length, slope and roughness of the catchment. Calibration of the models was again conducted with simulated discharge compared against recorded data.

At Waldron's Bridge adequate calibration correlation was achieved with differences attributed to reservoir attenuation in the upper catchment and spatial and temporal distribution of rainfall data within the River Dodder Catchment. Review of singular events provided closer correlation to gauged data.

Similarly, the calibration of the model draining to the Frankfort gauge was adequately calibrated with small changes in flows returning large errors. Again, review of singular flood events provided better results with difference attributed to spatial and temporal distribution of rainfall data.

3.1.3 Statistical Analysis – Simulated Results

Extreme value analysis was conducted to determine design discharges of known period for gauged catchments. The analysis was conducted on the rainfall runoff simulated discharge files using the Peaks over Threshold (POT) methodology. Multiple probability distributions were then fitted to the data with assessment conducted on the confidence for each event.

Statistical analysis conducted at the Waldron's bridge gauge using simulated discharge files informed the best fitting probability distribution was Exponential (EXP1/MOM). Statistical analysis at the Frankfort gauge using simulated discharge informed the best fitting probability distribution was Log Pearson Type 3 (LP3/MOM/LOG).

3.1.4 Statistical Analysis – Amax Data

Annual Maxima (Amax) data collected from multiple sources was subject to Extreme Value Analysis (EVA) using MIKE software module. Multiple distributions were assessed with Gumbel (EV Type 1) and Frechét (EV Type 2) providing the best results.

3.1.5 Flood Studies Report (FSR) Method

The FSR methodology, the predecessor to FSU using catchment descriptors to calculate estimated flows, was conducted as a comparison to other methods. A Factorial Standard Error (FSE) of 1.47 was applied to the flows equating to a confidence of approximately 86%, but no detail has been provided informing of the derivation of this percentage.

3.1.6 Results Summary

Results of the various methods were compiled for comparison at each of the relevant gauge locations, refer to Table 3-1 and Table 3-2.

Table 3-1 Waldron's Bridge Results Comparison

		Waldron's Bridge Gauged Catchment			
AEP	**Return Period**	**Sim (EXP1, MOM)**	**FSR**	**EV1 of Annual Maxima**	**EV2 of Annual Maxima**
(%)	**(Years)**	**(m^3/s)**	**(m^3/s)**	**(m^3/s)**	**(m^3/s)**
50	2	74.26	50.3	56	53
20	5	108.18	64.6	90	78
10	10	130.63	72.54	114	102
4	25	159.01	81.01	146	143
2	50	180.06	87.36	165	170
1	100	200.95	96.13	185	210
0.5	200	221.77	103.26	210	257

Table 3-2 Frankfort Results Comparison

		Frankfort Gauged Catchment			
AEP	Return Period	Sim (LP3/MOM/LOG))	FSR	EV1 of Annual Maxima	EV2 of Annual Maxima
(%)	(Years)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
50	2	3.88	3.99	3.41	3.13
20	5	5.57	4.94	4.52	4.35
10	10	6.82	5.3	5.6	5.45
4	25	8.65	5.82	6.92	7.08
2	50	10.25	6.1	7.96	8.41
1	100	12.07	6.55	N/A	N/A
0.5	200	14.15	6.88	N/A	N/A

Generally the FSR calculations provide lower flows than those derived using statistical analysis. The report informs that the simulated (Sim) design flows are favoured for use and that they compare well with the results of the EVA on the recorded annual maxima.

3.1.7 Application to Model

To apply the calculated hydrology, the whole catchment was split into 15 Sub catchments with rainfall-runoff models prepared for each. Simulations were conducted using weighted historic rainfall contribution relative to catchment area. Upon completion of the simulation's EVA analysis was conducted for each catchment with the best fitting probability distribution chosen for use.

The application of the rainfall was via point inflows and distributed lateral inflows. Detail was provided informing of the application, refer to Figure 3-1, which upon review notified that calculated flows for comparison could only be extracted for the Little Dargle and the River Slang. Extracted flows have been compiled for present day, Table 3-3, and Mid-Range Future Scenario (MRFS), Table 3-4 and

Table 3-5. No detail was provided for the High End Future Scenario (HEFS).

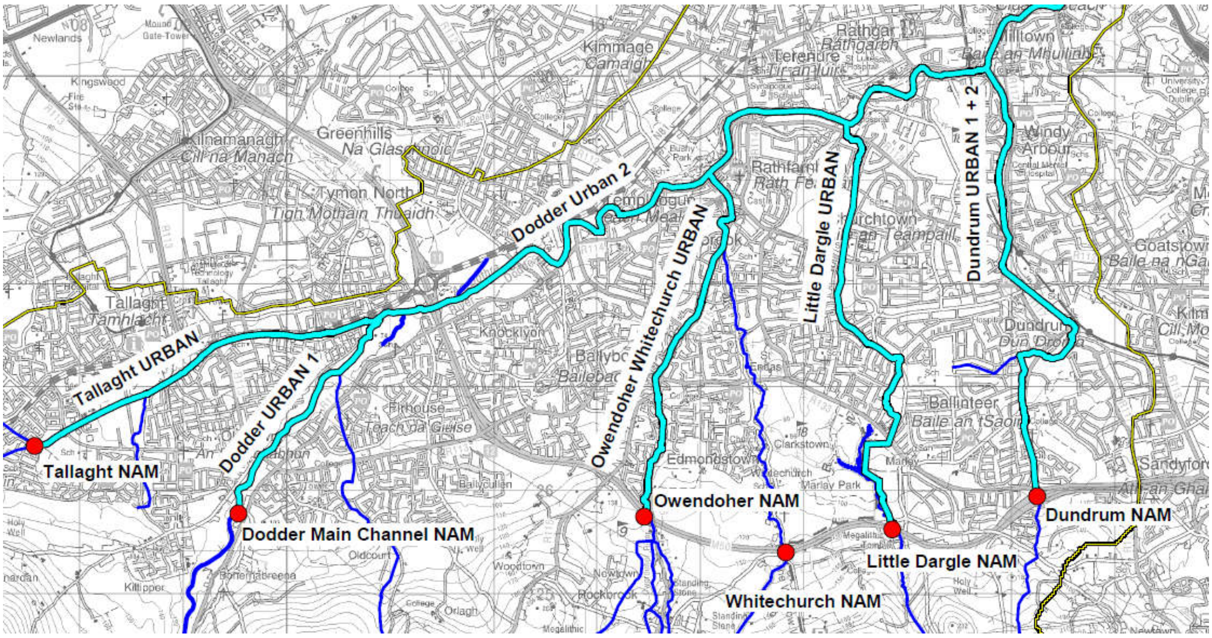

Figure 3-1 River Dodder CFRAM Application of Hydrology[2]

Table 3-3 CFRAM flows (Present Day)[3] in m³/s

		Return Period (years)							
RR Boundary Catchment	**Probability Distribution**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**1000**
Little Dargle NAM	Generalised Pareto (GP2/ML)	7.12	8.8	10.67	13.42	15.79	18.47	21.5	30.19
Little Dargle URBAN	Log Pearson Type 3 (LP3/LMOM)	4.54	5.89	6.89	8.37	9.65	11.09	12.73	17.47
Little Dargle Catchment Flow		**11.66**	**14.69**	**17.56**	**21.79**	**25.44**	**29.56**	**34.23**	**47.66**
Dundrum NAM	Log Pearson Type 3 (LP3/LMOM)	3	4.08	5.12	6.86	8.55	10.65	13.24	21.91
Dundrum URBAN 1+2	Generalised Pareto (GP2/ML)	6.59	9.60	11.99	15.58	18.69	22.25	26.32	38.18
River Slang Catchment Flow		**9.593**	**13.68**	**17.116**	**22.435**	**27.247**	**32.904**	**39.56**	**60.09**

[2] Extracted from River Dodder Catchment Flood Risk Assessment and Management Study - Hydrological Analysis Report – Issued 31.10.2008 - Figure E.1

[3] Extracted from River Dodder Catchment Flood Risk Assessment and Management Study - Hydrological Analysis Report – Issued 31.10.2008 – Table 6-3

Table 3-4 CFRAMs flows (MRFS) – Assuming FULL Implementation of SUDS in m^3/s[4]

		Return Period (years)							
RR Boundary Catchment	**Probability Distribution**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**1000**
Little Dargle NAM	Generalised Pareto (GP2/ML)	9.04	11.1	13.26	16.23	18.63	21.2	23.94	31.09
Little Dargle URBAN	Log Pearson Type 3 (LP3/LMOM)	6	7.8	8.94	10.46	11.66	12.95	14.33	17.96
Little Dargle Catchment Flow		**15.04**	**18.9**	**22.2**	**26.69**	**30.29**	**34.15**	**38.27**	**49.05**
Dundrum NAM	Log Pearson Type 3 (LP3/LMOM)	3.63	4.78	5.82	7.5	9.06	10.93	13.17	20.21
Dundrum URBAN 1+2	Generalised Pareto (GP2/ML)	8.78	12.46	15.78	21.35	26.78	33.58	42.13	71.47
River Slang Catchment Flow		**12.41**	**17.24**	**21.6**	**28.85**	**35.84**	**44.51**	**55.3**	**91.68**

Table 3-5 CFRAMs flows (MRFS) – Assuming NO Implementation of SUDS in m^3/s[5]

		Return Period (years)							
RR Boundary Catchment	**Probability Distribution**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**1000**
Little Dargle NAM	Generalised Pareto (GP2/ML)	6.19	8.19	9.77	12.14	14.2	16.54	19.22	27.02
Little Dargle URBAN	Log Pearson Type 3 (LP3/LMOM)	7.85	10.89	12.75	15.19	17.11	19.15	21.33	27.05
Little Dargle Catchment Flow		**14.04**	**19.08**	**22.52**	**27.33**	**31.31**	**35.69**	**40.55**	**54.07**
Dundrum NAM	Log Pearson Type 3 (LP3/LMOM)	2.11	2.76	3.34	4.27	5.12	6.13	7.32	11
Dundrum URBAN 1+2	Generalised Pareto (GP2/ML)	11.2	16	20.9	28	35.6	45	55.5	92
River Slang Catchment Flow		**13.31**	**18.76**	**24.24**	**32.27**	**40.72**	**51.13**	**62.82**	**103**

[4] Extracted from River Dodder Catchment Flood Risk Assessment and Management Study - Hydrological Analysis Report – Issued 31.10.2008 – Table 6-6
[5] Extracted from River Dodder Catchment Flood Risk Assessment and Management Study - Hydrological Analysis Report – Issued 31.10.2008 – Table 6-8

3.2 Historic Flood Data Analysis

A search for historic records of previous flood events has been undertaken, to determine new information over and above that considered by previous studies. The initial step was to define instances of peak flows thereby pinpointing specific floods to review. Peak flow data from the last 115 years was reviewed and ranked with findings presented in Table 3-6.

Table 3-6 Highest Estimated Peak Flows since 1905

Date	Peak Flow (m^3/s)
26/08/1986*	254.769
24/10/2011*	220.178
25/08/1905	198
03/09/1931	153
05/11/2000*	148.971
17/11/1965	138.75
05/09/2008*	119.443
19/12/1958	116.10
02/12/2003*	114.921
05/11/1982	105.630
14/11/2014*	99.755
03/02/1994*	92.056
16/01/2010*	85.624
02/11/1968	84.950
01/12/1983	82.270

*Note: Peak Flows have been updated using study rating curve

The highest recorded flows are attributed to Hurricane Charlie which caused extensive flooding on the 25th & 26th August 1986. It was summated that 400 properties on the River Dodder and 520 properties on the Little Dargle were affected by this storm. Review of available data uncovered a hand drawn extent of the flooding which has been digitised and will be used in historical calibration of the hydraulic modelling, refer to Figure 3-2.

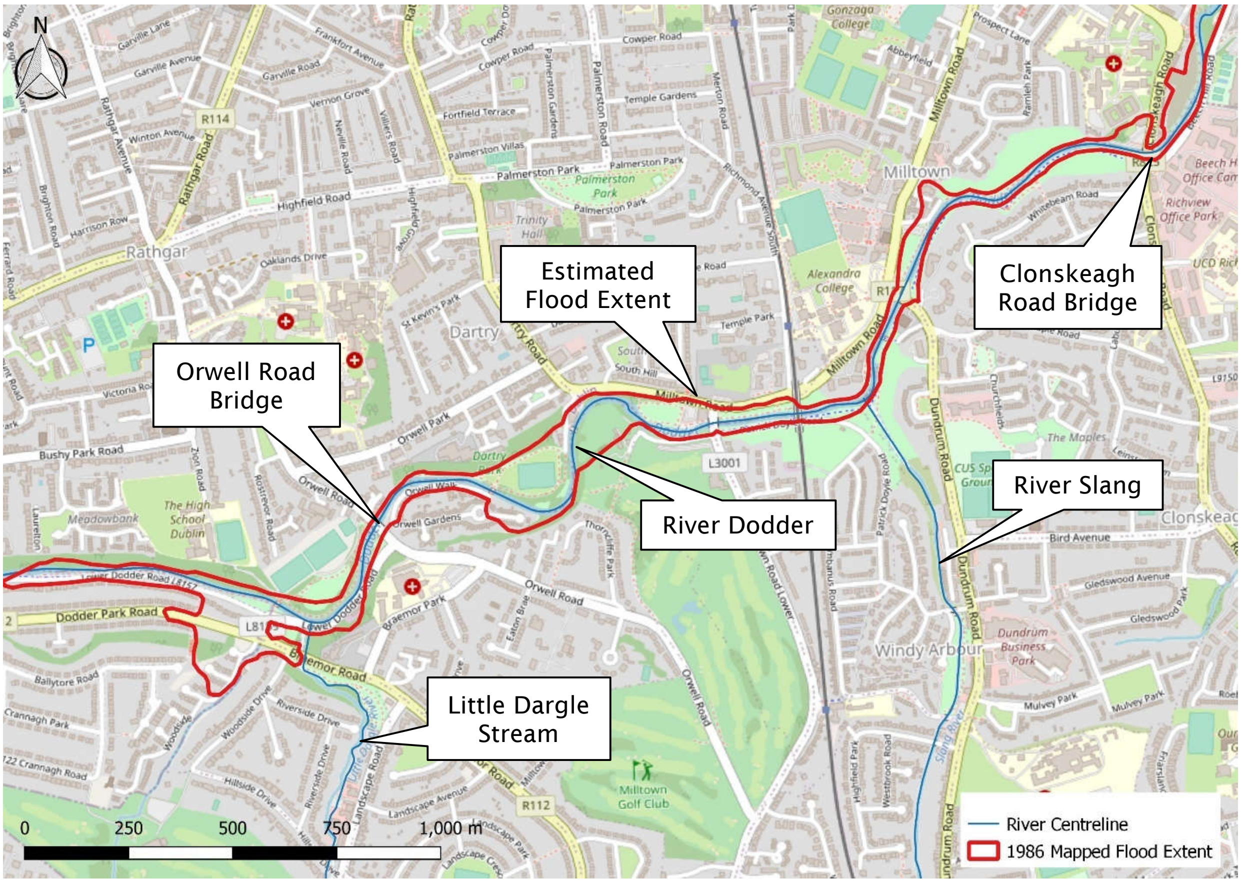

Figure 3-2 Extent of Flooding Hurricane Charlie August 1986

The second largest storm was experienced in October 2011 with peak flows of circa 220m³/s. It was recorded that Darty Road, Milltown area and Stillorgan Road in Donnybrook to Lansdowne Road Bridge were inundated. A total of 192 dwellings in addition to 136 other buildings/ non-residential ground floor units were affected by the flood. Extent maps for this event were produced and issued in the Overarching Report of the October 2011 Flood Event as part of the Eastern CFRAM Study (Doc reference IBE0600Rp0014). A review of the extent informs that it is not dissimilar to that of the 1986 event with flooding realised in the same locations, albeit not to the same magnitude. The extent maps have been collated to produce a singular extent which has been provided in Figure 3-3.

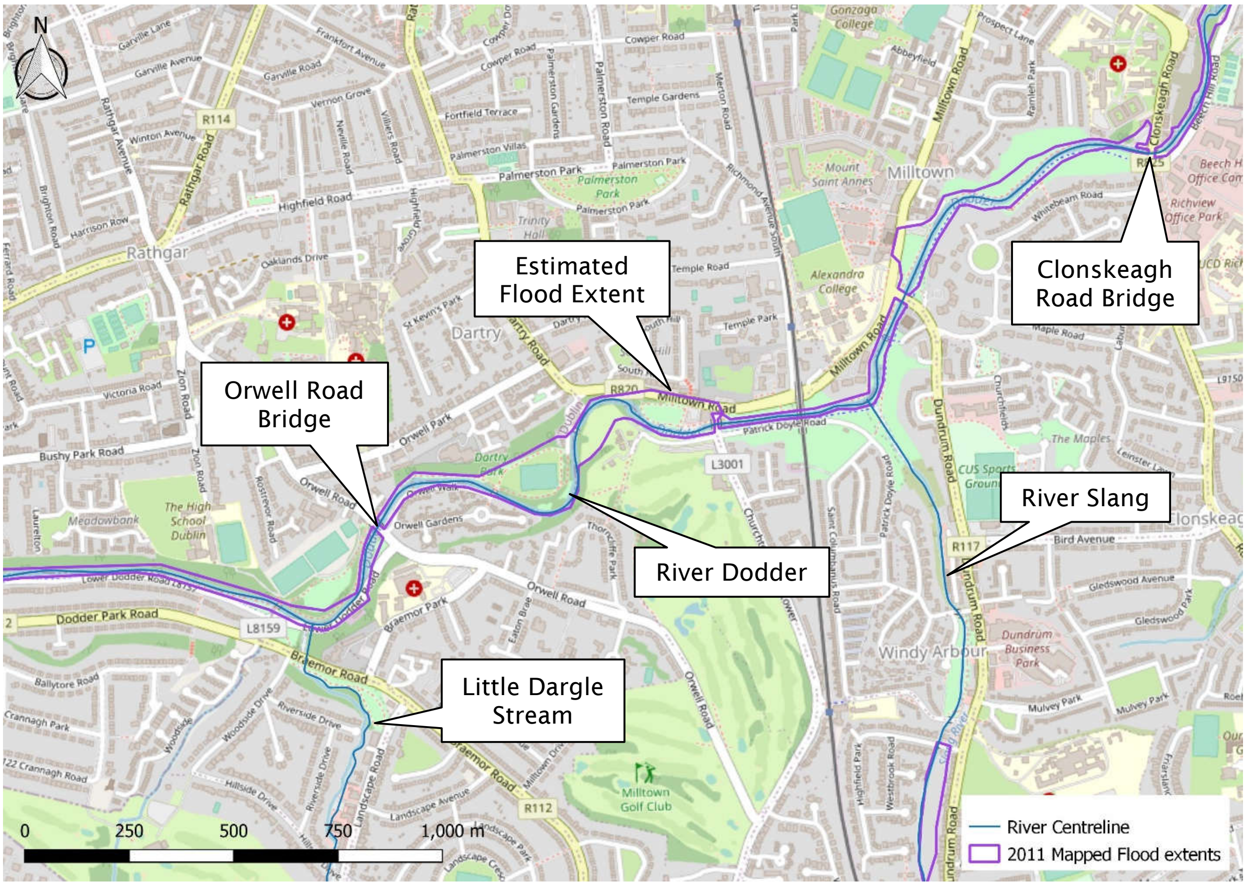

Figure 3-3 Extent of Flooding in 2011[6]

Although there are many documents detailing peak flow events in the study area it is noted that the detail required to enable calibration was difficult to attain. Mapping was produced at time of CFRAM defining areas of flooding that were detailed on floodmaps.ie. That mapping has been updated to account for occurrences of flooding post CFRAMs study thereby ensuring replication of flooding areas which have experienced flooding are appropriately represented within this study. This mapping will be used at time of hydraulic modelling as a means of calibration / validation.

3.3 Environmental Protection Agency (EPA) Data

The EPA provided data for two gauges:

3.3.1 09010 Waldron's Bridge

- Sited on the left-hand bank of the River Dodder approx. 25m upstream from the Orwell Road Bridge. EPA note upstream catchment size of 94.3km² and gauge datum of 30.105m OD (Poolbeg)
- The detailed data provided comprised of level gauging information and calculated flow datasets providing information from 01/08/1996
- EPA informed that the control for the station was the Orwell Weir, located 190m downstream from the gauge, and that the weir had been damaged in October 2000 with a stone removed causing a change to the rating curve. The stone was replaced in August 2003.
- Three rating curves were supplied associated with the gauge data
    - C1.2 – valid 24/04/1979 to 02/10/2000 (Prior to stone removal)
    - C4.2 – valid 02/10/2000 to 27/08/2003 (Stone removed)

[6] Overarching report of October 2011 flood event – Eastern CFRAM Study – IBE0600Rp0014 – Issued 01.05.2013 – Appendix A Flood Extent Maps A001 to A021

- C5.2 – Valid 07/08/2003 to present day (last updated 07/06/2020)

3.3.2 09011 Frankfort

- Sited on the River Slang on the left bank approx. 20m upstream of the road bridge to Frankfort Court. EPA note upstream catchment size of 5.5km² and gauge datum of 40.503m OD (Poolbeg)
- The detailed data provided comprised of level gauging information and calculated flow datasets providing information from 05/05/1982
- EPA informed the station was installed in 1982 as a velocity-area station with natural control. A non-standard flat v concrete control was installed in August 1986
- Two rating curves were supplied associated with the gauge data
  - C1.2 – valid 05/05/1982 to 23/07/1985 (natural control)
  - C2.2 – valid 14/08/1986 to present day (last updated 07/06/2020)

The gauging stations are well sited, refer to Figure 3-4, to inform of main channel and tributary flows for this study. Gauging information for several large magnitude storms has been captured at each location thereby allowing for appropriate calculation of flows using statistical analysis

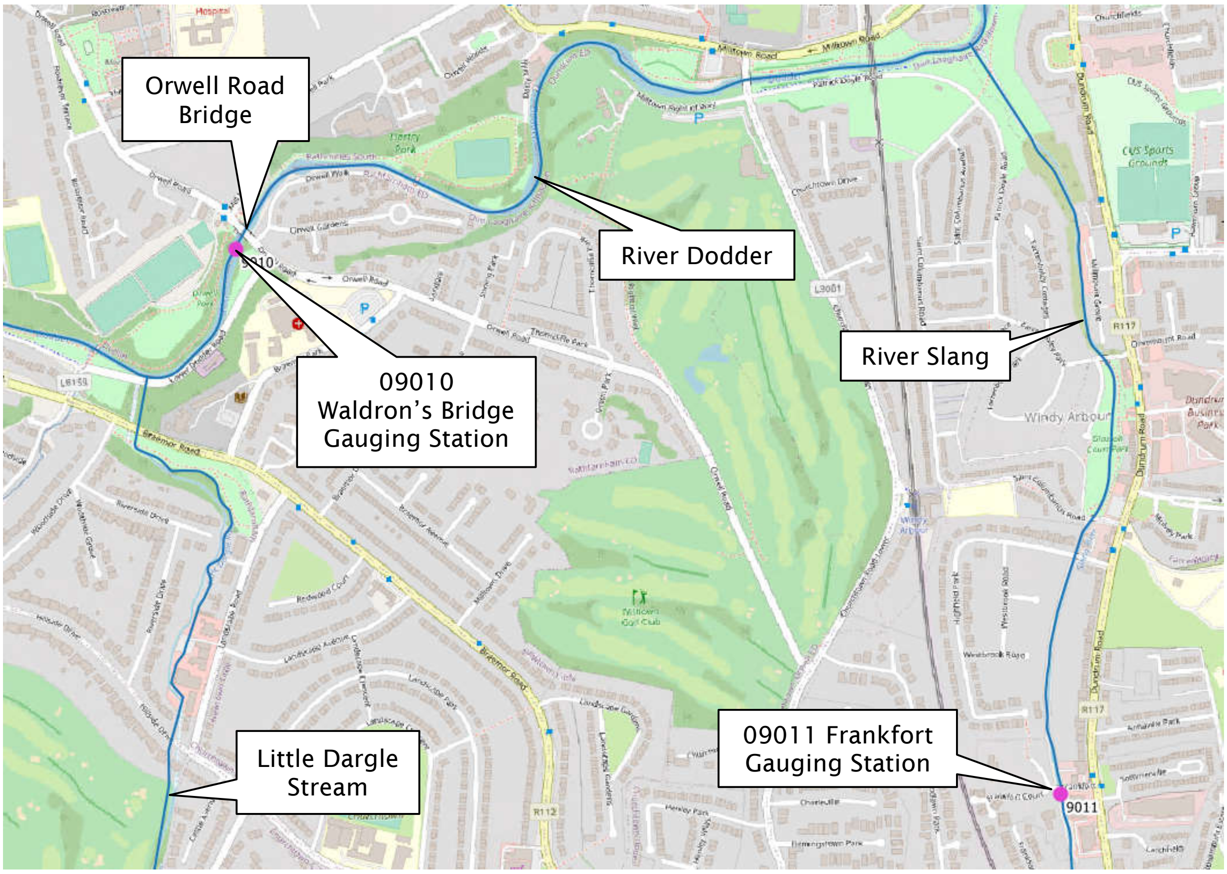

Figure 3-4 Gauging Station Locations

Review of the available survey data at Waldron's Bridge informed that it was not of suitable quality to allow a detailed rating review to be conducted. Therefore, the EPA rating curves have been used to progress the hydrological calculations until such times as updated survey data has been issued for use.

The River Slang was subject to a separate study, completed in December 2019, in which a calibrated and verified model was produced using catchment wide monitoring, refer to Section 3.4 for more detail. Due to this no rating review will be conducted at the Frankfort location as results from these works conducted can be applied to this study.

3.4 River Slang (Dundrum) Integrated Catchment Model

A study was commissioned by Dún Laoghaire-Rathdown County Council (DLRCC) to construct and verify an integrated catchment model of the River Slang catchment using InfoWorks ICM. The study was completed in December 2019 and was provided for information given relevance to the Dodder Phase 3 reach.

The model included the surface water network and the open watercourse within the catchment. Overland flow routes were simulated in 2D on ground levels taken from LiDAR data or topographical surveys. Details for the open watercourse were provided by previous studies conducted within the catchment coupled with new survey data collected to infill and enhance existing data along the watercourse.

3.4.1 Rating Review

The study reported that a rating review was conducted for the Frankfort Gauging Station as part of the study using EPA and newly collected survey information. The verified model was simulated for the 0.1% AEP with a rating curve developed from the simulated results, refer to Figure 3-5.

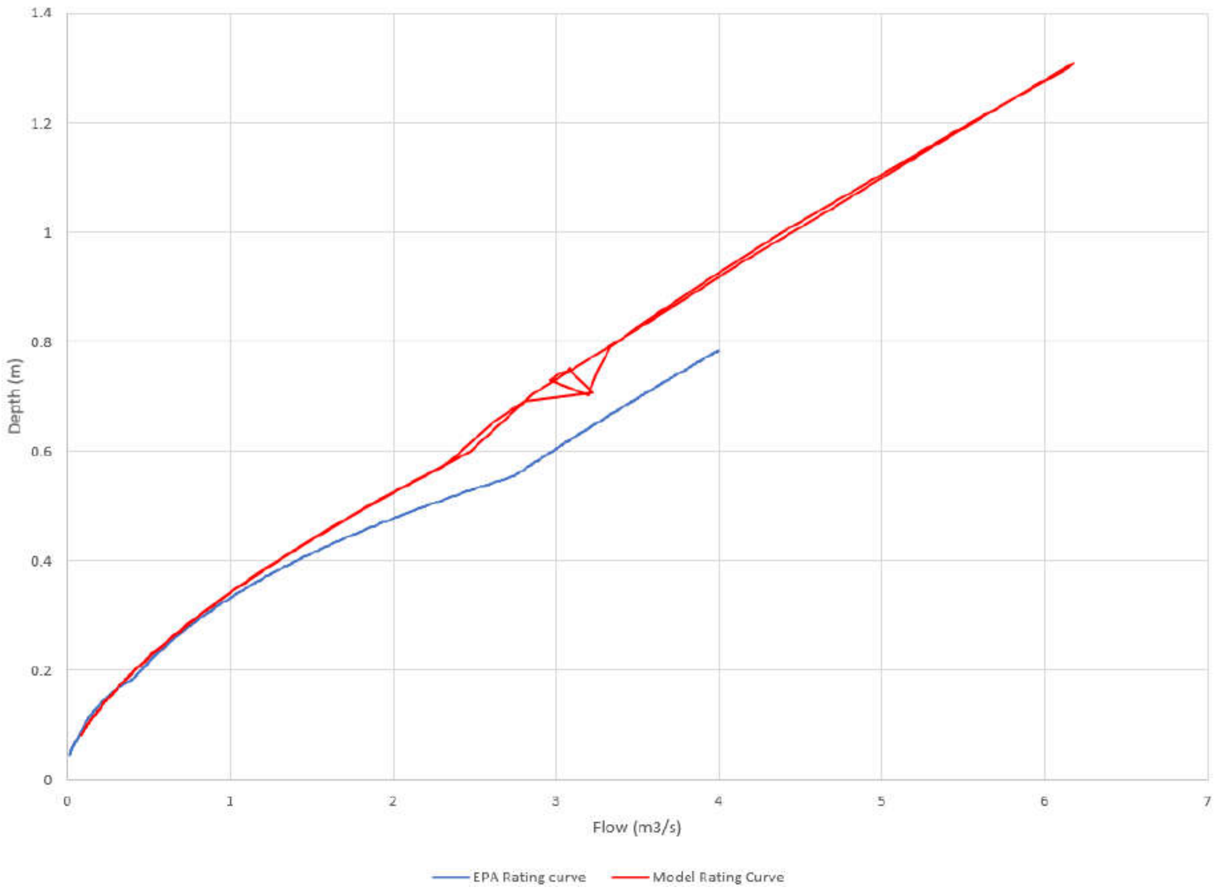

Figure 3-5 Rating Curve Comparison from River Slang Study[7]

Upon comparison with the EPA rating curve, reporting noted that flow depths up to 0.4m were as per EPA data, but the verified model diverged indicating a steeper stage discharge relationship at depths exceeding 0.4m when compared to the EPA rating curve. The graph in Figure 3-5 informs of a flow for the 0.4m level of approximately 1.4 m^3/s which equates to less than a 50% AEP.

The Slang study informed that no further assessment was conducted in the Slang study on the more conservative results given the location of the gauging station within the catchment and the flow verification conducted upstream of the gauge location. As the calibrated model results were deemed to hold greater confidence than that of statistical analysis at the gauge, the calibrated model was not updated subsequent to rating review.

[7] River Slang (Dundrum) Integrated Catchment Model – Hydraulic Modelling Report –Issued December 2019 – Figure 3-3 Comparison between the modelled and EPA rating curve

3.4.2 Modelled flows

The study report states that due to the highly urbanised nature of the catchment, typical methodologies such as FSU and IoH124 were not appropriate. Flows have been calculated at HEP locations, but they have been derived through the modelling process and therefore take account of restrictions in the surface water system, where the Slang essentially acts as an extension of the urban drainage network. Although discounted as a viable methodology, FSU was calculated and used for comparison against modelled (ICM) flows. Two HEPs have been selected from those where flows have been assessed for comparison in this study, namely

- 09_1381_6_01 Frankfort Gauging Station
- 09_1381_8_01 Confluence with the River Dodder

Table 3-7 HEP flows (FSU & ICM) for River Slang (as detailed in report)

	10% AEP (m^3/s)		1% AEP (m^3/s)		0.1% AEP (m^3/s)	
HEP	**FSU**	**ICM**	**FSU**	**ICM**	**FSU**	**ICM**
09_1381_6_01	4.68	4.76	7.26	8.73	9.79	10.82
09_1381_8_01	6.29	4.00	9.68	7.80	13.01	16.60

It is noted that modelled (ICM) flows exceed those at the gauging station in all instances whilst at the downstream confluence modelled flows are less, refer to Table 3-7. Reporting informs this is attributed to flow attenuation in pipes for the 10% and 1% AEPs with capacity being exceeded during the 0.1% AEP with most of the flow entering the watercourse form overland sources. Dublin County Council (DCC) have commented in relation to this stating that there would be typically no road flooding in a 1:10 year event and no property flooding for a 1:30 event. New developments should not flood in a 1:100 event with on-site attenuation required.

3.4.3 Report Conclusion

The study report concludes that the model has been successfully verified against flow survey data. The flooding experienced in October 2011 was successfully replicated with overland flow paths and extents represented. Testing of the downstream boundary informed that the application of a 1% flow level in the River Dodder increased levels in the River Slang for just over 700m but with limited impacts on predicted flooding. The full hydraulics report for this study has been provided in Appendix A.

3.4.4 Fitness for Inclusion

On review of the Slang project outcomes, the recency, nature of the project methodology and complexity, and confidence in its outcomes means it is reliable for use in the Dodder Phase 3 project.

3.5 GDSDS

The Greater Dublin Strategic Drainage Study (GDSDS) was conducted between 2000 and 2006 with the objective to carry out a strategic analysis of the existing foul and surface water systems in the local authority areas. As part of these studies modelling was conducted in Innovyze InfoWorks CS software producing verified models using survey and monitoring data. These models were provided for use in this study and included models for:

- F005 Rathmines & Pembroke
- F006 Dodder DLRCC
- F007 Dodder SDCC
- S2009 Dodder Owendoher
- S2010 Dodder Whitechurch

- S2011 Dundrum Slang
- S2013 Dargle

All the models were converted from InfoWorks CS to Innovyze ICM (v10.5) to allow high level review to ascertain content and relevance to this study. It was noted that the models were predominantly representing foul networks with only the S2011 Dundrum Slang model applicable for use due to replication of storm network.

The S2011 Dundrum Slang model was subject to closer inspection informing that model extent included the River Slang and Little Dargle catchments draining to the River Dodder. An audit informed the model had hydrology applied via contributing area (sub-catchments) and was to a standard allowing for simulation in the updated software. Simulations were conducted for 13 durations ranging from 30 to 720 minutes for both summer and winter profiles and all the required AEPs. Upon completion the critical duration was ascertained at key locations allowing comparison against other flow calculation techniques, refer to Section 7.3.3 for more detail.

3.6 Irish Water Data

Irish Water provided network data in GIS format for the study area for use in this study. Data received included but was not limited to pipe, manhole and outfall locations for the surface water network with checks informing it was last updated in February 2020.

An inspection of the data was conducted against the GDSDS dataset, specifically the 2011 Dundrum Slang model, and found that it correlated well where there was data overlap. Additional data was noted adjacent to the River Dodder and this was extracted from the Irish water Dataset for use in this study, refer to Figure 3-6.

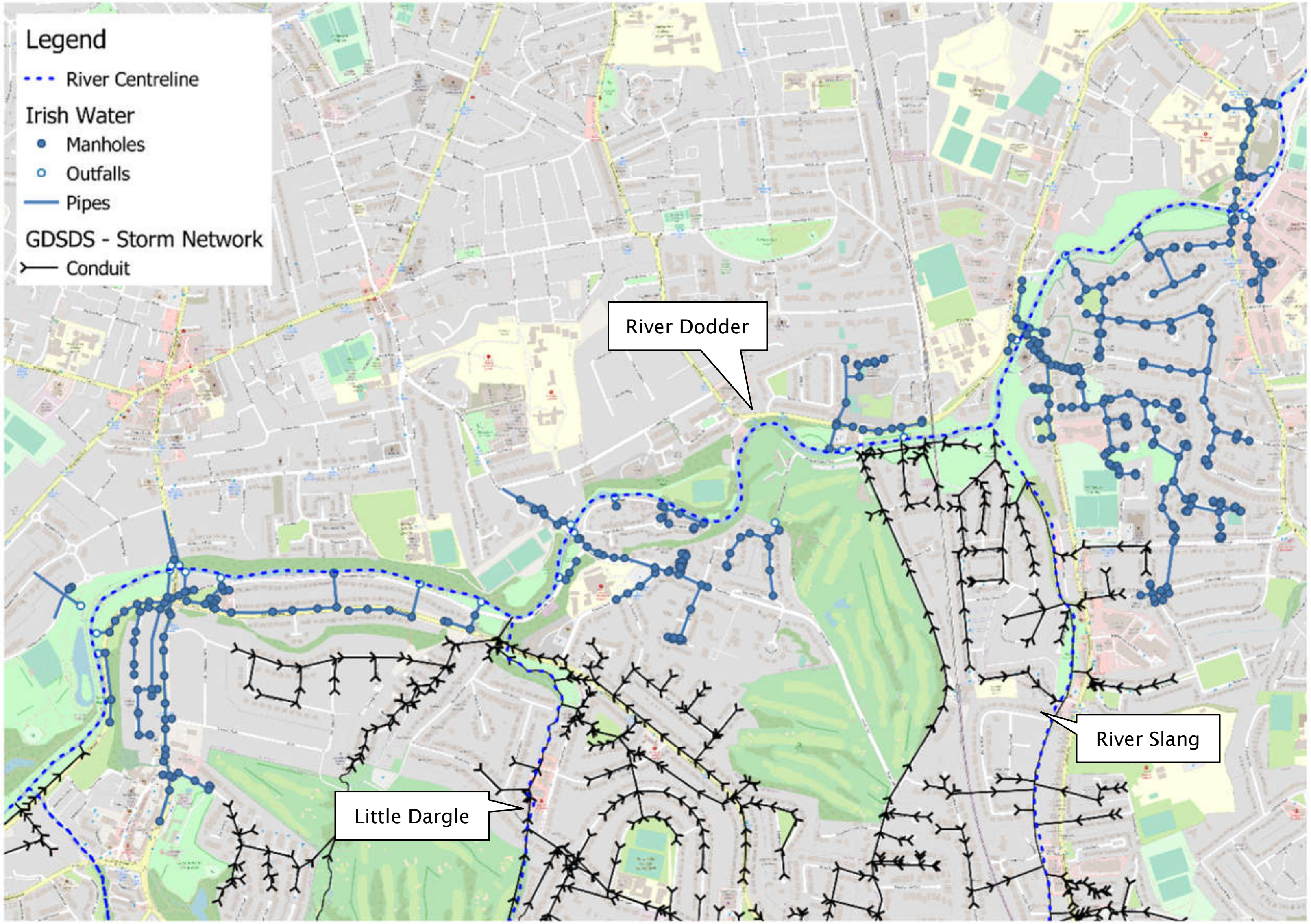

Figure 3-6 Irish Water Data for use in Study

3.7 Arterial Drainage

Review of OPW arterial drainage schemes[8] indicate there are no arterial drainage schemes or benefitting lands within the model catchment.

3.8 Previous Phases of River Dodder FAS

No substantial data has been provided for review on previous phases of the River Dodder Flood Alleviation Scheme (FAS) and thereby no comment can be made regarding works undertaken as part of those schemes.

The proposed downstream model extent for the present phase (Phase 3) is located between two substantial weir structures as shown on Figure 3-7. The elevation difference arising at those weir structures allows comfort that any backwater effect on water levels from previous or ongoing alleviation works at downstream phase phases would not affect predicted water levels used to inform Phase 3 . Figure 3-8 and Table 3-8 provide survey detail that informs of a difference in level between the upper weir, location 1, and bed level of downstream section of the lower weir, location 4, of 5.44m.

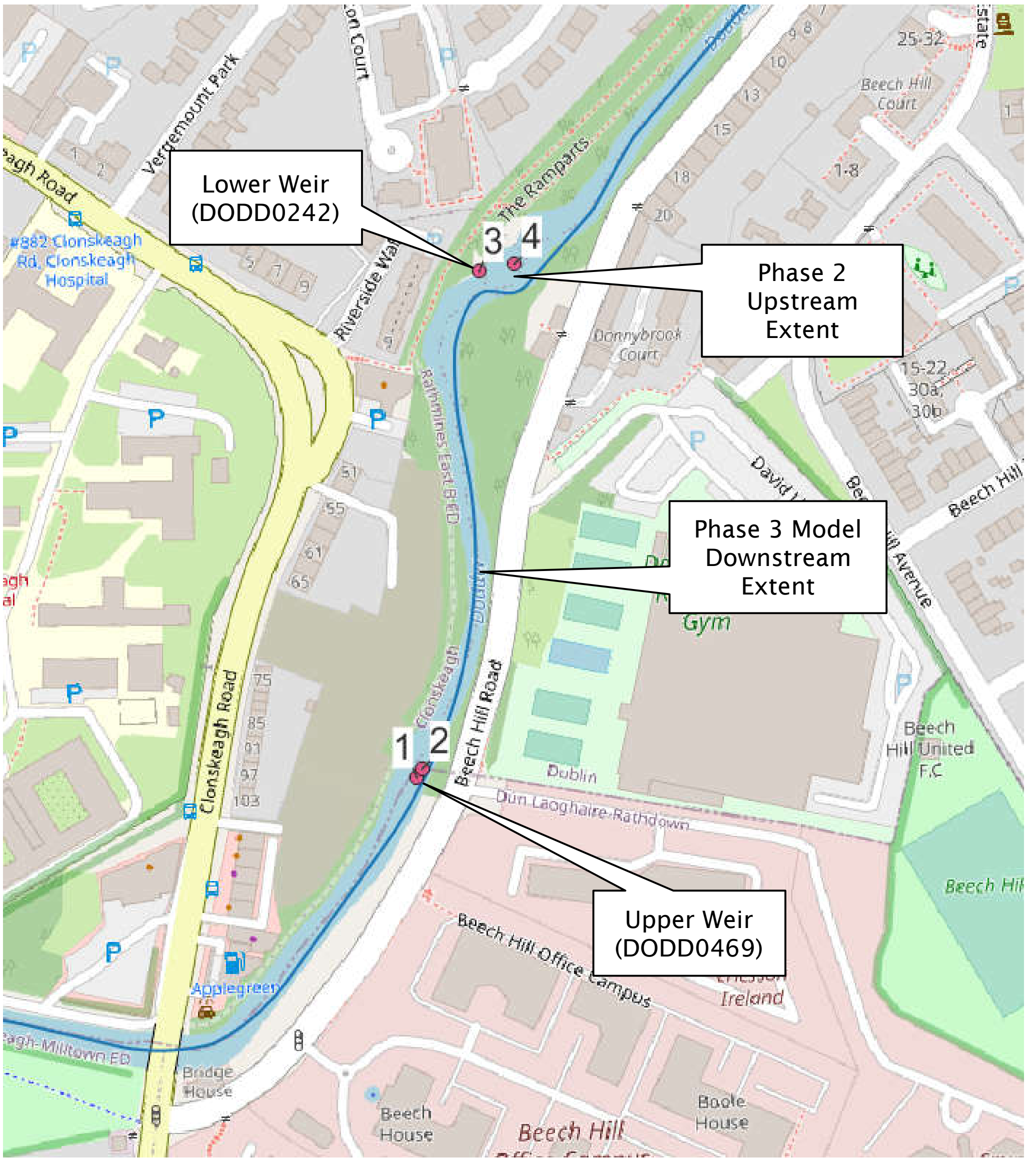

Figure 3-7 Location of Extent Relative to Downstream Weirs

8 OPW Floodinfo.ie. (2020). OPW Arterial Drainage Schemes. Available from: https://www.floodinfo.ie/. [Accessed: 2/9/2020].

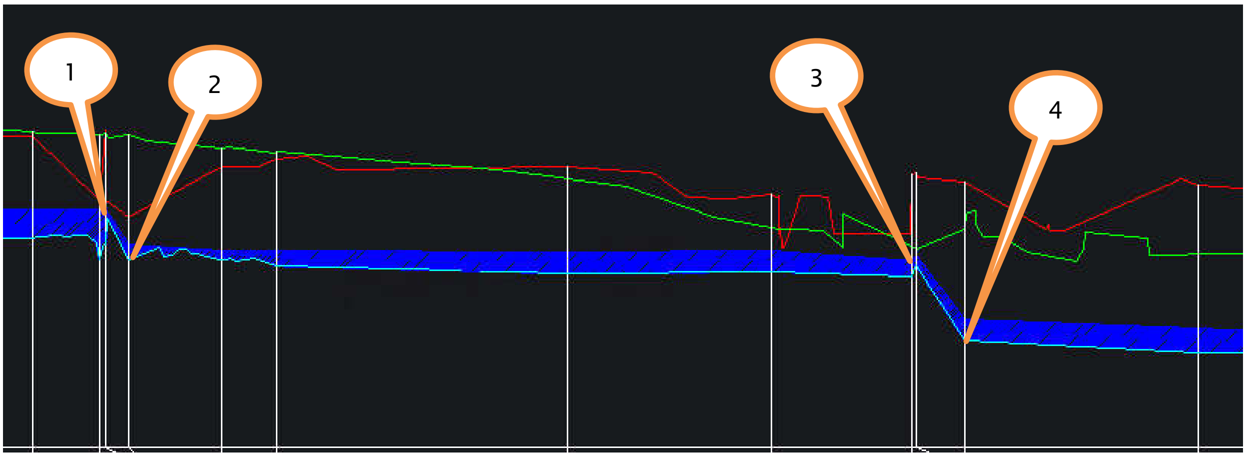

Figure 3-8 Surveyed Long Section of the Downstream Weirs

Table 3-8 Surveyed Weir Level Detail

Location	Cross Section No.	Level (m OD)
1	DODD0469	11.47
2	DODD0468	10.55
3	DODD0242	10.21
4	DODD0232	6.03

In conclusion; the Phase 3 scheme reach is hydraulically (in elevation terms) separate from previous Dodder flood alleviation phases.

Future assessment of Phase 3 options will assess effects at the downstream extent of the Phase 3 model reach as part of the Hydraulics / Options appraisal exercise to determine the significance of any hydraulic change leaving the scheme reach that may adversely affect earlier downstream phases.

4 FLOOD SOURCE SCREENING

4.1 Purpose

This chapter is an evaluation of sources of flooding and their influence on the hydrological setting. It will investigate sources of flooding and their significance in relation to the estimation of fluvial hydrology for the proposed study area.

4.2 Groundwater / Hydrogeology

Groundwater flooding occurs when water stored beneath the ground rises above the surface of the land. In Ireland, the most extensive form of groundwater flooding is related to prolonged rainfall causing water table rise in limestone lowland areas, primarily in the west of the country. A desktop review was completed to assess the influence of groundwater on the Scheme Area. This review was complete using available local data and national mapped datasets.

4.2.1 Bedrock Aquifers

The hydrological catchment draining to the study area lies over 3 distinct geological units.

- The north of the catchment (typically land lower lying than 150 m OD) lies over limestone and shale of the Lucan Formation.
- The west of the catchment lies over slate, quartzite and siltstone of the Butter Mountain and Aghfarrell Formations.
- The east and south of the catchment lies over granites.

Bedrock geology is shown on Figure 4-1, page 21.

Limestone and quartzite formations are classed as locally important aquifers (moderately productive only in local zones) while granites and slate are classed as non-aquifers. Aquifer distribution is shown on Figure 4-2, page 22.

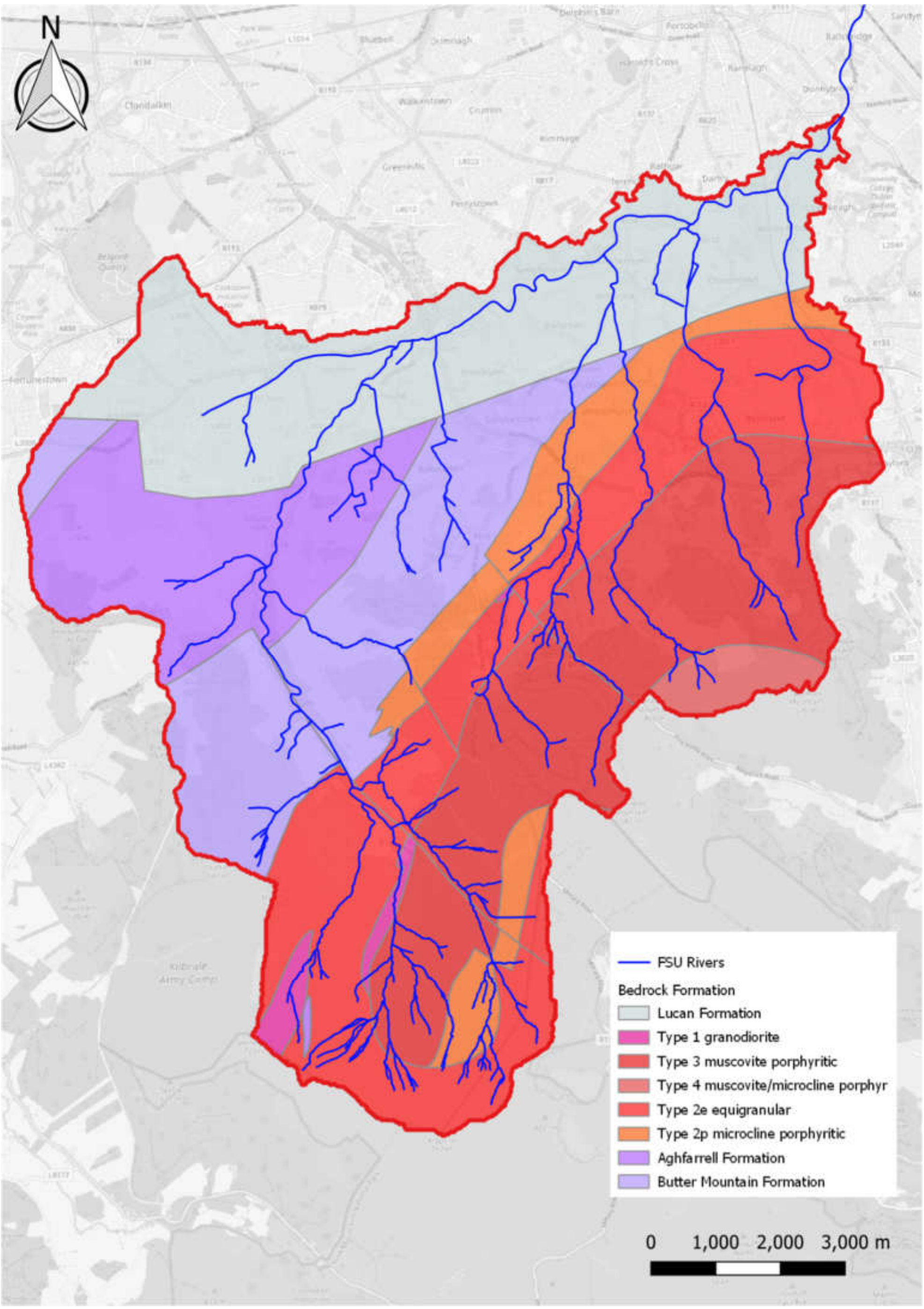

Figure 4-1 Bedrock Geology

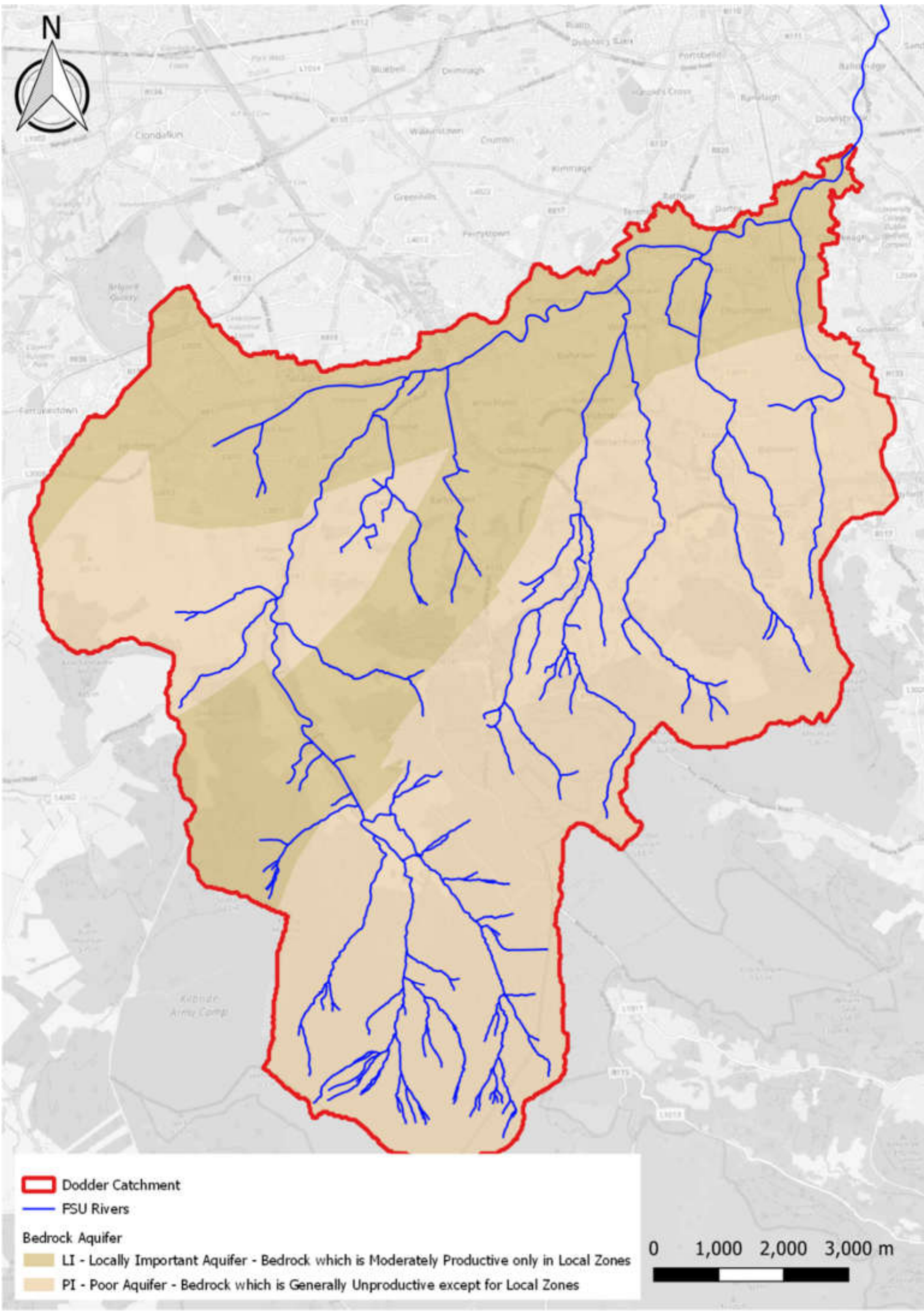

Figure 4-2 Bedrock Aquifers

4.2.2 Superficial Cover

Land within the catchment typically higher than 500m OD is overlaid by blanket peat, refer to Figure 4-3. Land lower lying that 150m OD is typically overlaid by glacial till derived from limestones with pockets of gravel derived from limestones. Land between those level bands (150 - 500m OD) typically comprises exposed bedrock outcrops and tills from limestones and metamorphic rock. Alluvium deposits coincide with the main channel of the River Dodder.

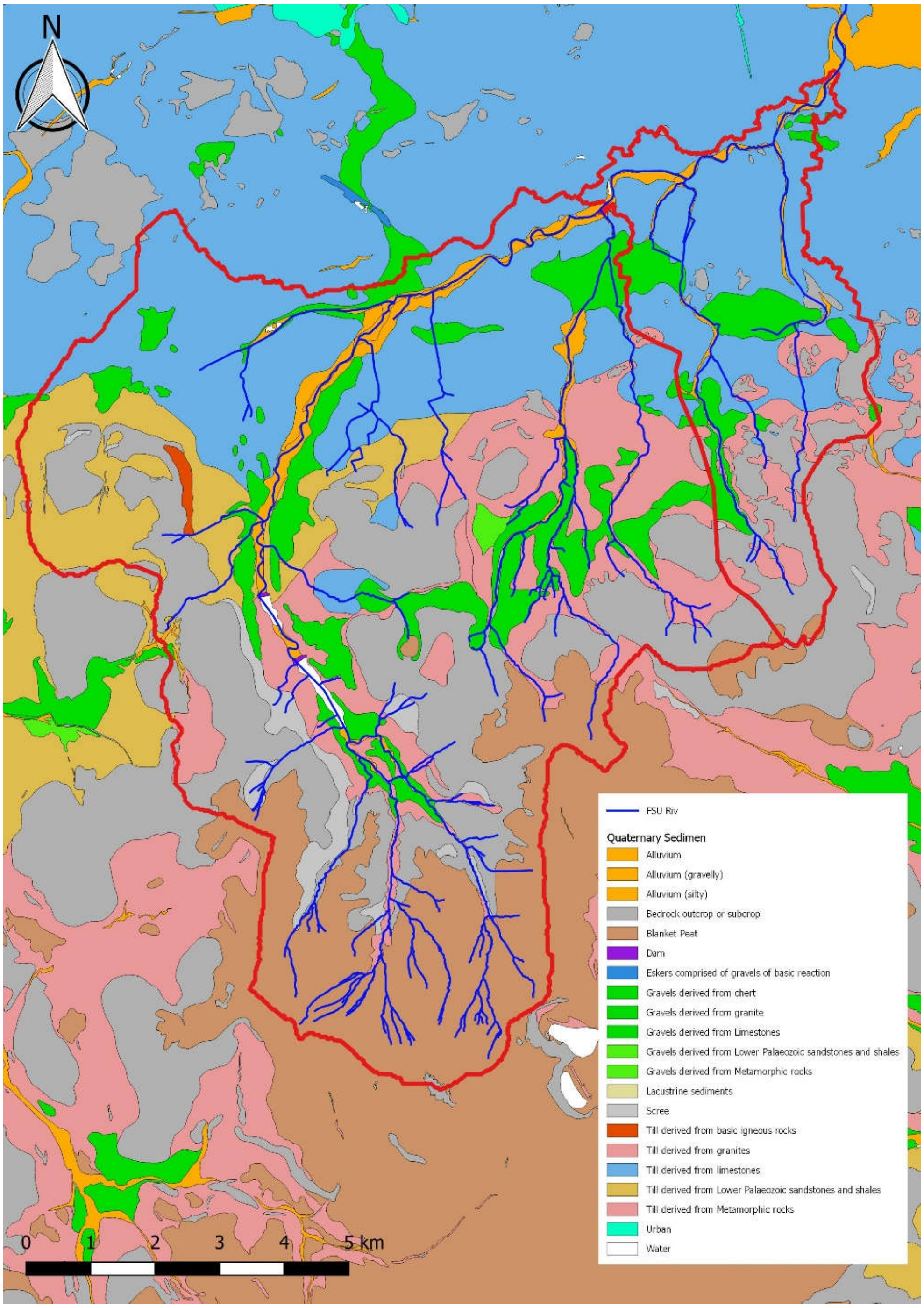

Figure 4-3 Superficial Cover (Quaternary Geology)

Subsoil permeability coinciding with tills is mapped as low, refer to Figure 4-4. Permeability coinciding with alluvial gravels is medium to high.

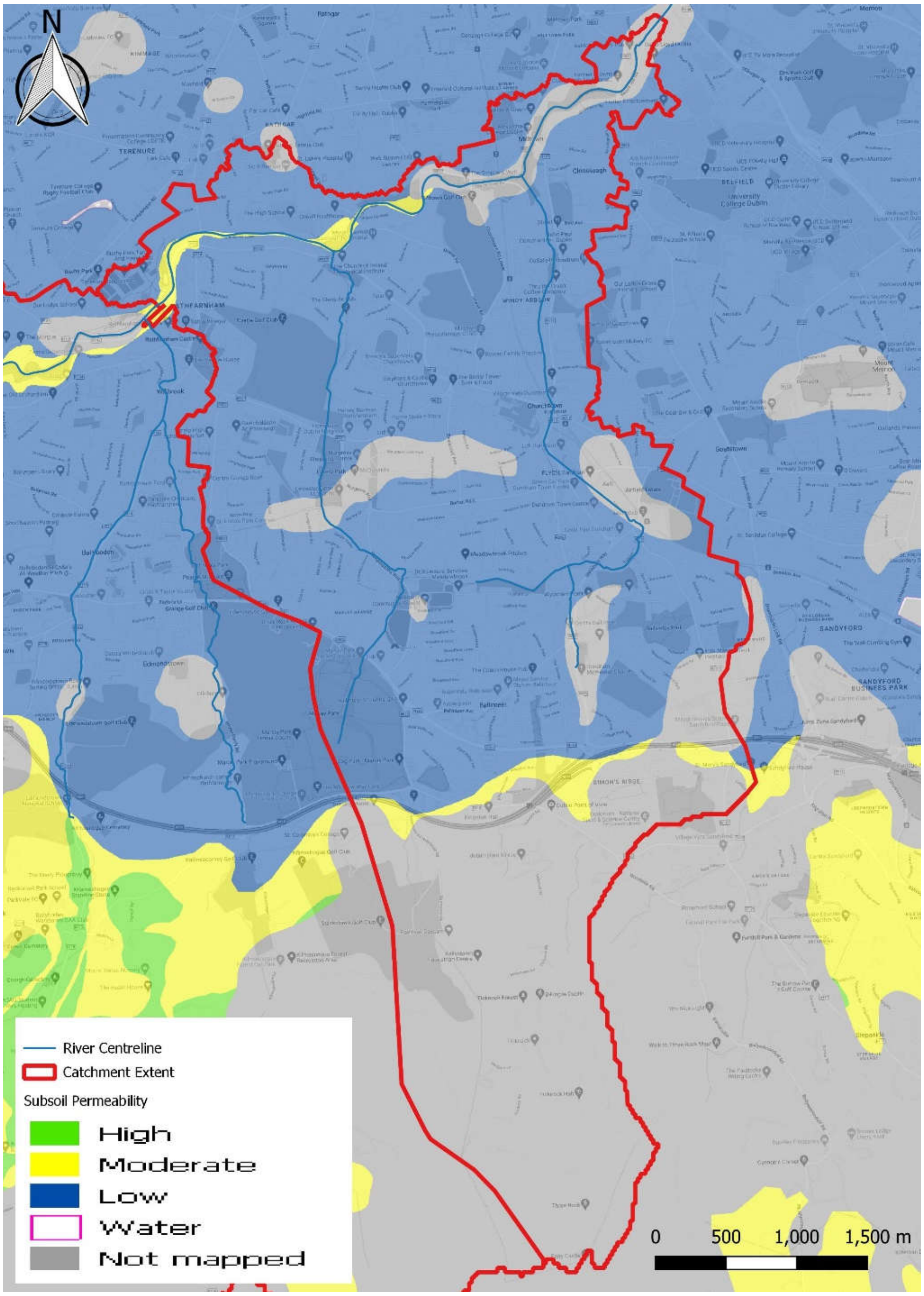

Figure 4-4 Subsoil Permeability

4.2.3 Karst Features

The GSI karst feature database indicates no karst features or known flows within the hydrological catchment.

4.2.4 Springs & Boreholes

Review of the GSI borehole and springs database inferred no springs within the hydrological catchment subject to assessment that would suggest significant groundwater at or near surface.

Boreholes and wells, refer to Figure 4-5, sited in the north of the catchment are drilled to a depth (>60m) implying that groundwater is at depth and has no interaction with surface hydrology.

Remaining boreholes are sited at elevations >200m OD with deep (>60m) boreholes into gravels adjacent to the Bohernabreena reservoir complex. A single spring exists at St Annes Well at an elevation of approximately 250m OD and is likely to be caused by shallow superficial groundwater emerging at the interface of till and gravel deposits.

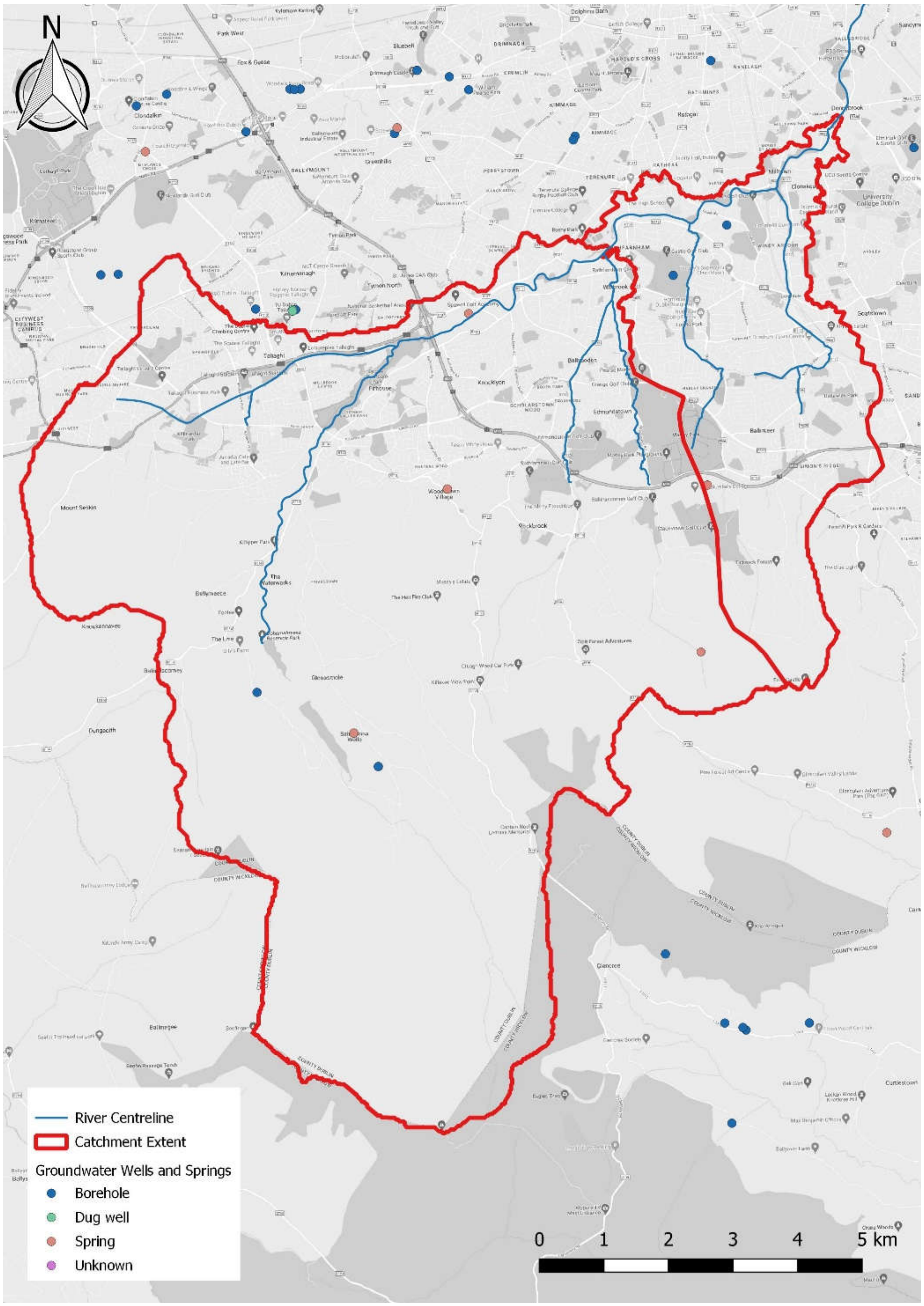

Figure 4-5 Springs & Boreholes

4.2.5 Groundwater Flood Database

Review of the GSI predicted groundwater flooding database indicates no areas of predicted flooding proximal to the area of investigation.

4.2.6 Summary

From a review of the available information and inspection of the catchment there is no evidence of significant groundwater influence on fluvial hydrology in the catchment. Ground conditions in conjunction with topography is likely to cause the risk of clearwater (above ground) or below-ground groundwater flooding to be insignificant in the Scheme Area and the upstream contributing catchment.

4.3 Pluvial / Surface Water / Urban Drainage

The catchment(s) that could contribute direct pluvial overland flooding to the River Dodder have been evaluated by determining the extents of the upstream hydrological catchment and associated significant flow paths within the extents of the proposed model build.

The analysis used a GIS evaluation of the terrain model formed using 25m resolution OSI LiDAR data provided for use in this study. The algorithm uses a Rho-8 type “rolling ball” hydrological analysis to determine key flow paths and drained areas. The analysis does not account for contributing area or network data provided through GDSDS or Irish Water datasets and as such has been updated at time of catchment delineation.

The analysis determined that the relevant cumulative pluvial catchment draining is 3.76 km², comprising 3.37 km² urban, 0.31 km² sport and leisure facility and 0.08 km² green urban area as defined by the Corine landcover dataset. The urban green area is Bushy Park providing an overland flow route at the upstream extent of the proposed model build and outside of the location of proposed defences, whilst the sport and leisure facility is the Milltown Golf Club which provides an overland flow route to the River Dodder upstream of the River Slang confluence and on the opposite side of the river to proposed defence locations. Catchments and predicted flow paths are shown on the following figures, please note that GDSDS networks have been omitted from these figures to retain clarity.

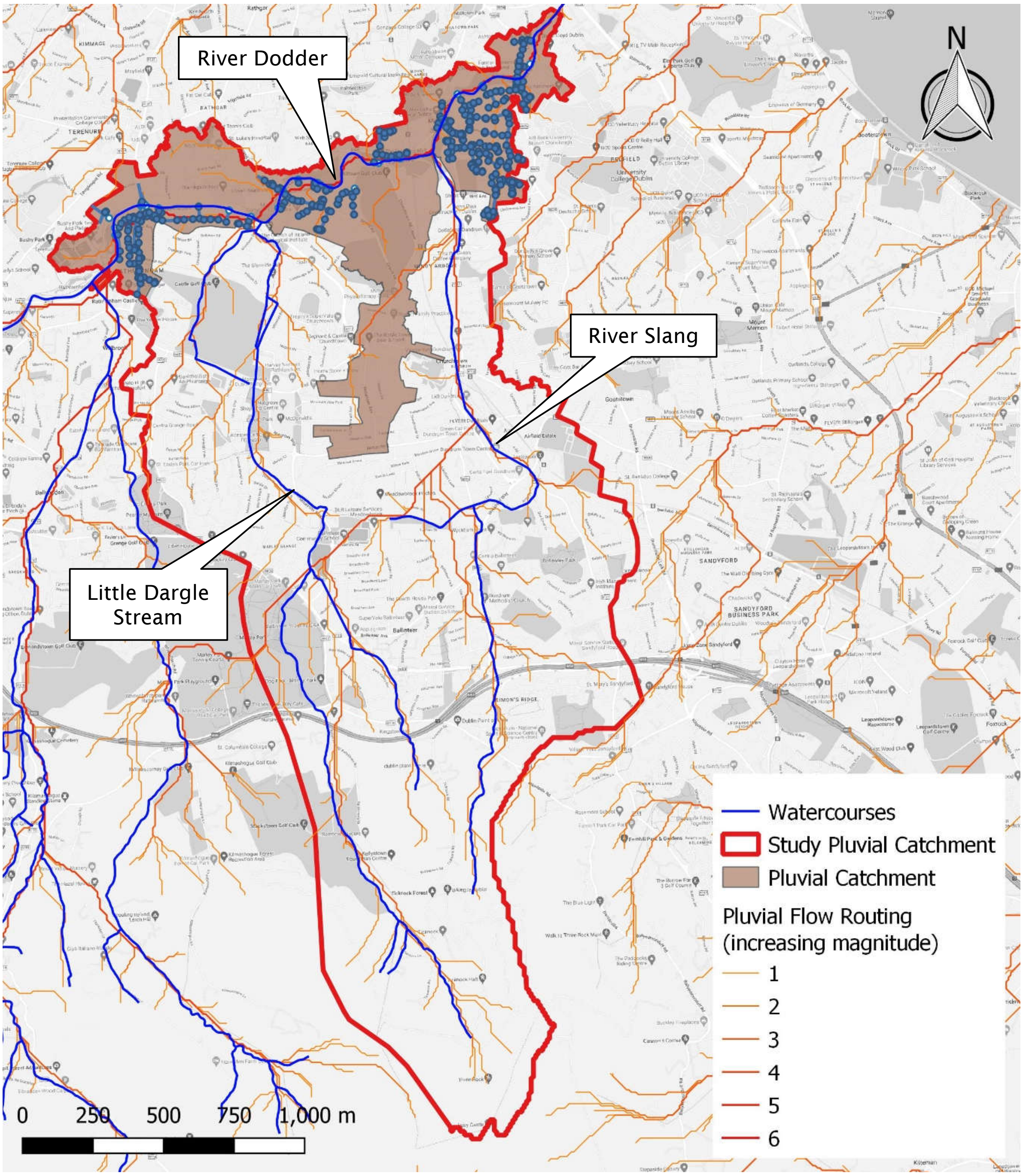

Figure 4-6 Pluvial Catchment Analysis

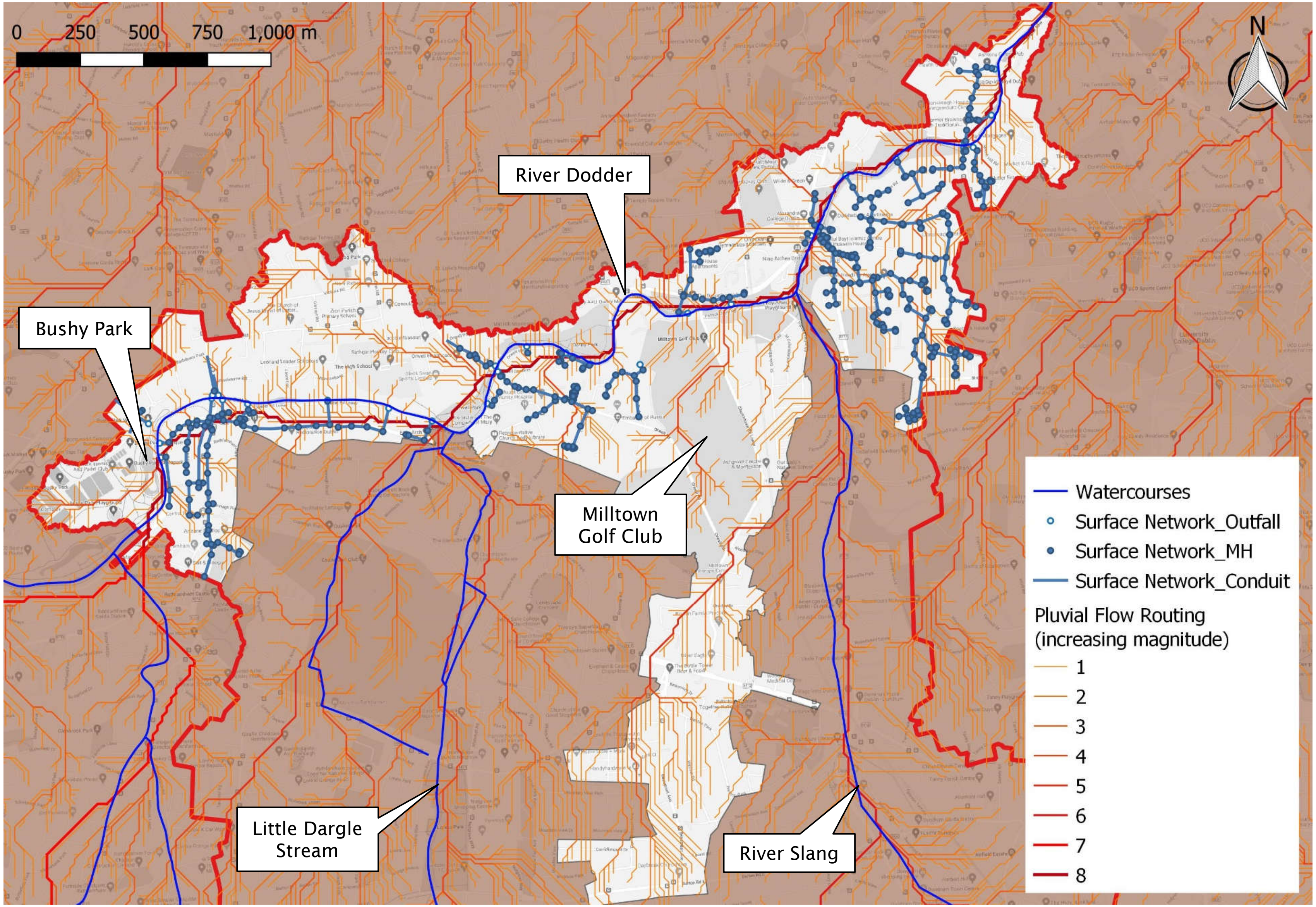

Figure 4-7 Pluvial Analysis (Study Scale)

The overland flow routes contributing flows directly to the River Dodder from the golf course and Bushy Park are most appropriately captured by distribution of hydrology in the model as inflows coinciding with flow paths entering the watercourse, refer to Section 8.5 for further detail on application of flows to the model.

For the urban areas, data capture to date informs that there are surface water collection and conveyance assets located within the pluvial catchment, refer to Figure 4-7. A review was conducted to ascertain locations of overland contributing flows to the River Dodder relative to this network. This found overland flow routes typically intersected network locations and a high coincidence between network and overland flow route discharge location to the River Dodder. It is therefore considered that replication of the network adjacent to the River Dodder is sufficient to replicate contributing flows from pluvial sources. For detail on the application of hydrology with regard to the surface water network, refer to Section 8.5.

It is considered that flow contributions from pluvial sources will be fully represented in the fluvial hydrology by the method described at Section 8.5. Detailed surface flow assessment to determine possible flood risk from pluvial generated surface flow is discrete from fluvial flooding and lies outside the scope of this hydrology report.

4.4 Artificial Sources

Review of Prime2 mapping and orthophotography indicated two reservoirs, namely Bohernabreena Upper and Lower, are located upgradient of the study area. The reservoirs are located in the upper catchment at an elevation of approximately 180m OD, refer to Figure 4-8.

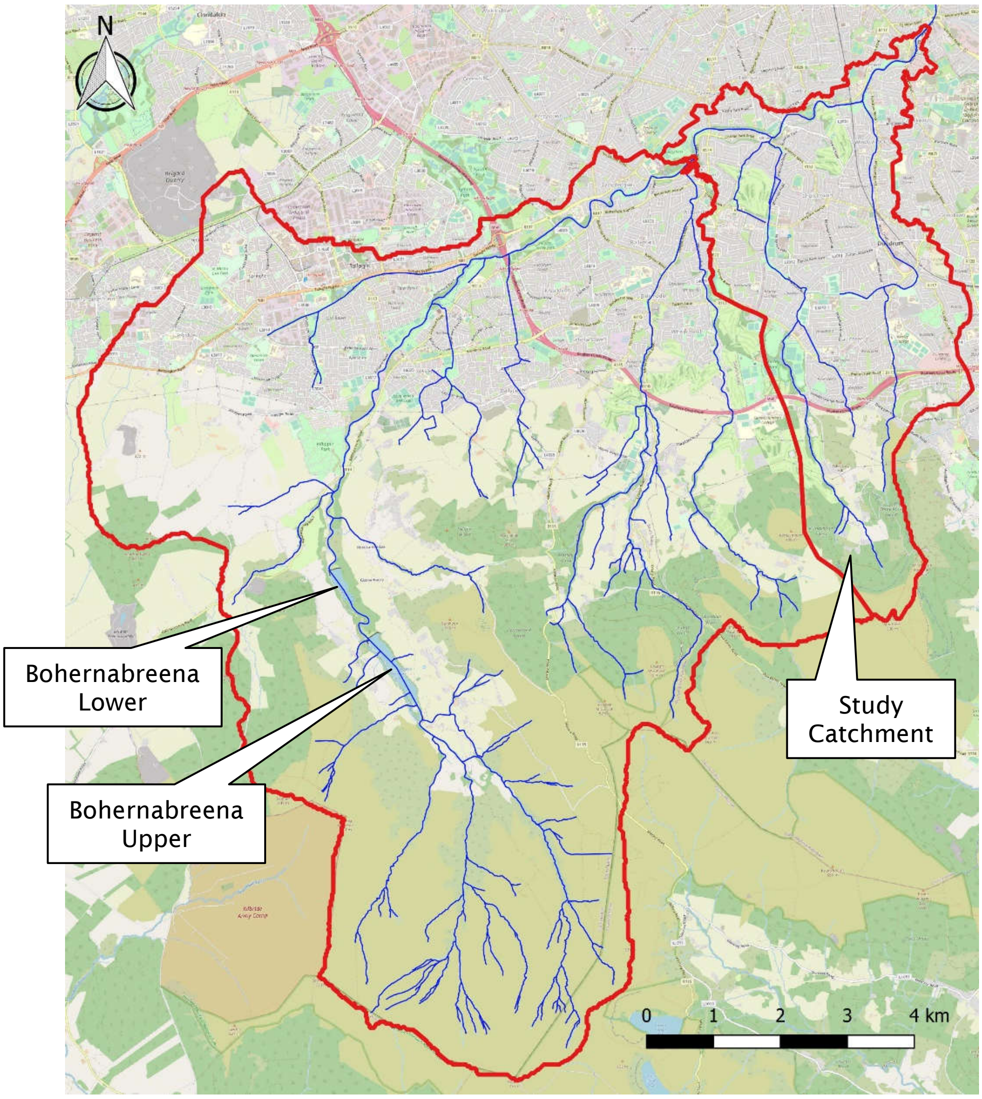

Figure 4-8 Reservoir Location in Upper Catchment

An estimated 28km² of the River Dodder Catchment drains to the reservoirs, coupled with an additional 65km², providing catchment area totalling 93km² to the upstream of the proposed model extent.

A review of detail provided regarding the operation of the reservoirs informed that it is typical that the level is manually drawn down when heavy rain is predicted but the extra capacity provided is minimal and is rapidly depleted thereby providing minimal attenuation affect. DCC have informed that the reservoir stores approximately 3% of the design flood. The attenuating effect of the reservoirs will be reflected in the calculation of the hydrology coupled with gauge data sited within the study reach.

Evaluation of likelihood or consequence of flood risk because of inundation following a dam breach or other uncontrolled release from the reservoirs lies outside the remit of the brief.

No other canals or other potentially impounded lakes have been located that would have potential to cause a risk of flooding in the event of a breach or other failure, or have an attenuating effect on fluvial flood hydrology.

4.5 Summary

Groundwater flooding is not deemed significant within the catchment or as having any significant effect on fluvial flood estimation.

The effect of the reservoirs in the upper catchment will have a small attenuating effect on flooding, which is most appropriately captured by giving weight to analysis of the gauge within the modelled reach as part of the fluvial flood estimation.

Pluvial effects to fluvial flood hydrology are most appropriately captured by application of point and lateral inflows with flow distribution to coincide with outflows from the urban drainage network.

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5 HYDROMETRIC DATA

5.1 Waldron's Bridge

Waldron's Bridge (09010) is a water level gauging station originally installed in 1952 with upgraded recording equipment installed in 2000. The gauge control structure is the Orwell weir which is sited approx. 190m downstream of the gauge location. Further detail on the gauge and data provided for use in this study was provided in Section 3.3.

5.1.1 Rating Review

A rating review model was required to provide flow discharge information for the gauge at Waldron's Bridge (09010) to inform the hydrological calculations. A rating review is typically a 1D model which is built and calibrated using valid spot gaugings with results used to update the upper (extrapolated) stage of the rating curve. The model has the primary purpose of achieving calibration with the highest gauged flow and extending the rating curve to increase confidence of predicted water levels for more extreme design events (where the design event has a magnitude greater than the highest gauged flow).

5.1.1.1 1D Rating Model Build

The rating review model extents are set relative to the Waldron's bridge gauge location and have been sited to ensure the effect of the structures represented is fully realised upstream and downstream of their location. The upstream extent is located approximately 275m upstream of the gauge location and is sited at the point of confluence between the little Dargle and the Dodder. The downstream boundary is located approximately 300m downstream from the gauge location, refer to Figure 5-1 detailing extents of the model.

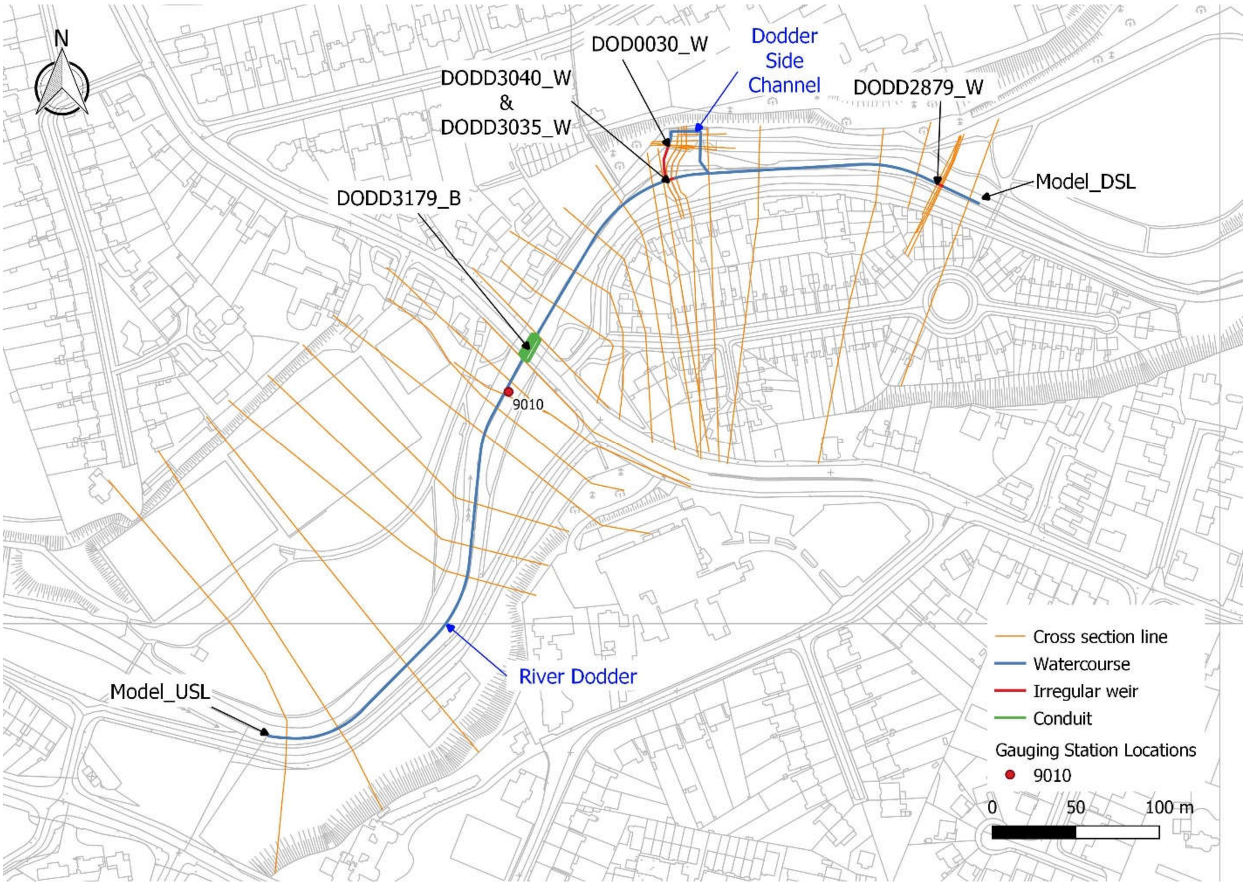

Figure 5-1 Rating Review Model Extent

Cross sectional survey data was collected in a topographical survey commissioned for use in this study and completed by a third-party surveyor. Where data gaps were present in the extended sections, best available LiDAR was used to infill with checks made to ensure correlation between the survey and LiDAR sources.

The river centreline was digitised based on a combination of survey information and OSi Prime2 Mapping, verified at time of site walkover. Survey information was imported directly into ICM. Naming of the sections within the model are as per survey naming.

The roughness of the 1D sections is represented through the application of the Manning's n roughness value. Roughness values change across the 1D cross sections to allow for the variance in roughness of the channel, banks and flood plain. Roughness values were initially applied per site observations and surveyor photographs at time of survey. Roughness was tested and updated as part of the calibration process ensuring appropriate values were applied. Testing informed that roughness varied from 0.03 in channel, representing clean straight watercourse with minimal stone and weed, to 0.045 out of channel representing maintained grass.

There are several structures, a bridge and several weirs, downstream of the gauge location that have been represented within the model due to their influence on stage levels at the gauge. The Orwell Road bridge, noted as DODD3179_B on Figure 5-1, is located approximately 20m downstream of the gauge location and is a clear span bridge with a higher-level pedestrian walkway to each side of the main channel. The bridge spans 17.2m with the upstream face of the structure captured at time of survey and used to inform of conveyance ability. The openings of the bridge are observed as non-linear prompting testing to assess representation methodology. The method adopted for use following testing was the representation of the bridge in modular form thereby ensuring appropriate conveyance at increasing stages, refer to Figure 5-2 for survey detail and modular blocks and Table 5-1 for corresponding geometries.

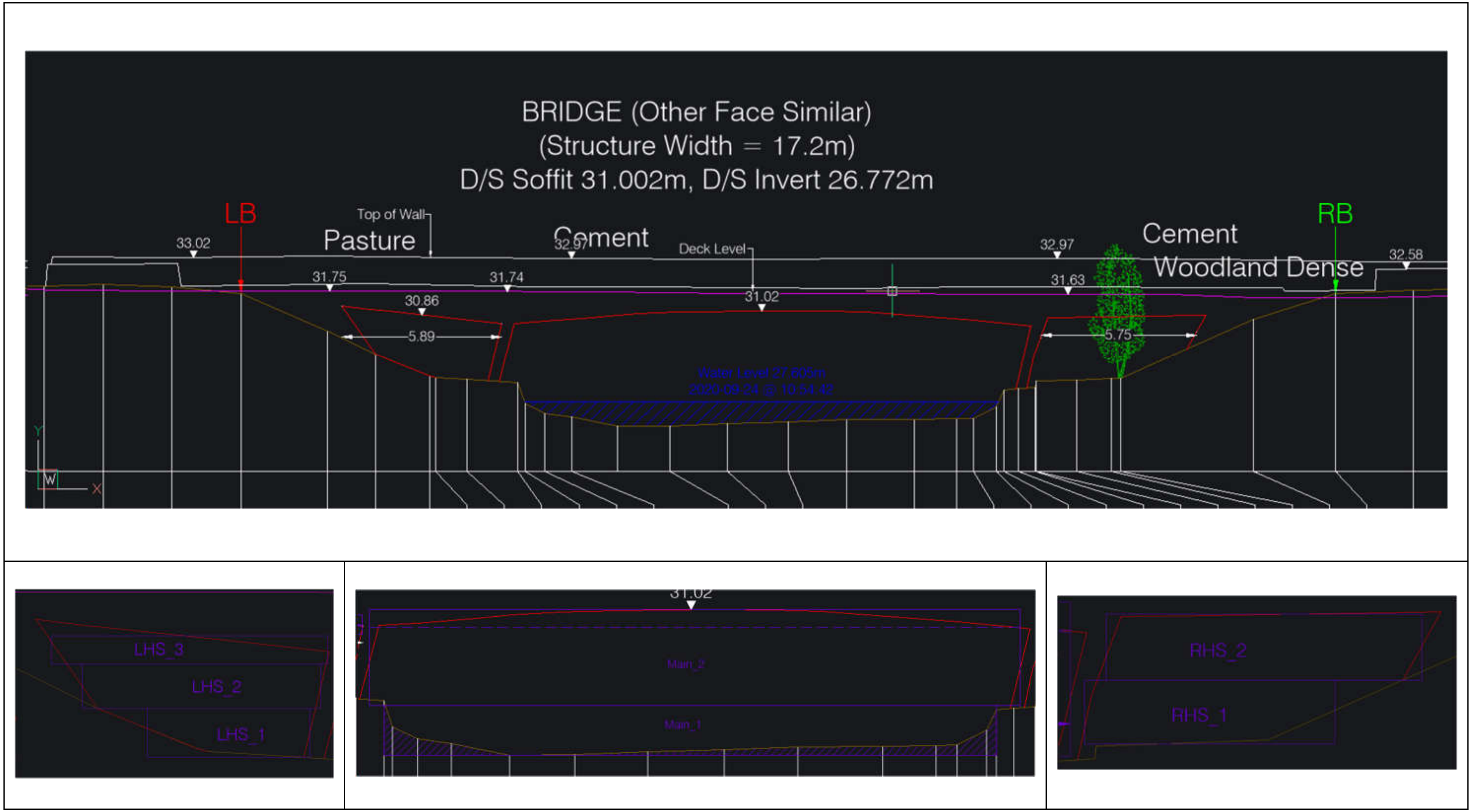

Figure 5-2 Survey Drawing of Orwell Bridge with Modular Units Displayed

BYRNELOOBY | McCloy Consulting

Table 5-1 Orwell Road Bridge Modular Representation

Structure	Type	Location	Link suffix	Shape ID	Width (mm)	Height (mm)	Springing height (mm)	Bottom roughness Manning's n	Top roughness Manning's n	US invert level (m AD)	DS invert level (m AD)	Conduit material
DODD3179_B	Multi Opening Bridge	LHS_1	1	RECT	3100	1000	-	0.012	0.009	28.379	28.379	Conc
		LHS_2	2	RECT	4500	910	-	0.009	0.009	29.379	29.379	Conc
		LHS_3	3	RECT	5500	580	-	0.009	0.012	30.289	30.289	Conc
		Main_1	4	RECT	18000	1500	-	0.03	0.009	26.684	26.504	Conc
		Main_2	5	ARCHSPRUNG	19100	2600	2300	0.009	0.012	28.184	28.004	Conc
		RHS_1	6	RECT	4550	1165	-	0.012	0.009	28.407	28.407	Conc
		RHS_2	7	RECT	5740	1200	-	0.009	0.012	29.572	29.572	Conc
		Deck	8	RECT	44311	2500	-	0.013	0.013	31.89	31.89	Asphalt

Notes:

- No inlet or outlet structures have been applied to this bridge due to the clear span construction of the bridge
- Non-linear shapes have been rationalised to rectangular to allow for application to the model
- When shapes are stacked the top/bottom roughness is set to the modelling software minimum roughness providing negligible impacts on flows
- For the main channel the downstream invert level has been calculated with a steady gradient applied between the bridge upstream face and the next surveyed section
- For the side channels and deck, conservatively representing the nature of the walkways and roads
- The deck has been represented as a conduit with width informed by survey and nominal heigh applied, allowing for any surcharged flows to pass
- Testing Conducted on the bridge included
    - Roughness (top and bottom)
    - Headloss
    - Conveyance (via application of sediment informed at time of survey)

Several weir structures are sited approximately 160m downstream of the gauge location which control the flow-level relationship at the gauge location. These weir structures comprise a multi crested crescent weir located on the main channel coupled with a side weir and channel which conveys flows around the main channel weirs discharging downstream of their location, refer to Figure 5-3 for layout.

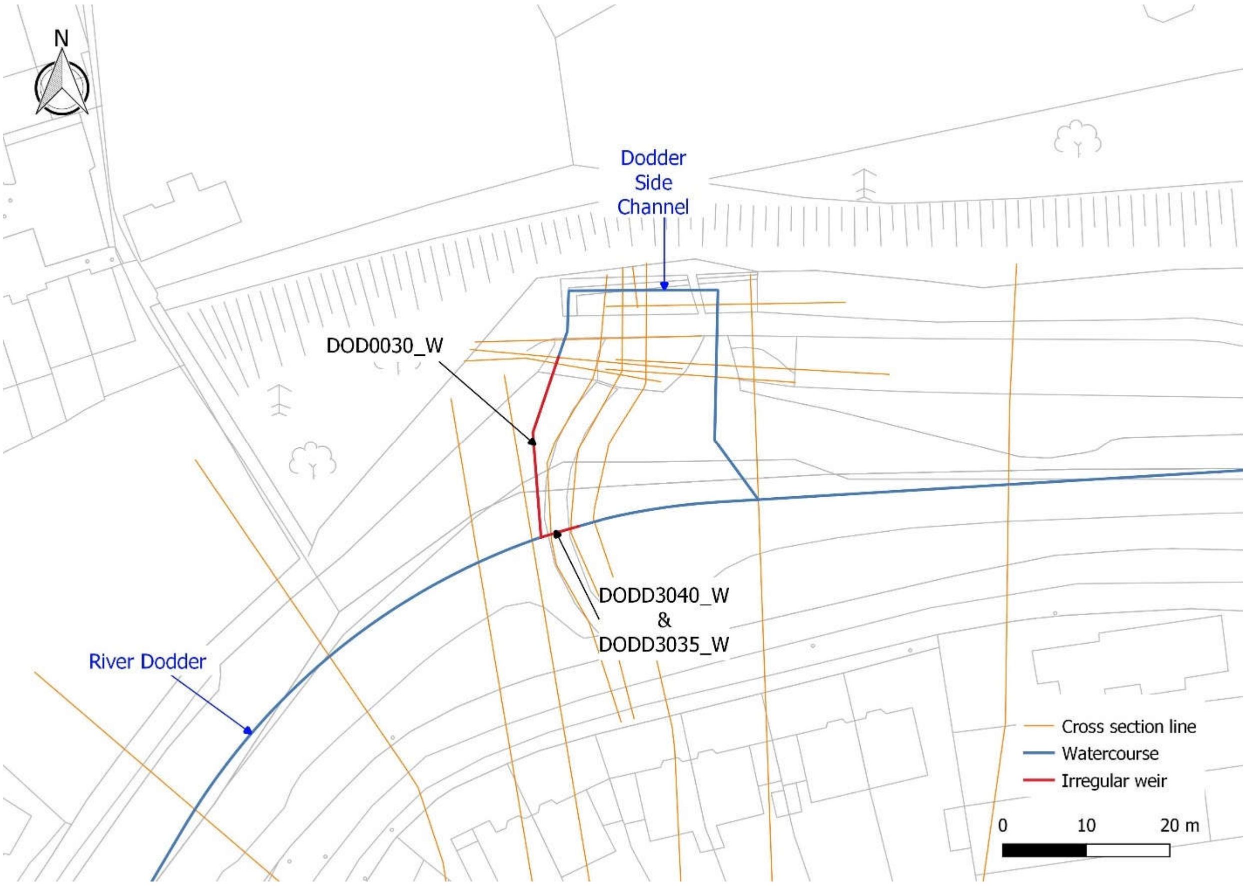

Figure 5-3 Orwell Weir and Side Channel Layout

The weirs in all instances have been represented within the model as irregular weirs allowing for surveyed crest level to be applied directly to the model and ease of testing of weir coefficients. The upper main channel weir, denoted as DODD3040_W in the above figure, is a crescent shaped sharp crested weir with a minimum crest level of 27.49m OD and has the primary influence on the flow level relationship. The secondary weir, denoted as DOSDD3035_W, is a crescent shaped round nose weir with a minimum level of 25.45m OD and was found to have no impact on the flows level relationship at the gauge upon testing.

Flows to the side channel are governed by weir DODD0030_W which is a sharp crested weir with two defined crest levels. A lower level of 27.37m OD is applied to 1.5m of the weir length allowing low flow conveyance to the side channel. The rest of the weir, approximately 4.5m, has a level of 27.55m thereby allowing for increased flow to the channel during higher flow events.

The last weir represented within the model is denoted as DODD2879_W and is located at the downstream extent of the model adjacent to Dodder Vale. This has been represented as an irregular weir with a minimum crest height of 24.30m OD. This weir has no impact on the flow level relationship at the gauge location but allows the completion of an overland flow path through Dodder Vale and therefore was retained within the model build.

5.1.1.2 Calibration Data

A review of the check gauging dataset to be used in the calibration of the model. An inspection of the data issued by the EPA informed that a total of 129 check gaugings had been captured between April of 1979 to May of 2019. When inspected it informed that there was disparity between gaugings captured before and after the October of 2000, refer to Figure 5-4.

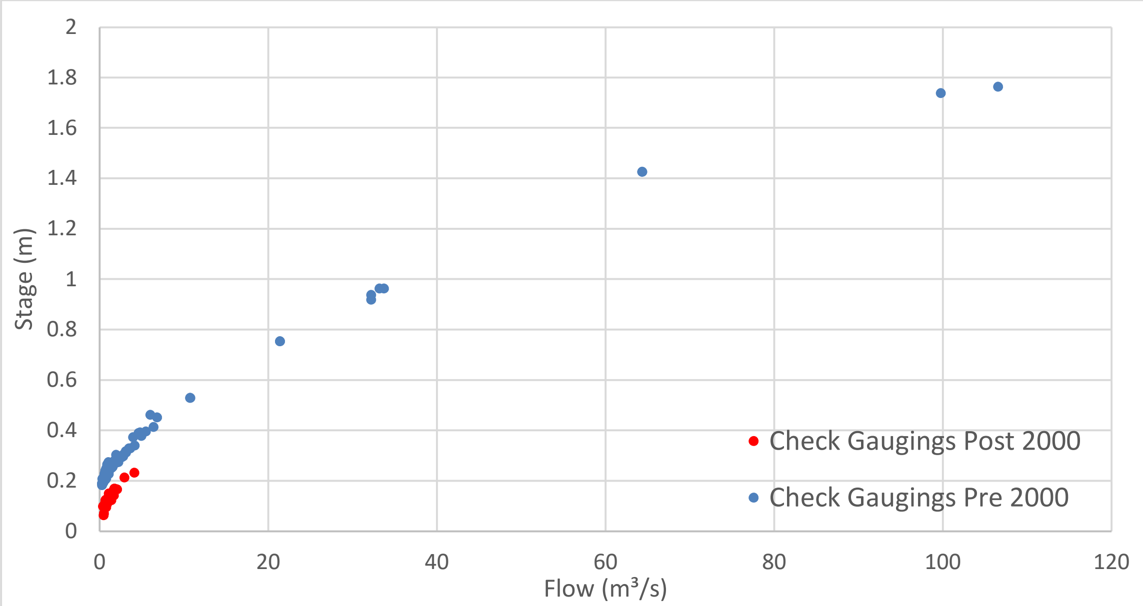

Figure 5-4 Check Gaugings - Full Dataset

Discussions with the EPA informed that the staff gauge that was originally at Waldron's Bridge was an imperial gauge which was changed to a metric staff gauge in October 2000. Further to this, it was informed that the backing board was also changed and the likelihood that it was erected in a slightly different place and the new staff gauge then secured to it. At that time the new staff gauge level was captured at 30.105m OD, a difference in SGZ of +0.105m compared against the pre 2000 level. This information was used to update the pre 2000 gaugings to provide a singular consistent dataset for use in model calibration, refer to Figure 5-5.

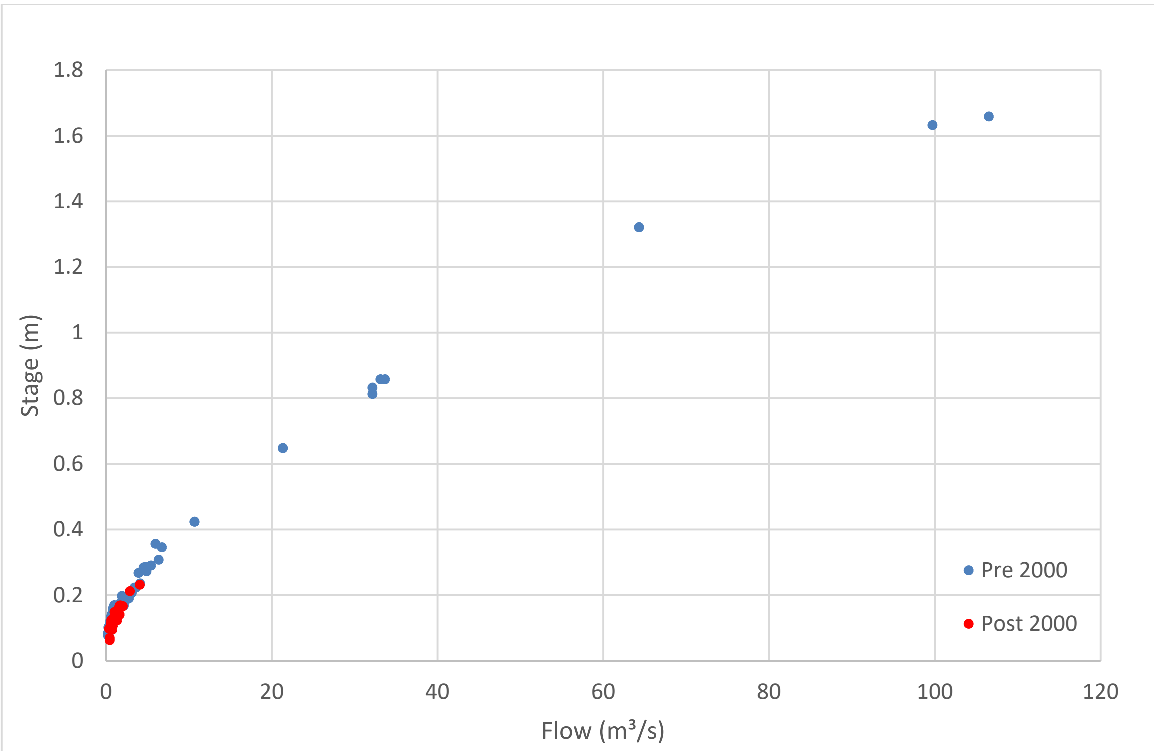

Figure 5-5 Check Gaugings - Full Dataset with Pre 2000 Datum Fix (30.105m)

The updated check gaugings dataset was used to inform the model calibration. Further to this, review of the data used to produce the EPA rating curve indicated that the discrepancy was included in the EPA rating curve, and so all sections of the curve are invalid / required updated as part of this study.

5.1.1.3 1D Rating Model Results

Review of calibrated 1D model results informed of a good correlation between check gaugings up to and inclusive of the highest gauged flow, i.e. the upper validated limit, refer to Figure 5-6.

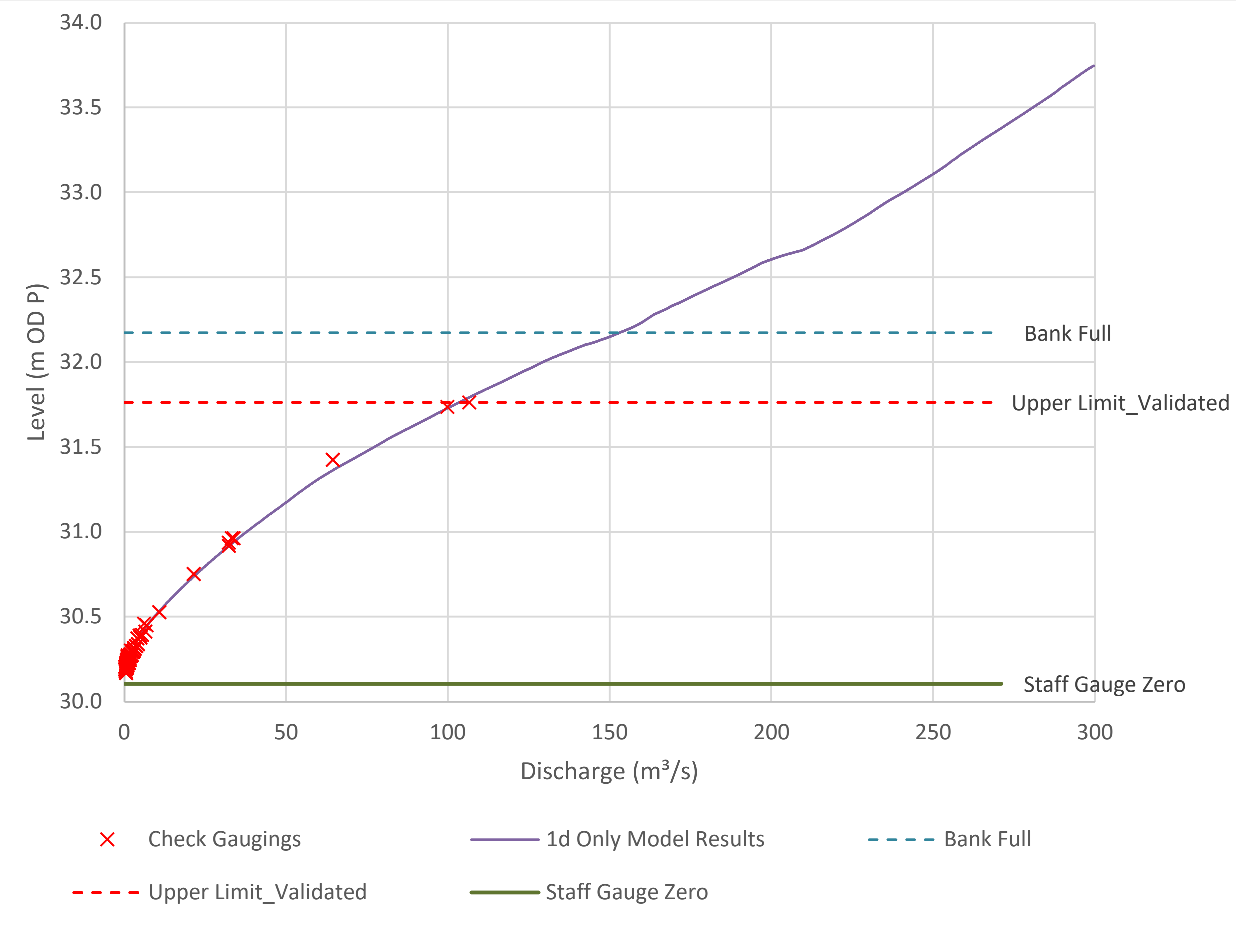

Figure 5-6 1D Calibrated Model Results

The calibration provided expected results up to the bank full level. Between the Bank full level and approximately 210m³/s a variance in the flow level discharge is observed with flows above 210m³/s increasing in trajectory. A further inspection of the model build informed that results from the 1D model may not be appropriate for use above the 210m³/s as they did not reflect out of channel flow regime and floodplain storage available upstream and downstream of the gauge location. As two historic extreme events have been collected at this gauge exceeding the 210m³/s it was prudent to conduct more analysis ensuring replication at these higher flows.

5.1.1.4 1D/2D Rating Model Build

Following the 1D model review the 1D model was progressed to a 1D/2D model with additional elements added ensuring appropriate representation of out of channel flows. The upstream extent of the model was reviewed at time of conversion to 1D/2D to ascertain if it was prudent to extend the model to a location where there was no out of channel flooding. This exercise informed of significant flooding upstream removing this possibility and therefore the first 2 cross-sections of the build have been kept as 1D only. This methodology allows for flows to be retained within the model, i.e. not lost from the 2D zone along the upstream boundary, and provides a stable solution for assessment of results.

The following table, Table 5-2, and figure summaries the additional works conducted with location information provided in Figure 5-7.

Table 5-2 2D model Build Elements

Element	Detail
2D Zone	1D model converted to 1D/2D 1D to represent in channel flows 2D to represent out of channel flows Flexible mesh used with Max triangle size 25m² Min triangle Size 5m² Terrain sensitive meshing selected with max height variation 0.25m applied 2D zone baseline roughness set to 0.035 reflecting maintained grass
Walls	Detail provided by 2021 topographical survey where available Where missing detail was noted, 1d cross section data informing of wall location was used Walls are represented as 2d baselinear structures with surveyed levels applied
Buildings	Location provided by simplified Prime 2 mapping Finished Floor levels (Mesh Level Zones) Threshold levels from 2021 survey applied where available Where unavailable a 0.3m uplift applied relative to adjacent ground level as per standard modelling practice Origin of threshold level detailed within model build Building porosity (Porous Polygon) Building porosity set to 0.1 as per standard modelling practice
Surface Roughness	Roughness Zones based on simplified Prime 2 mapping applied for Roads - Assumed to be smooth asphalt with Manning's n of 0.013 Hard Standing - Assumed cement paving with Manning's n of 0.016
Inline Banks	Applied at structure locations allowing for flow access and egress into the 1D

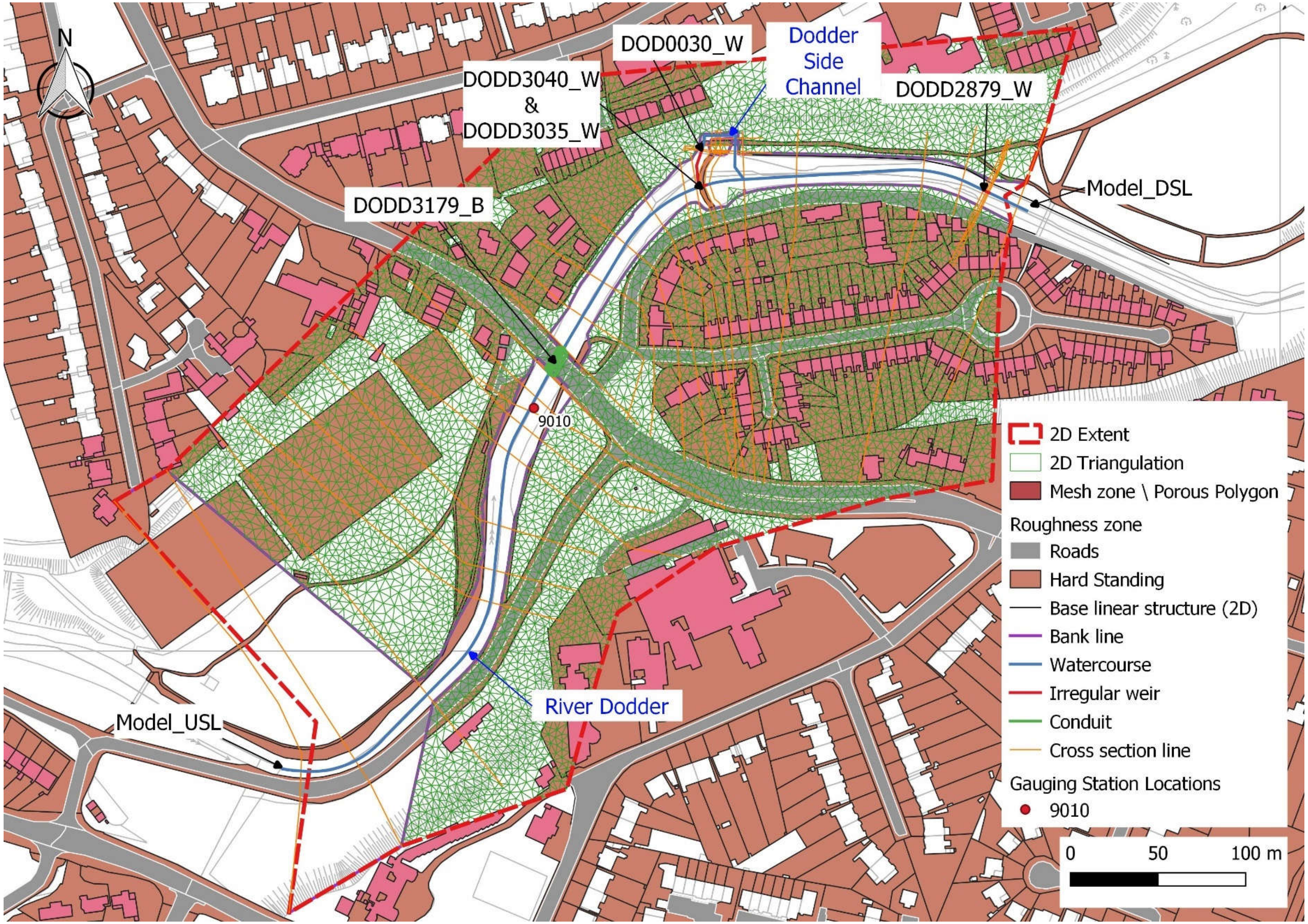

Figure 5-7 2D Modelled Elements

5.1.1.5 1D/2D Rating Model Results

The calibrated 1D/2D model provided results which were more representative of out of channel flows. A comparison against the check gauging calibration dataset and 1D model results has been provided in Figure 5-8.

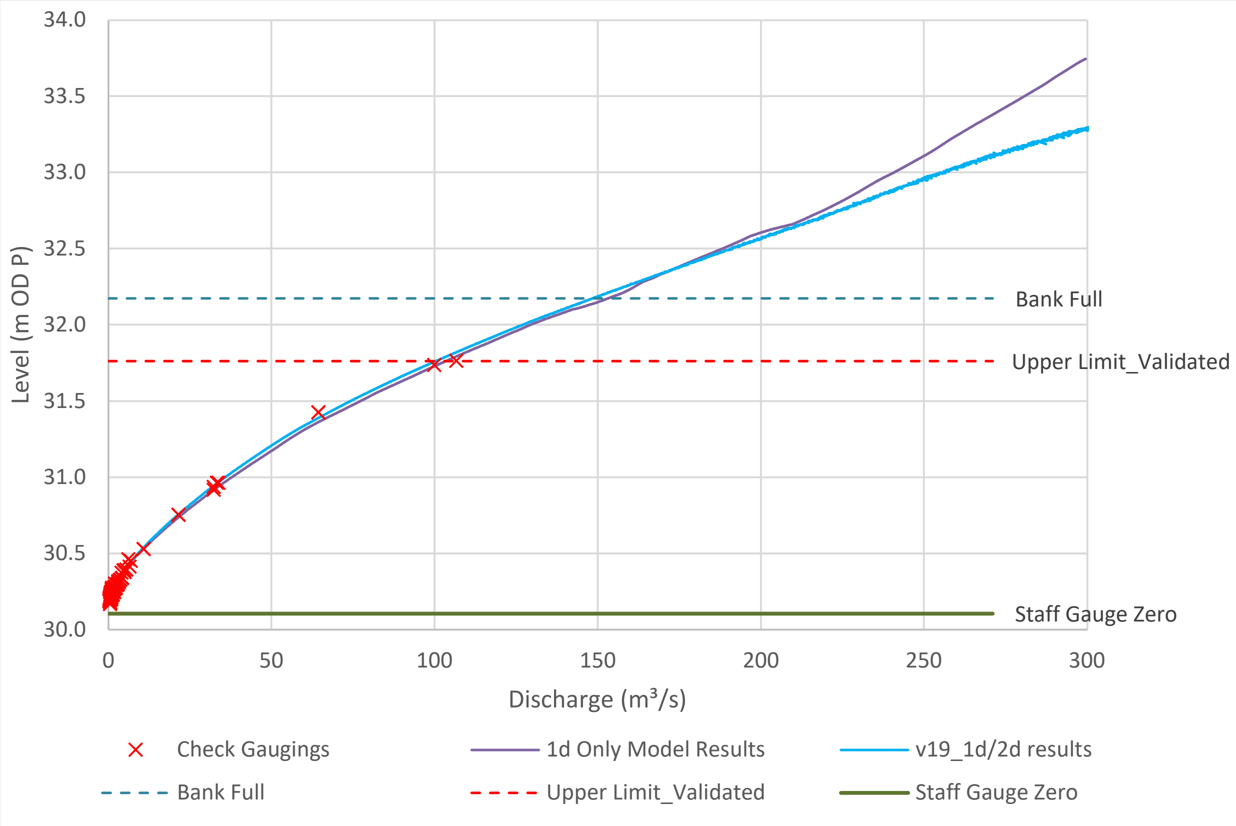

Figure 5-8 1D/2D Rating Model Results

Review of the calibrated model results against the check gaugings informs of a good correlation between the model and calibration dataset. Where flows are conveyed out of bank the level discharge relationship retains the increasing shape and does not deviate per the 1D model results from the 210m³/s discharge level. The addition of the 2D elements in the model, in terms of floodplain capacity and representation of overland flow regime, has added a greater confidence to the modelling outcomes. It is concluded that the 1D/2D rating review model provides results that can be taken forward for use in the rating curve analysis.

5.1.1.6 1D/2D Full Model

Upon completion of the full baseline model build the model was calibrated to historical events, namely the 1986 and 2011 events, refer to section 3.3.1 for more detail on these events. The full baseline model holds enhanced detail over the 1D/2D rating model in terms of application of distributed flows, replication of 1D storm networks and contains updates to model geometry for the representation of the Orwell Bridge and Dodder Side Channel located downstream of the gauge location. Due to these updates the calibration at the gauge was revisited to ensure correlation with the gauge data.

Review of results, refer to Figure 5-9, informed that calibration at the gauge was retained during the full model update and historical calibration process. The updates provided a more stable set of results and as such the full model results were taking forward for use in the rating curve analysis.

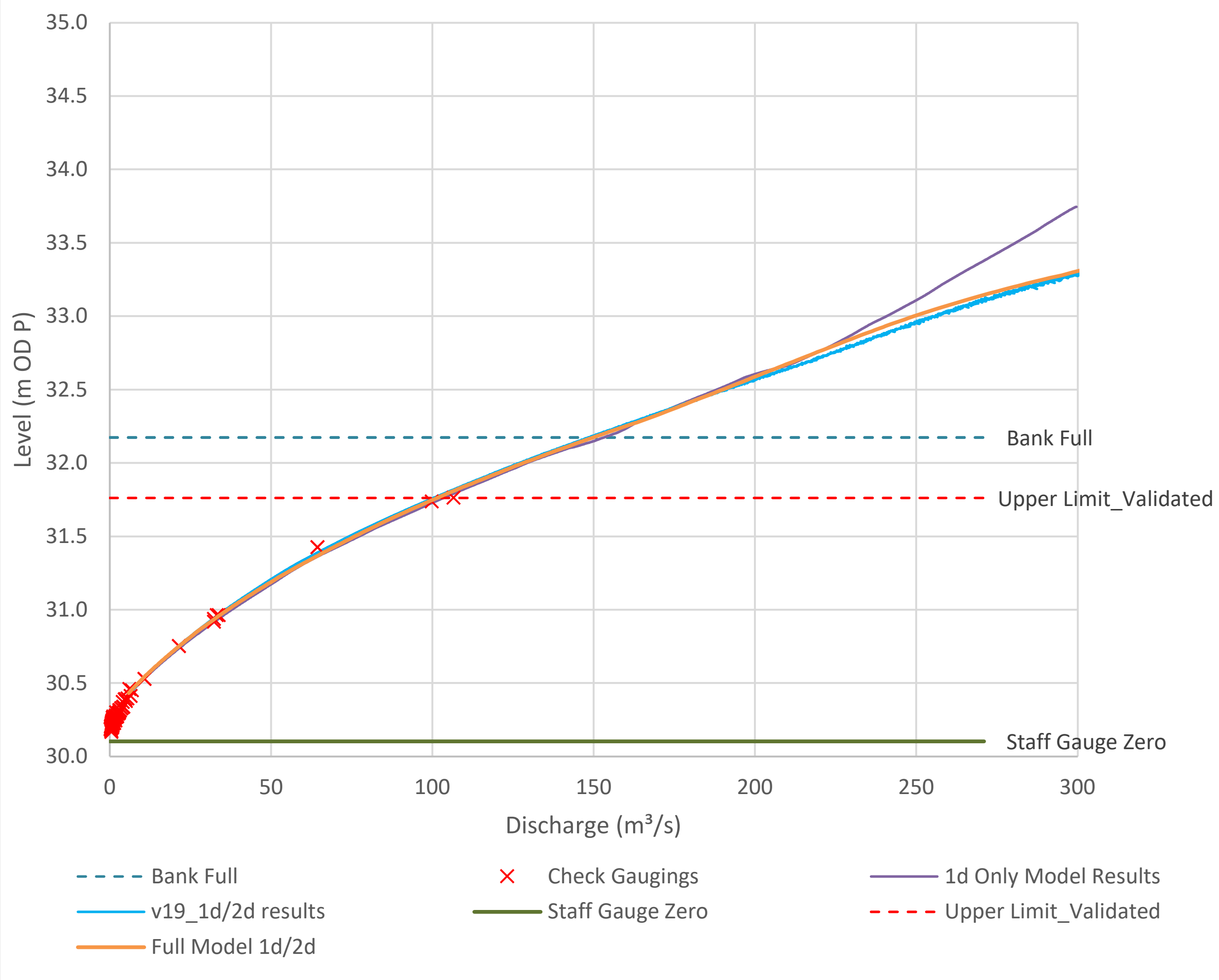

Figure 5-9 1D/2D Full Model Results

5.1.1.7 Rating Curve Analysis

The EPA provided 3 rating curves at time of data request, the most recent of which was rating curve C5.2 which was last updated June of 2020, please refer to section 3.3.1 for more detail. Table 5-3 informs of the EPA C5.2 flow stage relationship with for increasing stages, whilst Figure 5-10 depicts the rating curve with the deviations used to inform the curve. The stage levels informed are relative to the SGZ level which is to Poolbeg datum.

Table 5-3 EPA Rating Curve C5.2

Section	Min Stage (m)	Max Stage (m)	C	a	b
1	0.144	0.205	98.3396	0	2.25073
2	0.206	0.674	32.9461	0	1.56121
3	0.675	1.763	36.4077	0	1.81438

Where: Q = C(h+a)b and h = stage readings (metres)

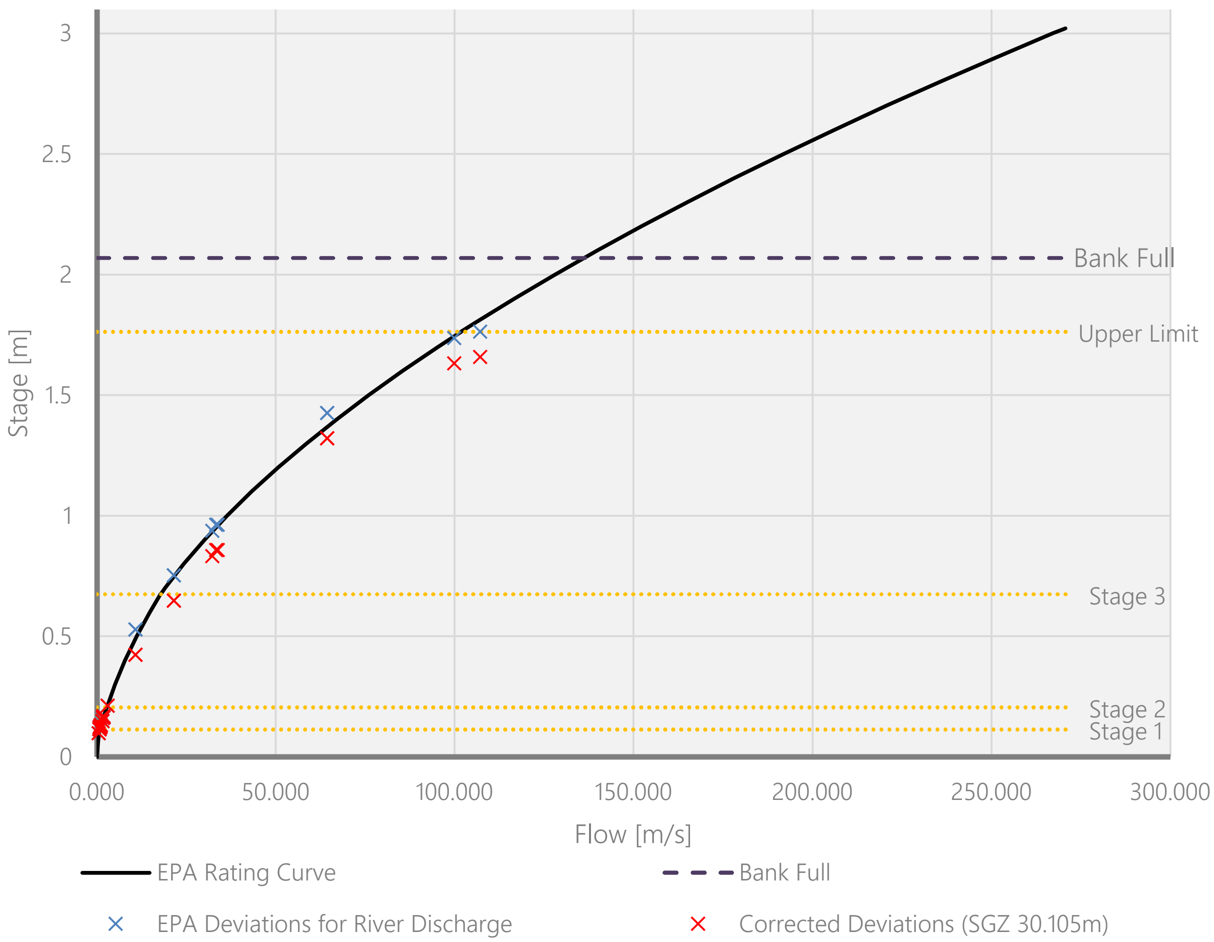

Figure 5-10 EPA Rating Review (C5.2) with Deviations

Stage levels below the minimum stage of 0.144m are extrapolated down using the section 1 formula, whilst stages above 1.763m are extrapolated up using the section 3 formula. The extrapolation of the values above the upper limit is where the modelling exercise can enhance the rating curve with calibrated model results used to inform these higher stages increasing the confidence in rating curve results.

Section 5.1.1.2 informed of the requirement to update the rating curve for all stages due to a discrepancy in the Staff Gauge Zero (SGZ) level pre and post the year 2000. Figure 5-10 depicts this with blue corrected deviations informing of the required location for the updated curve. During the update of the rating curve there was a requirement to update the stage levels in some of the sections allowing for a better correlation of the curve to the data. The resultant update is informed in Table 5-4 and depicted in Figure 5-11**Error! Reference source not found.**.

Table 5-4 Dodder FRS Phase 3 Rating Curve

Section	Min Stage (m)	Max Stage (m)	C	a	b	Comment
0	0	0.113	438.956	0.113	4.374	Lower Limit extrapolated using section 1 formula
1	0.114	0.205	438.956	0.113	4.374	Updated using corrected deviations
2	0.206	0.84	44.953	0.007	1.764	Updated using corrected deviations
3	0.841	2.118	30.861	0.198	1.928	Informed by model results
4	2.119	3.049	37.692	0.240	1.663	Informed by model results
5	3.050	4.521	31.951	0.000	1.937	Informed by model results

Where: Q = C(h+a)b and h = stage readings (metres)

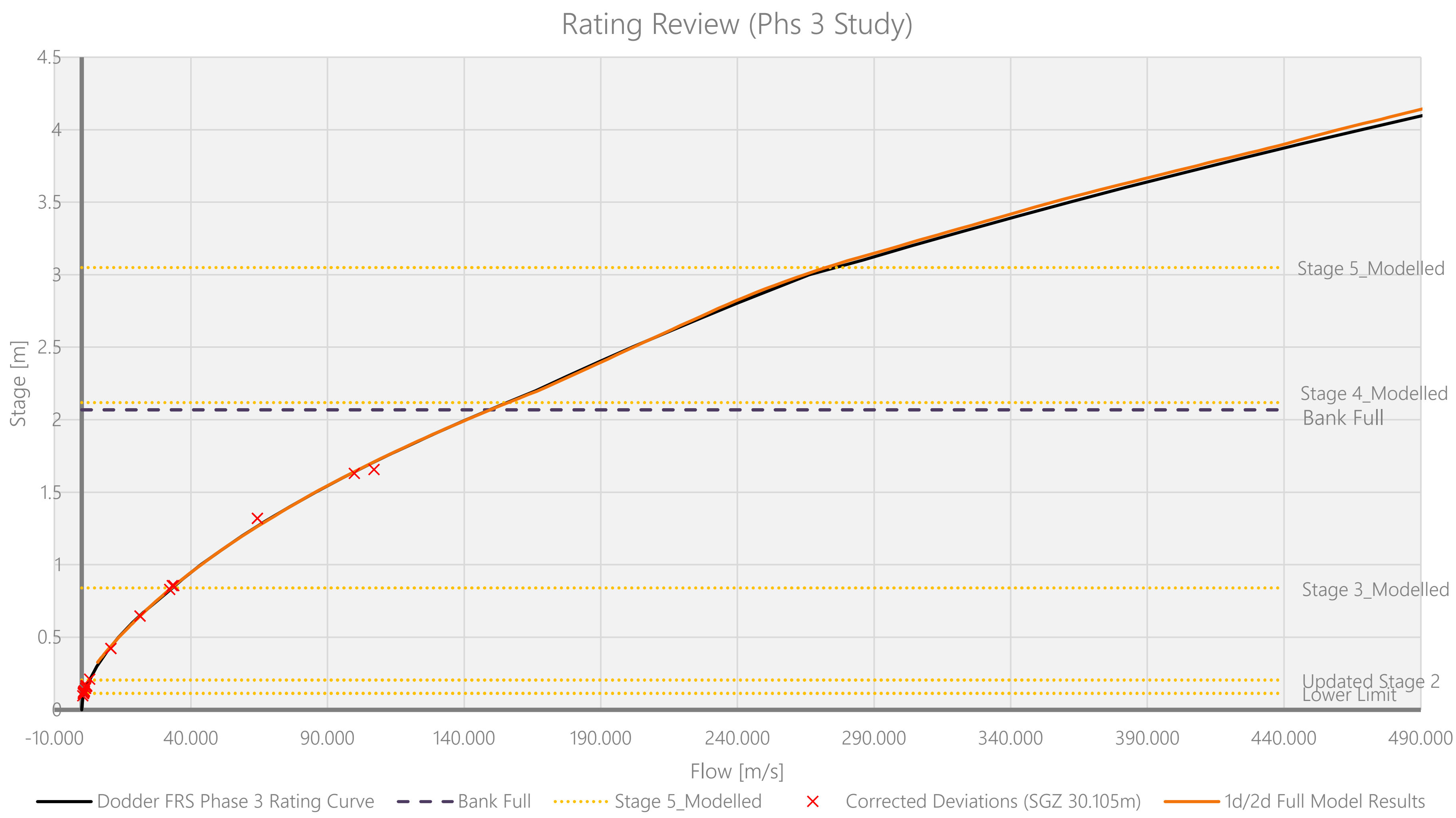

Figure 5-11 Dodder FRS Phase 3 Rating Curve

5.1.2 Annual Maximum (AMAX) data

The updated rating curve was applied to update the AMAX dataset issued by the OPW/EPA for use in this study, dated from 1985 to present day. Further to this, the corrected Staff Gauge Zero (SGZ) was used to update the stage of hydrometric years 1985 to 2000. The following table, Table 5-5, informs of the resultant estimated flows.

Table 5-5 Waldron's Bridge AMAX Data

Hydrometric Year	Date	Staff Gauge Reading (m)	Water Level (mOD) Poolbeg	Estimated Flow (m^3/s)
1985	26/08/1986	33.02	2.915	254.769
1986	05/04/1987	31.253	1.145	54.477
1987	21/10/1987	30.903	0.795	30.441
1988	01/10/1988	---	---	---
1989	19/02/1990	30.686	0.585	17.815
1990	12/04/1991	31.088	0.985	42.657
1991	03/07/1992	30.604	0.495	13.317
1992	11/06/1993	31.418	1.315	68.550
1993	03/02/1994	31.665	1.565	92.056
1994	27/01/1995	30.926	0.825	32.479
1995	06/01/1996	31.066	0.965	41.278
1996	11/06/1997	30.68	0.575	17.287
1997	09/04/1998	31.468	1.365	72.985
1998	30/12/1998	31.063	0.955	40.596
1999	25/04/2000	30.902	0.795	30.441
2000	05/11/2000	32.278	2.065	148.971
2001	07/10/2001	31.076	0.97	41.621
2002	14/11/2002	31.306	1.2	58.860
2003	02/12/2003	31.885	1.78	114.921
2004	29/10/2004	31.371	1.27	64.674
2005	04/11/2005	30.903	0.8	30.777
2006	22/06/2007	31.065	0.96	40.936
2007	05/09/2008	31.926	1.82	119.443
2008	06/06/2009	31.324	1.22	60.494

Hydrometric Year	Date	Staff Gauge Reading (m)	Water Level (mOD) Poolbeg	Estimated Flow (m³/s)
2009	16/01/2010	31.602	1.5	85.624
2010	17/11/2010	30.946	0.84	33.152
2011	24/10/2011	32.755	2.65	220.178
2012	22/03/2013	31.406	1.3	67.246
2013	24/05/2014	31.251	1.15	54.868
2014	14/11/2014	31.742	1.64	99.755
2015	03/12/2015	31.164	1.06	48.025
2016	23/03/2017	30.823	0.72	25.599
2017	14/03/2018	31.429	1.32	68.988
2018	29/09/2019	30.691	0.59	18.082
2019	07/11/2019	31.164	1.06	48.025
			Qmed	**51.251**
			Qmax	254.769
			HGF	106.528
			HGF/Qmed	2.079

It was informed by the EPA that in November/December 2000 the Orwell weir downstream of the gauge sustained damage resulting in a centre masonry block being dislodged. The damage to the weir was rectified in August of 2003 but has resulted in a different rating for the lower levels for this period. The impact of the damage at the weir has no influence on the AMAX data for this period as stage readings above 0.674m are noted as unaffected.

The updated rating curve and corrected stages have impacted previously calculated Qmed at the Waldrons' bridge gauge of 44.515 m³/s, increasing it by 6.736 m³/s to 51.251 m³/s. The Qmax value, captured at time of Hurricane Charlie in 1986, is observed as reduced from 270.46 m³/s to 254.769 m³/s, a decrease of 15.691 m³/s. Discussions with the EPA were conducted about this decrease who informed that the lower value was in line with their expectations and informed that the increased value of 270 m³/s had been calculated and reported externally of the EPA. For comparison, rating curve analysis at time of the CFRAMs study informed of an estimated peak flow of 251.08 m³/s.

Although the flood event with the maximum estimated flow has been reduced, discussions with the OPW and the EPA informed of a reduced confidence in the collection of the level used to inform the estimate. Detail issued by the EPA informed that for the peak level on the 26/08/1986 the autographic recorder was topped by flood waters and stopped working requiring the peak being obtained from debris marks via a crane measurement taken from Orwell bridge. The second biggest event in October 2011 was also informed to have reduced confidence due to a similar means of capture.

These two events have a large impact on the Qmed and single site analysis calculations due to their magnitude and the reduction of confidence in their capture methodology prompted further discussions with the OPW as to how to proceed. Agreement was made to increase the AMAX dataset prior to 1985 using data from EPA paper (Micheál Mac Cárthaigh, August 2005) and the IEI Dodder River Flood Study Report (P. Hennigan, J. McDaid, J. Keyes, Nov. 1988), refer to Table 5-6 for additional values.

Table 5-6 AMAX values from 1949 to 1984

Hydrometric Year	Estimated Peak Flow (m^3/s)	Comment
1949	58.05	
1950	36.81	
1951	50.69	
1952	35.68	Dublin Corporation (now Dublin City Council) installed an OTT water level recorder
1953	32	
1954	50.77	
1955	28.88	
1956	64.85	
1957	74	
1958	116.1	
1959	44.85	
1960	67.96	
1961	28.32	
1962	23.79	
1963	28.6	
1964	21.8	
1965	138.75	
1966	44.74	
1967	50.4	
1968	84.95	
1969	---	In the1970's Waldron's Br. was re-built, and the water level recorder was moved a short distance upstream of the bridge (on the left bank). The water level is controlled by Orwell Weir downstream.
1970	---	
1971	---	
1972	---	
1973	37.94	
1974	48.42	
1975	33.98	

Hydrometric Year	Estimated Peak Flow (m^3/s)	Comment
1976	40.21	
1977	46.44	
1978	43.7	
1979	41.78	
1980	72.09	
1981	57.62	
1982	105.63	
1983	82.27	
1984	53.3	

In the 1949 to 1984 AMAX dataset there is a further 28 years of AMAX data, with values ranging between 21.8 and 138.75 m^3/s. There is uncertainty as to whether the years are calendar years or hydrometric years, and further uncertainty on the calculation of the peak flows as no detailed information on flow measurements is given or how the rating value was determined. It is however considered that the inclusion of these values provides a more balanced overall dataset with uncertainties in high flows being offset by the larger and local dataset informing of more typical values at the gauge location.

The following table provides comparative values with the inclusion of the increased dataset, see Table 5-7.

Table 5-7 Comparison of Key Detail on increased Dataset

Comparison	Initial Dataset (1985 to 2019)	Increased Dataset (1949 to 2019)
Qmed (m^3/s)	51.251	48.222
Qmax (m^3/s)	254.769	254.769
HGF	106.528	106.528*
HGF/Qmed	2.079	2.21

*Highest Gauged Flow (HGF) dataset is as per initial in both instances as no captured gauged flows have been issued relative to years 1949 to 1984

The final AMAX dataset for use in the single site analysis dates from 1949 to 2019 with 66 years of usable AMAX values ranging from 13.317 to 254.769 m^3/s. This increased AMAX dataset returns a Qmed of 48.222 m^3/s providing a reduction in Qmed of 3.029 m^3/s over the initial dataset dated 1985 to 2019.

5.1.3 Single Site Analysis

Single site analysis was conducted using the 1949 to 2019 AMAX data to provide a flood frequency curve. The curve was reviewed for a range of statistical distributions with particular attention paid to Gumbel (EV Type 1) and 2-parameter lognormal (LN2) distributions as FSU Work Package 2.2 informed these are most applicable best for gauged locations.

Review of the gauge data informed it provided a linear relationship for the majority of the dataset but the two highest gaugings in 1985 and 2011 were noted to deviate from the body of values, refer to Figure 5-12**Error! Reference source not found.**. Analysis was conducted with these events removed from the dataset which improved the linearity, reflected in the increase of the R² value from 0.75 to 0.9.

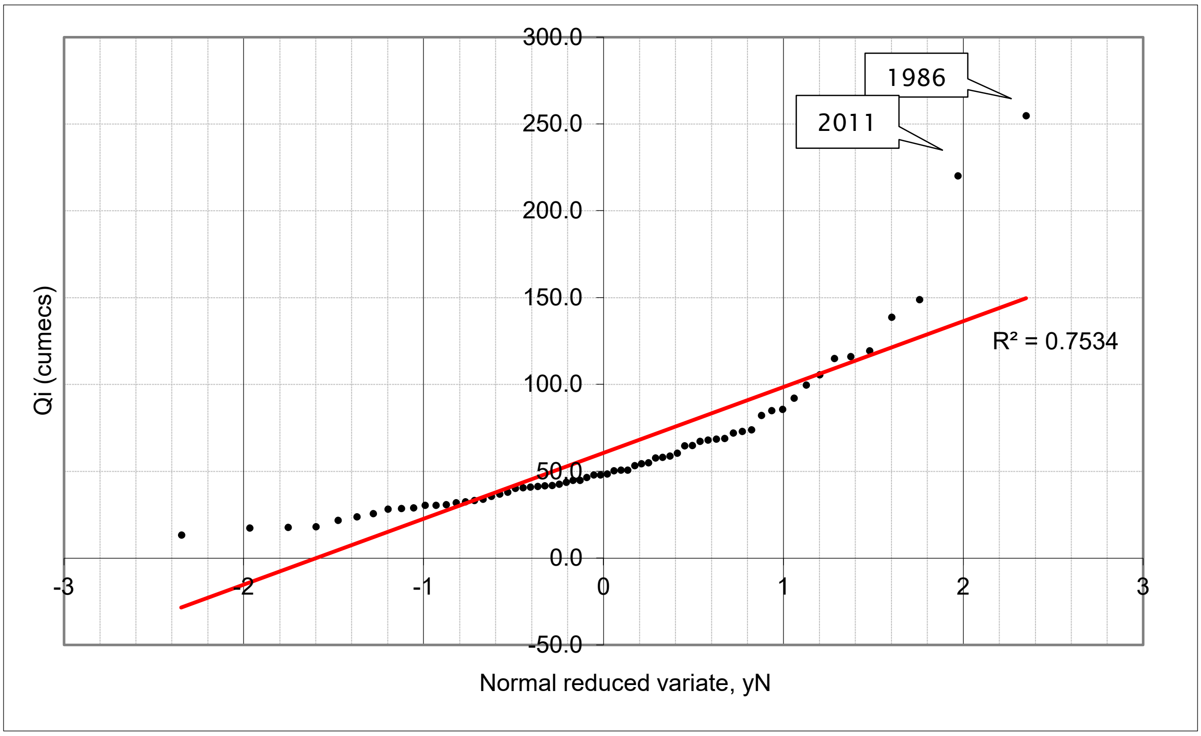

Figure 5-12 2-Parameter LN2 curve fitting for Linearity

The inclusion of the higher values prompted the review of 3 parameter distributions with Flood Frequency curves generated for multiple distributions. The best fit curves found to be the 3 parameter GEV and GLO distributions, refer to Figure 5-13.

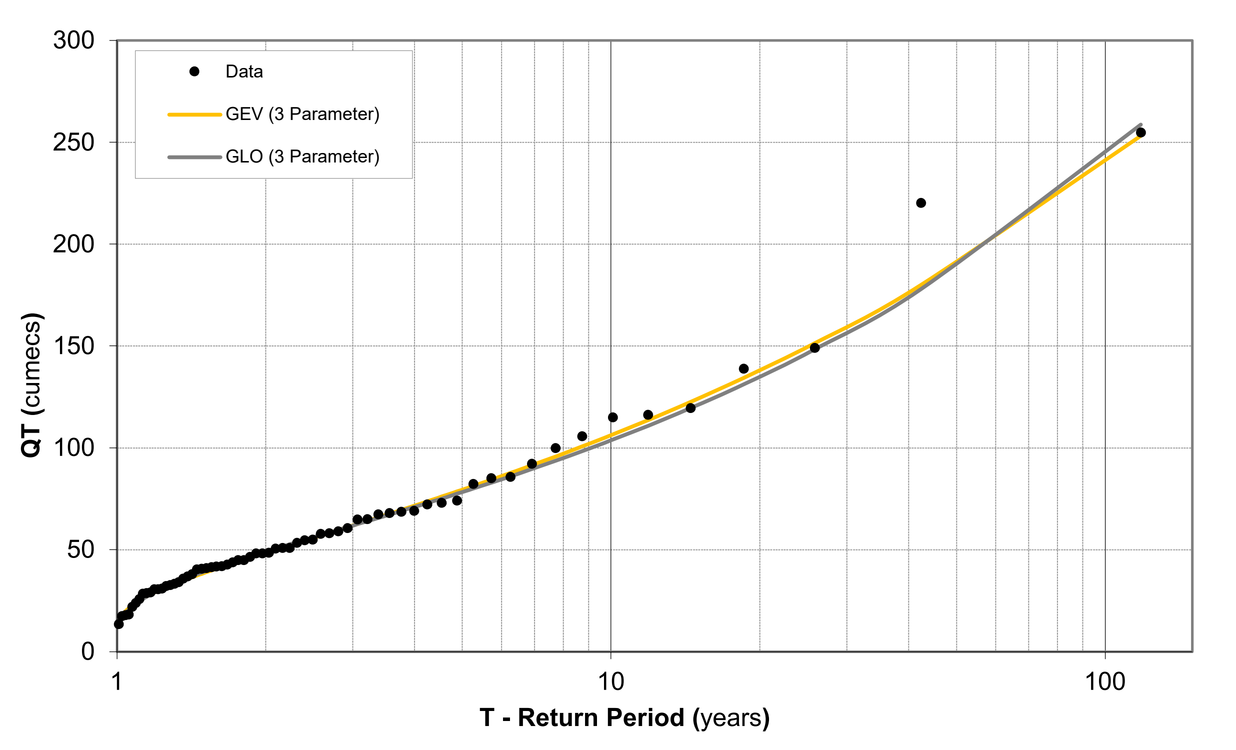

Figure 5-13 3 Parameter Flood Frequency Curves (GEV & GLO Distributions)

The GEV and GLO both distributions provide a very similar curve, but the GEV distribution has been selected for use due to goodness of fit, particularly through the 10 to 30 year return periods, and guidance provided by FSU work package documentation.

The resultant growth factors and calculated flows provided the following results shown in Table 5-8. The table also includes values extracted from the CFRAMs hydrology report at the same gauging station for comparison. Outcomes from the Single Site Analysis tend to vary significantly from CFRAM findings whereby the new analysis indicates lower magnitude for high probability flows than CFRAM predicted, and higher magnitude for lower probability (more extreme) flows than CFRAM predicted. The differences between the calculated values can be attributed to the dataset used for calculation of Qmed and flood frequency curve at time of CFRAMs, refer to Section 3.1 for more detail

The 1% AEP, for which the defences are to be designed for, sees an increase in flow of 37.4 cumecs equating to an increase of 19% over CFRAMs.

Table 5-8 GEV Growth Factors and Flows

AEP (%)	Return Period (T)	GEV Growth Factors	GEV Calculated Flows	CFRAMs Sim (EXP1)	% Difference
2	50	1.000	48.222	74.260	-35
5	20	1.638	78.979	108.180	-27
10	10	2.190	105.630	130.630	-19
20	5	2.846	137.224	-	-
50	2	3.923	189.174	180.060	5
100	1	4.943	238.349	200.950	19
200	0.5	6.185	298.247	221.770	34
1000	0.1	10.227	493.154	-	-

There are differences between the CFRAMs flows and study flows viewed in the table above. For the high probability events (50% to 10% AEP) the CFRAMs are noted to be in excess of those calculated for this study, whilst the lower probability events (2% to 0.5% AEP) the study are noted to be in excess of those at time of CFRAMS.

The differences can be attributed to several factors the foremost of which is the methodology applied for the calculation of flows. At time of CFRAMs the simulated results, from the Rainfall Runoff (NAM) model, were subject to statistical analysis and informed the probability distribution and thereby growth curves.

The rating curve for this study has been completely reworked due to the discrepancy noted in the Staff Gauge Zero (SGZ) level noted previously. Further to this it was found that a 1D only model, as used at time of CFRAMs, was not sufficient to replicate the stage discharge relationship with the requirement of a 2D zone to ensure out of channel flows and attenuation were fully appreciated.

The use of gauge data, over simulated model results, along with the inclusion of a larger dataset and selection of an alternative distribution coupled with the updated rating curve provides the differences viewed in the comparison of the CFRAMs values.

5.1.4 Historical Event Analysis

The top five events have been reviewed using the GEV growth factors to assess an estimated AEP. These events have been presented in Table 5-9 in descending order of magnitude.

Table 5-9 Estimated AEP of Top 5 Gauged events

Date	Peak Flow (m³/s)	Estimated AEP (%)	Estimated Return Period (Yrs)
26/08/1986	254.77	0.863	116
24/10/2011	220.18	1.370	73
25/08/1905	198.00	1.821	55
03/09/1931	153.00	4.089	24
05/11/2000	148.97	4.322	23

5.1.5 Pooled Analysis

As a further check of the estimated design flows, pooled analysis was investigated to assess if inclusion of flood data from other similar gauged sites could add additional confidence to the calculated flows. Pooled analysis is defined in FSU documentation as a method for creating a longer time series to estimate the peak flow at a subject site using observed flood data from other gauged catchments. Gauged catchments are identified that are hydrologically similar based on Physical Catchment Descriptors (PCDs) and as such can improve the robustness of design estimation.

The standard methodology for this assessment is the use of the FSU Portal, but this must be treated with caution in this instance; the portal includes water year records up to 29/10/2004 only. Pooled analysis outside of the portal, to bring the dataset to present day, was deemed to be beyond the scope of this study due to the significant works required in the update. Pooled analysis within the inherent limitations of the FSU portal was therefore conducted for comparative purposes only with the following results gained.

Table 5-10 Pooled Analysis using FSU Portal

	FSU_LN2		FSU_GEV	
AEP (%)	Growth Factors	Design Peak Flows (m³/s)	Growth Factors	Design Peak Flows (m³/s)
50	1	48	1	48
20	1.94	93.3	1.85	88.87
10	2.75	132.06	2.63	126.4
5	3.67	175.95	3.6	172.99
2	5.06	243.01	5.29	253.74
1	6.28	301.38	6.96	334.15
0.5	7.65	367.02	9.09	436.48
0.2	9.71	465.99	12.83	616.05
0.1	11.48	550.95	16.58	795.82

As informed in the table above, Table 5-10, the Qmed (50% AEP) used in the pooled analysis through the FSU portal was 48 m³/s which is not dissimilar to that used in this study, see previous sections. As per single site analysis, the LN2 and GEV distributions provided the best fit for the dataset within the FSU Portal. The growth factors obtained through the pooled analysis are noted to be significantly higher compared to those informed via the single site analysis providing 1% AEP values in excess of 300 m³/s.

As detailed earlier in this section, it is the PCDs of the location under review that are used to inform the pooling group carried forward for analysis. In this instance the Waldron's Bridge Gauge is noted to have the highest S1085 value (20.977 m/km) and one of the highest URBEXT values (0.2404) of the whole FSU database.

It is the considered opinion of this assessment that the Waldron's Bridge gauge location and catchment characteristic is distinctive within the FSU gauge database given its size and urbanisation and the lack of other urbanised catchments, and that a pooled analysis of substantially dissimilar sites causes a regression versus the single site analysis.

5.1.6 Summary

At Waldron's Bridge, the base water years evaluated through the FSU methodology extend from 1985 – 2004 (19 valid water years) and as such inclusion of a further 43 years AMAX data increases the available series by +226% substantially increasing the confidence in dataset. Checks using the FSU Portal to assess pooled analysis informed that results gained were overly conservative. The results of the single site analysis informed that the GEV distribution provided the best fit flood frequency curve and as such, growth factors have been calculated using that distribution.

Rating review and single site analysis calculation sheets have been provided in Appendix B with the rating review model provided in Appendix B.

5.2 Frankfort

A review of the study, provided by the client for use, completed in December 2019 informs that the study results provide greater confidence in flows at the gauging station location than statistical analysis could provide. This is due to the catchment wide gauging and calibration of the Slang catchment model, where the Slang performs as an extension of the urban drainage network. As such, no further analysis has been conducted at the Frankfort gauge, and model results are suitable to be extracted directly for use in this study.

6 HYDROLOGICAL ESTIMATION POINTS

6.1 HEP Selection

The location of existing FSU Hydrological Estimation Points (HEPs) within the proposed model extent was reviewed to assess which would be pertinent to the study. HEPs were adopted based on the following criteria:

- Upstream boundaries of all modelled watercourses,
- Gauge locations (09010 & 09011)
- Points on tributaries upstream of the confluence with the receiving channel,
- Points on receiving channels upstream/downstream of confluences of tributaries,
- Location of proposed defences indicated on the preferred CFRAM option.

The HEPs selected for estimation of flows within the model detailed in Table 6-1 and depicted in Figure 6-1. The model extent is as described previously in Section 2.2.

Table 6-1 Hydrological Estimation Points (HEPs) Selected for Review

HEP	Location
09_1373_2	River Dodder - Model Upstream Limit (USL)
09_1380_1	River Dodder - US of Little Dargle
09_1385_4	Little Dargle - Model USL
09_1385_8*	Little Dargle - Confluence with River Dodder
09010	River Dodder – Waldron's Bridge GS
09_1380_4	River Dodder - US of Slang
09011	River Slang - Frankfort GS & Model USL
09_1381_8**	River Slang - Confluence with River Dodder
09_587_1	River Dodder - DS of Slang
09_587_2	River Dodder - Main Channel
09_587_3	River Dodder - Model Downstream Limit (DSL)

Notes:

*The location of this HEP is noted as incorrect and is due to a discrepancy in the FSU database. Further detail is provided in subsequent sections

**HEP 09_1381_9 is located at the confluence with the River Dodder, but a box culvert is located between this HEP and 09_1385_8 which throttles flows and affects results. HEP 09_1385_8 has been selected for use in calculations to allow direct comparison in calculated and model results.

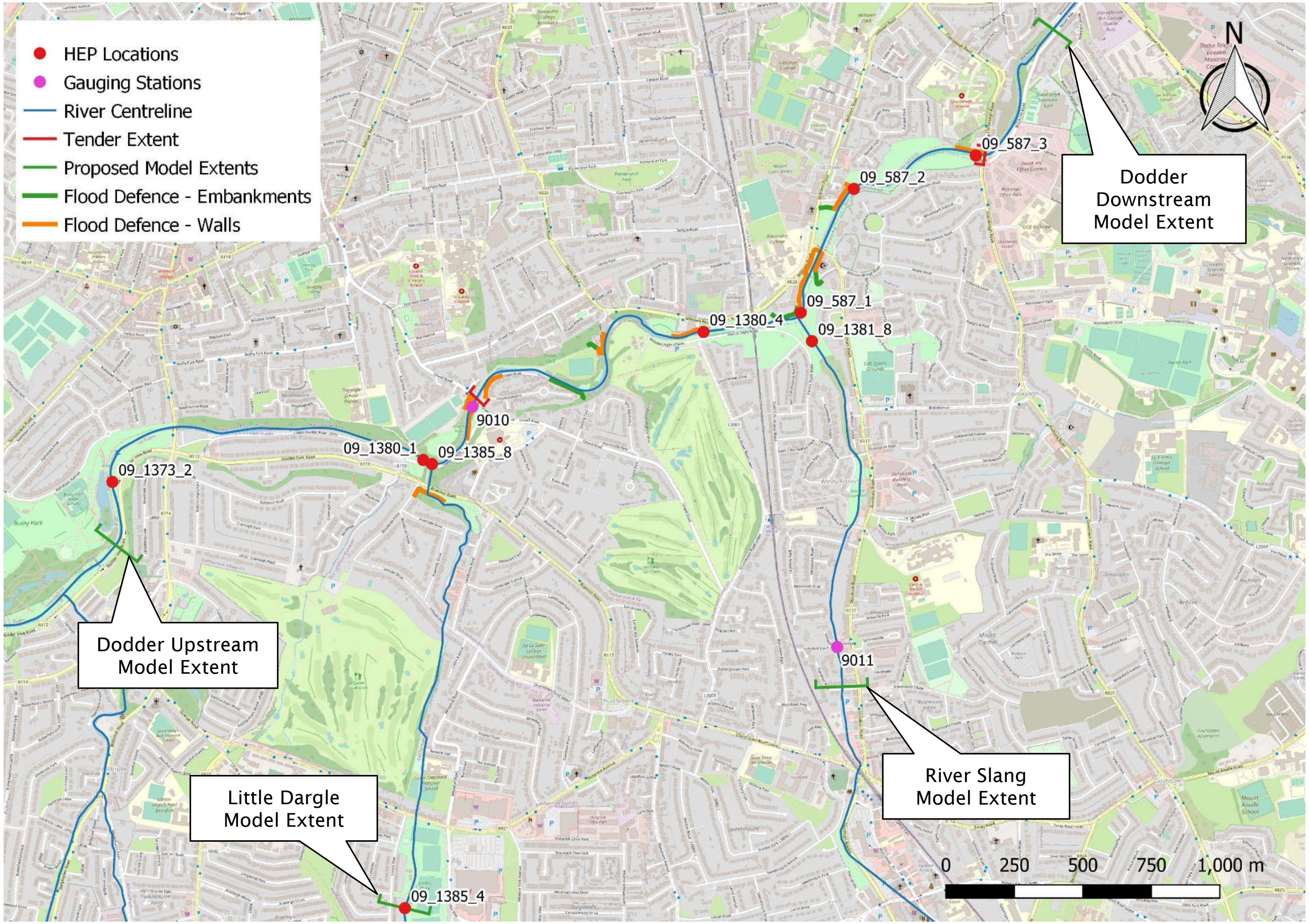

Figure 6-1 Hydrological Estimation Points (HEPs)

HEPs shall be used within the flood model to check and ensure flows are consistent with modelled flows, any instances where significant differences occur shall be investigated, remedied or reported on where there is suitable justification for the difference observed.

6.2 Physical Catchment Descriptors

Catchment descriptors, as provided by the OPW for use in this study, are derived from the FSU dataset and provide detail at each of the Hydrological Estimation Point (HEP) locations. Descriptors have been verified where possible using available GIS datasets with updates required where deemed necessary.

6.2.1 Catchment Extent (AREA)

The hydrological catchment draining to the downstream limit of the proposed model extent, HEP 09_587_3, has been calculated at 114km². This is a 9% increase in size over the FSU catchment measuring at 104km². The extent has been defined using a combination of data from

- 2m LiDAR for the extent of the study area
- 25m DTM for the remainder of the catchment
- GDSDS network data
- Irish Water sewer network data

The delineated catchment for the study area is shown on the following Figure 6-2 which also provides the River Dodder watercourse network which flows from the south west to the north east from an approx. elevation of 700m to 13m OD at the downstream model extent.

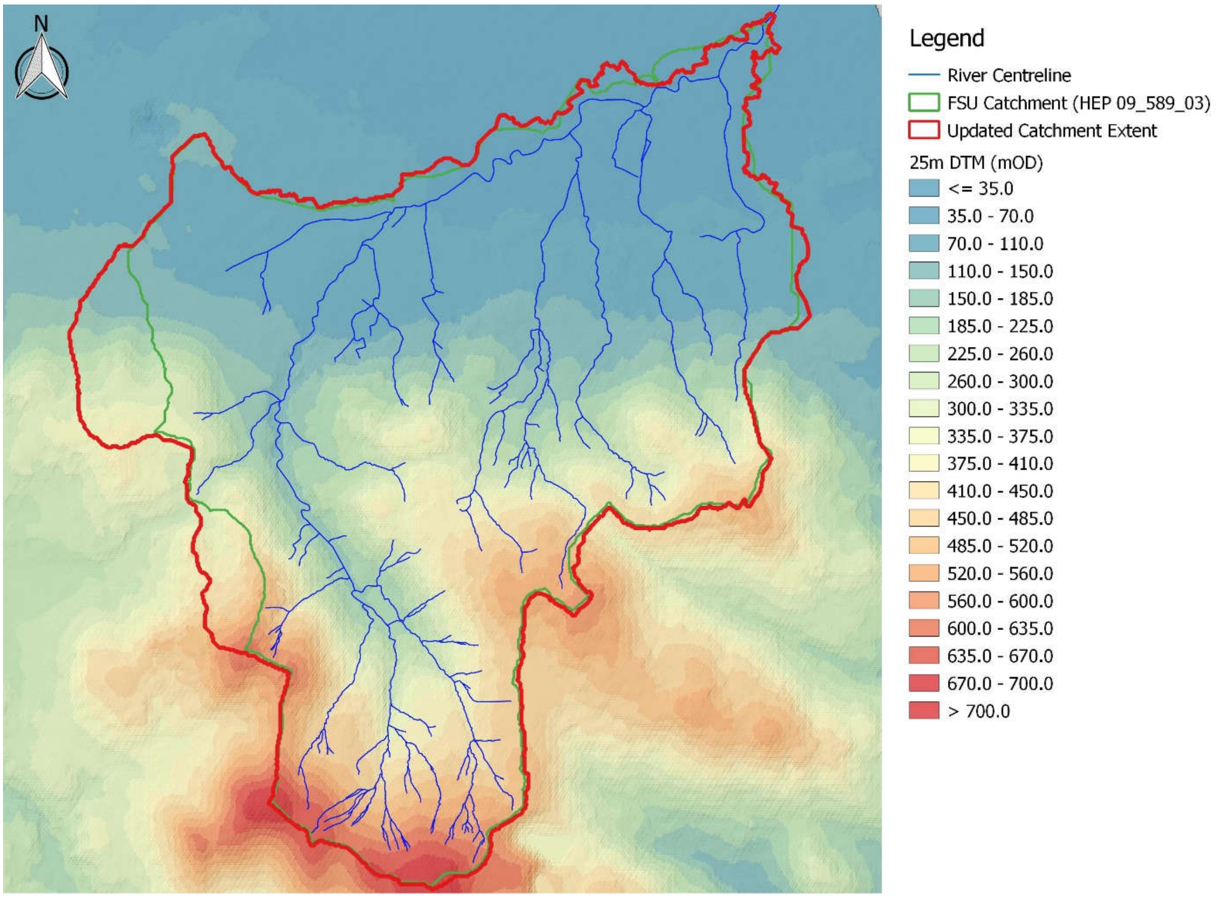

Figure 6-2 Comparative Catchment Extents with River Dodder Watercourse Network

The same methodology was applied at each of the HEP locations with catchment areas updated at each location for use in calculations with the following results, refer to Table 6-2.

Table 6-2 Comparison of Catchment Area at HEP Locations

HEP	Location	FSU Catchment (km²)	Updated Catchment (km²)	% Difference
09_1373_2	River Dodder - Model Upstream Limit (USL)	83.431	92.629	11.0%
09_1380_1	River Dodder - US of Little Dargle	93.275	93.165	-0.1%
09_1385_4	Little Dargle - Model USL	7.046	6.206	-11.9%
09_1385_8	Little Dargle - Confluence with River Dodder	9.088	9.036	-0.6%
09010	River Dodder – Waldron's Bridge GS	94.260	102.750	9.0%
09_1380_4	River Dodder - US of Slang	95.050	104.071	9.5%
09011	River Slang - Frankfort GS & Model USL	5.460	6.323	15.8%
09_1381_8	River Slang - Confluence with River Dodder	8.328	6.828	-18.0%
09_587_1	River Dodder - DS of Slang	103.451	111.094	7.4%
09_587_2	River Dodder - Main Channel	103.836	111.695	7.6%
09_587_3	River Dodder - Model Downstream Limit (DSL)	104.229	111.861	7.3%

6.2.2 Urban Extents (URBEXT)

The latest Corine Land Cover Dataset (2018)[9] was used as a baseline for the assessment of the catchment in terms of urban extent for Present Day analysis. There were 6 Corine Land Cover Classifications (CLCs) selected for use to represent what constituted as urban extent, refer to Table 6-3.

Table 6-3 Corine Land Cover Classifications

CLC code	Description
111	Continuous urban fabric
112	Discontinuous urban fabric
121	Industrial or commercial
122	Road and rail networks
123	Sea ports
124	Airports

Upon extraction and merge of polygons with these codes, the extents were reviewed against the best available aerial photography and extra areas were added where additional impermeable zones were viewed. For the assessment of urban extent for the High-End Future Scenario (HEFS), an elevation of 160m OD was selected as the urban rural divide for future development, a level confirmed by DCC relevant for planning and development. This elevation was set due to water supply restrictions and assumes 100% development below this elevation thereby providing a conservative urban area. The Mid-Range Future Scenario (MRFS) is calculate as the average between Present Day and HEFS, refer to Table 6-4.

[9] Data obtained from https://land.copernicus.eu/pan-european/corine-land-cover/clc2018

Table 6-4 Comparison of Catchment URBEXT at HEP Locations

HEP	Location	FSU URBEXT (Present Day)	Updated URBEXT (Present Day)	% Difference	Updated URBEXT (MRFS)	Updated URBEXT (HEFS)
09_1373_2	River Dodder - Model Upstream Limit (USL)	0.213	0.215	1%	0.283	0.350
09_1380_1	River Dodder - US of Little Dargle	0.234	0.219	-6%	0.287	0.354
09_1385_4	Little Dargle - Model USL	0.228	0.324	42%	0.402	0.481
09_1385_8	Little Dargle - Confluence with River Dodder	0.363	0.485	34%	0.564	0.642
09010	River Dodder – Waldron's Bridge GS	0.240	0.246	2%	0.314	0.382
09_1380_4	River Dodder - US of Slang	0.246	0.256	4%	0.324	0.392
09011	River Slang - Frankfort GS & Model USL	0.683	0.742	9%	0.793	0.844
09_1381_8	River Slang - Confluence with River Dodder	0.758	0.702	-7%	0.806	0.854
09_587_1	River Dodder - DS of Slang	0.288	0.293	2%	0.360	0.427
09_587_2	River Dodder - Main Channel	0.291	0.297	2%	0.364	0.430
09_587_3	River Dodder - Model Downstream Limit (DSL)	0.293	0.300	2%	0.366	0.432

6.2.3 Standard Period Average Annual Rainfall (SAAR)

A review of the SAAR values at the HEP locations informed they were prepared using the Met Éireann 1961-1990 SAAR grid. OPW informed that this grid had been updated and latterly Met Éireann provided the 1981-2010 grid for use.

A comparison of the two datasets informed that there was negligible change to the Little Dargle and River Slang SAAR values but there was a noted decrease averaging 20% in SAAR for the HEPs along the River Dodder. All values have been updated to reflect the 1981-2010 dataset with the following results, refer to Table 6-5.

Table 6-5 Comparison of SAAR at HEP Locations

HEP	Location	FSU SAAR	Updated SAAR	% Difference
09_1373_2	River Dodder - Model Upstream Limit (USL)	974	760	-22%
09_1380_1	River Dodder - US of Little Dargle	957	761	-21%
09_1385_4	Little Dargle - Model USL	843	814	-3%
09_1385_8	Little Dargle - Confluence with River Dodder	823	761	-8%
09010	River Dodder – Waldron's Bridge GS	955	757	-21%
09_1380_4	River Dodder - US of Slang	953	742	-22%
09011	River Slang - Frankfort GS & Model USL	773	772	0%
09_1381_8	River Slang - Confluence with River Dodder	760	743	-2%
09_587_1	River Dodder - DS of Slang	937	743	-21%
09_587_2	River Dodder - Main Channel	936	744	-21%
09_587_3	River Dodder - Model Downstream Limit (DSL)	935	746	-20%

6.2.4 Other Descriptors

Following review of the other descriptors required for hydrological calculations, no changes have been made to:

- Baseflow Index derived from soil data (BFI Soil)
- Flood Attenuation by Reservoirs and Lakes (FARL)
- Drainage Density (DRAIND)
- Mainstream Slope (S1085)
- Index of Arterial Drainage (ARTDRAIN2)

7 DESIGN EVENT FLOW CALCULATION

7.1 Preamble

7.1.1 Index Flood (Qmed)

Determination of the design flood relies on estimation of an index flood (Qmed - median annual flood discharge) and application of a flood growth factor estimated from a flood frequency curve for the T-year return period of the flood of interest.

This section reports in selection of appropriate index flood estimates. Calculations for values provided in Chapter 7 have been provided in Appendix C.

7.1.2 Growth Factors

Upon selection of a preferred method of index flood (Qmed) calculation, growth factors will be applied to obtain the T-year flood magnitude. A single Flood Frequency Curve (FFC) has been adopted to determine growth factors for the River Dodder and Little Dargle watercourses, refer to Section 5.1.3. Adoption of a single FFC is an acceptable rationalisation given the extent of the study area in combination with the available data and location of the Waldron's Bridge Gauge in relation to the study area and the Little Dargle Catchment.

The River Slang verified pluvial model was tested for 6 durations per season with results analysed to assess critical duration. For the length of the River Slang proposed for representation the reporting informs that the summer profile coupled with the 480-minute duration provided the most conservative flows. The flows extracted from the model reflect on-line attenuation and catchment lag and as such, any growth curve extracted from the model results would be unreflective of the hydrology applied.

7.2 River Dodder - Selection of Methodology

7.2.1 Flood Studies Update (FSU)

As prescribed in the agreed method statement, the initial analysis was to be conducted on the Waldron's Bridge Gauging Station (09010). The FSU portal was used for the extraction of catchment descriptors with subsequent review, refer to Section 6.2, informing of updates required to the dataset.

Calculations were conducted using the 7-variable equation to provide unadjusted Qmed rural values. These calculations were conducted using the original and updated catchment descriptors to allow direct assessment of the updates.

The uncertainty associated with the calculation of Qmed though the 7 variable equation is expressed in terms of two confidence levels, i.e. the 68% and 95%. The confidence levels are estimated via the application of the factorial standard error (FSE) which FSU documentation defines as 1.37. Upper and lower levels have been calculated for each confidence level percentage allowing comparison against the adjusted Qmed.

The adjusted Qmed rural values were reverse calculated using the Qmed urban values obtained from gauge results in combination with URBEXT values. For the original catchment descriptors, the Qmed urban values were obtained through the FSU portal for use and reflects the Qmed at the gauge calculated for dates 1985 to 2004 using AMAX data and the original URBEXT catchment descriptor. For the updated catchment descriptors the Qmed urban value was calculated using data from 1949 to present day, refer to Section 5.1 for more detail, with the updated URBEXT descriptor.

Table 7-1 Qmed Calculation using FSU 7 Variable Equation

Location	Unadjusted Qmed_rural (m³/s)	FSE Confidence Intervals 68% Lower (m³/s)	68% Upper (m³/s)	95% Lower (m³/s)	95% Upper (m³/s)	Adjusted Qmed_rural (m³/s)	URBEXT	Qmed_urban (Qmed @ Gauge)
09010 Original Catchment Descriptors	21.181	15.461	29.018	11.285	39.755	35.042	0.240	48.222
09010 Updated Catchment Descriptors	16.952	12.374	23.225	9.032	31.818	34.818	0.246	48.222

From review of Table 7-1 it is observed that the catchment descriptors have reduced the unadjusted Qmed rural value by approximately 20%. This correlates with updates to SAAR values applied, refer to Section 6.2.3, but the difference is minimal in the adjusted values.

It is also noted that the adjusted Qmed rural values for both the original and updated catchment descriptors are notably higher than those calculated using upper FSE confidence levels. At time of CFRAMs the Qmed (Q2) was calculated as 74.26 m³/s at the Waldron's Bridge gauge location reflecting a 35% decrease between CFRAMs and the 1949-present day Qmed dataset. The large difference observed relative to the CFRAMs value is attributed to calculation methodology, refer to Section 3.1 for methodology.

7.2.2 Statistical Analysis

Statistical analysis was conducted at Waldron's Bridge with results of the calculation of Qmed documented in Section 5.1.2. To summarise, the AMAX data for period 1949 to present day was used and a Qmed urban of 48.22 m³/s was calculated at the Waldron's Bridge gauge location.

7.2.3 Selection of Method and Application

The calculation of Qmed using updated catchment descriptors delivers a reduced estimate, primarily attributed to the reduction in SAAR values.

The calculation of the Qmed via catchment descriptors (original and updated) provides a lower estimated flow than those calculated through statistical analysis of the gauged flows, even when the 95% upper factorial standard error for the equation is applied.

The statistical analysis of gauge data when including an increased record length, pre 1985 and post 2004, results in a reduced estimate versus the records included in CFRAM and the FSU portal. However, the inclusion of the additional water years in the analysis increases the dataset by 226% thereby substantially increasing the confidence in the results.

Qmed was initially estimated at HEP locations on the River Dodder using the FSU 7 variable equation (including updated PCDs) with Waldron's Bridge (09010) used as a pivotal site for the adjustment of Qmed rural to align calculations with the gauge data. Table 7-2 informs the results of these calculations.

Table 7-2 FSU Qmed Values (7 Var) for River Dodder HEPs

HEP	Comment	Area (km²)	QMED Rural (m³/s)	QMED Rural Adjusted (m³/s)	URBEXT	QMED Urban (m³/s)
09_1373_2	Model USL	92.629	15.855	32.565	0.215	43.464
09_1380_1	US of Little Dargle	93.165	15.516	31.869	0.219	42.760
09010	Waldron's Bridge GS	102.750	16.952	34.818	0.246	48.222
09_1380_4	US of Slang	104.071	16.657	34.212	0.256	47.982
09_587_1	DS of Slang	111.094	17.562	36.071	0.297	53.041
09_587_2	Main Channel	111.695	17.627	36.205	0.302	53.505
09_587_3	Model DSL	111.861	17.567	36.079	0.305	53.549

Review of these calculations informed that Qmed Urban flows fluctuated and failed to ensure a consistent increase in discharge when moving downstream through the catchment. Therefore, this methodology was deemed not suitable for application in this study.

A review of the catchment descriptors informed that the most applicable methodology was the use of scaled catchment area relative to that of gauged flows at 09010 Waldron's bridge catchment. This methodology provides a Qmed rural value which is more reflective of catchment contribution and can then be adjusted using gauge data and urban extent (URBEXT) to provide a Qmed urban value that is fit for purpose. Refer to Table 7-3 for results.

Table 7-3 Qmed Values (Scaled Catchment Area) for River Dodder HEPs

HEP	Comment	Area (km²)	QMED Rural (m³/s)	QMED Rural Adjusted (m³/s)	URBEXT	QMED Urban (m³/s)
09_1373_2	Model USL	92.629	15.283	31.389	0.215	41.895
09_1380_1	US of Little Dargle	93.165	15.371	31.570	0.219	42.359
09010	Waldron's Bridge GS	102.750	16.952	34.818	0.246	48.222
09_1380_4	US of Slang	104.071	17.170	35.266	0.256	49.460
09_587_1	DS of Slang	111.094	18.329	37.645	0.297	55.357
09_587_2	Main Channel	111.695	18.428	37.849	0.302	55.936
09_587_3	Model DSL	111.861	18.456	37.906	0.305	56.260

7.2.4 Application of Growth Curves

Growth factors detailed in Section 5.1.3 have been selected for use in this study. Factors have been applied at the gauging station location to allow comparison against CFRAMs flows, refer to Table 7-4.

Table 7-4 Comparison of Design Event Flows at Waldron's Bridge

AEP (%)	Return Period (Years)	CFRAMs (m^3/s)	Single Site Analysis GEV (m^3/s)	Difference (%)
50	2	74.26	48.222	-35%
20	5	108.18	78.979	-27%
10	10	130.63	105.629	-19%
5	20		137.223	-
2	50	180.06	189.173	5%
1	100	200.95	238.348	19%
0.5	200	221.77	298.245	34%
0.1	1000		493.151	-

The comparison informs of a disparity between the flows with available data informing the 5% AEP as a pivotal design event in terms of the variances observed. The differences between the calculated values can be attributed to the dataset used for calculation of Qmed and flood frequency curve at time of CFRAMs, refer to Section 3.1 for more detail. The study design event, the 1% AEP, shows an increase in calculated flows of 19% informing of a more conservative flow than that calculated at time of CFRAMs.

Growth factors were applied to all the River Dodder HEP locations for the core events with results provided in Table 7-5.

Table 7-5 River Dodder Core Design Event Flows at study HEP Locations

HEP	Comment	10% AEP (10yr) (m^3/s)	1% AEP (100yr) (m^3/s)	0.1% AEP (1000yr) (m^3/s)
09_1373_2	Model USL	91.770	207.075	428.446
09_1380_1	US of Little Dargle	92.787	209.370	433.195
09010	Waldron's Bridge GS	105.629	238.348	493.151
09_1380_4	US of Slang	108.342	244.468	505.814
09_587_1	DS of Slang	121.259	273.616	566.122
09_587_2	Main Channel	122.527	276.475	572.039
09_587_3	Model DSL	123.236	278.076	575.350

7.3 Little Dargle - Selection of Methodology

7.3.1 Flood Studies Update (FSU)

Inspection of the FSU watercourse route and HEP locations for the Little Dargle informed of a flow route that diverted to the west at Nutgrove Avenue before flowing north through the Castle Golf Club. This was reviewed in conjunction with available network data and an alternative route was found. The alternative route was confirmed upon review of the CFRAMs survey and hydraulic data, refer to Figure 7-1.

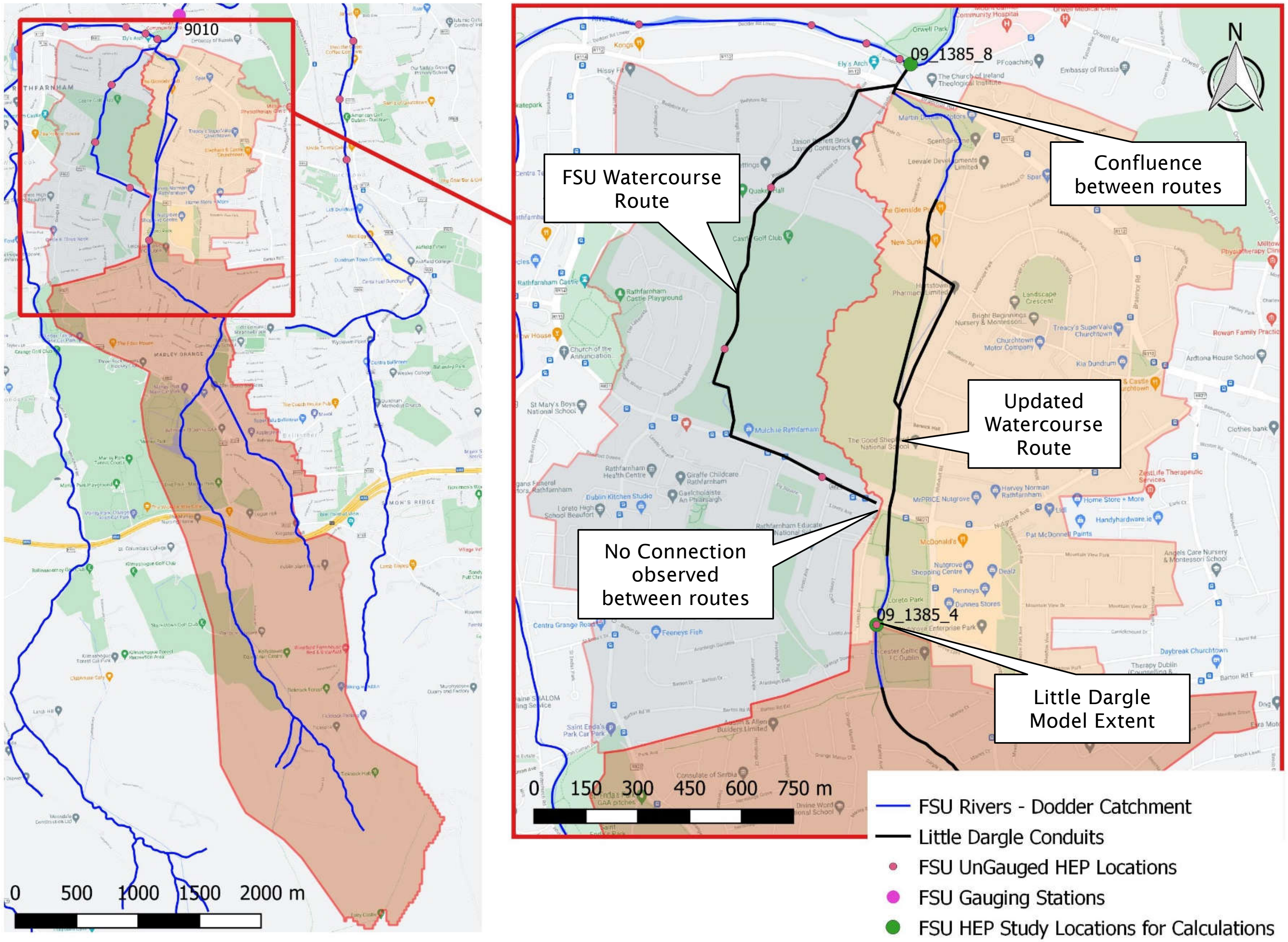

Figure 7-1 Little Dargle Watercourse Network and HEP Locations

An HEP upstream of the split in flow route, 09_1385_4, was selected as the upstream limit of the proposed modelling to enable appropriate application of hydrology due to culverting downstream of this location, whilst offering ease of flow comparison.

As the Little Dargle is sited upstream of the gauge at Waldron's bridge it was appropriate to adopt the same methodology applied to the HEP locations on the River Dodder where the Qmed rural value is adjusted to correlate with that of the gauge data, refer to Section 7.2.3.

The uncertainty associated with the calculation with of the Qmed though the 5 variable equation is expressed in terms of two confidence levels, i.e. the 68% and 95%. The confidence levels are estimated via the application of the factorial standard error (FSE) which FSU documentation defines as 1.674. Upper and lower levels have been calculated for each confidence level percentage allowing comparison against the adjusted Qmed. Calculation results have been provided in Table 7-6.

Table 7-6 Little Dargle Qmed Calculation using FSU 7 Variable Equation

		FSE Confidence Intervals						
Location	**Unadjusted Qmed_rural (m^3/s)**	**68% Lower (m^3/s)**	**68% Upper (m^3/s)**	**95% Lower (m^3/s)**	**95% Upper (m^3/s)**	**Adjusted Qmed_rural (m^3/s)**	**URBEXT**	**Qmed_urban (m^3/s)**
09_1385_4 Model Upstream Limit	1.613	1.178	2.210	0.860	3.028	3.314	0.324	5.022
09_1385_8 Confluence with River Dodder	1.951	1.424	2.672	1.039	3.661	4.006	0.484	7.189

Review of the results informs that the adjusted Qmed rural values at each location exceed those of the upper end of the 95% confidence bracket.

The total catchment size for the Little Dargle is c. 9 km^2 therefore the use of the FSU 5 variable equation is appropriate for assessment as catchment was <25 km^2. The uncertainty associated with the calculation of Qmed though the 5 variable equation is expressed in terms of two confidence levels, i.e. the 68% and 95%. The confidence levels are estimated via the application of the factorial standard error (FSE) which documentation provided by the OPW is defined as 1.674. Upper and lower levels have been calculated for each confidence level percentage allowing comparison against the adjusted Qmed. Calculation results have been provided in Table 7-7.

Table 7-7 Little Dargle Qmed Calculation using FSU 5 Variable Equation

Location	Unadjusted Qmed_rural (m³/s)	FSE Confidence Intervals 68% Lower (m³/s)	68% Upper (m³/s)	95% Lower (m³/s)	95% Upper (m³/s)	Adjusted Qmed_rural (m³/s)	URBEXT	Qmed_urban
09_1385_4 Model Upstream Limit	2.467	1.473	4.129	0.880	6.912	5.066	0.324	7.677
09_1385_8 Confluence with River Dodder	2.966	1.772	4.965	1.058	8.312	6.092	0.484	10.933

Review of the results informs that the adjusted Qmed rural values at each location exceed the 95% confidence bracket.

7.3.2 IoH124

Calculations using the Institute of Hydrology Report No. 124 (IoH124) methodology for the estimation of runoff from small catchments were conducted to allow comparison of flows. This methodology calculates Qbar rural, which is the mean annual flood flow from a rural catchment (approximately 2.3-year return period), using catchment descriptors. WRAP Soil type 4 was conservatively selected for use in this calculation to reflect the urban and rural elements of the catchment. To allow comparison Qbar rural has been converted to Qmed using a factor of 1.07.

The uncertainty associated with the calculation using with of the Qmed though this methodology can be calculated via the application of the factorial standard error (FSE) which is stated by OPW on the FSU portal as being 1.64. In line with FSU calculations this has been expressed in terms of two confidence levels, i.e. the 68% and 95%. Upper and lower levels have been calculated for each confidence level percentage allowing comparison against the adjusted Qmed.

Table 7-8 Little Dargle Qmed Calculation using IoH124

Location	Unadjusted Qmed_rural (m³/s)	FSE Confidence Intervals 68% Lower (m³/s)	68% Upper (m³/s)	95% Lower (m³/s)	95% Upper (m³/s)	Adjusted Qmed_rural (m³/s)	URBEXT	Qmed_urban
09_1385_4 Model Upstream Limit	2.304	1.405	3.779	0.857	6.197	4.733	0.324	7.172
09_1385_8 Confluence with River Dodder	2.975	1.814	4.880	1.106	8.338	6.111	0.484	10.967

The Qmed rural values have been adjusted as per previously defined methodology, refer to Section 7.2.1. Adjusted values are noted as exceeding the 95% confidence interval.

The IoH124 methodology does not account for urbanisation and as such is initially perceived as unsuitable for a catchment with physical characteristics such as that for the Little Dargle. There is no agreed protocol within the IoH124 to estimate the effect of urbanisation; in the absence of same an urbanisation uplift, applied in line with FSU methodology, has been accounted for in the calculations in order to provide a nominal comparable value to other calculation techniques.

7.3.3 GDSDS

The GDSDS model was simulated as detailed in Section 3.5. Results were extracted from the model at the locations correlating with the HEP locations selected for review for the 50% AEP (2 yr Return Period), for all durations and both seasons. Upon extraction of results the maximum flows were selected and are presented in

Table 7-9.

Table 7-9 Little Dargle GDSDS Max 50% AEP flow extraction at HEP locations

HEP	Comment	Critical Season	Critical Duration (min)	Flow (m^3/s)
09_1385_4	Model Upstream limit GDSDS node SO15273804	Winter	60	3.124
09_1385_8	Confluence with River Dodder GDSDS node SO15293421	Winter	120	5.457

It is noted that there are differing critical durations dependant on the review location within the catchment but the season providing the greatest flow was winter in both instances.

7.3.4 Selection of Method and Application

To adopt the correct methodology a review of the Little Dargle hydrology must be conducted in conjunction with flows expected from this watercourse to the River Dodder. This is due to the location of the gauge, the means for Qmed adjustment, downstream of the Little Dargle confluence. Calculation of expected flows, known as Top Up flows, is required across multiple AEPs thereby ensuring minimum and maximum flows are achievable using the selected methodology, refer to Table 7-10 for Top up flow calculation.

Table 7-10 Little Dargle Flows using River Dodder HEP data

HEP	Comment	50% (2yr)	10% (10yr)	1% (100yr)	0.1% (1000yr)
09_1380_1	US of Little Dargle	42.359	92.787	209.370	433.195
09010	Waldron’s Bridge GS	48.222	105.629	238.348	493.151
Top up Flows from Little Dargle		5.863	12.842	28.978	59.956

A review of the top up flows was conducted for each of the methodologies for the same exceedance probabilities with the following results, refer to Table 7-11. Flows calculated at time of CFRAMs have been included to allow for a complete comparison.

Table 7-11Little Dargle Comparative Catchment flows

HEP	AEP (%)	CFRAMS (m^3/s)	FSU 7 VAR (m^3/s)	FSU 5 VAR (m^3/s)	IoH124 (m^3/s)	GDSDS (m^3/s)
09_1385_8 @ Confluence With River Dodder	50	11.66	7.189	10.933	10.967	5.457
	10	17.56	15.748	23.948	24.022	7.170
	1	29.56	35.535	54.038	54.206	8.270
	0.1	47.66	73.524	111.806	112.153	-

A review of the tabularised results informs that the FSU 7 Variable equation provides the most comparable results overall. The GDSDS model was proposed in the method statement as the means for the application of lateral inflows but this review has informed that the GDSDS is unable to provide the flows required to achieve calibration. The FSU 5 variable and IoH124 are viewed as providing overly conservative flows that would not allow calibration to be achieved.

With all the results in mind it is proposed that the results using the FSU 7 variable are taken forward for use in this study

7.3.5 Application of Growth Curves

Growth factors detailed in Section 5.1.3 have been selected for use in this study. Factors have been applied at the downstream extent of the Little Dargle to allow comparison against CFRAMs flows, refer to Table 7-12.

Table 7-12 Little Dargle Comparison of Design Event Flows at Confluence with River Dodder

AEP (%)	Return Period (Years)	CFRAMs (m^3/s)	FSU 7 Var (m^3/s)	Difference (%)
50	2	11.66	7.189	-38%
20	5	14.69	11.775	-20%
10	10	17.56	15.748	-10%
5	20	-	20.459	-
2	50	25.44	28.204	11%
1	100	29.56	35.535	20%
0.5	200	34.23	44.466	30%
0.1	1000	47.66	73.524	54%

The comparison informs of a disparity between the flows with a pivotal point between the 10% and 2% AEPs noted in the design events in terms of magnitude. The differences between the calculated values can be attributed to the dataset used for calculation of Qmed and flood frequency curve at time of CFRAMs, refer to Section 3.1 for more detail. The study design event, the 1% AEP, shows an increase in calculated flows of c. 6 m^3/s equating to 20% increase in flows in comparison to those calculated at time of CFRAMs.

Growth factors were applied to each of the Little Dargle HEP locations for the core events with results provided in Table 7-13.

Table 7-13 Little Dargle Core Design Event Flows at study HEP Locations

HEP	Comment	10% (10yr)	1% (100yr)	0.1% (1000yr)
09_1385_4	Little Dargle - Model USL	11.001	24.823	51.359
09_1385_8	Little Dargle - Confluence with River Dodder	15.748	35.535	73.524

7.4 River Slang – Selection of Methodology

A review was conducted of a study completed by a 3rd party in December 2019 on the River Slang catchment. The study involved the production of a verified integrated catchment model for the River Slang, refer to Section 3.4 for more detail. As per Section 5.2, the results from the model have been accepted for use in this study. To ensure results from the model were appropriate for use comparison against other methodologies was conducted.

7.4.1 Comparison of Modelled Flows

Flow comparison was conducted on the verified Slang model at two locations correlating with HEP locations outlined in Section 6.1, i.e. the gauge location and the confluence with the River Dodder. Flows were extracted directly from the model for comparison.

To complete the analysis for the other methodologies, the AMAX data was reviewed between 1986 and 2019 which provided a Qmed of 2.48 m³/s at the Frankfort gauge. The data was then subject to a single site analysis with the LN2 distribution selected as the most appropriate for preparation of growth factors. With these calculations completed the Qmed_rural was calculated and adjusted using gauge data prior to conversion to Qmed_urban. The growth curves were then used to calculate the 50%, 10%, 1% and 0.1% values for comparison against model results for multiple methodologies, refer to Table 7-14 for results.

Table 7-14 Comparison of Modelled Flows Against Alternative Methodologies

HEP	AEP	River Slang Model (m³/s)	FSU 7 Var (m³/s)	FSU 5 Var (m³/s)	IoH124 (m³/s)
09011	50%	2.338	2.480	4.064	3.694
	10%	4.409	4.612	7.559	6.870
	1%	9.020	7.649	12.535	11.392
	0.1%	12.991	11.071	18.144	16.490
09_1381_8	50%	2.417	1.991	3.728	3.411
	10%	4.613	3.703	6.933	6.344
	1%	9.357	6.141	11.497	10.521
	0.1%	11.645	8.889	16.641	15.228

Extraction of modelled data at both locations found the flows loosely correlated with the calculated FSU 7 variable flows but lay below those of the IoH124 and FSU 5 variable values, the latter providing the most conservative flows as expected. A direct match of flows at the comparison locations was not expected due to calculation methodology, but confirmation that modelled flows are in line with expectations is accepted.

A comparison against flows calculated and simulated for the CFRAM study was also conducted. Table 7-15 informs that flows applied at that time are a multiple larger than all of those calculated for this study and as such provide an overestimate for the catchment contribution to the River Dodder, refer to Section 3.1 for more detail.

Table 7-15 River Slang CFRAMs Flows

AEP (%)	CFRAMs Hydrology Report (m³/s)	CFRAMs Mapping (m³/s)
50	9.593	-
10	17.116	11.546
1	32.904	27.713
0.1	60.09	51.208

7.4.2 Selection of Method and Application

To ensure the correct methodology has been adopted a review of the River Slang hydrology was conducted in conjunction with flows expected from this watercourse to the River Dodder. Calculation of expected flows, known as Top Up flows, is required across multiple AEPs thereby ensuring minimum and maximum flows are achievable using the selected methodology, refer to Table 7-16 for Top up flow calculation.

Table 7-16 River Slang Flows using River Dodder HEP data

HEP	Comment	50% (2yr)	10% (10yr)	1% (100yr)	0.1% (1000yr)
09_1380_4	US of Slang	49.460	108.342	244.468	505.814
09_587_1	DS of Slang	55.357	121.259	273.616	566.122
Top up Flows from River Slang		5.897	12.918	29.148	60.308

A review of the top up flows was conducted against the modelled River Slang flows for the same exceedance probabilities with the following results, refer to Table 7-17.

Table 7-17 River Slang Modelled Flows comparison to required River Dodder Top Up Flows

HEP	50% AEP (m³/s)	10% AEP (m³/s)	1% AEP (m³/s)	0.1% AEP (m³/s)
09_1381_8_01 River Slang Model	2.524	4.761	9.739	14.027
River Dodder Required Top up	5.897	12.918	29.148	60.308
Difference	3.373	8.157	19.408	46.281
Difference %	-57%	-63%	-67%	-77%

It is noted that the required Top up flows on the River Dodder are significantly higher than those provided by the River Slang model. The order of magnitude in terms of the River Dodder main channel flow is 7% and therefore the difference it is not deemed significant for flood estimation for the River Dodder flood relief scheme.

Review of the River Dodder HEP locations informs that HEP 09_1380_4 is located approx. 350m upstream of the River Slang confluence and that an additional c. 2.2 km² catchment area contributes flows which would reduce the disparity. The calculations on the River Slang inform of the conservative nature of the calculations conducted on the River Dodder due to the calculation methodology applied.

8 APPLICATION OF HYDROLOGY

8.1 Joint Probability

A review of joint probability was conducted for this phase of the Dodder flood alleviation scheme, relative to the flow contribution of tributaries to the River Dodder. It was concluded that due to the size and location of the catchment it is expected that rainfall would fall on the whole catchment. Thereby joint probability for contributing tributaries would be calculated as 1.

The focus of this study is the River Dodder with peak flows assessed at multiple instances along the watercourse. Tributary flows will be timed in accordance with these peak flows thereby ensuring calculated flows are achieved.

8.2 Model Flows – Present Day

A summary of design flood discharge for T-year floods for present-day are presented in the following tables. For clarity, the watercourses have been split.

Table 8-1 River Dodder - Design Flows per AEP (Present Day)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09_1373_2	41.895	41.895	68.616	91.770	119.218	164.352	207.075	259.113	428.446
09_1380_1	42.359	42.359	69.377	92.787	120.540	166.174	209.370	261.985	433.195
09010	48.222	48.222	78.979	105.629	137.223	189.173	238.348	298.245	493.151
09_1380_4	49.460	49.460	81.007	108.342	140.747	194.030	244.468	305.903	505.814
09_587_1	55.357	55.357	90.665	121.259	157.528	217.165	273.616	342.376	566.122
09_587_2	55.936	55.936	91.613	122.527	159.174	219.434	276.475	345.954	572.039
09_587_3	56.260	56.260	92.143	123.236	160.095	220.705	278.076	347.957	575.350

Table 8-2 Little Dargle - Design Flows per AEP (Present Day)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09_1385_4	5.022	5.022	8.225	11.001	14.291	19.701	24.823	31.060	51.359
09_1385_8	7.189	7.189	11.775	15.748	20.459	28.204	35.535	44.466	73.524

Table 8-3 River Slang - Design Flows per AEP (Present Day)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09011	-	2.338	3.470	4.409	5.433	7.500	9.020	10.443	12.991
09_1381_8	-	2.524	3.746	4.761	5.866	8.098	9.739	11.276	14.027

8.3 Model Flows – Climate Change

Mid-Range Future Scenario (MRFS) and High-End Future Scenario (HEFS) have been calculated using updated urban extent values, refer to Section 6.2.2, to provide an updated Qmed value for scaling as per the OPW Climate Change Sectoral Adaptation Plan guidance.

This methodology accounts for urban creep, and thereby increased runoff, within the study area and is deemed essential given the substantial influence that the urbanised area has on the contributing catchment. The following tables provide the results for the River Dodder and Little Dargle.

Table 8-4 River Dodder - Design Flows per AEP (MRFS)

HEP	Qmed	Flow Rate (m^3/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09_1373_2	45.391	54.469	89.211	119.314	155.000	213.681	269.226	336.883	557.039
09_1380_1	45.870	55.044	90.153	120.574	156.637	215.937	272.069	340.441	562.922
09010	52.169	62.603	102.533	137.131	178.147	245.590	309.430	387.191	640.223
09_1380_4	53.511	64.214	105.170	140.659	182.729	251.907	317.389	397.150	656.692
09_587_1	59.691	71.629	117.315	156.901	203.831	280.997	354.041	443.012	732.525
09_587_2	60.276	72.331	118.465	158.439	205.829	283.751	357.511	447.355	739.705
09_587_3	60.605	72.726	119.113	159.306	206.954	285.303	359.467	449.802	743.751

Table 8-5 Little Dargle - Design Flows per AEP (MRFS)

HEP	Qmed	Flow Rate (m^3/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09_1385_4	5.470	6.564	10.750	14.378	18.678	25.749	32.443	40.595	67.125
09_1385_8	7.759	9.311	15.250	20.395	26.496	36.526	46.021	57.587	95.220

Table 8-6 River Slang - Design Flows per AEP (MRFS)

HEP	Qmed	Flow Rate (m^3/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09011	-	2.852	4.262	5.369	6.931	9.021	10.461	11.643	13.503
09_1381_8	-	3.080	4.602	5.797	7.484	9.741	11.296	12.571	14.580

Table 8-7 River Dodder - Design Flows per AEP (HEFS)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
09_1373_2	48.977	63.670	104.280	139.468	181.183	249.775	314.703	393.789	651.134
09_1380_1	49.471	64.313	105.332	140.875	183.011	252.296	317.879	397.763	657.704
09010	56.216	73.081	119.694	160.083	207.964	286.695	361.220	451.996	747.379
09_1380_4	57.665	74.964	122.777	164.207	213.322	294.081	370.526	463.641	766.634
09_587_1	64.129	83.367	136.541	182.615	237.235	327.047	412.062	515.614	852.572
09_587_2	64.720	84.136	137.799	184.298	239.421	330.061	415.859	520.365	860.428
09_587_3	65.055	84.571	138.513	185.252	240.661	331.771	418.013	523.061	864.886

Table 8-8 Little Dargle - Design Flows per AEP (HEFS)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	**20% (5)**	**10% (10)**	**5% (20)**	**2% (50)**	**1% (100)**	**0.5% (200)**	**0.1% (1000)**
09_1385_4	5.930	7.709	11.646	15.576	20.235	27.895	35.146	43.978	72.719
09_1385_8	8.343	10.087	16.520	22.095	28.704	39.570	49.856	62.385	103.155

Table 8-9 River Slang - Design Flows per AEP (HEFS)

HEP	Qmed	Flow Rate (m³/s) per AEP							
		50% (2)	**20% (5)**	**10% (10)**	**5% (20)**	**2% (50)**	**1% (100)**	**0.5% (200)**	**0.1% (1000)**
09011	-	3.114	4.648	5.931	7.626	9.670	10.995	12.167	13.554
09_1381_8	-	3.362	5.019	6.404	8.235	10.441	11.872	13.137	14.635

8.4 Hydrograph Shape

The application of peak flows to the hydraulic model will be through the use of design hydrographs. The FSU recommended methodology was adopted in creation of these hydrographs for the River Dodder and Little Dargle watercourse. Similarly to deriving the index flow, hydrograph shape parameters are estimated for ungauged locations using pivotal catchments where gauge data is unavailable.

A hydrograph shape was extracted from the FSU portal for Waldron's Bridge gauge (09010) on the River Dodder as this is the pivotal location for the majority of the study hydrology. The shape extracted was not adjusted using any other pivotal site. Comparison of the shape was conducted against a spread of high magnitude storms which informed that the FSU shape was more conservative than those experienced, refer to Figure 8-1.

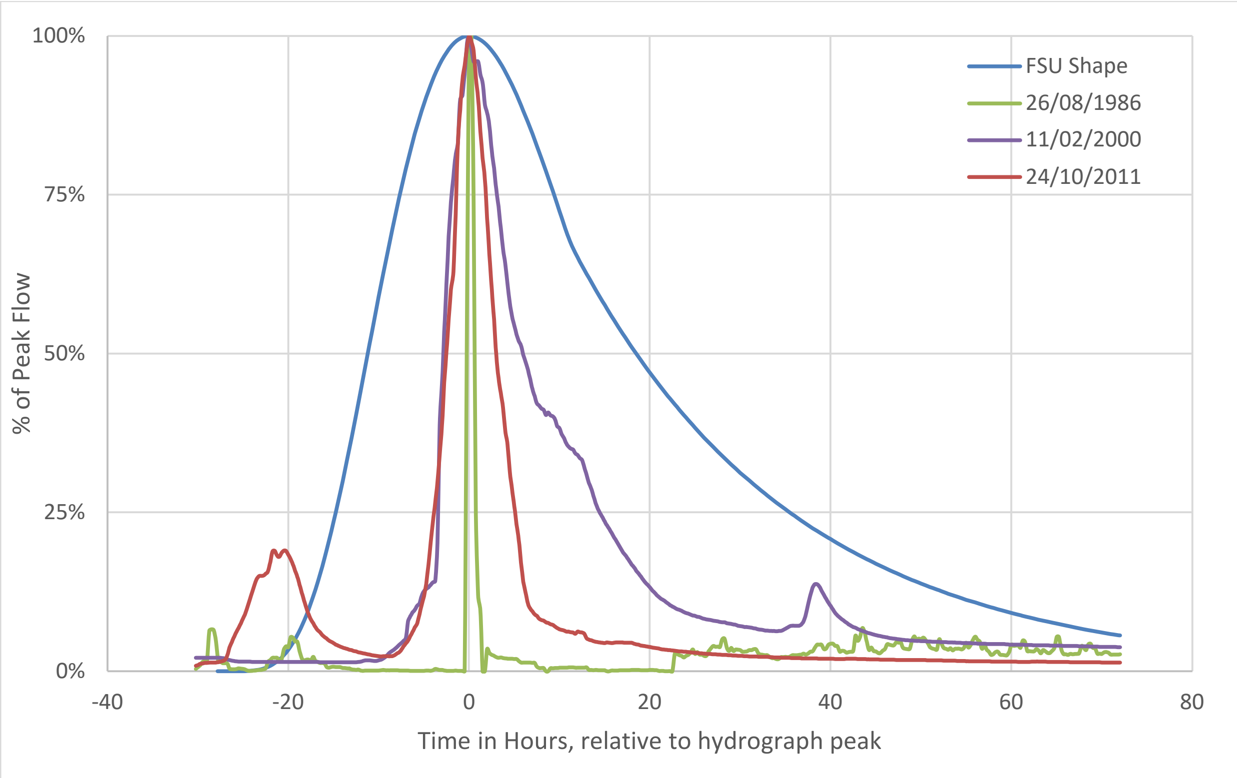

Figure 8-1 09010 Waldron's Bridge Hydrograph shape

The FSU portal was also used in preparation of a hydrograph for the Little Dargle. A pivotal site was, 25002 Barringtons Bridge, was selected for use for the Little Dargle as it was the best available site to reflect its flashy nature, refer to Figure 8-2.

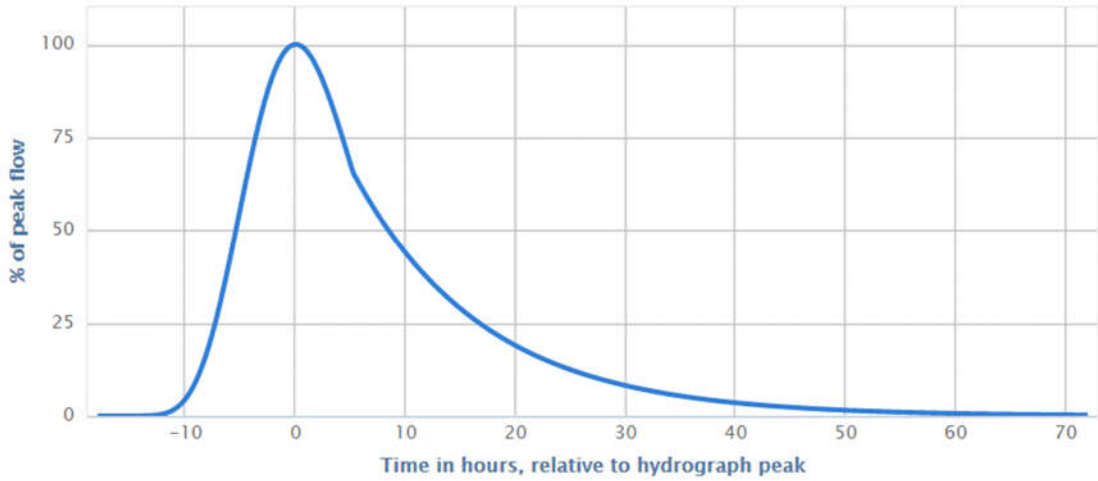

Figure 8-2 Little Dargle Hydrograph shape

As the Little Dargle is ungauged no comparison against storm data is available. FSU hydrographs typically are influenced by large, gauged catchments. The Little Dargle is expected to have a short duration flashy response to rainfall on the small catchment. Although all available measures have been taken to best replicate the expected response of the Little Dargle catchment, the hydrograph may be an overestimate in terms of flood volume due to a longer than reasonably anticipated receding limb.

The potential consequences to the project as a result of overestimation of volume within the extended hydrograph have been considered. Excessive volume will not affect model outcomes where peak flow rate is likely to determine maximum water surface elevations on the Dodder. Volume would influence evaluation of options that include storage or management of a flood volume.

Initial consideration of the options for the scheme indicate that upstream storage / natural water retention / online storage are not to be considered as part of the option assessment, refer to sections 2.3 and 4.3 for more detail. As such, selection of the adopted hydrograph can be deemed precautious without prejudicing the outcome of the project.

8.5 Application of flows to the model

The application of flows to the hydraulic model will be via point and lateral inflows to replicate location of contributing flows from upstream catchments, storm networks and overland flows. The application will ensure the magnitude of calculated hydrology is achieved thereby guaranteeing appropriate defence design on the River Dodder and Little Dargle.

Catchment hydrology will be appropriately split using contributing area. Point inflows will be applied at modelled upstream extents for all open channel watercourses represented as well as at storm discharge locations on the Dodder. Lateral inflows will be applied at two locations on the River Dodder replicating contributing flows from Bushy Park and Milltown Golf Club.

Storm networks adjacent to the River Dodder will be represented in the model to allow the assessment of proposed defences on those networks. Point inflow hydrographs will be applied to coincide with main drainage outfall locations, to suit upstream contributing areas. The previous pluvial analysis and GDSDS analysis determined that while GDSDS outflows are insufficient (and as such are likely to represent throttling by pipe capacity with surcharging of the network apparatus upstream), surface flood routing to the River Dodder that would result from surcharge broadly correlates with piped drainage outlets and thus application of cumulative estimates (representative of piped / network discharge and overland flooding) at those location is appropriate.

Similarly, provision of flows to the Little Dargle will be carefully considered allowing for appropriate application as assessment of flows relative to the proposed defence adjacent to Braemor Road.

Calculations for values provided in chapter 8 have been provided in Appendix D.

Appendix A

Dundrum Slang Hydraulic Modelling Report

Appendix B

Rating Review Calculation Sheets

Appendix C

Hydrology Calculation Sheets

Appendix D

GIS Files

Data	File Name	File Type
Study HEP Locations-Ungauged	Ungauged HEP Point [UPDATED].cpg	GIS Settings file
	Ungauged HEP_Point [UPDATED].dbf	GIS Database file
	Ungauged HEP_Point [UPDATED].prj	GIS Projection file
	Ungauged HEP_Point [UPDATED].qmd	GIS Metadata file
	Ungauged HEP_Point [UPDATED].shp	GIS Shapefile
	Ungauged HEP_Point [UPDATED].shx	GIS Index file
Study HEP Locations - Gauged	Gauged HEP_Point [UPDATED].cpg	GIS Settings file
	Gauged HEP_Point [UPDATED].dbf	GIS Database file
	Gauged HEP_Point [UPDATED].prj	GIS Projection file
	Gauged HEP_Point [UPDATED].qmd	GIS Metadata file
	Gauged HEP_Point [UPDATED].shp	GIS Shapefile
	Gauged HEP_Point [UPDATED].shx	GIS Index file
Study Catchment Extents	Catchment_Polygon.cpg	GIS Settings file
	Catchment_Polygon.dbf	GIS Database file
	Catchment_Polygon.prj	GIS Projection file
	Catchment_Polygon.qmd	GIS Metadata file
	Catchment_Polygon.qml	GIS Style file
	Catchment_Polygon.shp	GIS.Shapefile
	Catchment_Polygon.shx	GIS Index file