3. Hydrological estimation and tidal water level analysis

A detailed hydrological analysis of the various contributing catchments has been undertaken as part of the study and is reported on separately in the Hydrology Report (279365-HEL-1-RP-RP-HYD-000002). The objective is to provide reliable estimates of flood magnitudes and hydrograph shapes for various return period events for input as the inflow boundaries to the hydraulic flood model, as well as tidal design curves for input as the outflow boundary of the model. The analysis utilised a number of hydrological estimation methods to establish a range of design flows and tidal heights.

Below is a summary of the key outcomes of the hydrological analysis. The reader is referred to the accompanying hydrology report for a detailed description of the work.

3.1 Flood flow analysis

A review of historical fluvial and tidal flood events and statistical significance of these events was carried out and an estimate of the return periods of each was undertaken. Hydrometric data from several gauging stations were collated and reviewed and rating reviews were undertaken for Claregalway gauge (River Clare), Wolfe Tone gauge and Dangan gauge (Corrib River). The Index flood method was used to conduct the flood frequency analysis for the River Corrib.

The QMED value (1 in 2 year) peak flow was estimated using the AM series for Wolfe Tone and Dangan gauging stations, new rating curves for each and the LN2 distribution when performing at-site analysis. The average value of the two resulted in a QMED of 255.7m^3/s.

Several methods were used and compared to estimate the flood growth factors for the River Corrib:

- At-site frequency analysis of river AM series of the Corrib gauges (Wolfe Tone and Dangan);
- Averaging the flows from the above analysis; (this was used to produce the QMED)
- River Corrib Pooled Analysis using a regional pooling group of 9 Corrib gauging stations;
- Pooled Analysis using hydrologically similar sites and utilising 3 and 4 PCD parameters);
- OPW FSU 2021 Q-Atlas method.

The above methods were fitted to a series of 2-parameter or 3-parameter probability distributions (i.e., PE3, GLO, LO, GEV, EV1, LN3, LN2, Weibull or Wakeby).

The most appropriate growth factor estimation method and distribution were found to be the pooled analysis using hydrologically similar sites utilising 4 PCD parameters (AREA, SAAR, BFIsoil and FARL), fitted to the 2-parameter EV1 distribution. The final recommended return period growth factors and design flows for the River Corrib at Galway City (HEP Node 30-3419-2) are shown below.

Table 4 Recommended return period growth factors and flows

Return Period T years	X_T	QT m^3/s
2	1.000	255.8
5	1.189	304.0
10	1.313	335.7
20	1.433	366.4
50	1.588	406.0
100	1.704	435.7
200	1.820	465.4
1000	2.088	533.8

The approach followed to generate a realistic design hydrograph for the River Corrib was to match suitable observed Corrib flood hydrograph shapes using a gamma curve relationship, which is similar to the FSU hydrograph width methodology used in the OPW FSU Web Portal method. The events used were the November 2009, December2015 /January 2016, and February 2020 flood events. The data used was taken from the Dangan gauge during these events and are compared with the UPO Gamma curve in Figure 3.1.1. The final hydrographs for all the design flood events used for modelling are shown in Figure 3.1.2.

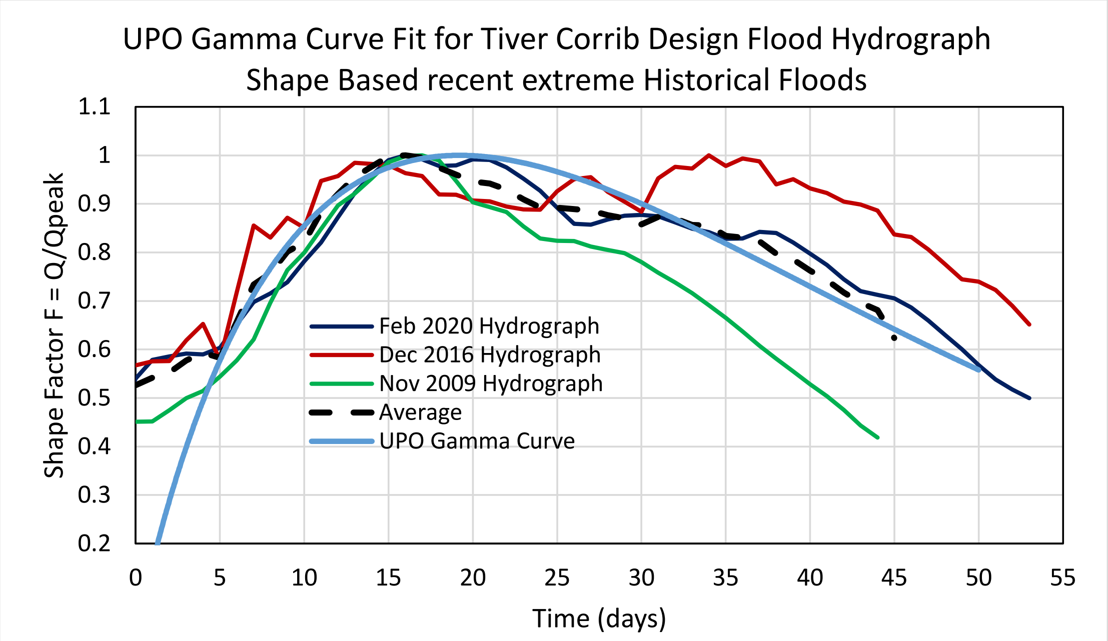

Figure 3.1.1 UPO Gamma Curve hydrograph fit for the Corrib at Dangan compared to historical events (copied from Hydrology report)

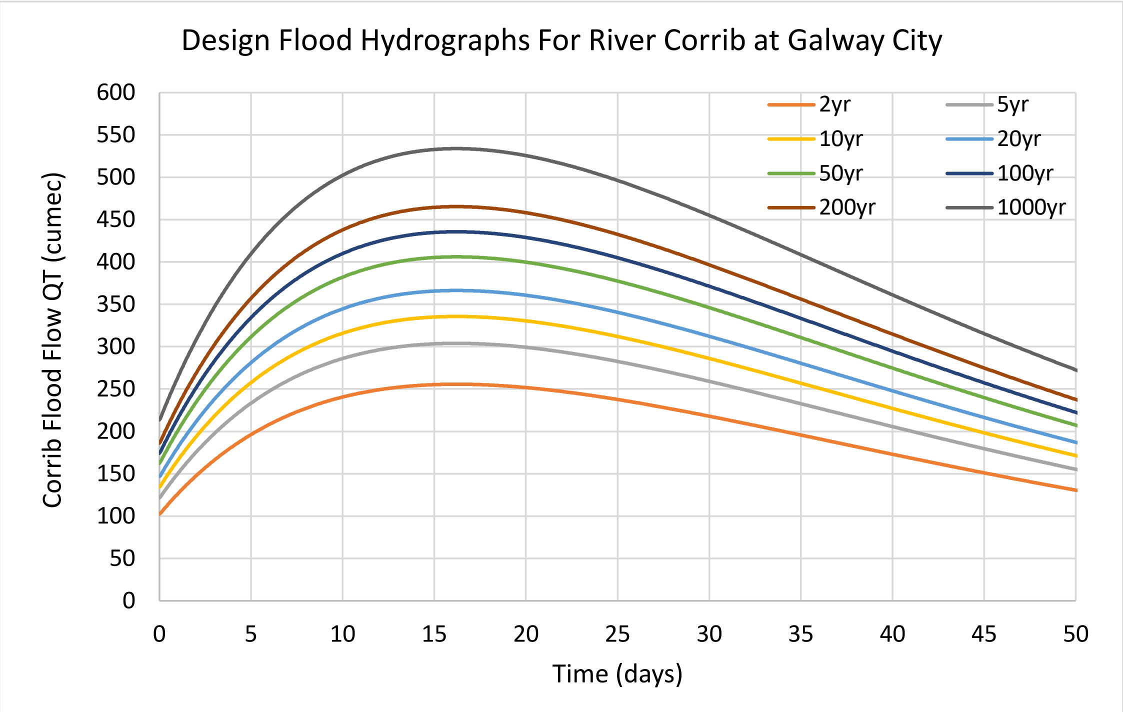

Figure 3.1.2 Return period design flow hydrographs for River Corrib at Galway City

3.2 Tidal flood frequency analysis

A tidal flood frequency analysis was also undertaken as part of the hydrological analysis to assess the potential tidal flooding in Galway City from the combined effect of tidal and storm surge events.

The analysis was undertaken on the annual maximum series of tide levels extracted from the Wolfe Tone Bridge (30061), Galway Port Gauge (30062) and Oranmore (29015) tidal gauges and fitting a variety of statistical distributions to the AM data. Oranmore station provided the most robust results with 40 years of data. The EV1 distribution was the preferred and recommended method to generate the peak levels.

The generated design highwater levels were compared with the levels generated by the ICWWS (Irish Coastal Wave and Water Level Study, RPS, 2020) for node W6 outside Galway Bay. The recommended design tide highwater level is the average of the two (Oranmore at-site frequency analysis and ICWWS). These are shown in Table 5.

Table 5 Recommended Return Period Design Tide Highwater Level HT

Return Period years	ICWWS W6 HT mOD Malin OGSM15	Gauged Oranmore HT mOD Malin OGSM15	Recommended Design HT mOD Malin OGSM15
2	3.29	3.09	**3.19**
5	3.44	3.29	**3.37**
10	3.55	3.42	**3.49**
20	3.66	3.55	**3.60**
50	3.80	3.71	**3.76**
100	3.90	3.83	**3.87**
200	4.01	3.96	**3.98**
1000	4.26	4.24	**4.25**

The return period design tide profiles are estimated using the astronomical spring tide profile, computed from tidal harmonics measured at the Galway Docks Gauge and applying a design surge profile that uplifts the tidal curve so as to achieve the required return period design flood levels. In order to determine a representative design surge profile for the study area the surge time series was extracted from the available gauged record for Galway Docks for the period March 2007 to May 2022.

Given the irregularity of the surges profiles and variation between events it is recommended that a smooth wide profile is used to represent the design surge profile. Different functions were tested, with the sine curve being selected. Different combinations of timing of the surge compared to the tidal curve were tested to identify the most appropriate to be used. The recommended tidal flood profile that retains a degree of conservatism but is not overly conservative is the combination of storm surge profile that peaks at mid-ebb stage coinciding with high spring tides.

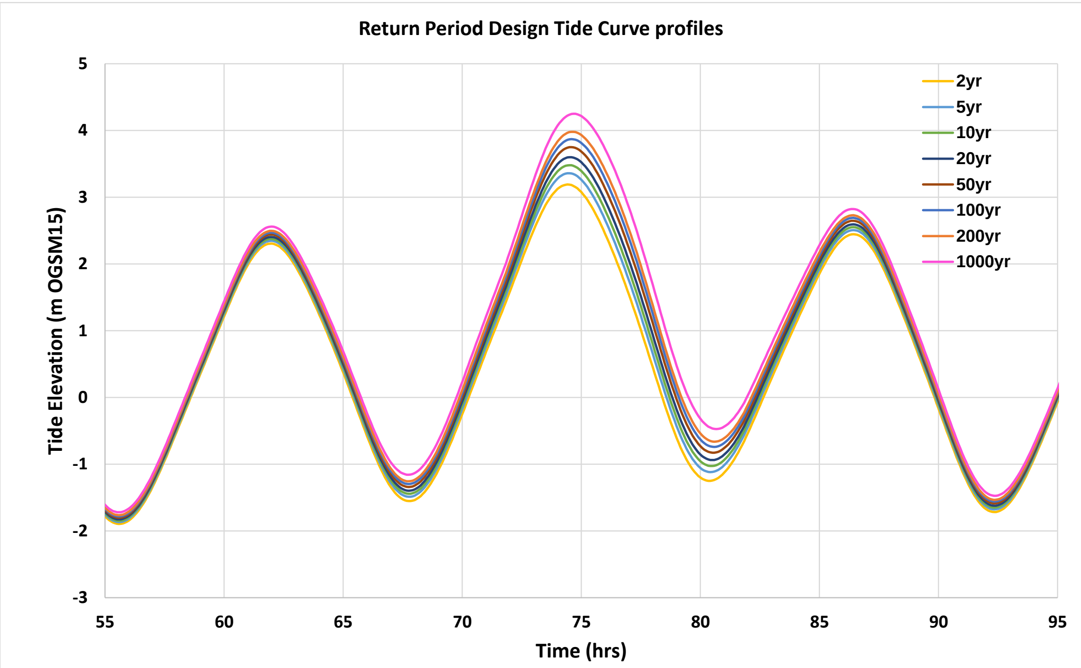

Figure 3.2.1 Return period tides for combination of storm surge peaking at mid-ebb and mean spring tide conditions

3.3 Joint probability

Joint probability analysis was undertaken for the fluvial – tidal conditions, between tide levels and river Corrib flows. Daily average fluvial flows at Dangan and daily maximum high water levels at Wolfe Tone and Galway Docks between 2009-2020 were used for the analysis. A poor correlation between tide levels and River Corrib flows was found, suggesting little or no correlation between the two. Utilising the Hawkes, 2004 relationship for combined probability of two variables, a series of peak tidal heights were produced for the peak flood flows and a series of flood flows were produced for the peak tidal heights. Please refer to the Hydrology report for further information on this.

3.4 Urban Flows

Urban design flows for a range of events on the Sruffnacashlaun and Terryland urban catchments were estimated as part of the study. The three key steps used in the derivation of the flows are summarised as:

- IH124 (1993) was used to estimate the Qbar for the catchments;
- The FSR (NERC 1975)[6] was used to estimate the growth curve;
- The FSR synthetic hydrograph method (based on the time to peak) was used to derive the hydrograph shapes.

The reader is referred to the hydrology report for a detailed description of the work.

3.5 Climate Change

The OPW recommends an increase in flows of 20% and 30% at the Mid-Range Future Scenario (MRFS) and the High-End Future Scenario (HEFS) be included in assessing the adaptation of the proposed Flood relief scheme to future climate change. An increase in sea level of 0.5m and 1m at the MRFS and HEFS is also recommended. It was agreed with the OPW that the above recommendations would be followed for the Galway City FRS.

3.6 Uncertainty in the hydrological estimation

The uncertainty associated with the hydrological estimation of the design flows and its impact on the outline design of the scheme and freeboard allowance will be considered as part of the optioneering for the scheme and will be reported on in the Options report.

4. Wave over topping analysis

4.1 Introduction

A Wave Over Topping (WOT) assessment for the coastline of the scheme area (Figure 1.3.3) has been undertaken as part of the project. The purpose of the assessment is to:

- Estimate the wave overtopping discharges at various cross sections for a range of AEP events for the current, MRFS and HEFS climate epochs
- Estimate the flood risk associated with WOT by modelling the propagation of overtopping discharges across the coastal floodplain using a hydraulic model.
- Develop options which mitigate the risk of overtopping across the study area (discussed in the Options report).

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[6] Flood Study Report (5 volumes), Natural Environmental Research Council, Wallingford, UK.

The WOT risk is to be considered in parallel with the tidal risk i.e. as WOT generally requires an elevated tidal water level to occur, the risk of WOT is typically accompanied by a risk of direct tidal inundation. The risk from both of these sources (i.e., WOT and tidal inundation) is therefore to be considered in parallel as part of the coastal flood risk analysis.

There is a high degree of uncertainty in undertaking any WOT assessment due to the complex hydraulic mechanisms associated with the overtopping process. The Eurotop manual notes that any WOT study achieves an order of magnitude level of accuracy. While this study has adopted a very rigorous approach to the WOT calculations and utilised the best available datasets as input, uncertainty over the results remain. The WOT/tidal inundation flood maps produced as part of the study need to be considered in this regard.

This chapter sets out the methodology adopted as part of the analysis. The datasets used to inform the analysis were presented in Section 2 of the report. The results of the hydraulic modelling of the WOT and tidal risk are discussed in Section 9. The full set of results from the WOT assessment, as well as detailed information on each of the individual profile sections, is presented in Appendix G.

4.2 Overview of the WOT methodology

4.2.1 WOT Calculation points

The first task of the WOT assessment is to determine the location of the points at which WOT calculations are to be undertaken. In doing so a balance needs to be found between (a) ensuring a sufficient number of points that capture the varying geometry and section types of the study area, and (b) avoiding having an excessive number of points which would entail very detailed calculations that ultimately would not add any extra value to the assessment.

A number of criteria were considered when determine the number and locations of the points:

- As per the project brief the maximum distance between two consecutive calculation points did not exceed 100m;
- It was ensured that any significant changes in the coastline and embankment geometry were accounted for when defining the points i.e., including a calculation point at any significant change in the coastline geometry;
- It was ensured that the CWWS data could be correctly applied to each of the calculation points (refer to Section 4.2.2).

A total of 89 calculation points across the study were selected for the assessment. The location of the points in the immediate vicinity of Salthill is presented in Figure 4.2.1. The location of all of the points is presented graphically in Appendix G.

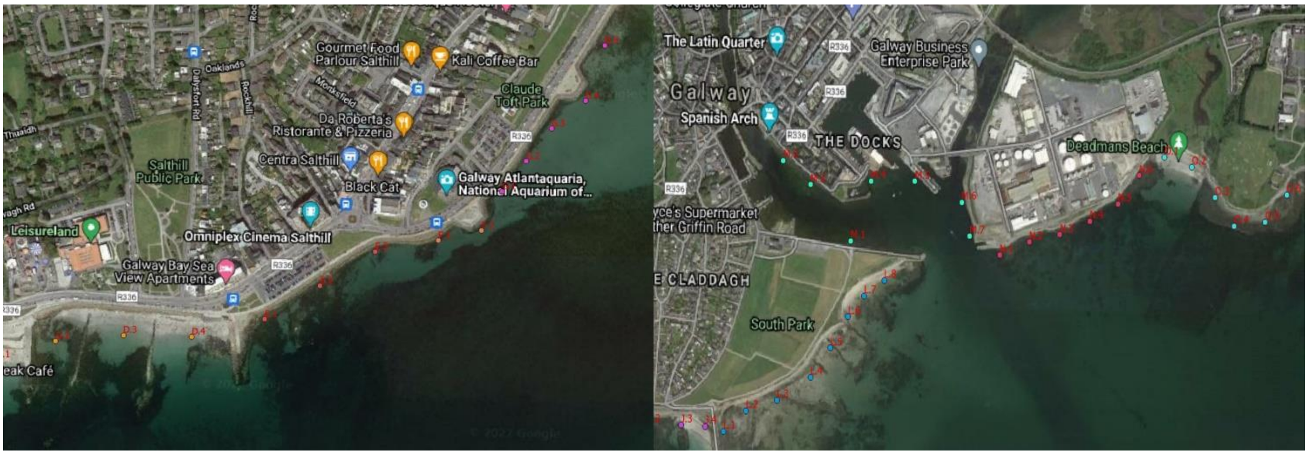


Figure 4.2.1 Overview of WOT calculation points in Salthill and the Docks

4.2.2 Datasets used to inform the analysis

Two different datasets were required to inform the WOT assessment:

- data on wave and tidal water level conditions;
- geometrical data of the coast and embankment profile at each cross section.

Both of these datasets are now discussed.

Wave and water level conditions

The Galway City Coastal Wave and Water Level (CWWS) was undertaken by the GCC/OPW and RPS in 2020 and forms part of Phase 3 of the Irish Coastal Wave and Water Level Modelling Study (ICWWS). The CWWS provides combinations of wave climate data[7] and water level (astronomical tide plus surge) data for six joint probability events for eight separate AEP events. The data is provided at circa 39 locations within the study area. Five separate climate epochs are considered: current, MRFS, HEFS, H+EFS, and H++EFS, which represent a 0.5m, 1.0m, 1.5m and 2.0m increase in sea level, respectively.

The findings of the CWWS have been used as part of this study to provide wave condition and still water level data at each of the 89 WOT calculation points. The assigning of the CWWS data to the calculation points was based on geographical proximity i.e., the data from the CWWS point closest to the calculation point was assigned as the wave/water level data for the point. At some locations however this approach was not suitable due to wave refraction processes around structures and ports (i.e., adjacent to the breakwater which connects Mutton Island to the mainland). In such locations the assigning of CWWS data was refined in order to ensure appropriate values were used at the calculation points.

[7] Four parameters are provided: Extreme Water Level, Spectral significant wave height (Hm0) Mean wave period (Tp) Mean wave direction (°)

Table 6 presents the wind generated wave data for a point adjacent to the Salthill promenade. It can be seen from the data that there is a considerable difference between the wave heights and still water levels for each of the six joint probability events – the SWL for JP6 is almost a meter higher than the SWL for JP1 while the wave height for JP6 is almost a meter lower. It is noted that the SWL for JP6 is almost identical to the 0.5% AEP still water level estimated as part of the hydrological estimation study. The larger waves have a longer period associated with them (i.e., Tp for JP1 is circa 1.7 seconds longer than the Tp of JP6) while the wave direction is very similar for all cases.

Table 6 0.5% AEP CWWS data for point G1 (Salthill)

Joint Probability	SWL ODM15	Hm0 (Wave Hgt in m)	Tp (Wave Period in s)	MWD (Wave dir in deg)
1	3.07	1.94	7.14	196
2	3.29	1.81	6.91	197
3	3.55	1.53	6.31	198
4	3.80	1.21	5.97	199
5	3.90	1.09	5.59	200
6	4.01	0.96	5.46	201

Geometric dataset

The geometrical data for the cross section at each of the 89 calculation points was derived by Arup by extracting information from in-situ geometric surveys of the area that were commissioned as part of the project (refer to Section 2). Where survey data was unavailable the relevant data was extracted from Lidar and Bathymetry datasets. The primary geometric parameters extracted for input are listed as:

- the bed level at the toe of the structure;
- the crest level of the embankment;
- the slope of the embankment.

Details on the primary geometric parameters at each of the 89 calculation points are presented in Appendix G. Cross section plots for each of the profiles are also presented in Appendix G.

4.2.3 Calculation methods

Three separate WOT methods were considered as part of the analysis:

- Eurotop Artificial Neural Network (ANN) method (Second Edition, 2018);
- Eurotop Empirical Equations (EE) method (Second Edition, 2018);
- Bayonet GPE Overtopping Method.

Each of these methods are now discussed.

4.2.4 Eurotop ANN

The Eurotop ANN uses a Neural Network approach to predict wave overtopping discharges at a particular cross section. The ANN reads in the input parameters (i.e., Wave conditions, water level and cross section geometry) at a particular section and then utilises the very extensive CLASH database[8] in order to predict the overtopping rates at the section.

[8] The CLASH database contains the results of more than 13,000 WOT test results from both experiments undertaken in hydraulic laboratories as well as recorded data from actual WOT events in the field.

It does this by searching the database for similar set ups to the input data (i.e., it finds similar geometries and wave/water level conditions) and uses the results associated with the similar set ups to predict the OT rates at the particular section.

4.2.5 Eurotop EE

The Eurotop EE method provides a set of equations that allow for wave overtopping discharges to be calculated at a particular cross section. Two different sets of equations can be utilised depending on the type of geometry of the cross section:

- Embankment equations;
- Vertical Wall equations;

The EE method allows for both mean and ‘design’ overtopping rates to be estimated where the ‘design’ rates are equivalent to the mean value plus 1SD.

It is noted that the EE method requires a lesser number of input parameters than the ANN method.

4.2.6 Bayonet

The Bayonet GPE Overtopping Method is an online overtopping tool developed by HR Wallingford in the UK. The method adopts a ‘Gaussian Process Emulator’ (GPE) statistical technique in order to predict overtopping at a particular section. The method also uses the CLASH database in order to derive OT rates at a particular cross section.

As with the other two methods noted above, data on the wave and water levels as well as cross section type and geometry are required as input to the Bayonet method.

4.2.7 Comparison of the three WOT methods

As part of the study Arup made a comparison between the WOT discharges estimated by each of the three methods noted above. The assessment was undertaken at 12 of the 89 calculation points for two separate Joint Probability scenarios (JP1 and JP6) for the 0.5% AEP current scenario event. The 12 points were taken from across the study area and were representative of the various types of coastline/embankments across the whole of the study area.[9]

It was evident from the analysis that at a number of the calculation points the ANN WOT discharges were significantly higher than the discharges derived using both the EE and the Bayonet methods. It can be seen from Figure 4.2.2 that the ANN discharges for JP1 at point O7 is an order of magnitude higher than either of the other two methods. For JP6 (Figure 4.2.3) the ANN discharges at point H3 are also far greater than the other two methods. The ANN is however more comparable to the EE for point O7 in this scenario.

[9] Ten out of twelve calculation points feature embankments. The remaining two points are vertical walls in the vicinity of the Spanish Arch.

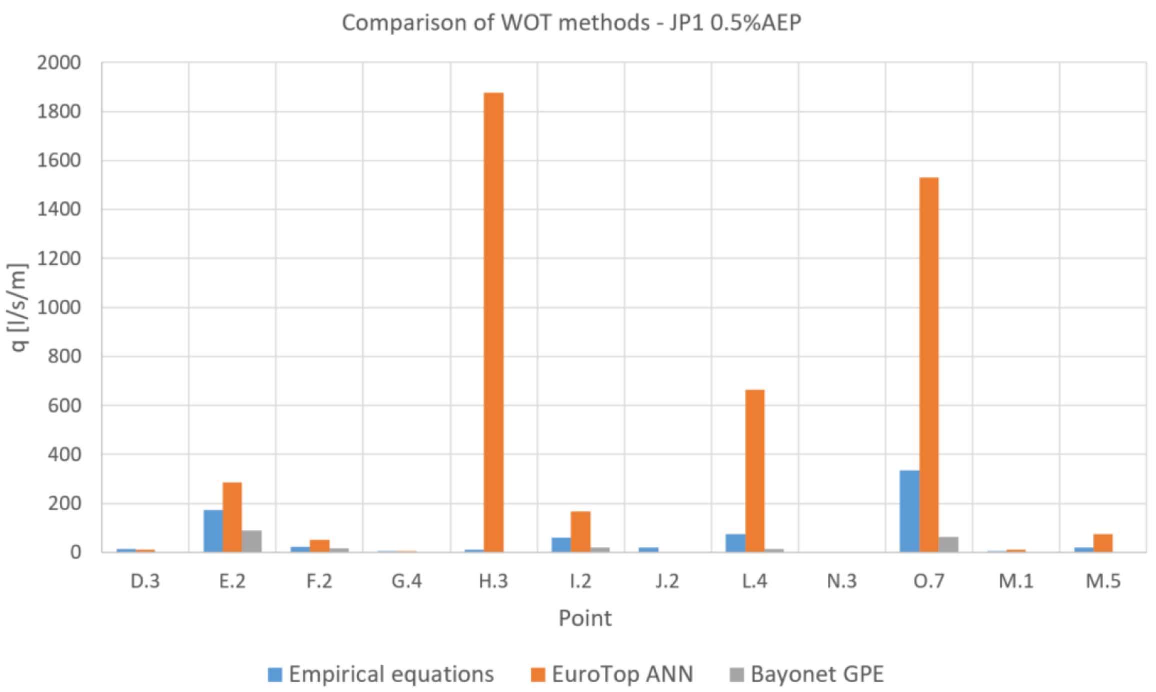


Figure 4.2.2 Comparison of the WOT discharges from three methods (JP1, 0.5% AEP)

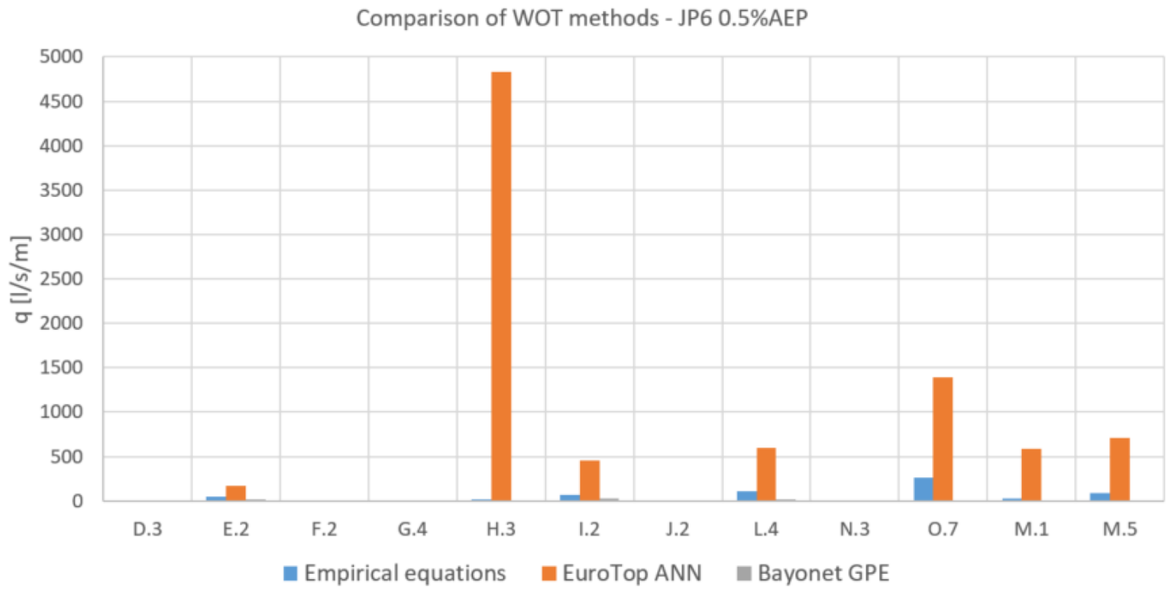


Figure 4.2.3 Comparison of the WOT discharges from three methods (JP6, 0.5% AEP)

The ANN results at calculation points O7 and H3 were deemed to be unrealistic and not in keeping with the historic record of flooding at the site.[10] Following discussion with the OPW and GCC the ANN method was therefore discarded from the assessment and not considered as part of the design runs for the study.

It can be seen from Figure 4.2.2 and Figure 4.2.3 that the EE WOT discharges are comparable but consistently higher than the WOT discharges as estimated by the Bayonet method. Following further discussions with the OPW and GCC it was agreed to adopt the EE method for all locations and return period events.

[10] This was assessed by simulating the coastal model with the ANN output set as the WOT boundary conditions of the model and then comparing the modelled extents with the observed WOT historic extents.

Its relative comparability with the Bayonet method gives us a degree of confidence in its suitability for use as part of the study and its conservatism over the Bayonet method also minimises the risk of the WOT flows being underestimated across the site.

Significant uncertainties over the estimated WOT discharges however remain.

4.2.8 Running the Empirical Equations

Both sets of empirical equations were used across the site - the embankment equations were used at 81 out of 89 calculation points while the vertical walls equations were used at the remaining 8 calculation points. Both the 'mean value approach' and 'design or assessment approach' formulas were adopted at each calculation point.[11] The design values were subsequently used for the design model runs as they were more conservative.[12]

The calculation process was fully automated using Python scripts in order to ensure that the very significant number of calculations were undertaken effectively and efficiently. This work involved automating the following tasks:

- reading in the CWWS data and assigning it the correct calculation points;
- reading in the correct geometric and parameter data for each of the sections;
- undertaking initial calculations which were required as input to the calculations (i.e., estimating the water depth at each section by subtracting the bed level from the design water level);
- Undertaking the EE calculations to derive the peak flows;
- Undertaking the EE calculations for other points on the WOT hydrograph (discussed later in this section);
- Creating output in a neat and easy to read format.

The following table presents the complete list of input parameters to the EE and their data source.

Table 7 List of EE input variables

Input	Units	Definition of the parameter	Source [clustered or not]
Name	[-]	Label/ID of the test	Assigned sequentially from East to West
B_l	[m]	Bed level	Calculated from survey where available and from LIDAR/bathymetric data
C_l	[m]	Crest level	Calculated from survey data
SWL	[m]	Level of still water during the event	CWWS dataset
$H_{m0,t}$	[m]	Significant wave height at the toe of the structure	CWWS dataset
$T_{m-1,0,t}$	[s]	Spectral wave period at the toe of the structure	CWWS dataset
β	[°]	Wave angle	CWWS dataset (wave direction) and angle of alignment of the coast –

[11] The design or assessment approach includes a partial safety factor in the empirical equation which is in effect accounting for the uncertainty in the predication. From inspection both the mean value and design values were comparable to each other. The design values were utilised to ensure an element of conservatism in our approach.

[12] The design values were on average circa 5 – 15% higher than the mean values

Input	Units	Definition of the parameter	Source [clustered or not]
$\cot\alpha_d$	[-]	Slope of structure face	Calculated from survey data and/or LIDAR
γ_f	[-]	Roughness factor	Appropriate values taken from the literature

4.2.9 Accounting for negative freeboard/tidal inundation

Figure 4.2.4 presents the crest height of the embankment/promenade/beach for each of the 89 WOT calculation points across the study area. The points plotted on the x axis from left to right correspond to the alignment of the calculation points from East to West across the study area. As the Mutton Island points are not on the mainland they have been included on the far right of the plot. The location of Salthill, Galway Port and Mutton Island are indicated with shading on the plot in order to provide a geographical orientation for the reader.

The 0.5% AEP SWLs for the six different JP events as derived by the CWWS are also plotted on the figure. It can be seen from the plot that the SWL exceeds the crest level of the embankment at a number of locations for each of the six JP events. As the SWL for JP6 is almost 1m higher than the SWL for JP1, the crest level is exceeded at a greater number of points and for longer lengths for JP6 than for JP1.

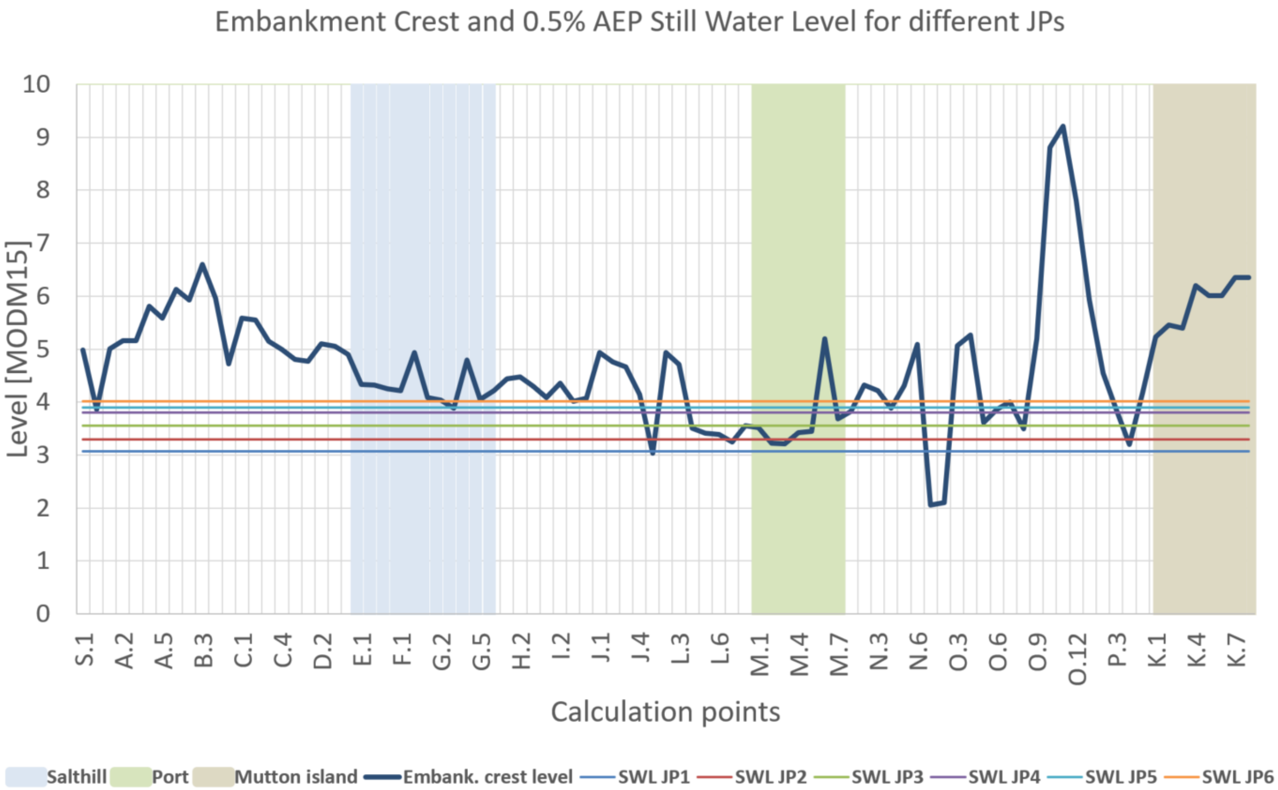


Figure 4.2.4 Embankment levels and 0.5% AEP still water levels

When the SWL exceeds the crest level of the embankment direct tidal inundation over the embankment will occur in parallel with the WOT. In this case the total discharge over the top of the embankment (Total Q) is the sum of the discharge from the wave overtopping (WOT Q) and direct tidal inundation (Tidal Q). This scenario is referred to as ‘negative freeboard’ in the Eurotop manual given that the distance from the SWL to the embankment crest level is effectively negative.

Equation 5.29 of the Eurotop manual provides a weir equation by which the Tidal Q (i.e., the direct tidal inundation) can be directly calculated at a section.

There are two ways in which the WOT Q and the Tidal Q can be considered as part of a hydraulic modelling study:

- The WOT Q and the Tidal Q are estimated separately using the EE method and equation 5.29 of the Eurotop manual, respectively. The results are then inserted as separate source discharge points in the hydraulic model;
- The WOT Q is calculated using the EE method and included as source discharge points in the model. The Tidal Q is not however estimated using eq 5.29 of the Eurotop manual but is instead explicitly calculated by the Tuflow model by specifying the tidal levels as an open sea time varying water level profile - when the water level of the open sea boundary exceeds the crest level of the embankment/promenade, the model calculates the direct tidal inundation (i.e., the Tidal Q) at each of the relevant grid cells of the model.

The second approach has been adopted in the study as it represents a more accurate approach to modelling the tidal inundation given that the tidal inflow is calculated at each individual grid cell of the model and therefore allows for the varying geometry in between the calculation points to be accounted for. In the first approach the direct tidal inundation is only calculated at each of the 89 calculation points.

The WOT Q component become negligible when the overflow depth is approximately one third of the incident wave height. In this case the WOT Q is effectively reduced to zero and the direct tidal inundation is the dominant mechanism of flooding at the cross section.

4.3 WOT Results

4.3.1 WOT discharges – 0.5% AEP event

The results in this section of the report should be considered in parallel with the detailed set of results presented in tabular format in Appendix G.

Figure 4.3.1 presents the QWOT discharges across the study area for the 0.5% AEP event for each of the six JP's[13]. It can be seen from the plot that the overtopping rates vary across the site for each of the six JP events. The discharges range from 0m^3/s/m (at numerous points) to circa 0.55m^3/s/m (point O7).

The differences between the six different JP events at individual points also varies – at some of the locations there is little differences (i.e., at E2 and E3 the rates vary from 0.22 to 0.3m^3/s/m for the six events) while there is a significant difference between the rates at other points (i.e., at I2, I3 and I4 the OT rates range from 0.14m^3/s/m to 0.47m^3/s/m). It is therefore evident that the WOT discharges are not equally sensitive to the input boundaries for different sections.

[13] Only 30 of the 89 calculation points are labelled on the x-axis of Figure 4.3.1 due to the space constraints of the figure. The reader is therefore referred to Appendix G which presents a detailed breakdown of the WOT results. The appendix should be considered in parallel with the results presented in this section of the report.

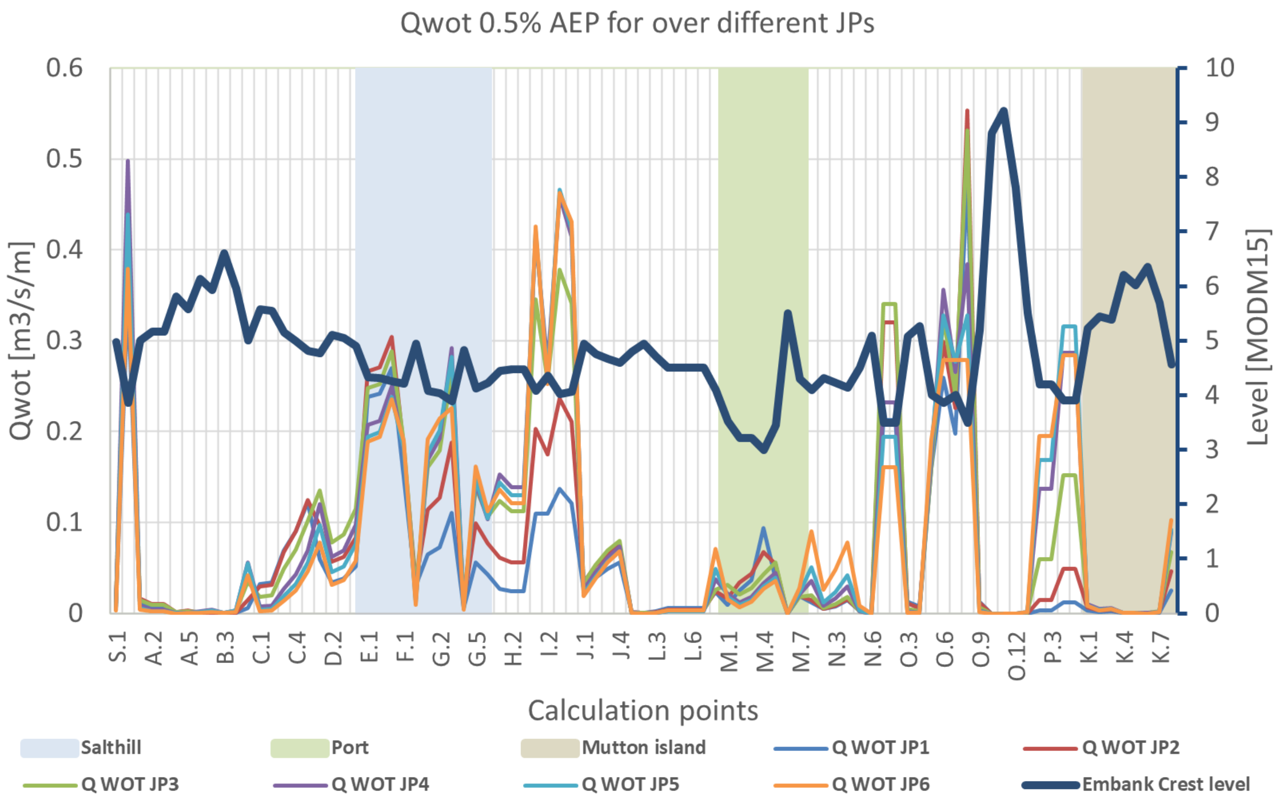


Figure 4.3.1 Wave-overtopping discharge over different JPs - embankment EE

As noted above the hydraulic modelling undertaken as part of the study considers the WOT and Tidal inundation as separate boundary conditions and the direct tidal inundation will therefore be explicitly calculated by the Tuflow Quadtree model. It is however very useful to calculate the Q Tidal as per equation 5.29 of the Eurotop manual and plot it with the Q WOT in order to assess the relative contribution that both of these sources of flood risk make to the Total Q.

Figure 4.3.2 presents the Q WOT and Q Total for the 0.5% AEP event for JP1. The Q Tidal can be inferred from the difference between the two flow rates. Figure 4.3.3 presents the equivalent plot for the 0.5% AEP JP6. It can be seen from both figures that as expected, the Q WOT equates to the Total Q for all the points where the SWL falls below the crest level of the embankment. It can also be seen from comparing both plots that the Q WOT is comparable for both of the JP events i.e., the Q WOT for JP1 is similar to the Q WOT for JP6.

The most significant difference in the Total Q between the JP events therefore relate to the Q Tidal component. As the 0.5% AEP SWL for JP6 is circa 1m higher than the 0.5% AEP SWL for JP1, the Q Tidal for JP6 is significantly higher than the Q tidal for JP1. While there are only two points where the crest level is exceeded in JP1 (O1 and O2) there are multiple locations where the crest level is exceeded in JP6 due to the much higher SWL of 4.0mOD associated with this JP event. The total Q for JP6 is therefore significantly larger than for JP1 as a consequence of the much greater tidal water level.

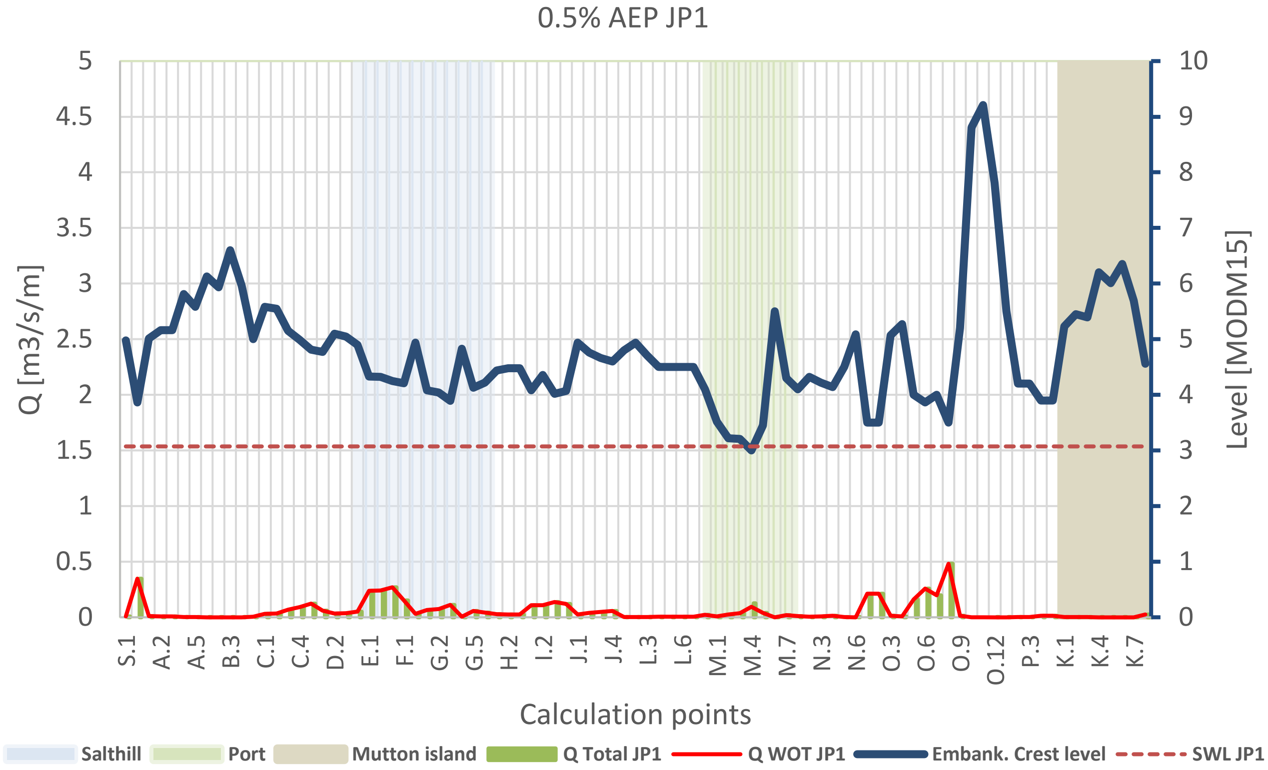


Figure 4.3.2 0.5% AEP event – JP1

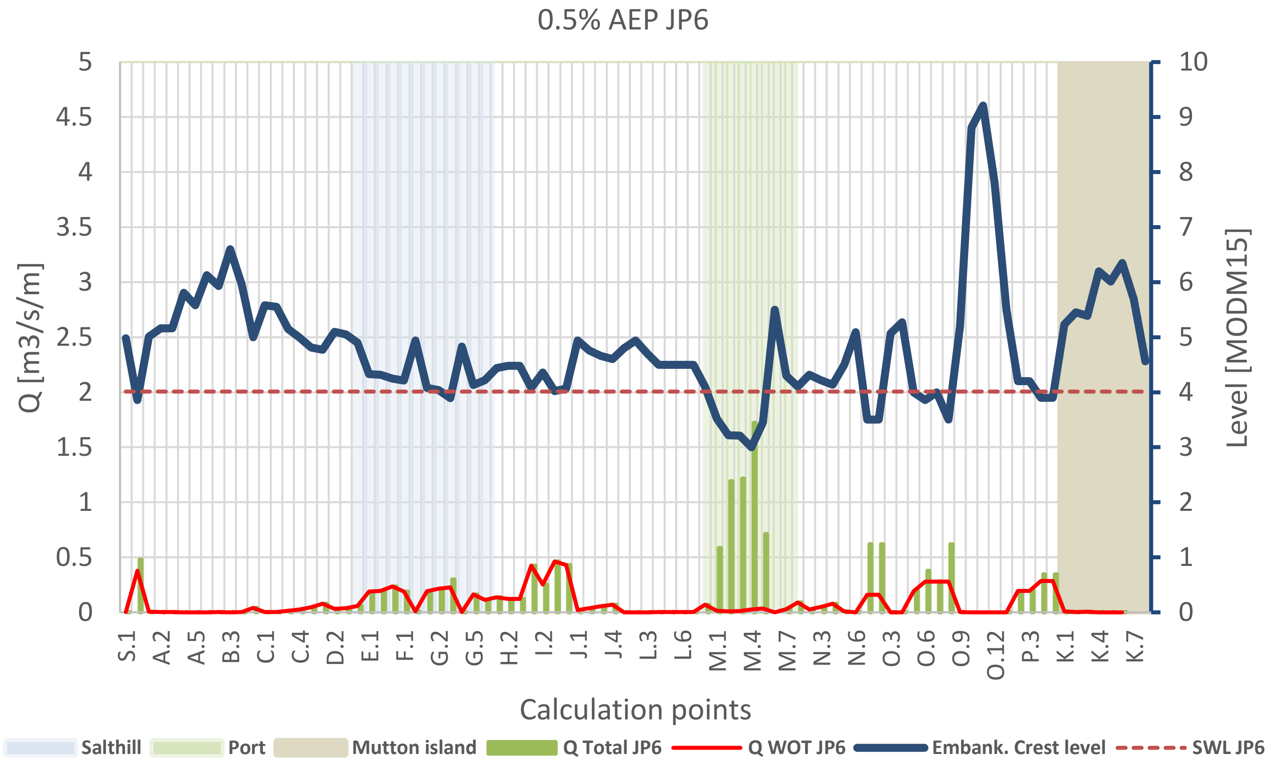


Figure 4.3.3 0.5% AEP event – JP6

4.4 WOT Hydrograph and Tidal Curve development

Once the peak WOT Q has been derived using the EE method, a WOT hydrograph for each event needs to be derived. Arup has followed the methodology outlined in the EA's Coastal Flood Boundary 2018 report in order to generate the WOT hydrographs. The steps in the process are given as:

- The maximum wave overtopping discharge is first calculated. This value equates to the peak WOT Q and correlates with the peak of the SWL tidal curve (these were the values presented in the previous section of the report);
- The EE equations are re-run for lower SWLs i.e., for points on both the rising and falling limbs of the tidal curve. It is assumed that the wave height is not reduced for these lower SWLs such that the wave height used to inform the WOT at the peak of the tide is also used to inform the WOT for the lower water levels. This is a conservative approach as the wave height is likely to be reduced for lower SWLs.

The tidal water level profiles estimated by the hydrological study undertaken as part of the project have been used to define the rising and falling limbs of the tide. These curves are plotted in Figure 4.4.1. As the peak water levels estimated by the hydrological study differ to the CWWS peak water levels[14] the tidal curves were scaled in order to match the SWLs of the CWWS data.

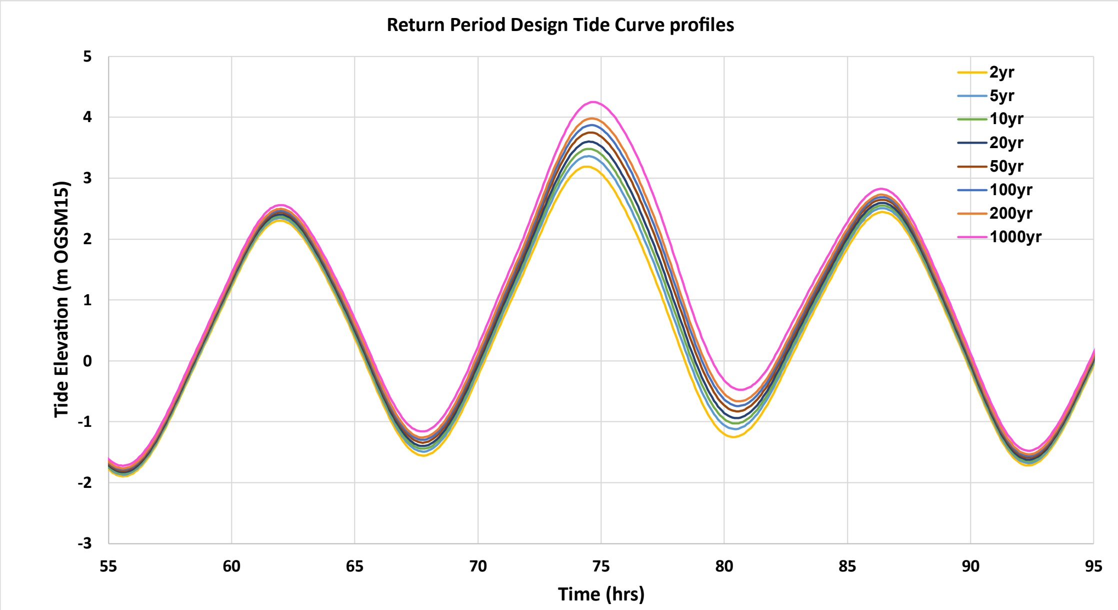


Figure 4.4.1 Current scenario tidal curves for the eight AEP events as estimated by the hydrological study

The point at which WOT stops was set relative to the water depth at each of the calculation points - when the wave height is greater than the water depth * 0.78, the WOT was assumed to be zero. Figure 4.4.2 presents the WOT hydrograph for point E3 (adjacent to Salthill promenade) for the 0.5% AEP current scenario event for JP6.

[14] The SWL of the CWWS's JP6 is approximately the same as the SWL as estimated by the hydrological study. The SWL for the other 5 JP events are however all lower than the levels estimated by the hydrological study.

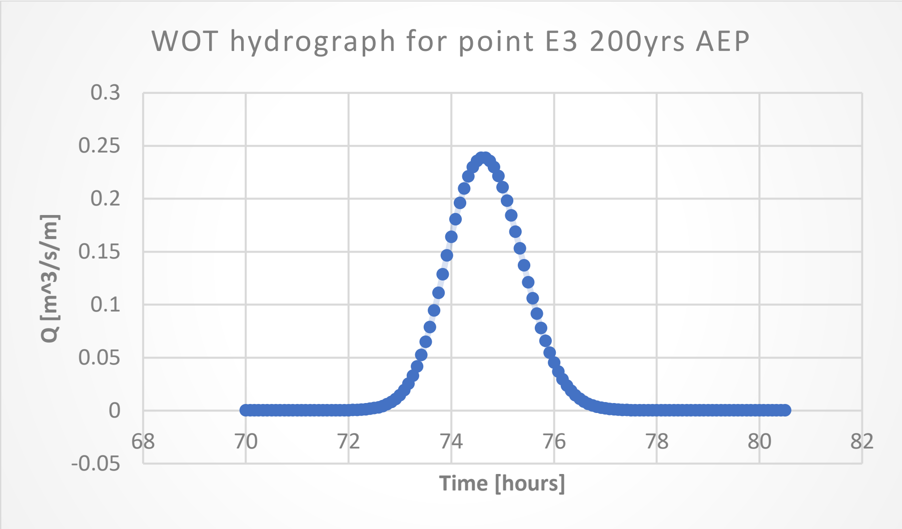


Figure 4.4.2 Example WOT hydrograph input for point E3

4.5 Coastal Model development

A detailed 2D Tuflow Quadtree model of the coastal floodplain has been developed as part of the study in order to simulate the WOT discharges and tidal inundation across the full study area. An overview of the development of the model is presented in Section 5 of the report.

4.6 Model results

The results of the WOT hydraulic modelling are presented later in the report in Section 9.

5. Model development

5.1 Introduction

A one-dimensional (1D) and two-dimensional (2D) fluvial/tidal model of the River Corrib and its main tributaries has been constructed as part of the study to simulate fluvial and tidal flood risk along the River Corrib in the scheme area. The 1D model simulates the in-bank flows within all the watercourses and has been constructed in Flood Modeller Pro (v4.6) software. The 2D model simulates the out of bank floodplain flows and it has been developed in Tuflow software (v2018). Both the 1D and 2D models are dynamically linked and run together as a coupled hydraulic model – once the water level in the 1D model exceeds the bank level, it spills into the 2D grid and acts as a source discharge along the 1D/2D model interface. This model is referred to as the Fluvial/tidal hydraulic model.

A separate 2D only model has also been constructed to simulate the tidal surge and wave overtopping conditions in the scheme area. The model reaches from the Seashore Caravan Park at the western end of the scheme area to Curragreen on the eastern end. This model is built entirely in Tuflow software (v2018). This model is referred to as the Coastal model in this report. Both of the models together are referred to as the Galway City FRS hydraulic models.

It is noted that the Sruffnacashlaun stream/culvert was included in the original project brief. Following a detailed review of the flood extents from the stream/culvert however, it was agreed by the Steering Group to exclude the catchment from the Flood Relief scheme as the Flood risk within this particular small and heavily urbanised catchments is typically a function of the drainage system and therefore falls outside of the scope of this flood relief scheme.

The subchapters below describe the development of the Fluvial/tidal hydraulic model. The final section describes the development of the Coastal hydraulic model.

5.2 Fluvial/Tidal Model Development

A coupled 1D/2D hydraulic model of the River Corrib and some of its tributaries was developed as part of the Western CFRAM Study. The fluvial/tidal hydraulic model was developed from the Western CFRAM hydraulic model and represents a more refined and detailed version of it.

The purpose of refining the model was to ensure that the level of detail and accuracy of the model is appropriate to the needs of flood relief scheme project. It should be noted that the Western CFRAM model was first transformed from ING to ITM geospatial system and from OD Malin OSGM02 to OD Malin OSGM15 vertical datum.

The refinements and updates of the fluvial/tidal hydraulic model can be summarised as:

- Infill Survey – As detailed in Section 2.3, Arup identified a number of areas where additional river survey data would improve the performance and accuracy of the fluvial/tidal hydraulic model over the Western CFRAM model. All of these areas were subsequently surveyed as part of the Infill Survey Management and incorporated into the model, with the most significant area being the Distillery channel, which was entirely replaced in this model;
- Inclusion of additional tributaries – Some additional tributaries/mill races in the city centre were included in the revised model;
- Modification of 1D/2D interface – The 1D/2D interface has been modified across the scheme area in order to (1) accommodate tributaries that were not considered as part of the Western CFRAM, and (2) to provide a more accurate representation of the spilling from the river to the floodplain in a number of key urban areas;
- Updated structures – The Salmon Weir crest levels has been updated to reflect the recent river channel surveys. The Salmon Weir Bridge openings have been updated to reflect the pier widths more accurately and one of the bridge arches has been removed to reflect on ineffective flow areas – see Section 6.5. A number of weirs spilling from mill races to the River Corrib were updated using information from the infill surveys and culverts were updated to incorporate additional details provided by the infills;
- Model Parameters – A number of the model parameters used in the Western CFRAM model were altered in the fluvial/tidal hydraulic model. These include channel roughness and structure coefficients which are described in Section 5.3.3 of this report.
- Defences –Effective flood defences in the city such as the Dyke road embankment have been included in the model. Ineffective flood defences such us the defences along the Leonardo Hotel have been represented as per their surveyed geometry. The openings in these walls have been defined where applicable.

5.3 Model Extents

5.3.1 Fluvial/tidal model

The watercourses included as part of the fluvial/tidal hydraulic model are listed in Table 8. The alignment of the watercourses is presented in Figure 1.4.2. The upper extents of the River Corrib from Lough Corrib to Dangan are represented in 1D only and the sections for this reach extend across the entire floodplain.

Table 8 Watercourses modelled for the existing scenario

Watercourse name	Modelled name	Upstream extent (ITM)		Downstream extent (ITM)		EPA River ID
		Easting	Northing	Easting	Northing	
Corrib	CORR, FCUT, CLOP	525358	730743	529939	724677	CORRIB_020
Terryland	CAST	529341	726571	531821	727546	TERRYLAND_010

Watercourse name	Modelled name	Upstream extent (ITM)		Downstream extent (ITM)		EPA River ID
		Easting	Northing	Easting	Northing	
Distillery Channel	SALW	529170	726249	529554	725327	n/a
Eglinton canal	EGLI, SHEA	529554	725327	529570	724941	n/a
Claddagh basin	EGLI	529527	724909	529662	724795	n/a
Gaol river	NUNS	529355	725685	529533	725195	n/a
St Clare river	GMRA	529231	725455	529538	725124	n/a
Parkavara river	MACT, SMRN	529377	725272	529400	725012	n/a
Madeira river	VARA	529426	725145	529400	725012	n/a
Dominic canal	DOMI	529442	725125	529411	724995	n/a
Middle river/ Friar's river	SALR	529651	725705	529590	725063	n/a
Slaughterhouse river/Friar's river	FRIA	529691	725402	529574	725166	n/a

The 2D domain of the fluvial/tidal model has been split into the urban and rural domains in order to allow different cell sizes to be applied at each domain. A fixed grid cell size of 4m was used in the urban domain while a size of 8m was used in the rural domain. This set up allows for higher model definition in the key area while keeping the model run time reasonable. The urban and rural 2D domains, 1D nodes and Scheme extents are shown in Figure 5.3.1.

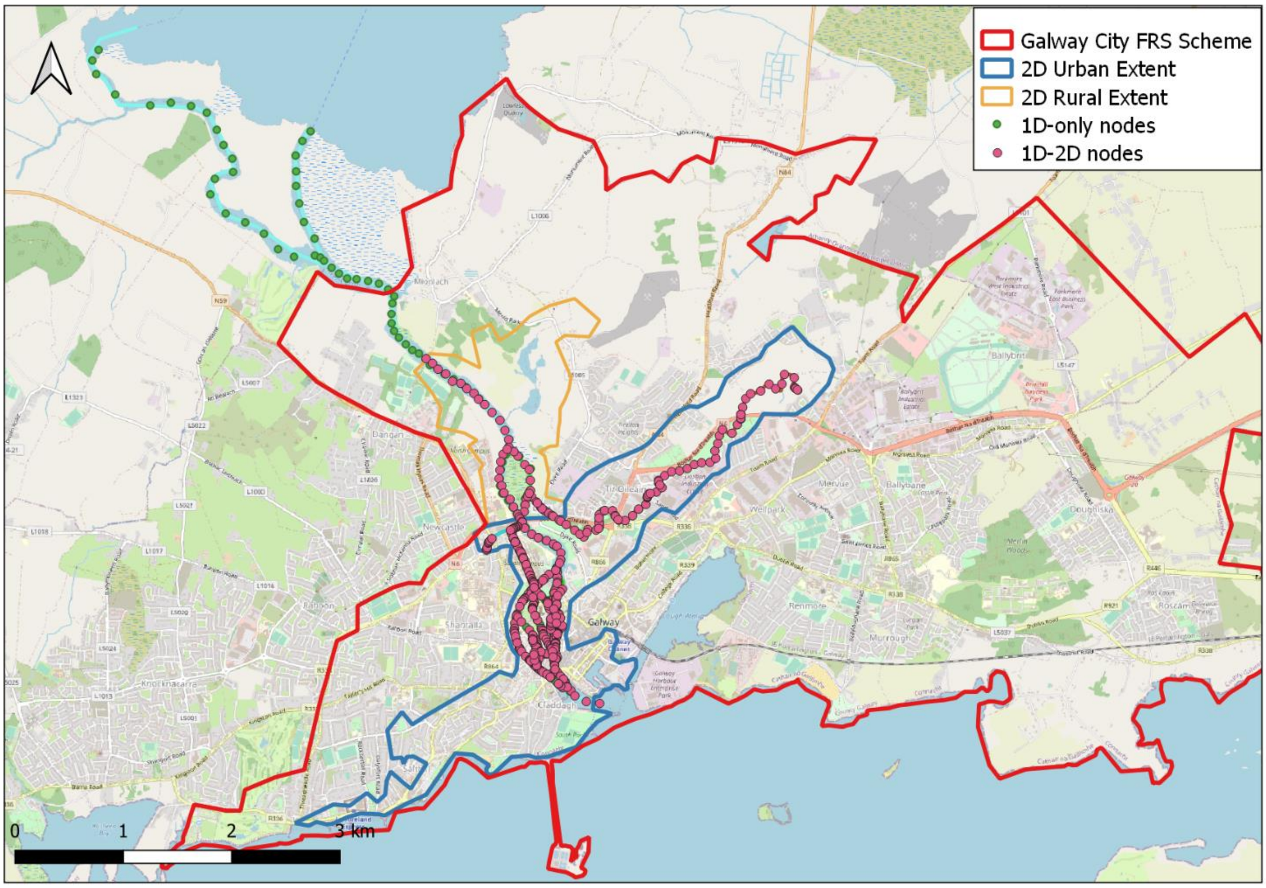


Figure 5.3.1 Fluvial/tidal model - 2D model domains (rural and urban) and 1D model cross sections

5.3.2 Model boundaries

There are no major tributaries discharging into the Corrib within the scheme area downstream of Lough Corrib. The catchment area of the scheme model is circa 7km^2 which represents 0.6% of the total catchment area of the Corrib. As such, lateral inflows have not been used within the model build and the total catchment flow as calculated at Galway Bay has been applied at Lough Corrib. This builds an element of conservatism into the design flows at the upstream end of the model.

The model has a number of inflow boundaries:

- an inflow applied at Lough Corrib to represent flows within the Corrib at Galway City;
- an inflow applied to represent runoff from the Sruffnacashlaun stream;
- an inflow applied upstream of the Distillery channel to represent urban flows from the Dangan catchment;
- an inflow applied at Terryland representing the urban flows from the Terryland catchment.

A number of other low ‘dummy’ flows have been applied at some the mill races and canals in order to prevent drying out of channels which was a source of significant model instabilities during the model build phase.

There are two other boundary conditions applied to the model:

- a stage-time boundary has been applied at the downstream end of the model to represent tidal conditions in Galway Bay (see Figure 5.3.2);
- a flow-stage boundary has also been applied ay the downstream end of the Terryland stream in order to represent the influence of groundwater and tide locking that occurs at the sinkholes as described below.

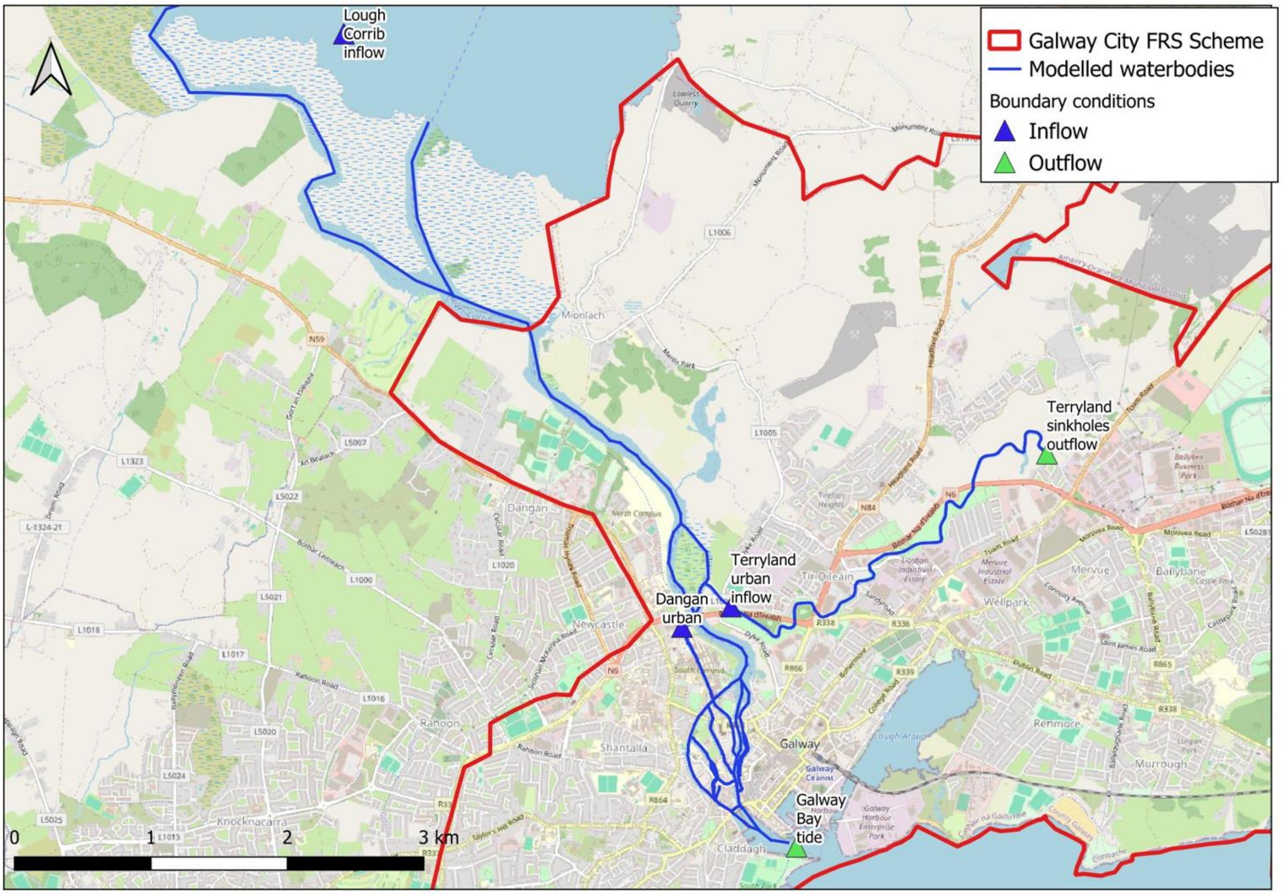


Figure 5.3.2 Fluvial/tidal model – model boundaries

5.3.3 Terryland sinkholes

The Terryland drainage system is a karst catchment. Two sinkholes at the eastern part of the catchment act as outflows for the Terryland stream. Following installation of a new gauge within the Terryland River by OPW (Terryland, 30117), a clear tidal pulse has been observed within the river, as shown in Figure 5.3.3. It is therefore highly likely that the sinkholes and karst catchment allow for connectivity with the tide.

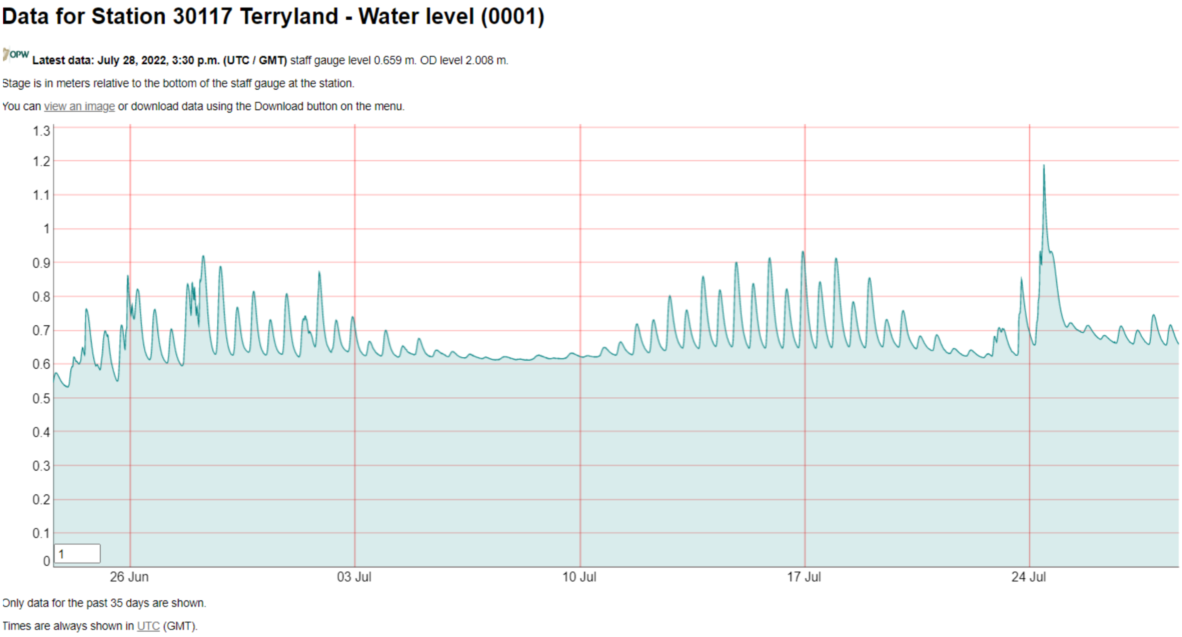


Figure 5.3.3 Terryland gauging station signal showing tidal influence (waterlevel.ie)

The tidal signal and channel stage - storage relationship of the Terryland stream were used to create a flow-stage relationship for the sinkholes which as noted above was applied as the downstream boundary of the stream in the model.

It was found from our assessment that the baseflow in the catchment correlates with a level of circa 1.72m AOD at the sink hole. All discharge out of the sinkholes below that level were therefore set to 0. The flow increases linearly between 1.72m AOD to 2.4m AOD from 0 to 1.5m^3/s at which point it is assumed to level off.

5.4 Model Parameters

5.4.1 Labelling System

The model nodes derived from the infill and validation survey followed the same labelling format as used for the Western CFRAM survey (e.g., The River Corrib labels are provided in the form of 30CORR0000, with chainage starting from 0 at Galway Bay).

For the fluvial/tidal model though a different approach was followed. When a new cross section was introduced from the 2021 Infill surveys, the prefix S was added to the cross section (e.g., for the new sections along the Distillery channel the labels are S30SALW00000).

This approach was adopted to ensure that a direct reference can be made to both the CFRAM and Infill surveys.

5.4.2 Model resolution

The 1D model resolution is determined by the distance between adjacent cross sections which changes throughout the model domain. For the key urban area, this distance is on average between 30-50m which is deemed sufficiently accurate to assess water levels for both the existing scenario and the optioneering. For upstream areas in the Corrib the spacing increases to 70-100m.

The 2D model resolution is defined by the spacing of the 2D grid. Defining this parameter involves a trade-off between accurately resolving the two-dimensional flow in an urban environment using a high-resolution grid and the computational run time of the model which is reduced with the lower resolution grids.

A 2D cell size of 4m in the urban part and 8m in the rural part has been selected. These are deemed to provide sufficient accuracy in the model for each part. It is noted that a smaller resolution (2m urban/4m rural) was also tested as part of the model sensitivity, and it was found that the modelled water levels are not sensitive to the finer grid (refer to Section 10).

5.4.3 Manning's n for the 1D and 2D Models

The roughness values of the 1D model have been defined for three separate sections of each cross section: left bank, main channel, and right bank. These sections of each cross section in the model are defined through the use of panel markers.

The spatially varying Manning's n roughness values of the 1D model were selected based on a detailed analysis of a number of datasets:

- The values previously used in the Western CFRAM study[15];
- Notes on survey drawings and photographs from the Western CFRAM surveys and Infill surveys;
- Site visits undertaken by Arup;
- Relevant literature[16] and

[15] Office of Public Works (2016), *Western CFRAM Study – Hydraulics Report Unit of Management 30 and 31 – Corrib and Owengowla,* JBA consultants, September 2015

[16] Chow, V.T., 1959, *Open-channel hydraulics*: New York, McGraw-Hill, 680 p.

- Model calibration.

An overview of the values used in the study are presented in Table 9.

Table 9 1D Manning's n values typically used in the study

Channel Characteristics	Characteristics	Manning’s n values
Main Channel bed	Mud or silt material	0.02 – 0.03
	Stone and mud material, sometimes with silt or concrete	0.035 – 0.04
Banks	Stone/concrete walls	0.025
	Grassy/reed banks or floodplains	0.04 – 0.045
	Vegetated floodplain with trees	0.05
	Vegetated banks (ivy, bush, shrubs, or trees)	0.06

The Manning’s coefficient values used in River Corrib are detailed in Table 10.

Table 10 1D Manning's n values used in the study – River Corrib

Reach	Roughness values (manning's n)	Photograph
River Corrib – CORR, FCUT, CLOP		
30FCUT00116 - 30FCUT00008	Bed – 0.03 (mud material) Banks – 0.04 (grassy floodplains)	 30FCUT00060_RB
30CORR00951 – 30CORR00442	Bed – 0.03 (mud material) Banks – 0.04 (grassy floodplains)	 30CORR00881_DS
30CORR00434 – 30CORR00241	Bed – 0.035 (stone, mud, and silt material) Bank – 0.05 (vegetated floodplain with some trees) Some banks set to 1 for stability	 30CORR000404_US
30CORR00236 - 30CORR00110A	Bed – 0.04 (mud and stones) Bank – 0.045 - 0.05 (vegetated floodplain with some trees) 30CORR00235 side banks set to 0.025 (for Quincentennial bridge concrete abutments) 30CORR00170A – 30CORR00158 side banks set to 0.025 (for stone walls)	 30CORR000124_LB

Reach	Roughness values (manning's n)	Photograph
30CORR00110B – 30CORR00000	Bed – 0.035 (stone and mud or silt material) Side banks – 0.025 (stone walls)	 30CORR00070_RB
30CLOP00064 – 30CLOP00001	Bed – 0.04 (stone mud material) Bank – 0.04 (grassy read banks and floodplain	 30CLOP00002_US

A long section of the River Corrib showing the bed slope, bed material recorded during the surveys, the Manning's n values assigned during the Western CFRAM and the Manning's n value assigned as part of the Galway City FRS is shown in Figure 5.4.1. It should be noted that the Manning's n coefficient for the bed of the River Corrib was adjusted as part of the calibration analysis described in Section 6.

- The bed manning's coefficient between Dangan Gauge and Quincentennial Bridge was decreased from 0.04 to 0.035 to represent the mud and silty bed upstream Quincentennial Bridge. This enabled a close calibration at Dangan gauge during the calibration events (it is noted the bed material is rougher near Dangan – rocks/stones, however this was adjusted to a smaller coefficient to achieve the purposes of the calibration).
- The bed Manning's coefficient was increased from 0.035 to 0.04 between the Salmon Weir and node 30CORR00110A to represent the rougher bed material (mud & stones) and allow an improved calibration at Barrage D/S gauge.

Details of the manning's coefficient for the other watercourses can be seen in Table 11.

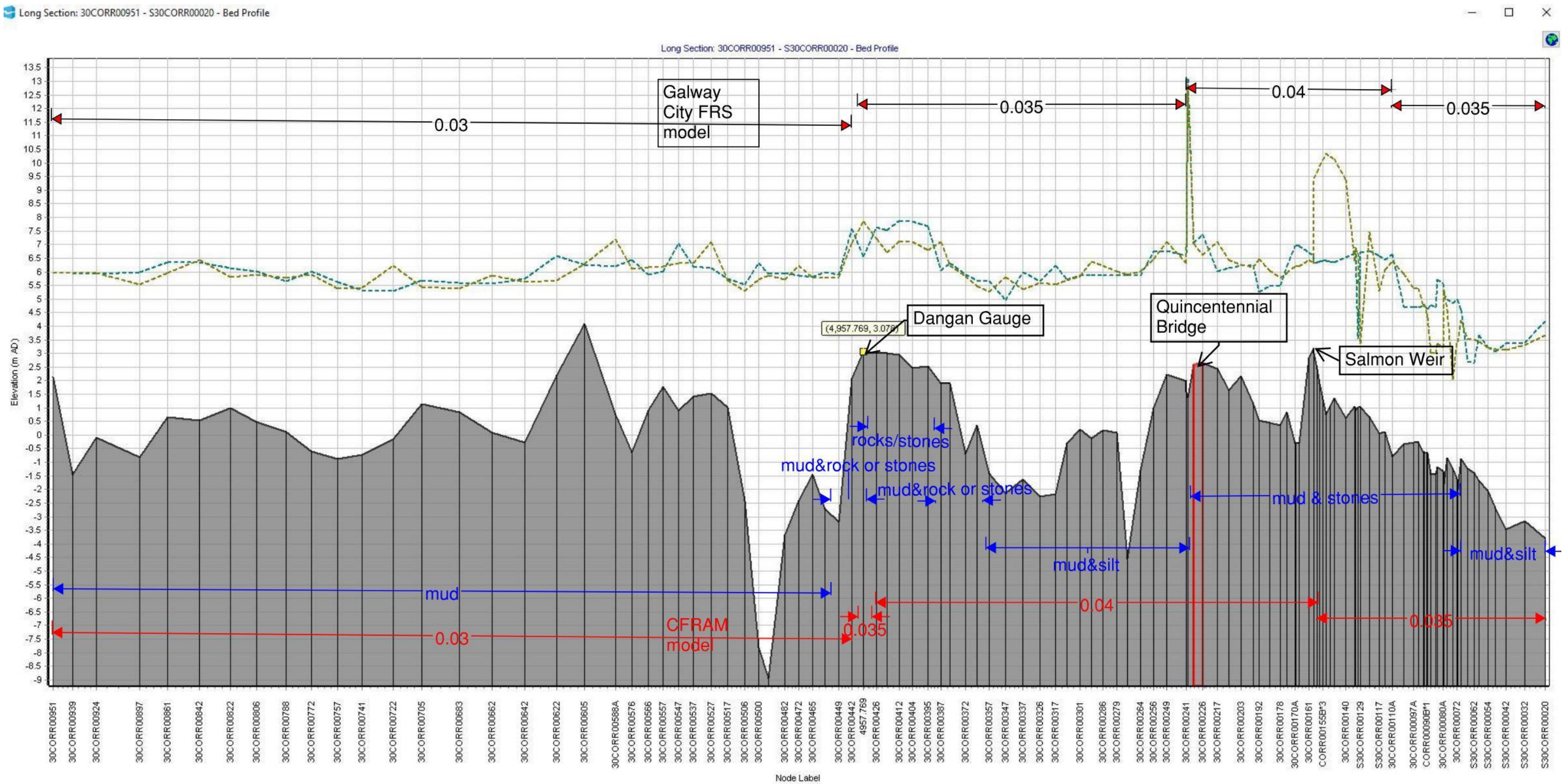
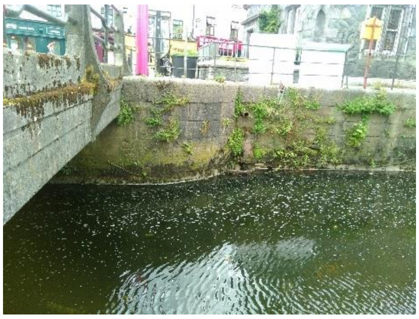


Figure 5.4.1 Long section showing Manning's n values along River Corrib's bed (dashed lines represent top of left and right banks)

Table 11 1D Manning's n values used in the study – Other tributaries and watercourses

Reach	Roughness values (manning's n)	Photograph
Eglinton Canal – EGLI, SHEA		
30EGLI00135 - 30EGLI00103A	Bed – 0.03 (silt material) Side banks – 0.025 (stone walls)	 30EGLI00116_RB
30EGLI000102B - 30EGLI00000	Bed – 0.035 (silt stone) Side banks – 0.025 (stone walls)	 30EGLI_00026D_LB
30SHEA00004 - 30SHEA00000	Bed – 0.04 (silt stone) Banks – 0.06 (heavily vegetated banks)	 30SHEA00016_DN
Gaol River - NUNS		
30NUNS00052 - 30NUNS00000	Bed – 0.035 (stone mud) Side banks – 0.025 (stone walls) Banks – 0.06 (heavily vegetated banks)	 30NUNS00017_UP

Reach	Roughness values (manning's n)	Photograph
St Clare River - GMRA		
30GMRA00048 - 30GMRA0000N	Bed – 0.035 (stone silt material) Side banks – 0.025 or 0.065 (stone walls or bushed, shrubs and reeds) Banks – 0.06 (heavily vegetated banks)	 30GMRA00010_UP
Dominic canal - DOMI		
30DOMI00009D – 30DOMI00001	Bed – 0.04 (stone material) Side banks – 0.025 (stone walls) Banks – 0.025or 0.06 (stone walls or heavily vegetated banks)	 30DOMI00008_DN
Terryland river - CAST		
30CAST000001	Bed – 0.03 (mud material) Banks – 0.04 (grassy banks)	 30CAST00001_RB
30CAST00008A – 30CAST00018B	Bed – 0.03 (mud material) Banks – 0.025 (stone walls for Terryland works)	 30CAST00008O_US

Reach	Roughness values (manning's n)	Photograph
30CAST00019 - 30CAST000398	Bed – 0.03 Banks – 0.04 or 0.06 (grassy/reed banks floodplain)	 30CAST00057_UP
Distillery Channel - SALW		
S30SALW00111 – S30SALW00000	Bed – 0.04 (stone bed) Side banks – 0.025 (stone walls) – included for some Banks – 0.06 (vegetated banks)	 30SALW00053_DN
Middle River - SALR		
30SALR00071 – 30SALR00000	Bed – 0.035 (stone silt and concrete) Side banks – 0.025 (stone walls)	 30SALR00022W_DN
Slaughterhouse river/Friar's - FRIA		
30FRIA00040- S30FRIA00000W	Bed – 0.035 (stone silt and concrete) Side banks – 0.025 (stone walls)	 30FRIA00016_DN

Reach	Roughness values (manning's n)	Photograph
Parkavara river (Madeira Court) - MACT		
30MACT000011 - 30MACT00001	Bed – 0.04 (stone material) Banks/side banks – 0.06 or 0.025 (vegetated ivy banks or bare concrete /stone walls)	 30MACT00001_DN
Parkavara river - SMRN		
30SMRN00015 – 30SMRN00000	Bed – 0.04 (stone material) Side banks – 0.06 or 0.025 (vegetated ivy banks or bare concrete /stone walls)	 30SMRN_00014E_LB
Madeira river - VARA		
30VARA00103 – 30VARA00008	Bed – 0.04 (stone material) Side banks – 0.06 or 0.025 (vegetated ivy banks or bare concrete /stone walls)	 30VARA00040D_US
Sruffnacashlaun Stream - NEWC		
30NEWC00120 - 30NEWC00000	Bed – 0.04 (stone material) Banks – 0.06 (heavily vegetated)	 30NEWC_00090_UP

The Manning's n floodplain values were selected based on an analysis of datasets:

- Land use derived from OSi Prime2 mapping;
- Site visits undertaken by Arup;
- Relevant literature[17];
- The values used as part of the Western CFRAM[18].

Typical values used in the study are presented in Table 12. Figure 5.4.2 presents the manning's values for the City centre area of the study.

Table 12 2D Manning's values used in the study

Floodplain land use type	Manning's n values
Stability patches	0.5
Buildings	0.3
Non-coniferous woodland	0.07
General natural surfaces	0.04
Inland waterbodies	0.035
Roads, tracks, and paths	0.015

[17] Chow, V.T., 1959, *Open-channel hydraulics*: New York, McGraw-Hill, 680 p.

[18] Office of Public Works, 2016, *Shannon CFRAM Study – Hydraulics Report Unit of Management 25-26,* consultants Jacobs.

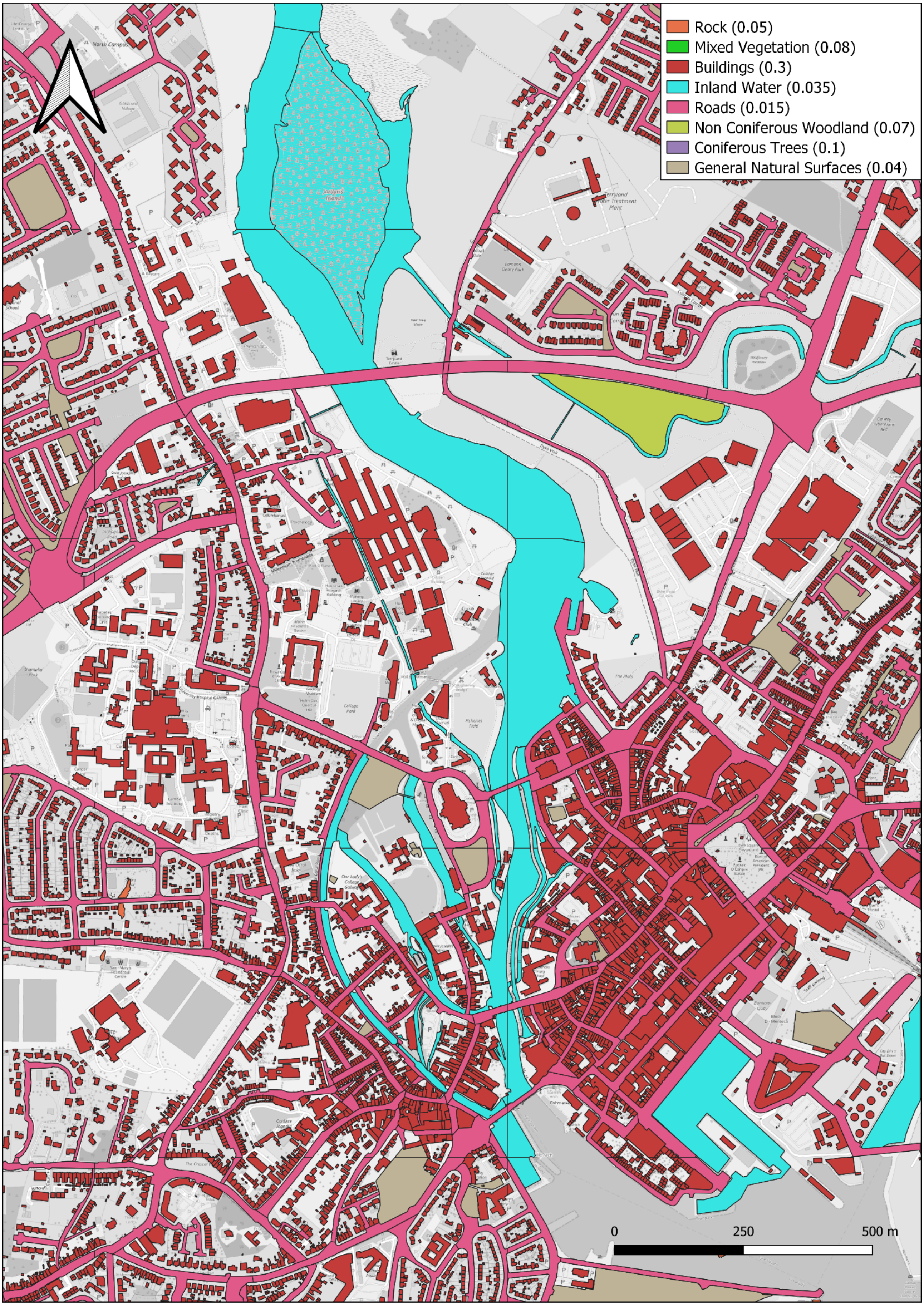


Figure 5.4.2 Map of Manning's n-values for various categories in the city centre

5.4.4 Representation of the river structures

The majority of bridges in the model have been modelled using the Bridge ARCH unit. The N6 Quincentennial Bridge has been modelled using the USBPR unit as the opening is relatively larger when compared with the channel geometry. A small number of other bridges located in the Distillery channel were also modelled as USBPR.

Overtopping of the bridges has been accounted for through the use of a spill unit in the 1D domain of the model. In-line weirs have been modelled using the weir unit in Flood Modeller Pro. In some cases, a spill unit was used instead to aid with model stability.

Culverts have been modelled through use of the culvert units (conduits) in Flood Modeller Pro combined with culvert inlets and outlets to represent head losses.

The dimensions of all the hydraulic structures have been taken from the surveyed data. The reader is referred to Appendix C which presents a datasheet for all the key structures included in the Galway City FRS fluvial/tidal model.

5.4.5 Salmon weir barrage

The Salmon Weir barrage is located close to the centre of Galway City, 800m upstream from Wolfe Tone Bridge and immediately downstream of the point where the Eglinton Canal separates from the Corrib.

The weir consists of 14 steel hydraulic gates and 2 older wooden lift gates. The weir forms an arc in plan view of 114.8m in length. The weir connects to a sloping concrete spillway apron which is designed to act as an energy dissipator by generating a hydraulic jump within the protected apron area downstream of the weir crest. The 14 gates are hinge gates that are lowered by tilting forward from their sill and are operated hydraulically.

A fish pass and two elver passes are also incorporated into the weir configuration

Figure 5.4.3 Example of 1 of 14 new steel hinge gates in closed position

Figure 5.4.4 Example of 1 of 2 old wooden gates in closed position

A detailed paper on the Galway Salmon Weir Sluice Barrage by Arthure (1960) describes the design and construction of the barrage. The paper provides dimensions of the gates and crest levels and information on its operation.

The Salmon weir gates have been represented as vertical sluice gates in the FMP model. This arrangement allows the gates to be modelled as sharp crested weir while they are closed and broad crested weir when they are opened. The parameters used for the gates are summarised in Table 13.

Table 13 Salmon weir geometric parameters used in the model

Parameter	2 old wooden gates	14 new steel gates	Source of information for Galway City FRS model
Elevation of weir crest	4.37 m OD	4.37m OD	Average of all levels from new Murphy's surveys, section 30CORR00151U
Length of weir	1.32m	1.32m	1960 As-built drawings
Breadth of weir	3.05m	6.13m	Old gates: Arthure report (10’ = 3.048m) New gates: Western CFRAM surveys 2013, as the variation in the Infill 2021 surveys is not consistent with other reports. Arthure report states a value of 6.1m
Crest level of gate	6.67m OD	5.79m OD	Old gates: Channel Surveys 2021 New gates: Arthure report (28' = 5.79m AOD). Infill surveys record a varying level between 5.76-5.79m OD.
Gate height	2.3m	1.42m	Gate height = Crest level of gate – Elevation of weir crest

Information has been collected from several sources with regard to the operation of the Salmon Weir gates in the present day. A meeting was held with the operator of the weir on 16th May 2022 to discuss the day-to-day operation. The following items are considered by the gate operator when controlling the gates on the weir:

- Gauge readings from several stations in Lough Corrib (Anglinham, Cong Pier, and Annaghdown), Clare River (Corrofin) and River Corrib upstream of the Weir (Dangan, Barrage U/S, and Woodquay);
- Long term weather forecast, and flood warnings issued by Met Eireann;

- Wind direction;
- Precondition from other river users and stakeholders, such as Western Regional Fisheries board, Galway City Council and river and lake boat users;
- Seasonal considerations.

The majority of gates are kept open during the winter months as the operator aims to create storage within the Lough Corrib to prepare for potential winter floods. The gates are typically closed during summer months in order to direct more flow towards the canal system and maintain levels higher upstream for navigation.

Figure 5.4.5 Downstream face of the weir with gates closed

The Gate log of the Salmon Weir gauge has been provided to Arup by the OPW and has been reviewed to compare the current operation of the gates with gauge levels and flows. It was found that the gates have been fully opened when flows on the Corrib correlate with circa Q2 fluvial event. It has therefore been assumed that the gates are fully open for all of the design flood events considered as part of the study. A sensitivity check of keeping some gates closed during the Q100 fluvial event has been undertaken and the outcomes are described in Section 10.

The Salmon weir discharge coefficient varies across the weir in the model in order to represent the variation in flows through the different gates. The higher coefficients (circa 1.05) are defined towards the western part of the weir (central to the River Corrib channel) and the lower coefficients (circa 0.85) are defined along the eastern part of the weir.

5.4.6 Salmon weir bridge

Salmon Weir Bridge is located approximately 80m downstream of the Barrage D/S gauge (Figure 5.4.6). This is a significant structure that has direct impact water levels in the channel.

It was observed from our site visits that there is an ineffective flow area along the left bank of the bridge which corresponds approximately to the first arch of the bridge (Figure 5.4.7) looking downstream. It was deemed that the conveyance through the arch would be very minimal in flood conditions. It was therefore decided to effectively remove the arch from the model and instead only include the other four arches of the bridge.

Furthermore, it was also noted that the width of the piers of the bridge vary with level. A varying pier width cannot be represented using the standard bridge model units in FMP. Various configurations of the bridge geometry were tested as part of the model calibration and the following approach was adopted:

- The February 2022 calibration model utilised the wider extent of the pier given that water levels during the event did not exceed the lower part of the bridge where the piers are widest.
- For all the design runs, the bridge pier widths were modelled using an average width for the piers as the water levels for these events are higher than the February 2022 event. Comparison of the actual conveyance area and modelled area during the Q100 event showed good agreement with a difference between them of circa 2%.

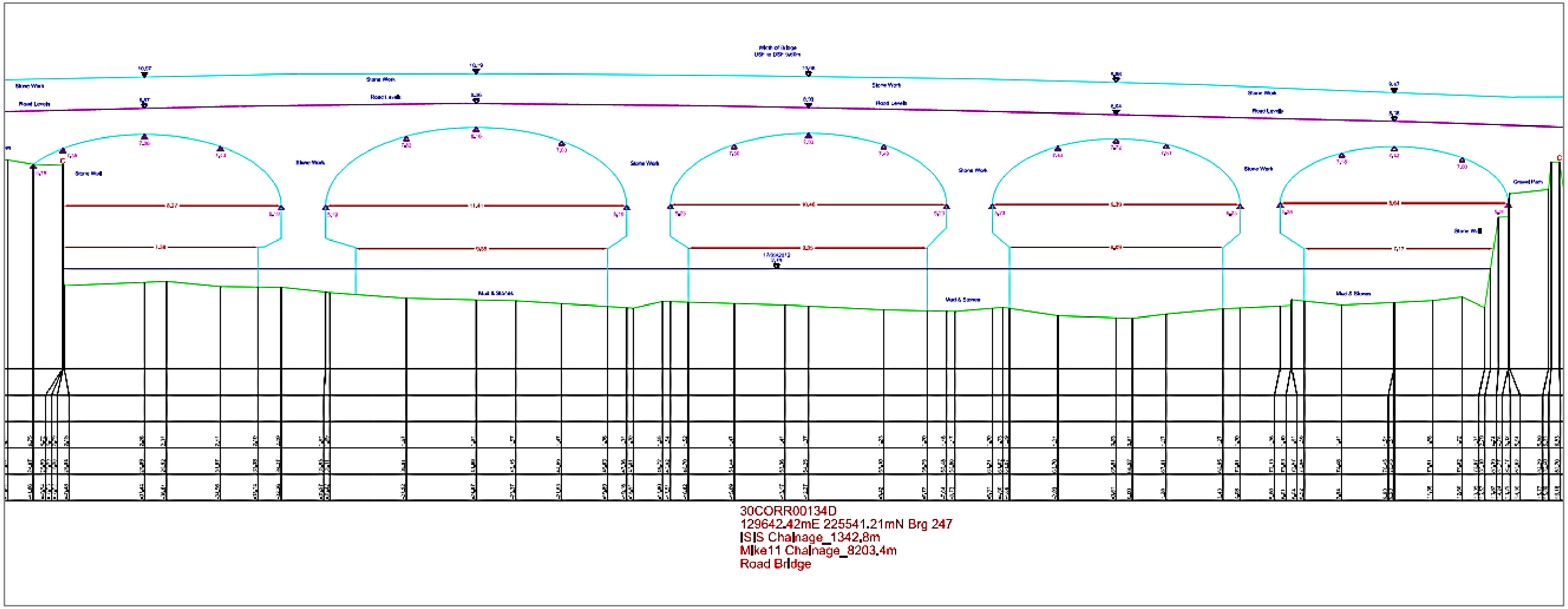


Figure 5.4.6 Salmon Weir Bridge survey (2013)

The bridge head loss coefficient was also tested as part of the calibration process. The final value was set as 1.5 for all the design runs.

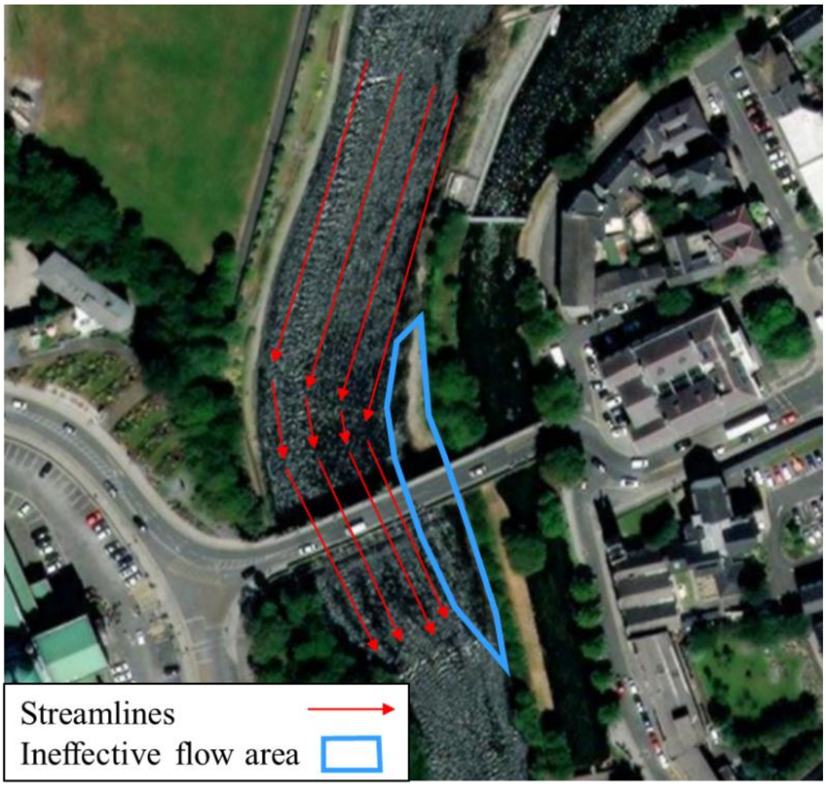


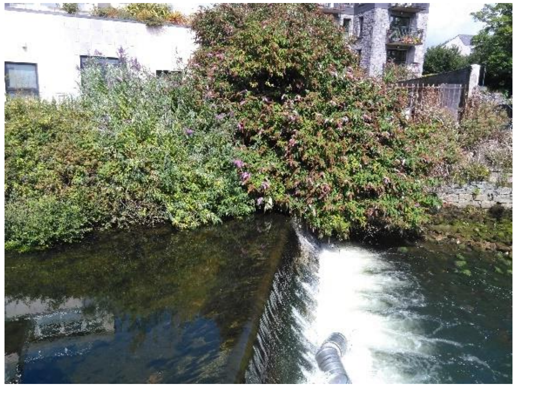
Figure 5.4.7 Ineffective flow area at Salmon Weir Bridge

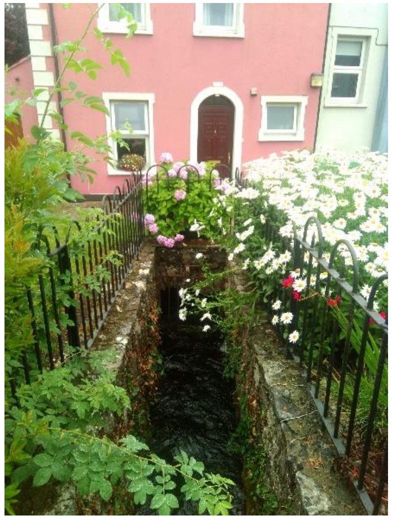
5.4.7 Other key hydraulic structures

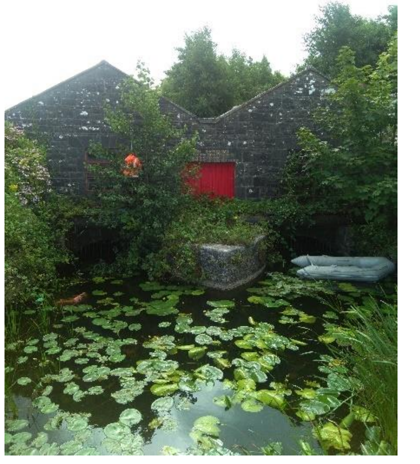
Aside from the Salmon Weir a number of other sluice gates and penstocks regulate the flow in Galway City along the various millraces and channels. These were originally constructed in order to manage water levels and flows through the mill races for hydropower. While consultation has been undertaken with all main parties, it has not been possible to establish who is responsible for the maintenance and operation for most of the structures in the present day. However, this investigation is ongoing and will be closed out during the Optioneering stages.

The structures have been represented in the model using various hydraulic units in FMP. The dimensions of the structures have been taken from the various topographic surveys. The structures are summarised in Table 14. It is noted that all the key structures in the scheme area are detailed in Appendix C.

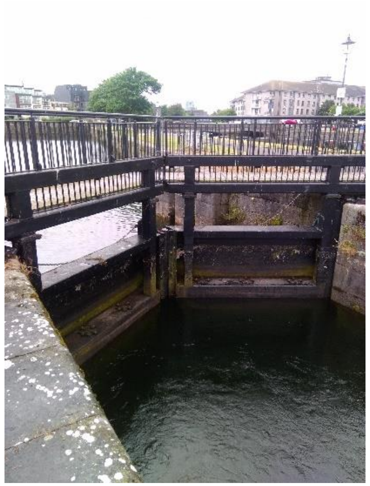
Table 14 Key hydraulic structures

Structure name and location	Description / Operation regime and responsibility	Galway City FSR model assumptions	Photograph
Sruffnacashlaun Siphon 30SALW00047O	Culvert that conveys Distillery channel flows underneath the Eglinton Canal. As seen in photograph, small openings allow flows from the Eglinton canal to the channel. These are not included in the model. A partial trash screen is also present at the inlet. The Siphon is heavily silted with large parts of debris.	Culvert modelled as inverted siphon, no trash screen. Information taken from recent Surveys 2021 and OPW long section from 2014. Opening area adjusted following review of below surveys. Surveys: Divers surveys (2010), CCTV and milk bottle survey, visual insptections and other surveys.	 Photograph looking downstream
Weir at Eglinton Canal 30SHEA0000W	Controls flows from Eglinton to the River Corrib. It is drowned out by tide during tidal events allowing flow from the Corrib to the Eglinton Canal.	Modelled as surveyed Surveys: Channel survey 2021	 Photograph looking at LB

Structure name and location	Description / Operation regime and responsibility	Galway City FSR model assumptions	Photograph
Culverts at Madeira Court MACT0006LB and MACT0006RB	Two sets of culverts under Madeira Court buildings with trash screen. Flow constriction. Inlets of culverts are smaller (1m x 1.2m). The RB culvert splits into two arch culverts with outfalls of 2m width each and LB culvert also widens to arch culvert of 2m width.	Modelled as per 2013 surveys but checked against Channel survey 2021. Modelled uniformly as per inlet details, as it is more conservative (smaller bore area). No trash screen included. Surveys: Channel surveys 2013	 Photograph looking downstream
Parkavera weir 30EGLI00039Y and 30EGLI00038T	Series of two weirs that have replaced the Parkavera gates along Eglinton canal. They restrict tidal influence on upstream areas.	Represented in model as two sharp crested weirs in sequence. Modelled as surveyed. Surveys: Channel survey 2021	 Photograph looking upstream

Structure name and location	Description / Operation regime and responsibility	Galway City FSR model assumptions	Photograph
Old Terryland waterworks 30CAST00018O	The old waterworks building is located in line on Terryland river. It significantly restricts the flow from Corrib to the river. A site visit confirmed that very little flow can pass through the turbines located underneath the building and that the openings are possibly blocked.	Represented in model as two orifices with very small openings and restricted bore area. Surveys: Chnnel survey 2021 and 2013, GCC layout plan, archaeological diving report of Terryland river, Arup Staff site visit March 2022.	 Photograph looking downstream
Gaol river - Old engineering building 30NUNS00013O1	This derelict building consists of 3 disused sluices and 2 boarded off culverts. Gate 1 (left) stuck in fully raised position (no restriction to flow) however little flow goes through, indicating blockages. Gate 2 (middle) also fully raised. Gate 3 (Right) is lowered. In general, when observing the flows downstream, little flow goes through all the building structures.	Not represented in model as a structure, limited flows are transferred from upstream section to downstream using an abstraction unit. Surveys: Channel survey 2021, JBA Condition Assessment on GC Sluices 2016 - no information on operation	 Photograph looking downstream

Structure name and location	Description / Operation regime and responsibility	Galway City FSR model assumptions	Photograph
Kingfisher's channel (Connection channel between Distillery and Corrib) 30SALW00107D	Sluice gates that Control flows from Corrib during winter. Closed when Corrib flows are high. Opened during summer months. The University of Galway facilities responsible for its operation.	Penstock assumed closed during winter months, Kingfisher channel not included in 1D model, included in 2D model only.	 Photograph looking downstream
Distillery Sluice 30SALW00099D	A set of 2 sluice gates introduced by the University of Galway. Penstocks always in open position. Penstocks closed only for maintenance. Control of high flows from Sruffnacashlaun.	Modelled as surveyed, penstocks assumed open. Surveys: Channel survey 2021	 Photograph looking downstream

Structure name and location	Description / Operation regime and responsibility	Galway City FSR model assumptions	Photograph
The Bish school sluices 30gmra00021B	2 penstocks. The operation of these gates is unknown. One was open and one closed during surveys.	Modelled as surveyed, with one gate open and one closed. Surveys: Channel survey 2021	 Photograph looking dowsntream
Gates to Claddagh basin 30EGLI00003	The gates typically get opened in April/May to allow boats exit for the Summer months. The gates are opened again in late Oct to allow boats in on order to shelter for the winter. The gates can also be opened at any stage for short durations outside of this time to facilitate festival events, clean ups etc. Anticipated operators are Claddagh users and Port of Galway.	Modelled as a weir set at the level of the opening. Assumes gates are closed during flood conditions. Surveys: Channel survey 2021	 Photograph looking upstream

5.4.8 Representation of buildings and other structures in the 2D grid

Buildings within the floodplain (2D space) impact on overland flow paths and as such it is important to consider how they are represented in the model.

The buildings in the Galway City FRS models (both fluvial/tidal and coastal models) were represented by specifying a high manning's value (0.3)[19] across the footprints of all the buildings which were identified from the OSi Prime2 dataset. This was to achieve a balance between allowing flows to enter buildings while ensuring that the resistance offered by the fabric of the building is accounted for.

The FFL of the buildings were also set to the surveyed threshold level from the recent Infill threshold surveys (Lot1b, 2021). The polygons of buildings that were not surveyed were set to the average Lidar value over the entire polygon.[20]

Other structures such as walls, roads and railway embankments can also influence the movement of water in the floodplain and must be correctly represented in the model. The N6 road embankment situated to the east and west of the Quincentennial bridge is captured by the Lidar and is therefore represented. In order to account for the underpasses at the local road crossings under the embankment, Z shapes were added to the Tuflow model. The ground levels therefore matched the underpass levels and allow flow to pass under the embankment.

Some bridge decks in the model have also been added as Z shapes/points to allow flow within the 2D space between crossed riverbanks.

5.4.9 Representation of flood defences

A number of flood defences have been identified within Galway City, the majority of which are considered informal ineffective. The two formal and effective flood defences are listed as:

- Dyke Road embankment along the left bank of the River Corrib
- Retaining wall on the right bank of the River Corrib, downstream the Salmon Weir.

Dyke Road Embankment

The Dyke Road embankment consists of a stone embankment of 600m length and runs in a northwest to southeast direction between the River Corrib and the Terryland/ Castlegar area. The embankment consists of a large stone crest, which also forms part of a local footpath.

The elevation of the crest of the embankment varies from 6.6m AOD to 7.73m AOD and prevents flooding up to the 1% AEP fluvial event[21] with no allowance for freeboard. The embankment overtops during the 0.5% AEP event (refer to Section 7), as well as the 1% AEP MRFS and HEFS events.

The embankment has history of seepage problems and experienced some damage from a flood event around 2007 which was subsequently repaired by the OPW.

The Dyke road embankment has been modelled in the 2D space as a Z line set to the surveyed levels from the recent flood defence survey undertaken by Murphy's Geospatial in 2021.

[19] https://tuflow.com/media/4997/2008-flooding-in-urban-areas-2d-modelling-approaches-for-buildings-and-fences-syme-hwe-aus.pdf

[20] A threshold has not been applied to the FFL of the non-surveyed buildings. From Arup's experience on other flood scheme projects it has been found that the LIDAR data generally provides a good estimate of the FFL for most buildings constructed on relatively flat ground. Adding a threshold therefore tends to overestimate the FFL. This approach for Galway is supported from our observations on the site visit and also by inspecting various buildings on street view. This assumption will however be further assessed as part of the damages analysis.

[21] This is a conclusion from the design model runs which are presented later in the report

Figure 5.4.8 Stone dyke road embankment at centre and adjacent footpath on right of photo

Figure 5.4.9 Dyke road embankment shown on the far top of photo, with Dyke road on far right

The Dyke Road embankment ties into a second earth embankment known as the Clifden Rail embankment which is running east to west. The Clifden Rail embankment is also represented in the model as per the LiDAR DEM data. The embankment has a base level set at approximately 6-7m AOD and crest levels varying between 11.5-12.5m AOD. This embankment functions together with the Dyke Road embankment to provide flood defence east of the Corrib.

Figure 5.4.10 View of the Clifden Rail embankment from Dyke Road. Photo facing west. Dyke road embankment located directly to the right (north) of the embankment in above photo.

A small culvert was observed on a recent site visit at the base of the Clifden Rail embankment (Figure 5.4.11). This culvert has been considered as part of the sensitivity analysis modelling and is described in Section 10 of the report.

Figure 5.4.11 Culvert under Clifden Rail embankment

5.5 1D and 2D model linkage

The two key variables which control the volume of water that spills onto the floodplain from the river channel are listed as:

- The water level in the river channel;

- The elevation of the bank of the channel i.e., the elevation at which water spills from the river to the floodplain.

The water level in the river channel is calculated by the 1D model. The elevation of the bank however is defined in the model by the user using the topographic survey data. It is a very important dataset in the model as it controls the volume of water that spills into the 2D domain of the model. Its correct specification is essential in ensuring an accurate and credible hydraulic model.

The elevation of the left and right banks throughout the 2D model domain of the model were defined from actual surveyed elevations from the river channel survey and were accounted for in the model through the use of Z lines in Tuflow. These Z lines were defined for the entire 1D-2D reach of the model and ensured an accurate representation of the volume of water spilling from the 1D to the 2D domain.

It is noted that a WrF (weir calibration factor) also forms part of the calculation of water spilling from the 1D to the 2D model and in effect acts as a discharge coefficient. The default values of the WrF parameter have been used in the model set up.

5.6 Blockage risk

A qualitative blockage risk assessment of all the key hydraulic structures in the scheme area has been undertaken as part of the study and is presented in Appendix F.

Following the qualitative analysis, a number of structures might be assessed through a quantitative blockage analysis as part of the study. This analysis will be undertaken as part of the optioneering of the study.

5.7 Hydrological Estimation Points

The design flows estimated by the hydrological study act as the upstream boundary conditions of the model and therefore need to be incorporated into the model.

5.7.1 Overview

As noted in the hydrology report, the hydrological estimation points (HEPs) were selected at key locations along the River Corrib within the Scheme Area following the guidance outlined in the Tender Brief. HEPs were located at the following points:

- Upstream boundaries of all modelled watercourses;
- Points on receiving channels upstream and downstream of the confluence of any tributary;
- Points on tributaries upstream of the confluence with the receiving channel;
- Locations as necessary to accurately represent the inflows, additional to tributaries, along the modelled watercourses;
- Other points at suitable locations as necessary to ensure that there is at least one HEP every 1km along all modelled watercourses.

The location of the HEPs is indicated in Figure 52.

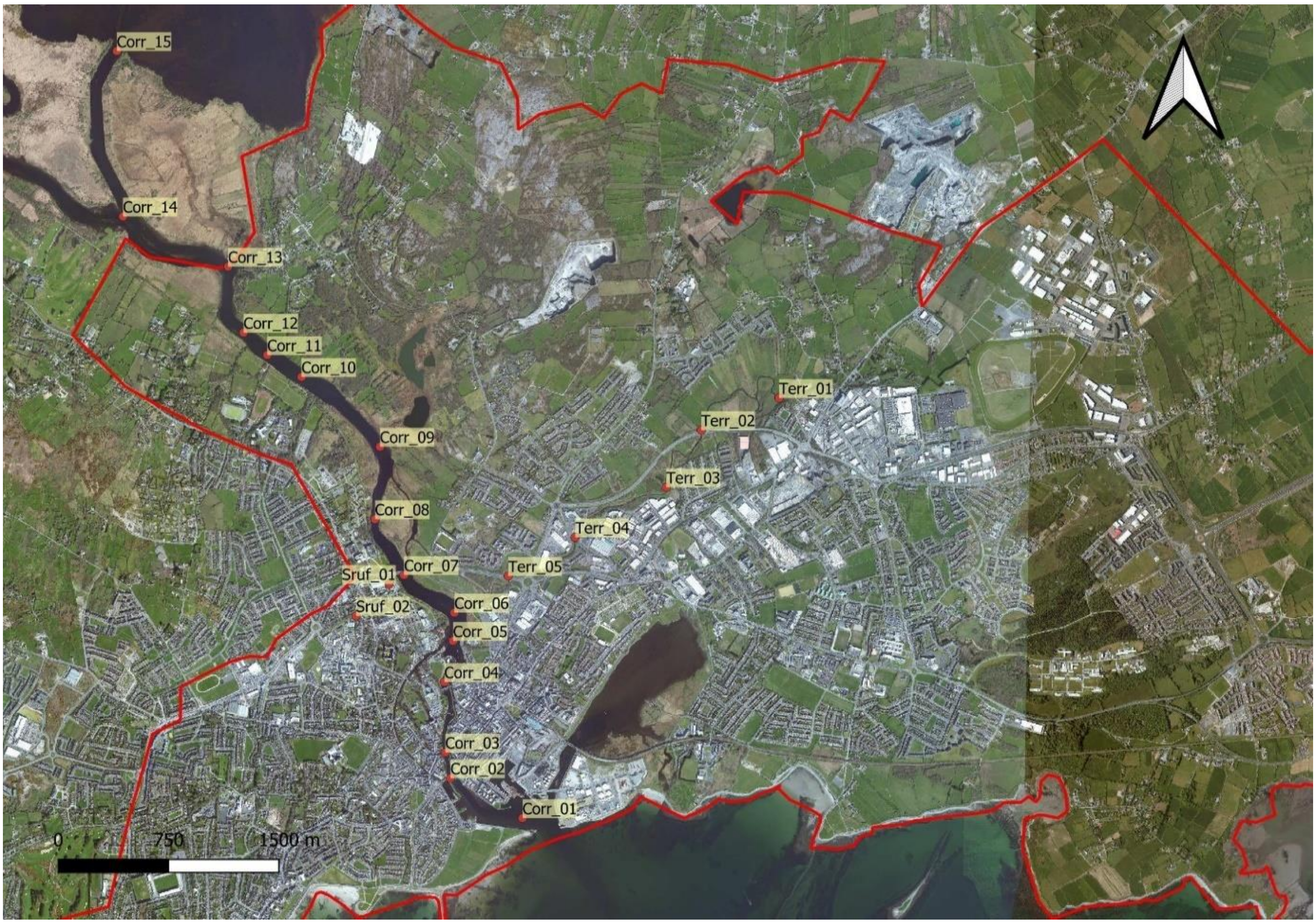


Figure 5.7.1 Hydrological Estimation Points (HEPs) location

5.7.2 Design flows

The design flows through the reach at the various HEPs are very similar for two main reasons:

- There are no significant sub-catchment inflows downstream of the Lough Corrib i.e., downstream of the Northern most HEP on the river;
- The total length of the reach in the study area is relatively small (circa 6.5km).

For example, the peak Q100 flow at HEP Corr_12 at the upstream end of the scheme area is 427.8 m^3/s, while the peak Q100 flow at HEP Corr_01 at the downstream end is 435.7 m^3/s. There is therefore a difference of circa 8m3/s between both ends of the scheme area which represents a difference of less than 2%.

The design flows at the various HEPs on the Corrib are therefore not subject to any significant variation throughout the reach.

5.7.3 Insertion of the HEPs in the Hydraulic model

Design flows calculated at the Corr_06 HEP has been used as the upstream flow boundary of the model. Flows upstream of this point will therefore be very marginally overestimated in the model. Flows downstream of the point will not however be underestimated by any significant amount as the difference in the design flow between Corr_06 and Corr_01 is very minimal.

Figure 53 presents the Q100 hydrograph from two locations in the model:

- At the upstream boundary of the model
- At model node 30CORR00185 which equates to the location of the Corr_06 HEP.

It can be seen from the plot that the peak flow at the location of the Corr_06 is maintained in the model.

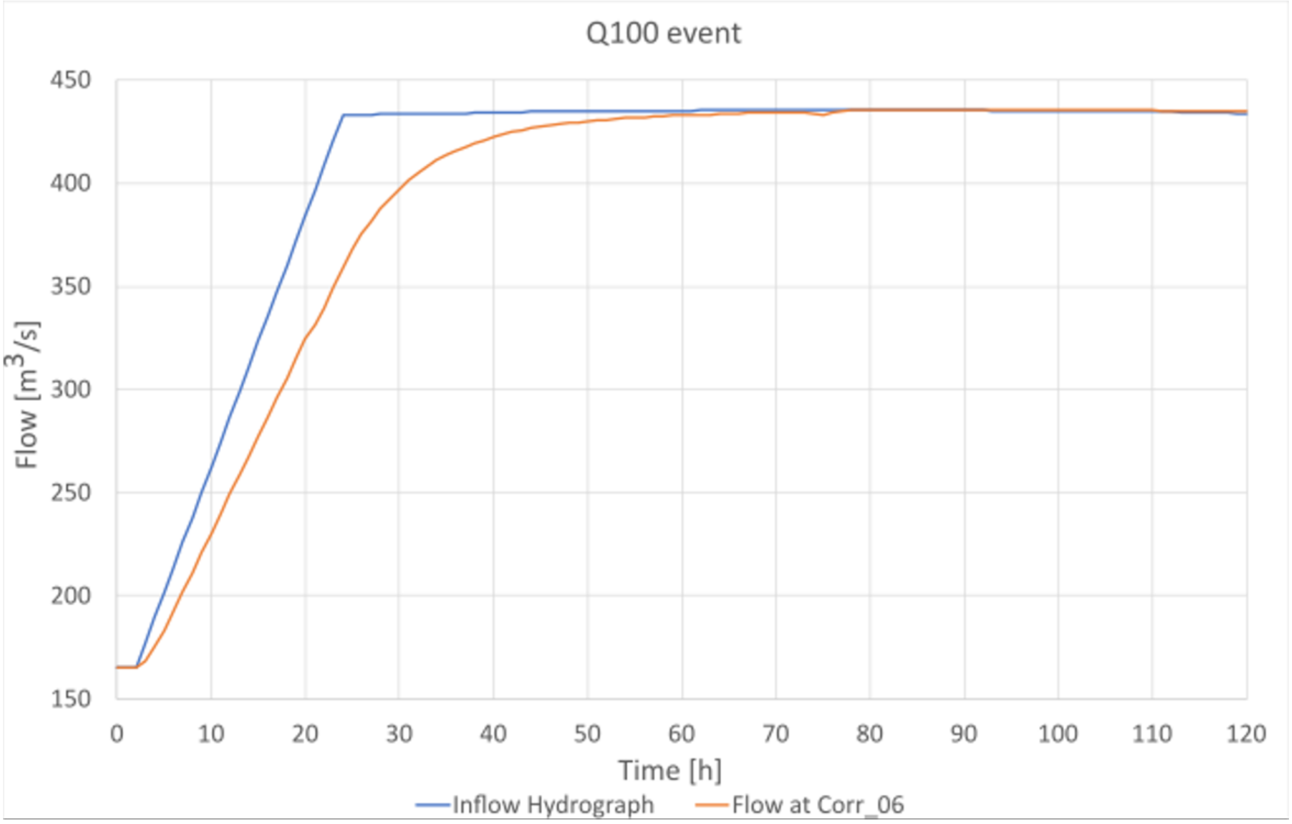


Figure 5.7.2 Comparison of the inflow hydrograph derived from the HEP, and the modelled flow at Corr_06 for the Q100 event

5.8 Coastal Model build

5.8.1 Overview

A standalone 2D Tuflow Quadtree model of the coastal floodplain has been developed as part of the study. The model was developed from the Western CFRAM coastal floodplain hydraulic model and represents a more refined and detailed version of that model. The key refinements include:

- model boundaries have been altered to include the WOT input from the 89 calculation points;
- building thresholds have been updated to ensure that the surveyed FFL of the building are explicitly represented in the model;
- A spatially varying computational mesh has been adopted in order to resolve the flow in the vicinity of Long Walk with finer resolution than the area in the rest of the model (refer to section 5.8.6)

The objective of the model is to estimate the coastal flood risk across the study area by modelling the propagation of overtopping discharges and direct tidal inundation across the coastal floodplain. This section of the report provides an overview of the coastal model development.

5.8.2 Model domain

The 2D coastal model domain is presented in Figure 5.8.1. It can be seen from the plot that the model covers a large area from the seashore caravan park in the East to Rosscam point in the West.

A more detailed schematic of the model is presented in Appendix G.

Figure 5.8.1 TUFLOW model formulation

5.8.3 Model boundaries

The coastal model uses two separate boundary conditions:

- Open Sea boundary – a time varying WL profile is applied at the interface with the sea. The WL profile does not vary spatially along the length of the sea boundary;
- WOT source discharges – The Q WOT discharges are included in the model through the use of 2d_SA polygons. Each individual polygon represents the flow from an individual WOT calculation point. The WOT flow is estimated in $m^3/s/m$ from the empirical equations and are therefore required to be multiplied by the full length of the individual boundary polygon in order to derive the total flow across the line in m^3/s. By then using the READ GIS SA ALL command in Tuflow the total flow across the line is evenly distributed between all the grid cells across the polygon whose centre point is covered by the area bounded by the polygon. The individual WOT source discharge points defined from the polygons for four separate WOT calculation points is illustrated in the following figure for Salthill.

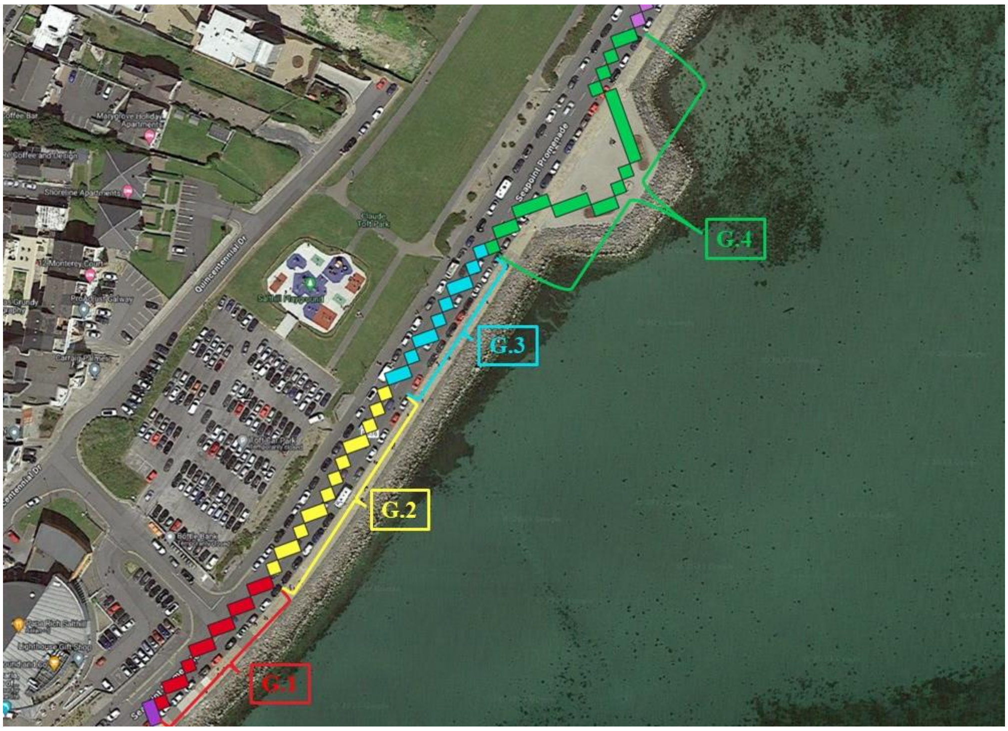


Figure 5.8.2 Schematic of the WOT discharge polyline boundaries

As the River Corrib is not explicitly represented in the model there is no 1D/2D model interface and hence no 1D/2D model boundaries.

5.8.4 Model parameters and representation of the buildings

The 2D coastal model adopts the same approach to the model parameter schematisation and representation for the buildings in the floodplain as the coupled 1D/2D model. The reader is referred to Section 5.4 for a description.

5.8.5 Representation of structures

As the coastal model is a standalone 2D model the river structures (i.e., bridges and weir) along the Corrib are not represented. This approach does not however impact on the results of the coastal model as the structures along the Corrib only impact on water levels locally in the River which is considered by the 1D/2D model as part of the study.

5.8.6 Grid resolution

The coastal model utilises a 4m grid resolution for most of the area of the domain. The area in the vicinity of Long Walk was however resolved with a 2m grid in order to avoid the WOT discharge polygon encroaching on the footprint of the buildings. This is illustrated in the following figure. The spatially varying resolution model was run as a single model using the Quadtree version of Tuflow.

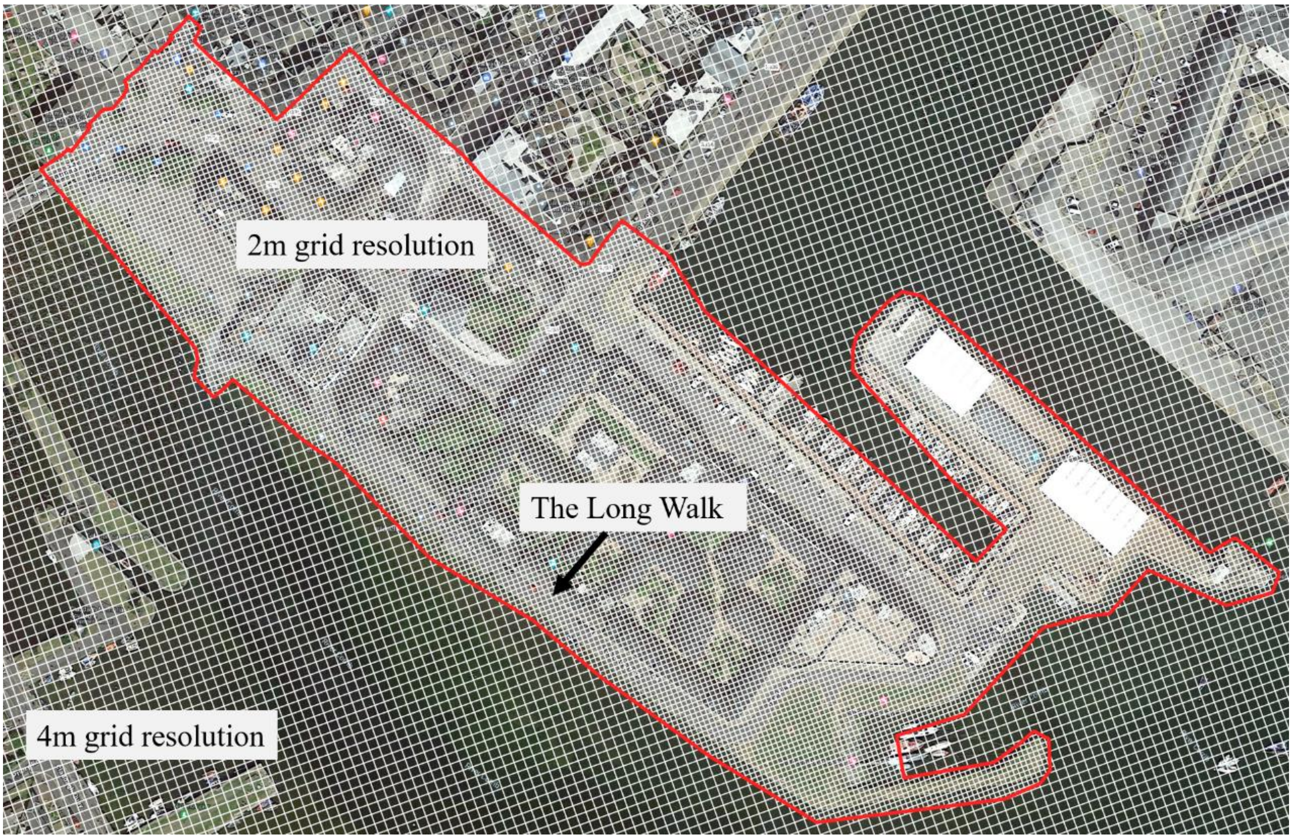


Figure 5.8.3 Location of the higher resolution (2m) Quadtree mesh

5.8.7 Model calibration

As noted earlier in the report, the coastal model has not been calibrated against any historic event. Accuracy in the model development has been ensured by utilising high quality input geometric data and ensuring best practice in model build.

5.9 Hydraulic modelling of the options

The fluvial/tidal and coastal hydraulic models will be modified to model the various flood relief options considered as part of the development of the scheme. This work will be reported in a separate options development report.