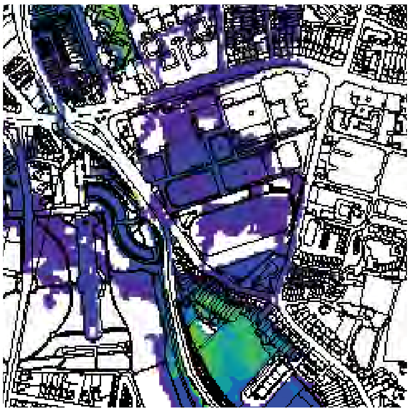
RPS

NORTH WESTERN
NEAGH BANN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

North Western - Neagh Bann CFRAM Study

UoM 06 Hydraulics Report
4.8 Annagassan

IBE0700Rp0012



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NWNB CFRAM Study

HA06 Hydraulics Report – Annagassan Model

DOCUMENT CONTROL SHEET

Client	OPW
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Model Name	Annagassan

Rev	Status	Author(s)	Modeller	Reviewed by	Approved By	Office of Origin	Issue Date
D01	Draft	T. Carberry	D. Irwin	I. Bentley	G. Glasgow	Belfast	13/03/2014
D02	Draft	T. Carberry	D. Irwin	S. Patterson	G. Glasgow	Belfast	26/06/2014
F01	Draft	E.Holland	E.Holland	L. Arbuckle	G. Glasgow	Belfast	20/02/2015
F02	Draft	E.Holland	E.Holland	L. Arbuckle	G. Glasgow	Belfast	13/08/2015
F03	Draft	E.Holland	E.Holland	S. Patterson	G. Glasgow	Belfast	07/07/2016

Table of Reference Reports

Report	**Issue Date**	**Report Reference**	**Relevant Section**
North Western Neagh Bann CFRAM Study UoM06 Inception Report	March 2013	IBE0700Rp0003_UoM 06 Inception Report	4.3.2
North Western Neagh Bann CFRAM Study Hydrology Report UoM06	October 2013	IBE0700Rp0008_UoM 06 Hydrology Report	3.2, 4.8, 6.2
North Western Neagh Bann CFRAM Study Flood Risk Review	May 2012	2011s5232 NW&NB CFRAM FRR Report	4
North Western Neagh Bann CFRAM HA01_06_36 Survey Contract Report	October 2013	IBE0700Rp0007_HA01_06_36 NWNB_CFRAM_Survey Contract Report	ALL
North Western – Neagh Bann CFRAM Study UoM 06 - Hydraulics Report	May 2014	IBE0700Rp0012_UoM 06_Hydraulics Report	3

4 HYDRAULIC MODEL DETAILS

4.8 ANNAGASSAN

4.8.1 General Hydraulic Model Information

(1) Introduction:

The NWNB CFRAM Flood Risk Review (2011s5232 NW&NB CFRAM FRR Report_Final_v2.0) highlighted, Annagassan, in the River Dee/Glyde catchment, as an AFA for fluvial and 'mechanism 1 tidal' and 'mechanism 2 wave overtopping' flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

Annagassan AFA is located on the southerly shore of Dundalk Bay in County Louth. In terms of fluvial flood risk it is located at the mouth of the River Glyde, which is also joined by the River Dee within the AFA extent (refer to Section 4.8.2, Figure 4.8.1). To this end, it is linked to both the Carrickmacross AFA model and the Ardee AFA Model (refer to Section 4.3 and 4.4 respectively). The total catchment area at its downstream limit is 750km^2. Both rivers meander through relatively flat lowlands before reaching the AFA itself.

Hydrometric station 06014 at Tallanstown forms the upstream limit on the River Glyde. Data from the gauge at Tallanstown was used to adjust initial Q_{med} estimates (based on catchment descriptors) on the upper reaches of the River Glyde. Hydrometric Station 06021 at Mansfieldtown is also located on the River Glyde (8km downstream from Tallanstown). A rating review of the station was undertaken as part of the CFRAM Study project brief (refer to Section 4.8.5 (4)). The review confirmed the gauged Q_{med} value of 21.87m^3/s based on over 50 years of data. It was not rated under FSU but would be considered an A1 station based on the rating review. To this end, the station's data was used to adjust initial Q_{med} estimates on the lower reaches of the River Glyde. The use of the Tallanstown and Mansfieldtown stations as pivotal sites resulted in significant downward adjustment of initial Q_{med} estimates of 0.65 and 0.6 respectively.

Charleville weir Hydrometric Station (06013) is located on the River Dee almost 2km upstream of the model limit (it is within the Ardee Model, refer to Section 4.4). Again a significant downward adjustment factor comes from this data (0.64) which was applied to initial Q_{med} estimates on the River Dee. Full details of hydrology estimation of design flows are included in the UoM 06 Hydrology Report (Rp0008 Chapter 4.8).

The lower reaches of the River Dee and River Glyde are HPWs, as well as approximately 4 km of the River Glyde in the vicinity of Mansfieldtown hydrometric station (06021) as this was subject to a rating review (refer to Section 4.8.5 (4)). The River Glyde Tributary is also a HPW due to its close proximity to hydrometric station 06021. The upper reaches of the River Dee and River Glyde (except for the gauging station section) are MPW. All HPW reaches of the River Dee and River Glyde were modelled as 1D-2D using the MIKE suite of software. The remaining watercourses (MPW) were modelled as 1D.

Mansfieldtown (Station 06021) was also modelled in 1d as the water level does not typically reach the bank levels meaning a 2d domain is not required.	
(2) Model Reference:	HA06_ANNA8
(3) AFAs included in the model:	Annagassan
(4) Primary Watercourses / Water Bodies (including local names):	

Reach ID	**Name**
0601M	River Glyde
0601A	River Glyde Tributary 1
0602M	River Dee

(5) Software Type (and version):

(a) 1D Domain:	**(b) 2D Domain:**	**(c) Other model elements:**
MIKE 11 (2012)	MIKE 21 - Flexible Mesh (2012) MIKE 21 - Flexible Mesh (2012) ('Mechanism 2 wave overtopping' flooding)	MIKE FLOOD (2012)

4.8.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

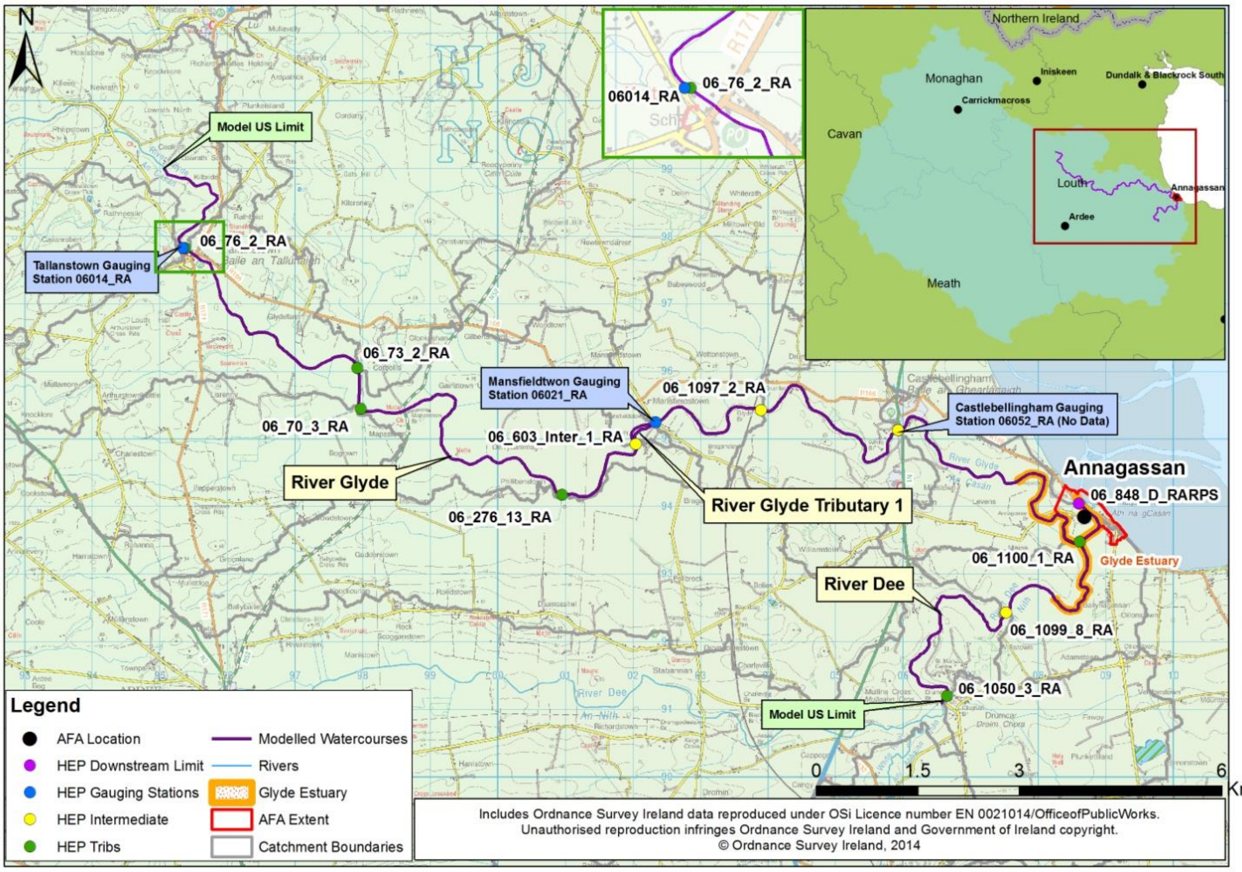


Figure 4.8.1: Map of Model Extent

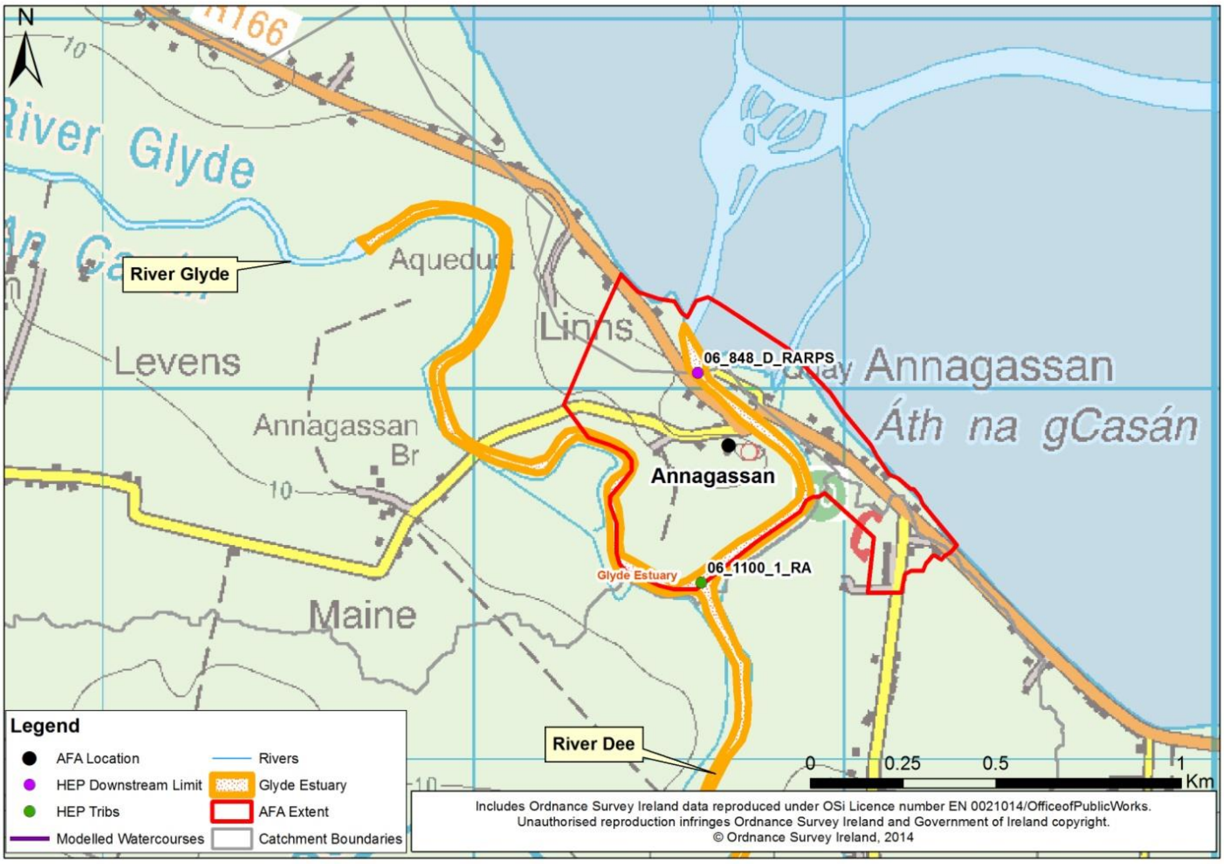


Figure 4.8.2: Map of AFA Extent

Figure 4.8.1and Figure 4.8.2 illustrate the extent of the modelled catchment, river centre line, HEP locations and AFA extents. The model contains 2no. Gauging station HEPs, one of which act as Upstream Limit HEP on the River Glyde (06014_RA). There are 4no. Intermediate HEPs and 7no. Tributary HEPs. There is 1no. Downstream Limit HEP. Castlebellingham Gauging Station (06052_RA) has no water level or flow data available and so was redefined as an Intermediate HEP. The Gauging Station, Intermediate, Downstream Limit and Tributary HEPs (where modelled) are used in anchoring the model to observed / estimated flows as detailed in Appendix A.3.

(2) x-y Coordinates of River (Upstream extent):

River Name		Easting	Northing
0601M	River Glyde	295352	297834
0601A	River Glyde Tributary 1	302040.5	294932
0602M	River Dee	304411	290763

(3) Total Modelled Watercourse Length:		31.0 km (approx.)	
(4) 1D Domain only Watercourse Length:	22.3 km (approx.)	**(5) 1D-2D Domain Watercourse Length:**	8.7 km (approx.)
(6) 2D Domain Mesh Type / Resolution / Area:		Flexible / 5-300 metres / 90 km^2 (approx.)	

(7) 2D Domain Model Extent:

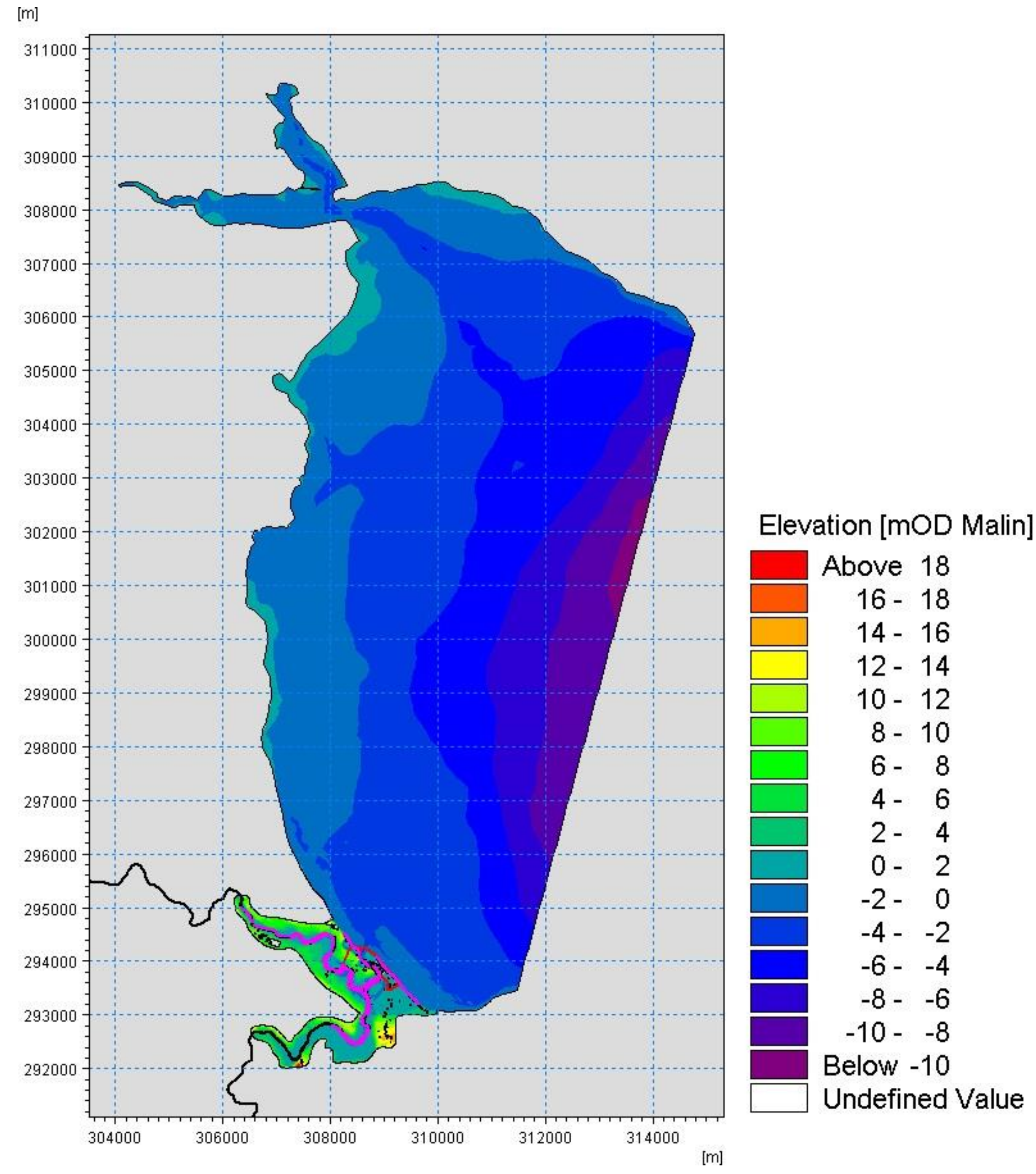


Figure 4.8.3: 2D Model Extent

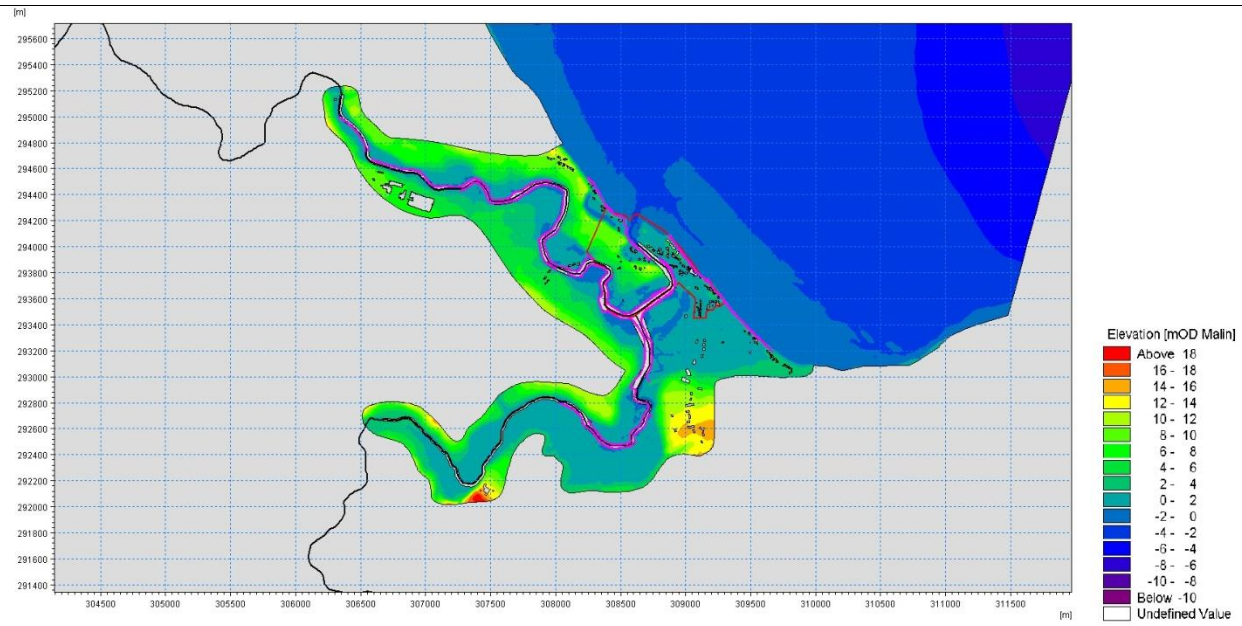


Figure 4.8.4: 2D Model Extent - Zoomed to Area of Interest

Figure 4.8.3 and Figure 4.8.4 illustrate the modelled extents, general topography and 2D extent. Buildings are excluded from the mesh and therefore represented as white spaces. Refer to Chapter 3 for details on representation of buildings in the model.

Figure 4.8.5 shows an overview drawing of the model schematisation. Figure 4.8.6 shows detailed views. The overview diagram covers the model extents, showing the surveyed cross-section locations, AFA boundary and river centre line. It also shows the area covered by the 2D model domain. The detailed areas are provided where there is the most significant risk of flooding. These diagrams include the surveyed cross-section locations, AFA boundary and river centre line. They also show the location of the critical structures as discussed in Section 4.8.3(1), along with the location and extent of the links between the 1D and 2D models. For clarity in viewing cross-section locations, the model schematisation diagram shows the full extent of the surveyed cross-sections. Note that the 1D model considers only the cross-section between the 1D-2D links.

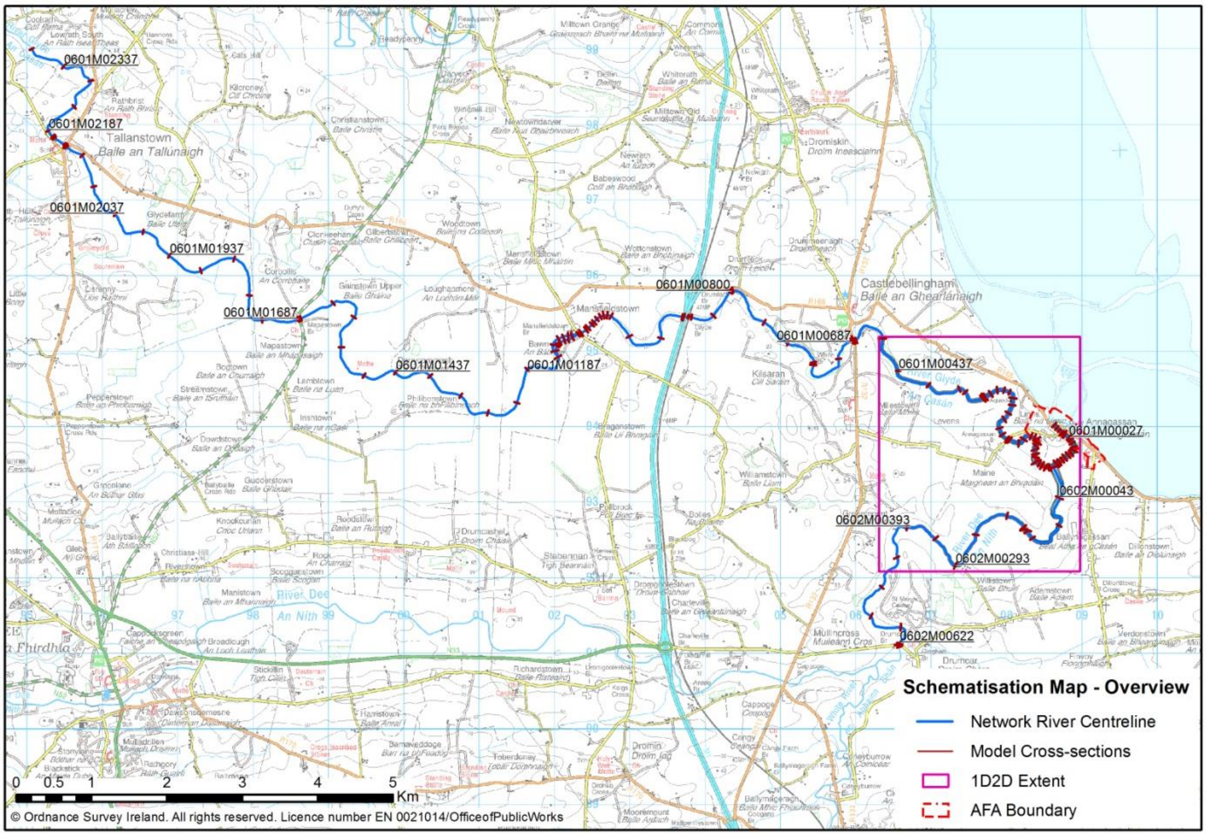


Figure 4.8.5: Overview Drawing of Model Schematisation

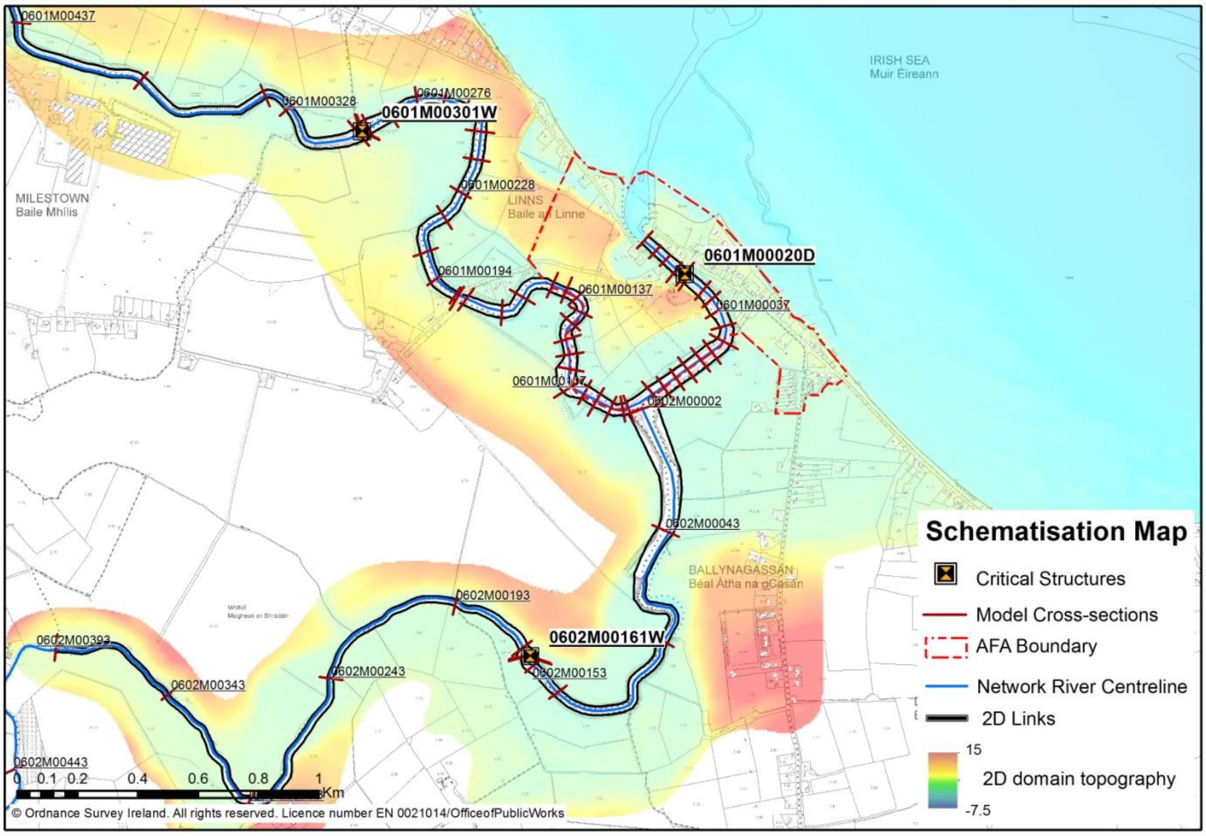


Figure 4.8.6: Detailed Area of Model Schematisation showing Critical Structures*

** For clarity in viewing cross-section locations, the model schematisation diagram shows the full extent of the surveyed cross-sections. Note that the 1D model considers only the cross-section between the 1D-2D links.*

Wave Overtopping Model

Figure 4.8.7 illustrates the extents of the specific 2D domain used to analyse 'mechanism 2 wave overtopping' flooding in Annagassan AFA. There are two ICWWS CAPO Prediction Locations applicable to the coastline as shown in Figure 4.8.7. It should be noted that this model area is considerably smaller than the overall model for analysing fluvial and 'mechanism 1 tidal' flooding since the area of interest is much more localised.

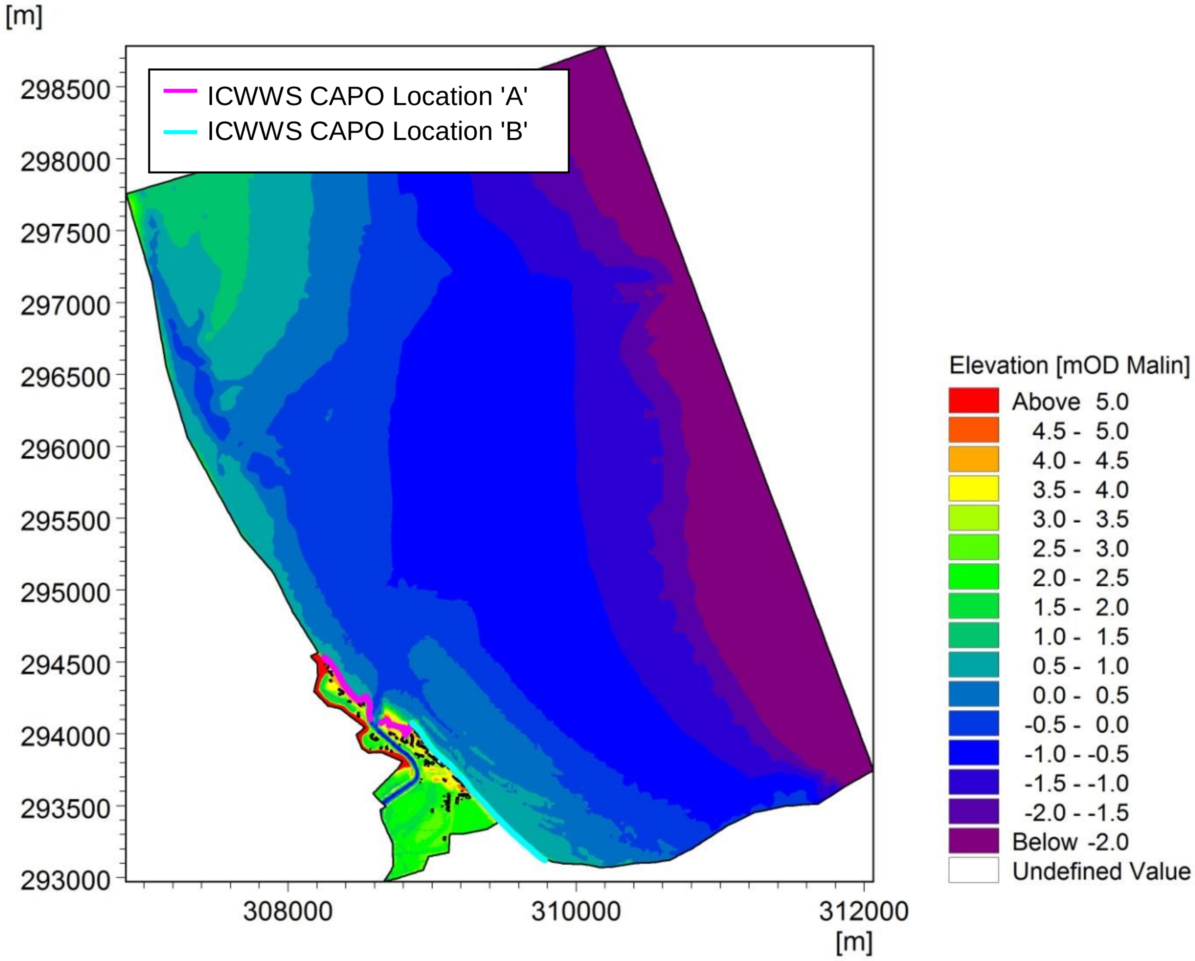


Figure 4.8.7: 2D Model Domain for 'Mechanism 2 Wave Overtopping' design runs

(8) Survey Information

(a) Survey Folder Structure:

First Level Folder	**Second Level Folder**	**Third Level Folder**
Murphy_NW6_M08_WP1_V1_0601A_130308 Annagassan Murphy: Surveyor NW6: North Western CFRAM Study area Hydrometric Area 6	V0_20130306_Ascii	
	V0_20130306_GIS	Flood_Plain_Photos_and_Shapefile
	V0_20130306_Other	Floodplain Photos
	Photos *(Naming*	

M08: Model Number 2 0601A: River Reference 130308; Date Issued (08th MAR 2013)	*convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank)*	

(b) Survey Folder References:

Reach ID	**Name**	**File Ref.**
0601M	River Glyde	Murphy_NW6_M06_M08_WP6_V1_0601M_A_130313 Murphy_NW6_M08_WP1_V1_0601M_130308 Murphy_NW6_M08_WP6_0601M_130306
0601A	River Glyde Tributary 1	Murphy_NW6_M08_WP1_V1_0601A_130308
0602M	River Dee	Murphy_NW6_M07_M08_WP6_V1_0602M_A_130426

(9) Survey Issues:

None.

4.8.3 Hydraulic Model Construction

(1) 1D Structures (in-channel along modelled watercourses):	See Appendix A.1 Number of Bridges and Culverts: 16 Number of Weirs: 5

The survey information recorded includes a photograph of each structure, which has been used to determine the Manning's n value. Further details are included in Chapter 3.5.1 A discussion on the way structures have been modelled is included in Chapter 3.3.3.

On the River Glyde, weir 0601M00301W (Figure 4.8.8) at chainage 36696m drowns out and causes flow to back up during large fluvial events (0.5% AEP and 0.1% AEP), resulting in flooding upstream, see Longitudinal Profile, Appendix A.2.

Figure 4.8.8: River Glyde, Weir 0601M00301W

On the River Glyde, bridge 0601M00020D (Figure 4.8.9) at chainage 39495m impacts upon the flow regime during large coastal events (0.5% AEP and 0.1% AEP), although its capacity is sufficient so that it never becomes submerged, see Longitudinal Profile, Appendix A.2.

Figure 4.8.9: River Glyde, bridge 0601M00020D

On the River Dee, weir 0602M00161W (Figure 4.8.10) at chainage 26932m drowns out and causes flow to back up during large fluvial events (0.5% AEP and 0.1% AEP), resulting in flooding upstream.

Figure 4.8.10: River Dee, Weir 0602M00161W

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):	None
(3) 2D Model structures:	7 dike structures were included in order to model defences in the 2D domain. This included the coastal embankments and walls highlighted in Section 4.8.3(4) below (Defence References 12-15), as well as the ends of embankments which extend into the 2D domain (Defence References 4, 6, 9 and 11). See Figure 4.8.11

(4) Defences:

Defence Reference	Type	Watercourse	Location
1	Embankment	River Glyde	Right bank, approx. model chainage 34990-35315.
2	Embankment	River Glyde	Left Bank, approx. model chainage 35285-35315.
3	Embankment	River Glyde	Left Bank, approx. model chainage 35440-37000.
4	Embankment	River Glyde	Right bank, approx. model chainage 36170-37830.
5	Embankment	River Glyde	Left Bank, approx. model chainage 37440-37830.
6	Embankment	River Glyde	Left Bank, approx. model chainage 37865-38160.

7	Embankment	River Glyde	Left Bank, approx. model chainage 38350-39170.
8	Embankment	River Glyde/ River Dee	Right bank, approx. model chainage 37940-38830 River Glyde and left bank 26930-28510 River Dee.
9	Embankment	River Dee	Right bank, approx. model chainage 26640-27630.
10	Embankment	River Dee/ River Glyde	Right Bank, approx. model chainage 28030-28510 River Dee and 38920-39460 River Glyde.
11	Embankment	N/A	Parallel to Harbour Road.
12	Sea Wall	N/A	Parallel to Harbour Road. (Please view Section 4.8.5(2) which discusses model updates for Final)
13	Embankment	N/A	Adjacent to The Saltings Apartments.
14	Sea Wall	N/A	Parallel to Strand Road.
15	Embankment	N/A	Parallel to Strand Road.

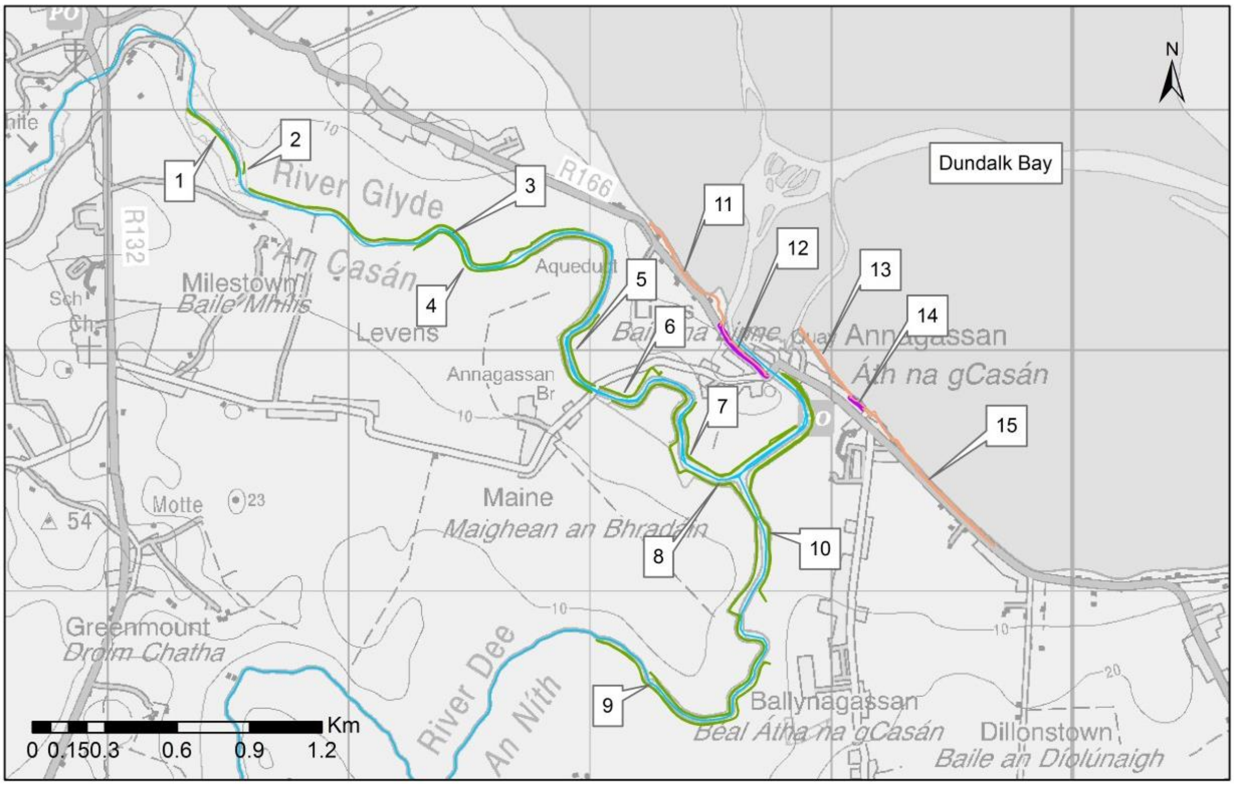


Figure 4.8.11: Location of Formal Defences in Annagassan

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the UoM 06 Hydrology Report IBE0700p0008_HA06 Hydrology Report_D01 - Section 4.8 and Appendix D). The boundary conditions implemented in the model are shown in Table 4.8.1.

Table 4.8.1: MIKE11 Model Boundary Conditions

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Open	Inflow	River Glyde	15805.582	0		06014_RA
2	Open	Inflow	River Dee	15087.086	0		06013_RA
3	Point Source	Inflow	River Dee	21609.36	0		06_550_5_RA
4	Point Source	Inflow	River Dee	22318.06	0		06_1050_3_RA
5	Distributed Source	Inflow	River Dee	19707.05	26081.6		Top-up flow between 06013_RA & 06_1099_8_RA
6	Open	Water Level	River Glyde	39666.1285	0		06_848_D_RARPS
7	Distributed Source	Inflow	River Dee	26081.6	28512		Top-up flow between 06_1099_8_RPS & 06_1100_1_RA
8	Point Source	Inflow	River Glyde	17881.25	0		06_76_2_RA
9	Point Source	Inflow	River Glyde	21272.65	0		06_73_2_RA
10	Point Source	Inflow	River Glyde	21842.74	0		06_70_3_RA
11	Point Source	Inflow	River Glyde	26323.83	0		06_276_13_RA
12	Distributed Source	Inflow	River Glyde	17862.91	28303.19		Top-up between 06014_RA & 06_603_Inter_1_RA
13	Distributed Source	Inflow	River Glyde	28303.19	28912.22		Top-up between 06_603_Inter_1_RA & 06021_RA
14	Distributed Source	Inflow	River Glyde	28912.22	30954.87		Top-up between 06021_RA & 06_1097_2_RA
15	Distributed Source	Inflow	River Glyde	30954.87	34213.8		Top-up between 06_1097_2_RA & 06052_RA
16	Distributed Source	Inflow	River Glyde	34213.8	39614		Top-up between 06052_RA & 06_848_D_RARPS

Figure 4.8.12 shows the 1% AEP design hydrographs for the upstream inputs to River Dee and River Glyde at HEPs 06013_RA and 06014_RA respectively. These hydrographs were derived based on observed data at the hydrometric stations in accordance with FSU Work Package 3.1 and as detailed in the UoM 06 Hydrology Report (Rp0008) Chapter 6.

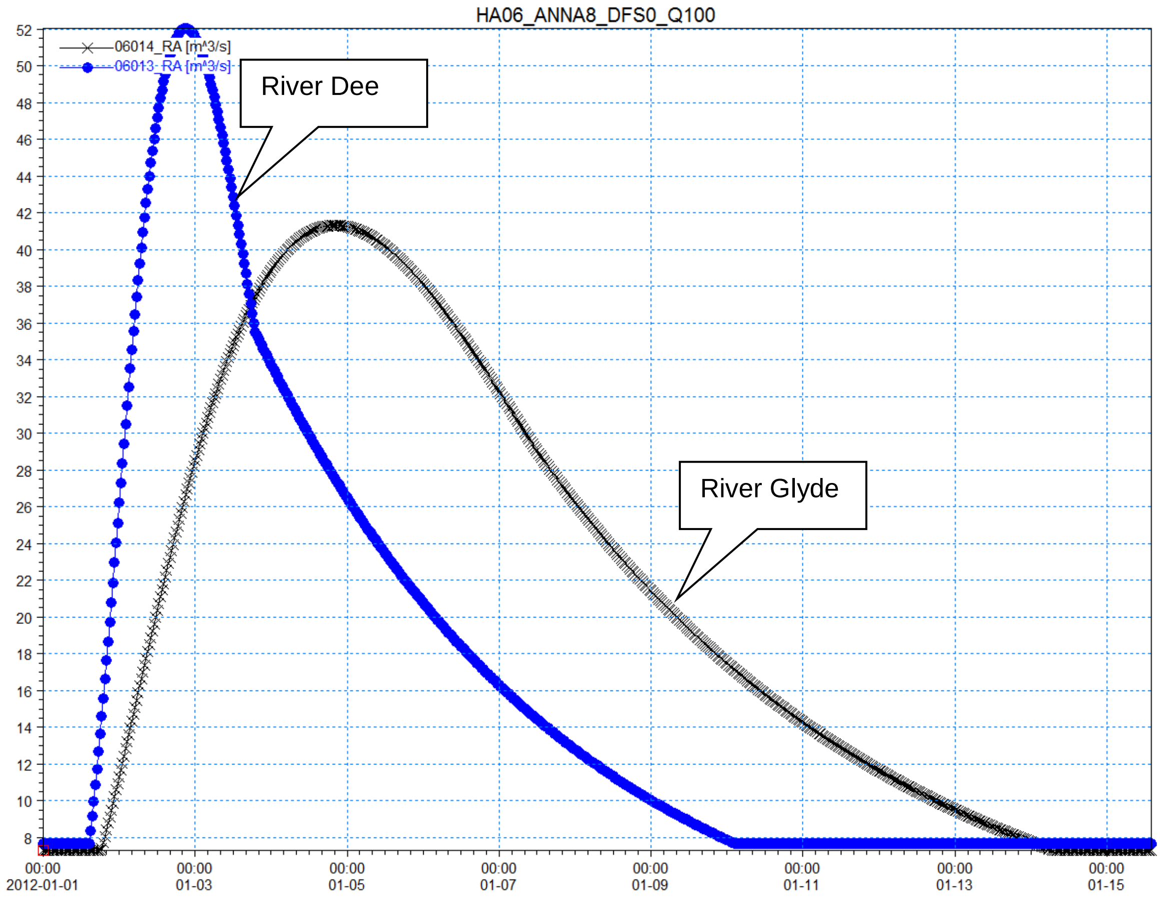


Figure 4.8.12: Upstream Inflow Design Hydrographs on River Glyde and River Dee (1% AEP)

To achieve anchoring of model flows to estimated flows following initial model runs input hydrographs for Glyde tributaries at HEP 06_550_5_RA and 06_1050_3_RA were delayed by 3 hours and 6 hours

respectively. This significant delay in the hydrograph timing was essential in ensuring the peak flow and design hydrograph were achieved within the model downstream on the main channel of the River Glyde at check points HEPs 06_1100_1_RA and 06_848_D_RARPS(refer to Appendix A.3).

Outputs from the Irish Coastal Protection Strategy Study (ICPSS) have resulted in extreme tidal and storm surge water levels being made available around the Irish Coast for a range of AEPs. The locations of the ICPSS nodes along with the relevant AFA locations are shown in Figure 4.8.13. The associated AEP water levels for the relevant node at Annagassan are shown in Table 4.8.2. The coastal boundary for this model is set across Dundalk Bay; therefore the values for node NE_26 were used for the still water inundation modelling in Annagassan.

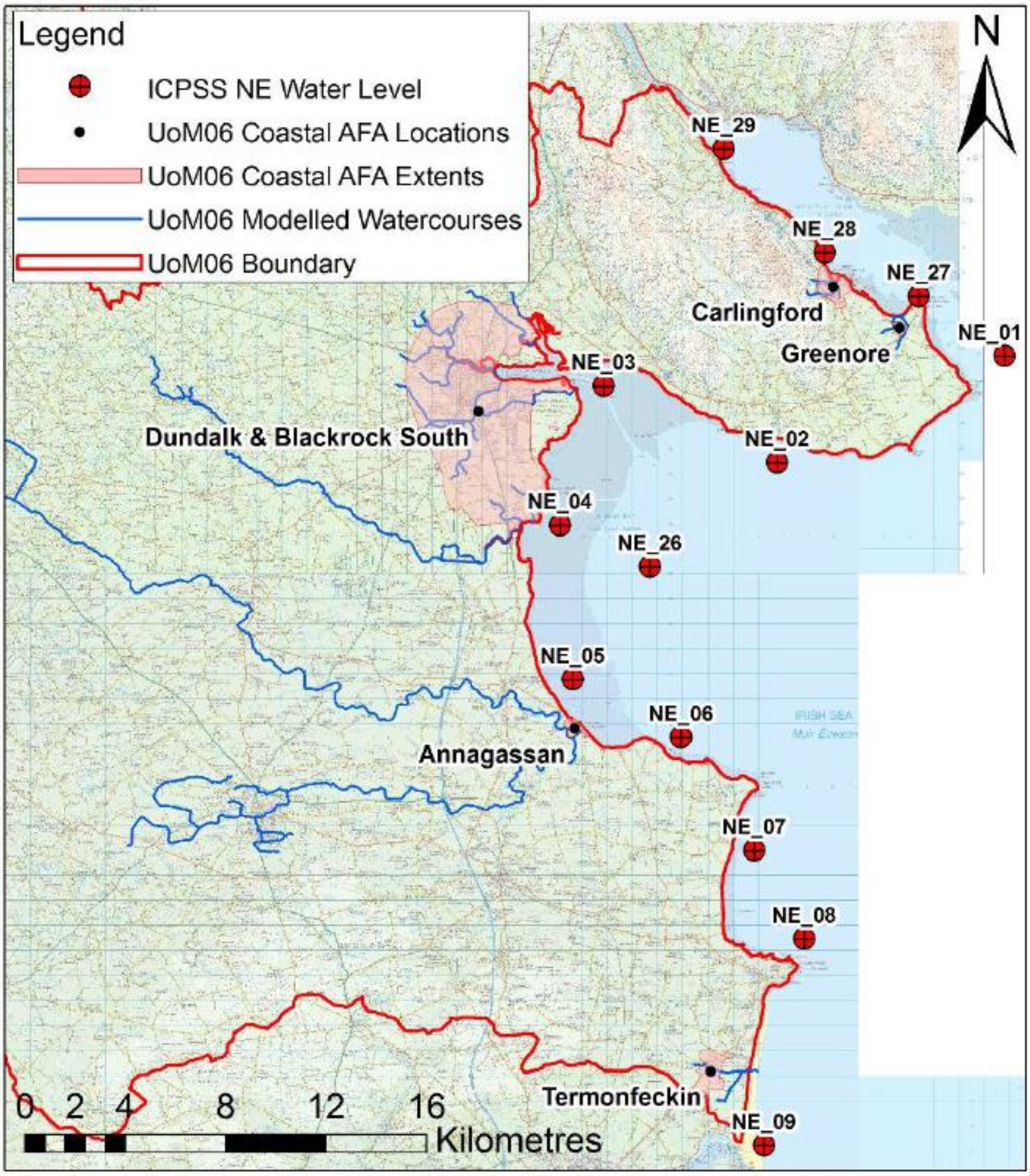


Figure 4.8.13: ICPSS Node Locations (IBE0700Rp0008_UoM06 Hydrology Report_F01)

Table 4.8. 2: ICPSS AEP Total Water Levels for Relevant Model Node

ICPSS Node	**Annual Exceedance Probability (AEP) %**							
	50	**20**	**10**	**5**	**2**	**1**	**0.5**	**0.1**
	Highest Tidal Water Level to OD Malin (m)							
NE_26	3.04	3.17	3.27	3.37	3.51	3.61	3.71	3.94

The ICPSS water levels are total water levels, comprising tidal and surge components which together yield a joint probability event of a particular AEP.

A representative tidal profile for Dundalk Bay was generated based on Admiralty Tide Table data for Soldiers Point.

A normalised 48 hour surge profile was scaled based on the difference between the peak water level of the generated tidal profile and the target extreme water level from the table above. The scaled surge profile was then appended to the tidal profile to achieve a representative combined tidal and storm surge profile for the required AEP events. Figure 4.8.14 illustrates the tidal profile, storm surge profile and resultant combined water level profile.

The water level profile was applied as a level boundary to the eastern edge of the 2D domain, representing Dundalk Bay.

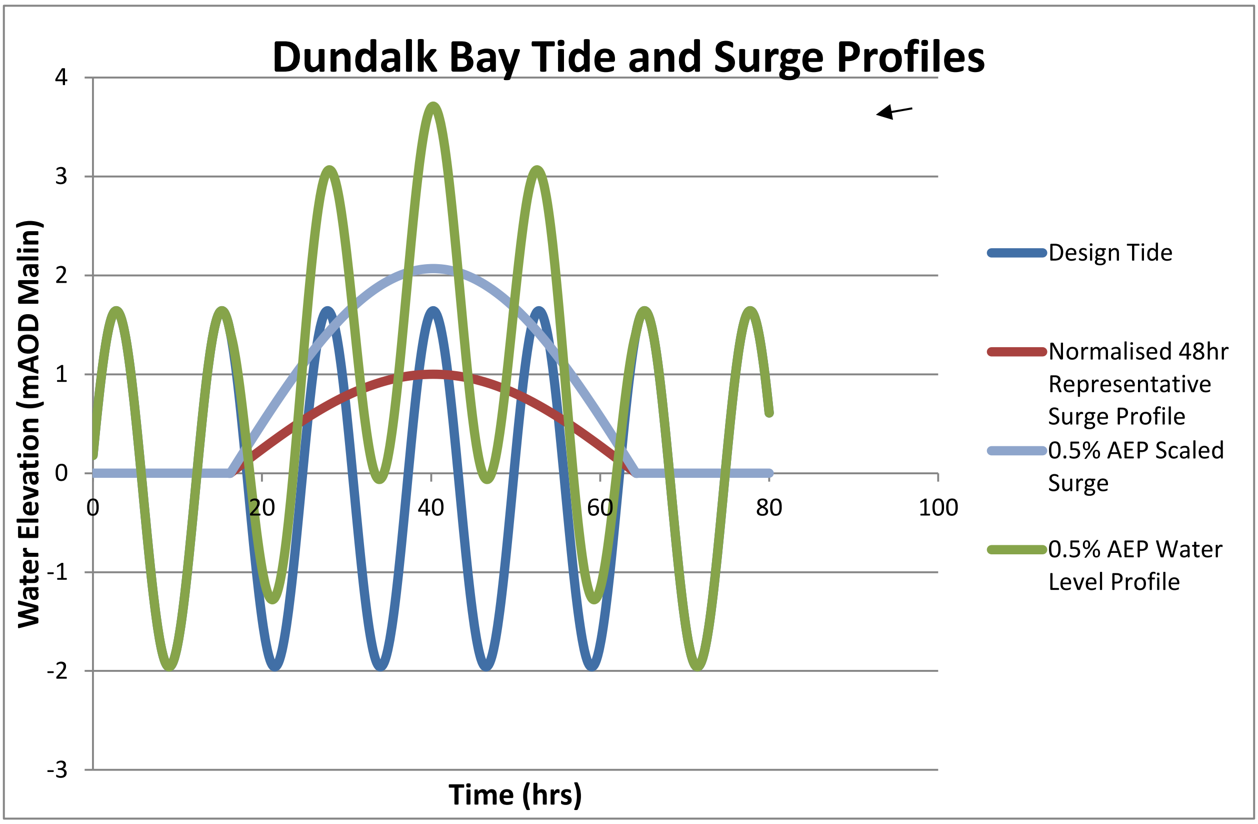


Figure 4.8.14: Tidal, Surge and Total Water Level Profiles for the Annagassan Model Boundary at 0.5% AEP

Assessment of the dependence between fluvial and coastal extreme flows was undertaken during hydrological analysis as detailed in the UoM06 Hydrology Report (Rp0008_F01, Chapter 6.3.2). Initial screening indicated that there was a large overlap between the PFRA flood extents and the ICPSS flood extents within Annagassan AFA. This was further investigated by looking at available data to ascertain if there was a dependence relationship. The long term tidal water level gauge record at Dublin was compared with matching time series flow data at Charleville (06013) and Tallanstown (06014) hydrometric stations on the River Dee and Glyde respectively. No obvious correlation was detected from the scatter plots and the dependence was found to be negligible. As such, a simplified conservative approach was applied to the model whereby the 50% AEP coastal design event was used with each fluvial design event (all % AEPs) and vice versa. This will be sensitivity tested prior to finalising the models and flood maps.

Wave Overtopping Model

To simulate 'mechanism 2 wave overtopping' flooding at the Annagassan AFA, data from the ICWWS was used including peak shoreline water levels and wave heights, periods and directions for each AEP event. The locations at Annagassan for which this data was calculated are shown in Figure 4.8.15. An example of this data for the Annagassan AFA is shown below in Table 4.8.3.

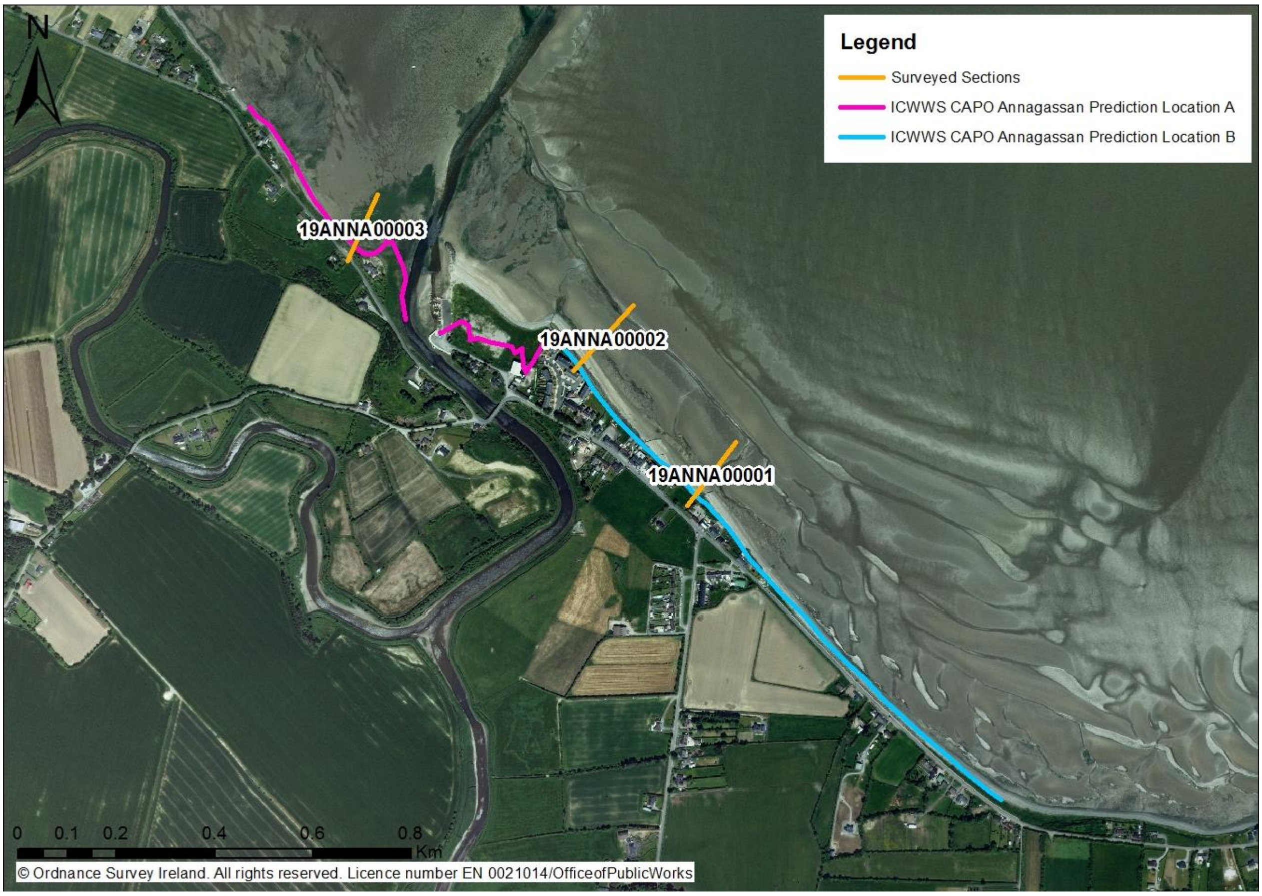


Figure 4.8.15: ICWWS CAPO Annagassan Prediction Location and Topographic

Table 4.8.3: ICWWS CAPO Annagassan Wave Climate and Water Level Data

Prediction Location Reference: Annagassan_Location A				
Bed Level 0.79m OD Malin				
		Combined Wave Component		
AEP	**WL (OD Malin)**	**Hm0 (m)**	**Tp (s)**	**MWD (°)**
0.1%	2.304	0.748	5.841	63
0.1%	2.534	0.823	5.526	66
0.1%	2.764	0.903	5.484	68
0.1%	2.974	0.975	5.524	69
0.1%	3.204	1.046	5.472	70
0.1%	3.534	0.960	5.171	70

To calculate the overtopping discharge rate for each scenario at various locations along the shoreline, the Neural Network overtopping calculator tool outlined by the EurOtop manual was used in addition to levels of the structures to be overtopped. The locations of the three surveyed sections taken at Annagassan are shown in Figure 4.8.15. All of the surveyed profiles at Annagassan are armoured slopes, and section 19ANNA00002 also contains a small vertical wall at the crest of the slope.

The largest calculated discharge rate out of the six possible combinations of water levels and wave heights, periods and directions was used for each design AEP event. When the peak discharge rate was less than 0.03l/s/m, no further analysis was required. The locations of the final discharge boundaries within the model are shown in Figure 4.8.16. None of the boundary locations analysed in Annagassan were found to have a peak discharge less than the threshold value of 0.03l/s/m during design runs of 0.5% AEP and 0.1% AEP; so all locations were taken forward for modelling. The peak discharge at boundary 19ANNA00002 was found to be below the threshold value during design runs of 10% AEP.

Once the discharges for simulation had been ascertained, an idealised water level profile was produced in order to calculate the discharge rate across the tidal cycle, as the rate determined by EurOtop was specific to the peak water level only. A storm duration of 12 hours beginning and ending at low-water was assumed. The discharge rate profile was then scaled based on the length of the exposed shoreline in order to produce a discharge profile in m^3/s, as shown in Table 4.8.4 and Figure 4.8.17

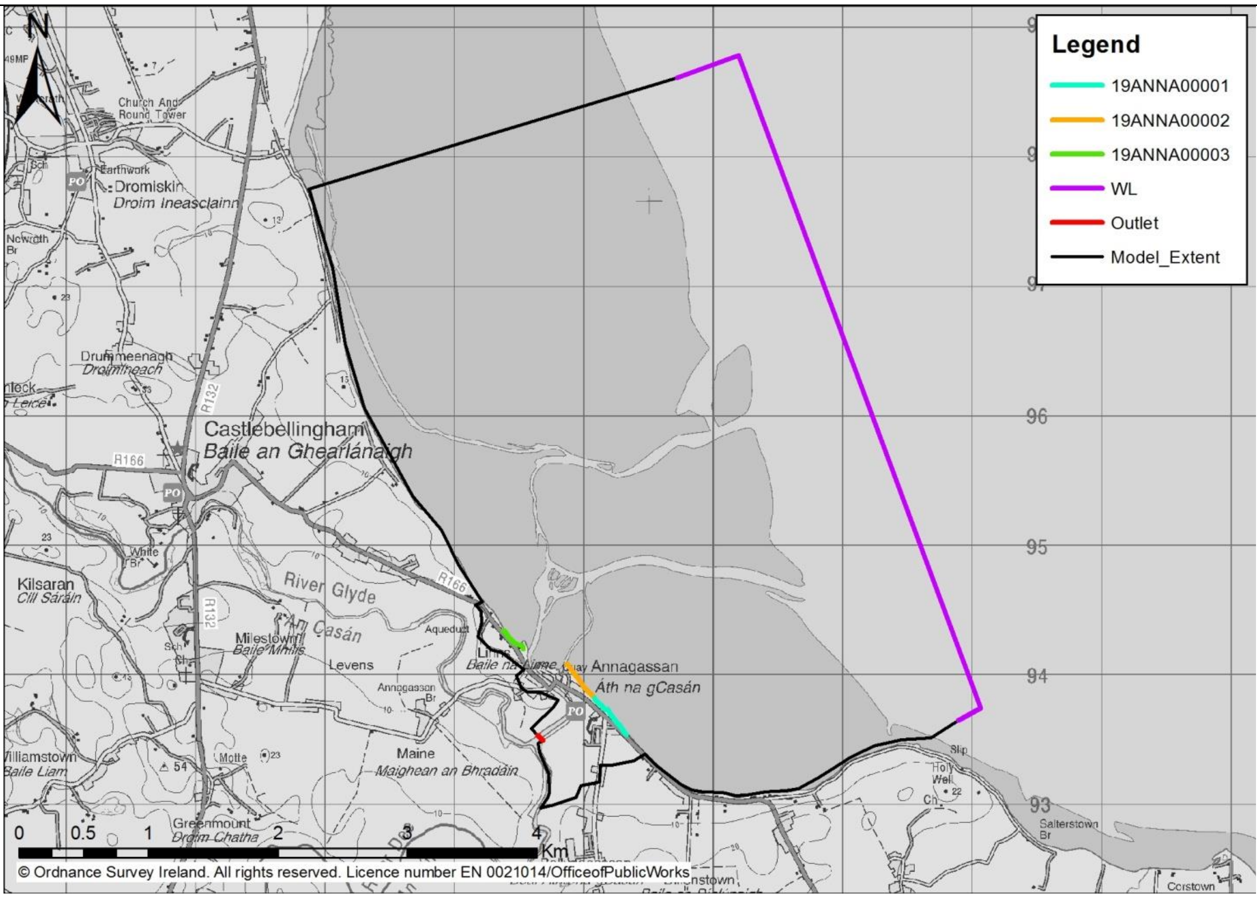


Figure 4.8.16: Annagassan Modelled Wave Overtopping Locations

Table 4.8.4: Peak Wave Climate and associated Wave Overtopping Discharges for Modelled Sections

Boundary	AEP	WL (OD Malin)	Hm0 (m)	Tp (s)	MWD (°)	Discharge Rate (l/s/m)	Discharge (m^3/s)
19ANNA00001	**10%**	2.864	0.718	4.682	62	0.085	**0.032**
19ANNA00001	**0.5%**	3.304	0.895	5.206	63	1.973	**0.753**
19ANNA00001	**0.1%**	3.534	0.979	5.428	64	7.480	**2.857**
19ANNA00002	**0.5%**	3.304	0.895	5.206	63	0.278	**0.095**
19ANNA00002	**0.1%**	3.534	0.979	5.428	64	1.459	**0.501**
19ANNA00003	**10%**	2.864	0.696	4.463	68	0.133	**0.030**
19ANNA00003	**0.5%**	3.304	0.872	4.956	69	8.829	**2.004**
19ANNA00003	**0.1%**	3.534	0.960	5.171	70	44.150	**10.022**

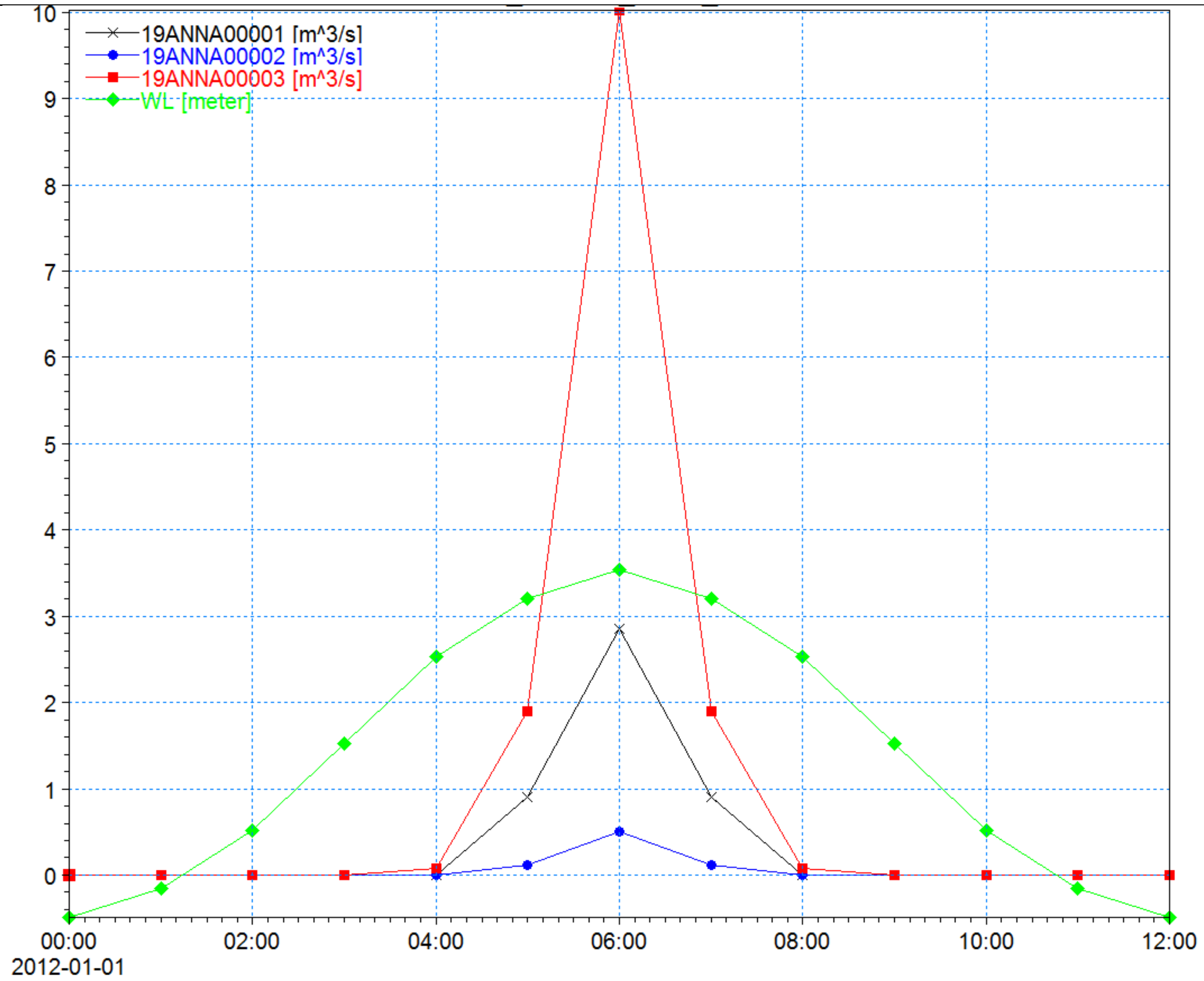


Figure 4.8.17: Wave Overtopping Discharge Profiles at Annagassan for the 0.1% AEP event

The water level boundary 'WL' was applied to simulate tidal levels in Dundalk Bay and therefore model flooding arising from the combination of waves and the ICWWS water level. An Outlet boundary 'Outlet' was also applied to the edge of the mesh at the location of the River Glyde in order to prevent water hitting the edge of the mesh and unrealistic flood depths being calculated.

(6) Model Boundaries – Downstream Conditions:	Water level boundaries were applied at the downstream extent of the River Glyde where it discharges to the 2D model domain (chainage 3966.1285m). This boundary is given a 'dummy' water level value of 0mOD Malin. This value is ignored once the simulation commences and the level of this boundary varies in time based on calculations driven by interaction of the water levels in the River Glyde and Dundalk Bay at that location.	
(7) Model Roughness: (see Section 3.5.1 'Roughness Coefficients')		
(a) In-Bank (1D Domain)	Minimum 'n' value: 0.030	Maximum 'n' value: 0.050
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.030	Maximum 'n' value: 0.060

(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.033 (Inverse of Manning's 'M')	Maximum 'n' value: 0.034 (Inverse of Manning's 'M')

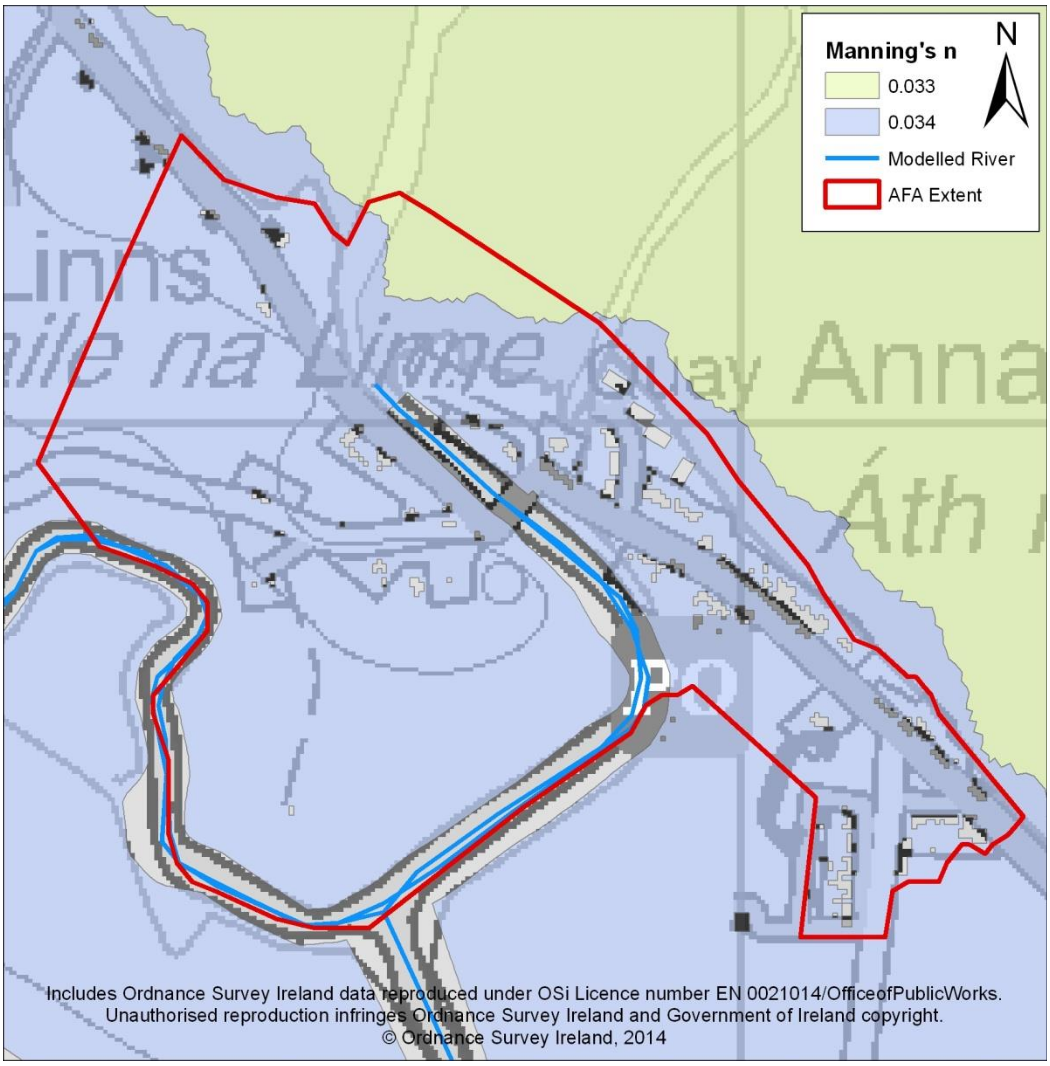


Figure 4.8.18: Map of 2D Roughness (Manning's n)

Figure 4.8.18 illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. Null Manning's n values on inland waterbodies were corrected to Manning's n of 0.033. Any values seaward of the high water mark were taken as 0.033 unless otherwise specified.

(d) Examples of In-Bank Roughness Coefficients:

Figure 4.8.19: ***River Glyde - 0601M02163_UP***

Manning's n = 0.03

Natural stream - clean, straight, full stage, no rifts or deep pools.

Figure 4.8.20: ***River Glyde - 0601M00587_UP***

Manning's n = 0.04

Natural stream - clean, winding, some pools and shoals.

Figure 4.8.21: ***River Dee - 0602M00693_UP***

Manning's n = 0.03

Natural stream - clean, straight, full stage, no rifts or deep pools.

Figure 4.8.22: ***River Glyde Tributary - 0601A00040_UP***

Manning's n = 0.03

Natural stream - clean, straight, full stage, no rifts or deep pools.

4.8.4 Sensitivity Analysis

To be completed for final version of report.

4.8.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0700Rp0003_UoM 06 Inception Report_F02 unless otherwise specified):	
(a) Oct 2004.	Details were found on www.floodmaps.ie which indicated that flooding occurred in Annagassan and Dundalk in October 2004. In Annagassan, high tides and wave action caused coastal flooding. Strand Road was flooded. However no further details were available. The nearest tidal gauge with data for 2004 is Dublin Port, which is approximately 60km South of Annagassan. The date of this event is not given, but the highest water level at Dublin Port in October occurred on the 27th with a level of approximately 2.4mOD Malin. This total water level equates to an AEP of approximately 50% at ICPSS point NE_22, which is close to Dublin Port. The peak flow at gauging station 06013 (River Dee) was 14.9m^3/s and occurred on the 29th October. The peak flow at gauging station 06014 (River Glyde) was 12m^3/s and occurred on the 30th October. These flows are considerably more frequent than a 50% AEP fluvial event. Flow data at station 06021 (River Glyde) for October 2004 is not available. Coastal flooding at Strand Road is recreated in the model during the 0.1% AEP due to inundation via the embankment on the right bank of the River Glyde (see Figure 4.8.23). The flood event in October 2004 is described as being due to a combination of high tides and wave action, so it is likely that flood water overtopped the coastal defences during this event. Also the total water level at Dublin Port indicated a coastal surge of approximately 50% AEP, it is reasonable to assume that the predominant cause of flooding during the October 2004 event was wave overtopping. The wave overtopping 10% AEP design event results indicate that waves do overtop the coastal defences at Annagassan. The results also indicate flooding of Strand Road as reported, see Figure 4.8.23. This provides qualitative support for the model.

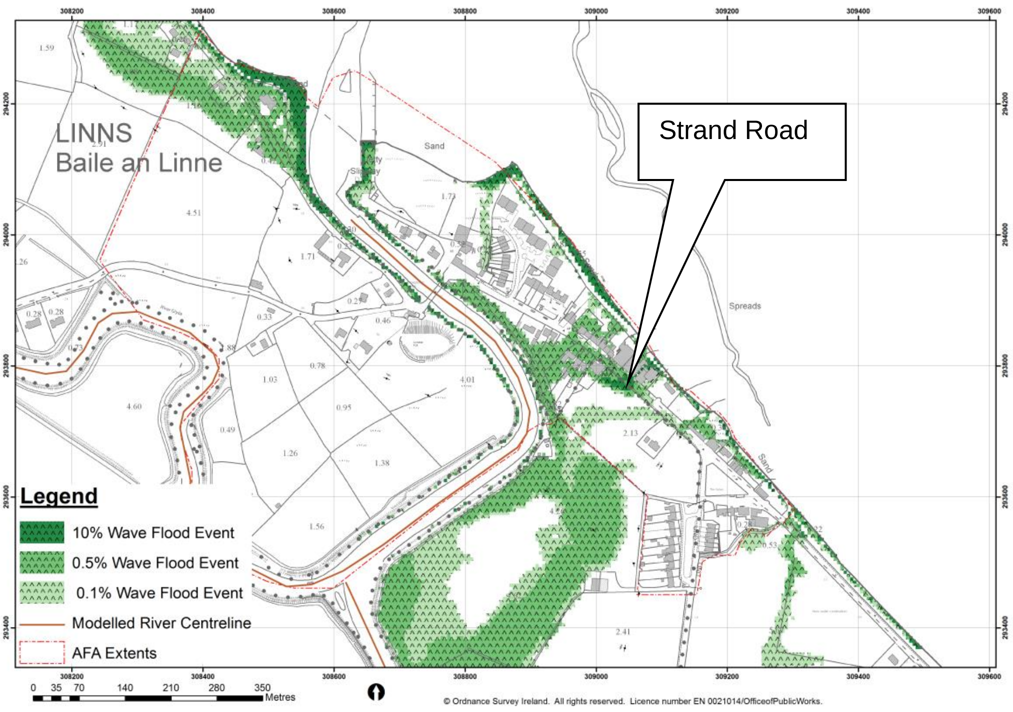


Figure 4.8.23: Coastal flooding due to wave overtopping at Strand Road

(b) Feb 2002.

Information was found on www.floodmaps.ie indicating that flooding occurred on 2nd February 2002 in Annagassan, Carlingford and Dundalk & Blackrock South due to heavy rain, high tides and strong easterly winds. Photographs of flooded farmland in Annagassan were found dated 2nd February 2002, following breaching of an embankment at two locations.

The peak flow at gauging station 06013 (River Dee) on 2nd February was 19m^3/s. The flow at gauging station 06021 (River Glyde) on 2nd February was around 15m^3/s, although this continued to increase before peaking at 16.5m^3/s on 4th February. The flow at gauging station 06014 (River Glyde at model upstream limit) on 2nd February was approximately 16m^3/s, although this continued to increase before peaking at 19.7m^3/s on 6th February. These flows are of higher frequency than a 50% AEP fluvial event.

Design rainfall frequency was estimated using the FSU DDF model (FSU WP 1.2 'Estimation of Point Rainfall Frequencies'). No hourly rainfall gauges are located close to Annagassan. The closest hourly gauge with data for 2002 is Dublin Airport, which recorded 10.8mm of rain in a three hour period on 2nd February. However this only equates to a rainfall event with an AEP of around 90% when applied to the Dee/Glyde catchment. Data from the daily rainfall gauge at Castlebellingham Lynns was also interrogated due to the flat nature of the catchment. Rainfall on the catchment was analysed over a 12 day duration preceding the flood event, with 80.7mm of rain falling in this period. The rainfall event was found to have an AEP of around 77%.

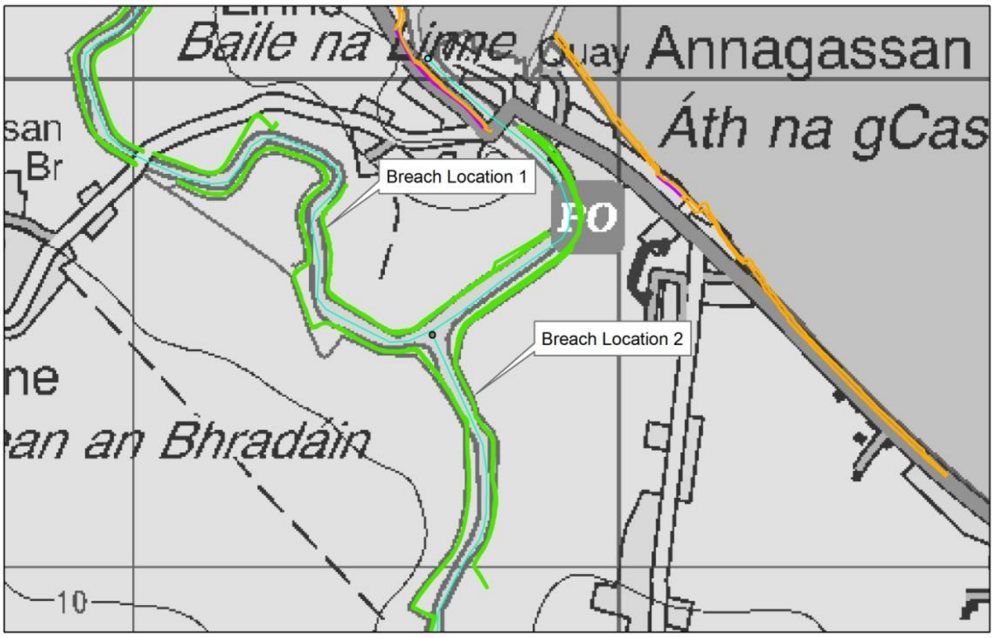
	Tidal gauge data at Dublin Port was also analysed for this event. A water level of approximately 2.75 mOD Malin was recorded on 1st February, which equates to an AEP of around 5% at ICPSS point NE_22 at Dublin Port. The fluvial element to this event is relatively low, so it is probable that the main cause to flooding was due to high tidal levels causing damage to embankments. The location of the breaches according to www.floodmaps.ie is shown in Figure 4.8.24.  **Figure 4.8.24: Embankment breach locations** Photographs of the breaches do not show flood extents as they appear to have been taken after flood waters have receded, as shown in Figure 4.8.25 and Figure 4.8.26.

Figure 4.8.25: Breach location 1

Figure 4.8.26: Breach location 2

The coastal event had an AEP of approximately 5% at Dublin Port, so it is reasonable to assume that the magnitude of the event was similar at Annagassan. A direct comparison between the model flood extents and the reported flooding cannot be

made as flooding occurred due to breaches in the embankments. The land adjacent to breach location 1 floods during the 0.5% AEP coastal model and the land adjacent to breach location 2 floods during the 0.1% AEP coastal model however, so it is expected that flooding would occur during a 5% AEP event if the embankment was damaged.

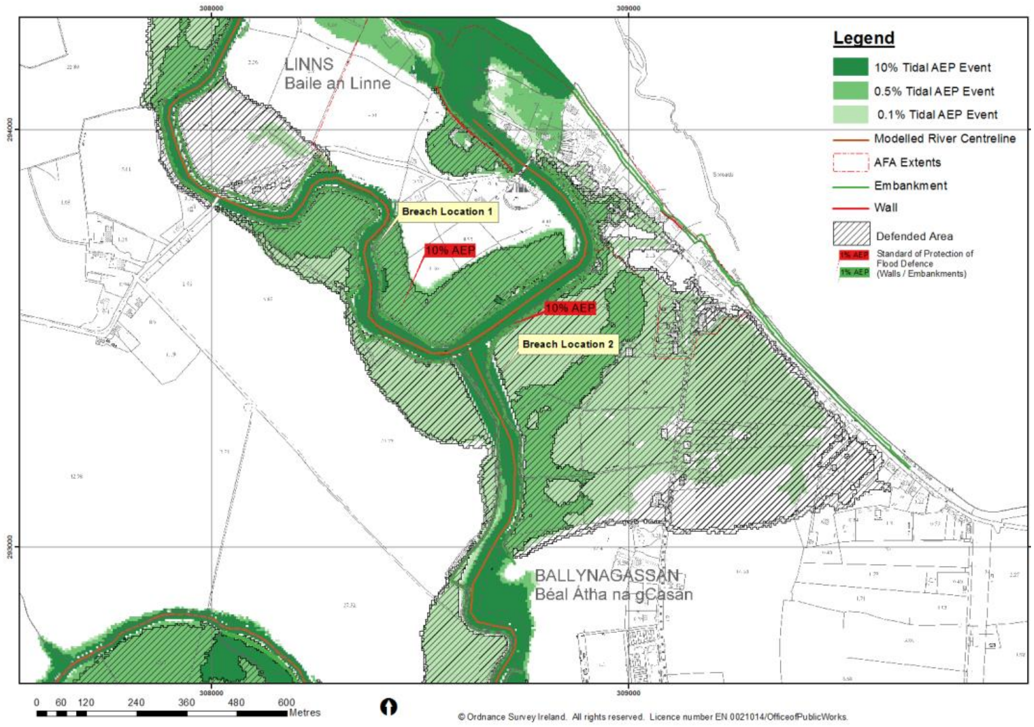


Figure 4.8.27: Coastal Flood Extents at Annagassan

This also demonstrates that this land would be prone to flooding if it was not protected by embankments. Calibration of this event will be further reviewed during breach analysis.

(c) Dec 1981.

Flooding occurred in Annagassan, Carlingford and Dundalk & Blackrock South on 3rd December 1981 due to heavy rainfall, high tides and strong winds. In Annagassan, the sea came up over the road, flooding houses and business premises. The Dundalk Democrat stated that "it was the highest flood level in living memory".

The peak flow at gauging station 06013 (River Dee) on 3rd December was 5.5 m^3/s. The peak flow at gauging station 06021 (River Glyde) on 3rd December was 7.1m^3/s. The peak flow at gauging station 06014 (River Glyde, upstream extent of model) on 3rd December was 9.3m^3/s. These flows are of greater frequency than a 50% AEP fluvial event.

Hourly rainfall gauges at Dublin Airport and Clones recorded no rainfall on 3rd December 1981. Castlebellingham daily rainfall gauges do not have data available for 1981, so the nearest daily gauge with data is at Ardee. 30.6mm of rainfall fell over a

	16 day period and a design rainfall frequency was estimated using the FSU DDF model. This rainfall event was found to have an AEP of 100%. From gauging station records and rainfall data it is clear that this event was not fluvially driven. No tidal gauge data was available for this event so it was not possible to estimate an AEP of the coastal water level. The report suggests that coastal inundation occurred and flooded a road, although it does not state what road. The report states that this flooding affected houses and businesses, so it is likely that the affected road was Strand Road, although may have been Harbour Road. Coastal inundation affecting Strand Road and adjacent properties was found to occur during design events of 0.5% and 0.1% AEP due to the embankment on the right bank of the River Glyde being overtopped and flood water entering from the coast west of the Saltings Apartments. This is shown in Figure 4.8.23 As previously discussed for the October 2004 flood event the wave overtopping 10% AEP design run indicates that waves do overtop the coastal defences at Annagassan. The results also indicate flooding of Strand Road as reported, see Figure 4.8.23. This provides qualitative support for the model. The Saltings Apartments were constructed around 2005. Information found on www.boards.ie suggests that this site was historically prone to flooding, but one property owner suggests there has been no flooding at the site due to construction of coastal defences as part of the development of this site.
Comparison with ICPSS	
A comparison was made between the modelled 0.5% AEP extents and the ICPSS 0.5% AEP extents as shown in Figure 4.8.28 overleaf. It can be seen that the modelled flood extents and the ICPSS mapped extents are in good agreement in the lower Ballynagassan area. However throughout the model extent there are significant differences between with the ICPSS outlines showing significantly more flooding of Annagassan. This is considered to be due to how the ICPSS flood extents are mapped. These are a level projection onto a digital terrain model and as such the ICPSS is not capable of taking into account barriers / restriction to flow inland and temporal effects. In the case of the centre of Annagassan, propagation of coastal flood waters up the Glyde and Dee Rivers is likely to be partially contained in-channel by the embankments which are represented in the CFRAMS 1D-2D model. Propagation of flow inland is not modelled in ICPSS and as such the CFRAM Study model represents a more detailed analysis of inland flooding.	

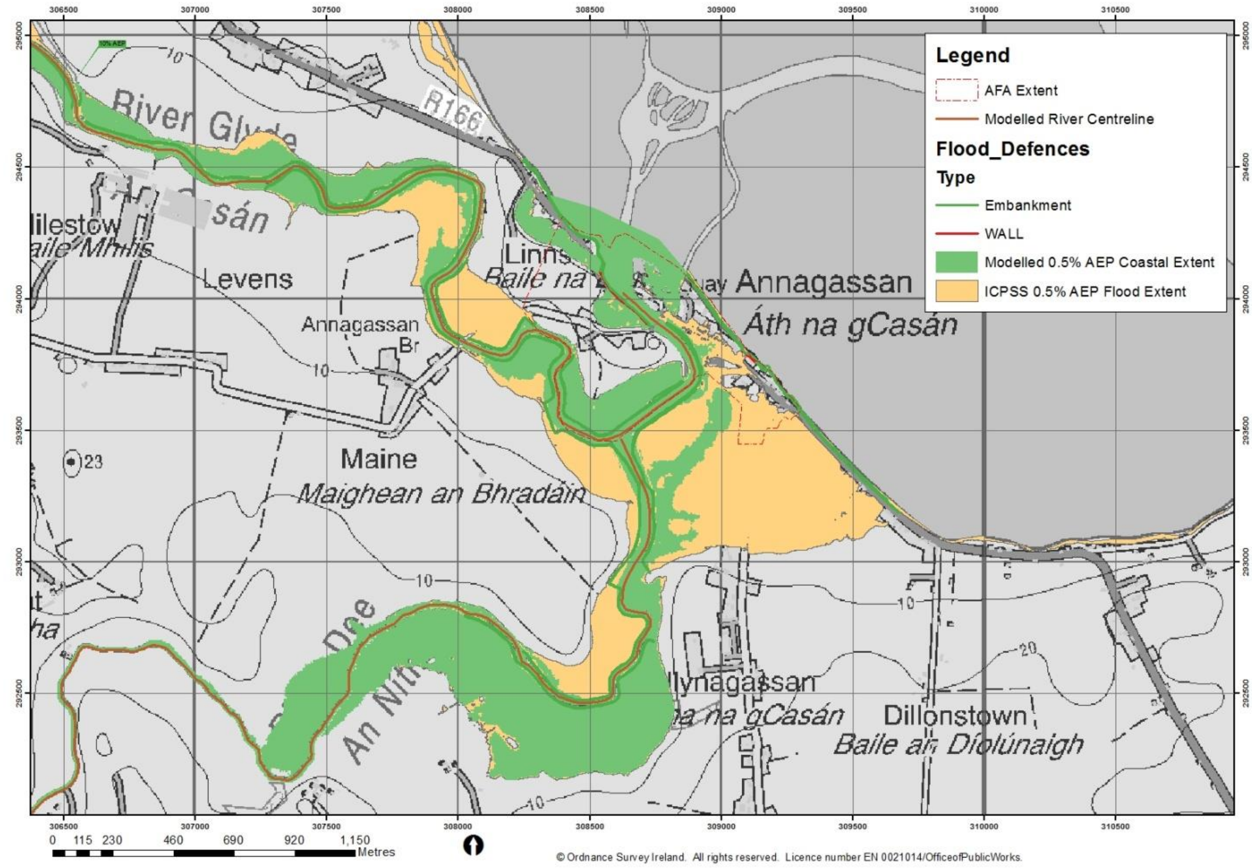


Figure 4.8.28: Comparison of ICPSS extents with modelled extents for 0.5% AEP event

Summary of Calibration

The fluvial component of the flood events in 2004, 2002 and 1981 were all estimated using flow data at gauging stations 06013 (River Dee) and 06021 and 06014 (River Glyde). Daily and hourly rainfall data was also used to estimate a rainfall frequency using the FSU DDF where possible. In each case, the fluvial component was found to equate to an event with a frequency greater than 50% AEP.

Tidal gauge data at Dublin Port was used to estimate the frequency of the coastal water level for the 2004 and 2002 events at 50% and 5% AEP respectively. This is useful to give an indication of the magnitude of the event, but the true magnitude may differ as Dublin Port is approximately 60km away from the area of interest.

The rating curves and spot gaugings at stations 06013, 06014 and 06021 were all used to calibrate the model at these locations. Good correlation between the existing OPW rating curve and the rating curve produced by the model was achieved at all stations, as described in section 4.8.5(4).

Model flows were checked against the estimated flows at HEP check points (intermediate, downstream limit and gauging station HEPs) where possible to ensure anchoring of the model to hydrological estimates and gauged flows. For example at HEP 06_603_Inter_1_RA, the estimated flow during the 1% AEP event is 45.02m^3/s and the modelled flow is 45.79m^3/s. The flow at downstream HEP 06_848_D_RARPS could not be checked during the final runs as it is in a tidal reach. Full flow tables can be found in Appendix A.3

The mass error in the 1D and 2D models combined were calculated for each scenario to ensure they were within an acceptable range. Refer to Chapter 3.11 for details of acceptable limits. The table below summarises the mass errors of each model run which are negligible and indicate that the model is robust.

Table 4.8.5: Model Mass Balance

Model	Mass Error
10% AEP Fluvial	0.00%
1% AEP Fluvial	0.02%
0.1% AEP Fluvial	0.03%
10% AEP Coastal	0.04%
0.5% AEP Coastal	0.05%
0.1% AEP Coastal	0.05%

Due to the lack of flood event data available, it was only possible to conduct a limited verification exercise on this model. However the model is shown to be a reasonable representation of the flood mechanisms described from the available flood event records and considered to be performing satisfactorily for design event simulation.

(2) Post Public Consultation Updates

All recorded comments were investigated following informal public consultation and formal S.I. public consultation periods in 2015. Model updates were applied to increase flood risk to a property on the leeward side of the wall along Harbour Road. The wall has been deemed ineffective and was therefore removed from the model.

These changes resulted in increased flooding at Harbour Road as shown below in Figure 4.8.29. The model results are further supported by the Key Historical Flooding discussed in Section 4.6.5 (1). The model was updated and check flows recalculated with a revised set of flood hazard and risk mapping issued as Final to reflect this change.

Figure 4.8.29: Increased flooding at Harbour Road

(3) Standard of Protection of Existing Formal Defences:	**Type**	**Watercourse**	**Bank**	**Modelled Standard of Protection (AEP)**
Defence Reference	Embankment	River Glyde	Right	10% AEP still water level
1	Embankment	River Glyde	Left	10% AEP still water level
2	Embankment	River Glyde	Left	<10% AEP still water level
3	Embankment	River Glyde	Right	10% AEP still water level
4	Embankment	River Glyde	Left	0.5% AEP still water level
5	Embankment	River Glyde	Left	0.5% AEP still water level
6	Embankment	River Glyde	Left	10% AEP still water level
7	Embankment	River Glyde/ River Dee	Right/ Left	10% AEP still water level
8	Embankment	River Dee	Right	<10% AEP still water level
9	Embankment	River Dee/ River Glyde	Right	10% AEP still water level
10	Embankment	N/A	N/A	<10% AEP still water level
11	Sea Wall	N/A	N/A	10% AEP still water level

12	Embankment	N/A	N/A	0.1% AEP still water level (Please view Section 4.8.5(2) which discusses model updates for Final)
13	Sea Wall	N/A	N/A	0.1% AEP still water level
14	Embankment	N/A	N/A	0.1% AEP still water level
15	Embankment	N/A	N/A	0.1% AEP still water level

There are 15 formal wall and embankment defences in Annagassan, as shown in Figure 4.8.11. Defences 1-7 are embankments situated along the Glyde River and prevent both fluvial and coastal flooding in the Castlebellingham, Lynn and Maine areas with crest levels of circa 3.0-4.2m OD Malin. These defences in total are 18.5 km in length. (Please view Section 4.8.5(2) which discusses model updates for Final)

Defences 8-10 are embankments situated along the Dee and along the right bank of the lower reaches of the Glyde. These defences protect the Maine and Ballynagassan areas from coastal and fluvial flooding, with elevations of 2.9-4.1 m OD Malin and a length of 9.2 km in total.

Defences 11-15 are a combination of walls and embankments along the coastline at Annagassan. These defences protect the Annagassan town area from coastal inundation, with elevations of 2.9-5.2 m OD Malin and a length of 5.2 km in total. (Please view Section 4.8.5(2) which discusses model updates for Final)

To simulate an undefended scenario, all defences were removed from the 1D and 2D elements of the model. This was achieved by removing the walls where relevant from the 1D model, whilst refining the LiDAR data in the 2D model to ensure no sections of the walls were picked up by the mesh.

Figure 4.8.30 to Figure 4.8.32 show that the flood defence would reduce the flood risk of an area predominately during 10% AEP flood event. The grey hatching identifies the area that would flood during a particular AEP event if the defence was removed.

The majority of the embankments adjacent to the River Glyde and River Dee were found to have a SoP of 10% AEP still water level, while the coastal embankments and walls (Defence Reference 13-15) were found to have an SoP of 0.1% AEP still water level. The critical flood mechanism on these defences is wave overtopping, see Figure 4.8.30

A large area of Ballynagassan was found to benefit from embankment Defence Reference 10, as shown in Figure 4.8.30. This embankment, which was found to have a SoP of 10% AEP still water level, protects numerous properties, as well as the L6225 road.

The coastal embankment reference 11 was found to have a low point, reducing its SoP to less than 10% AEP. This flooding may affect properties and the Harbour Road.

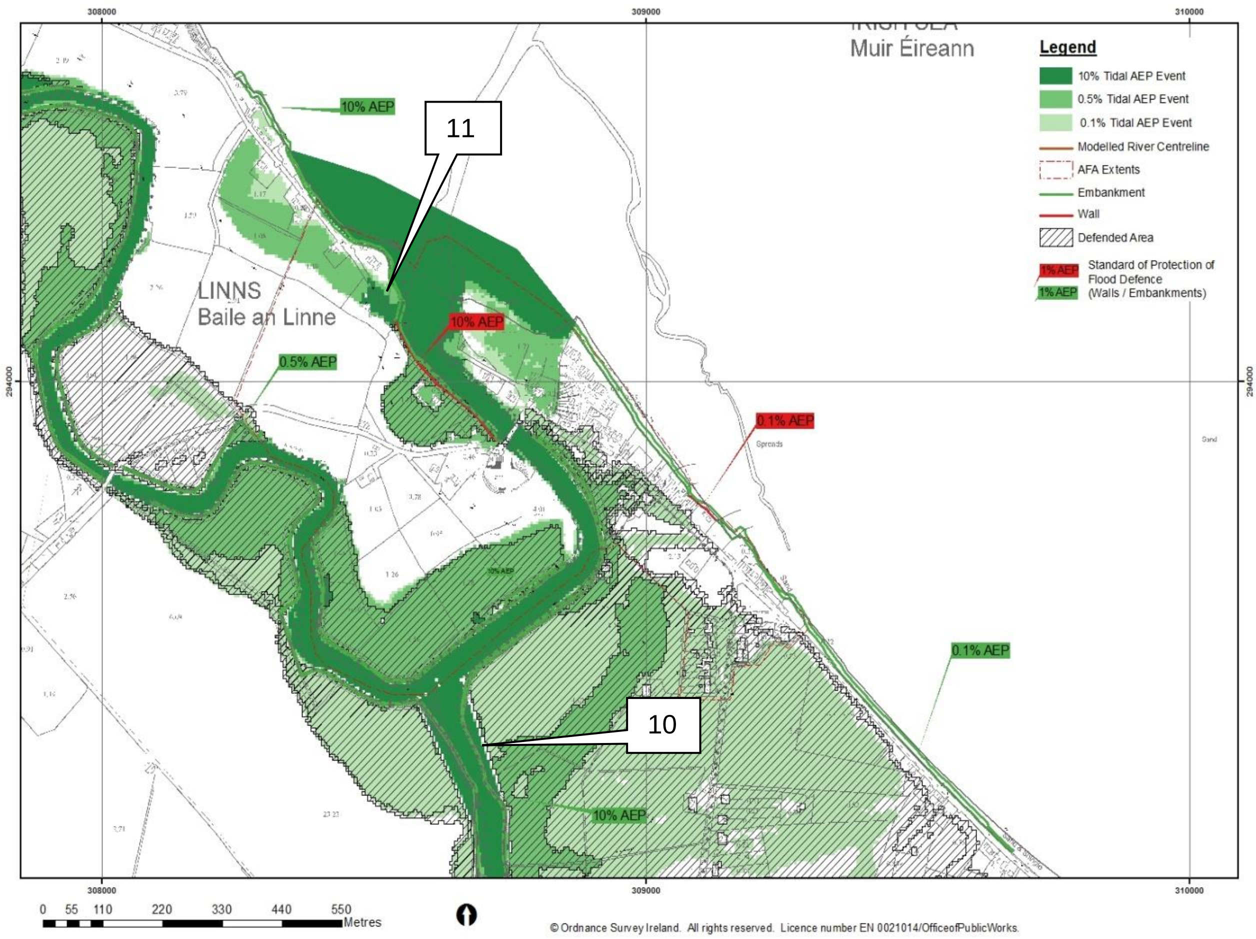


Figure 4.8.30: SoP and Defended Areas in Annagassan coastal area and Glyde Estuary

The area benefitting from embankments Defence References 2 and 3 were found to overlap, as shown in Figure 4.8.31. The SoP of Defence Reference 2 is 10% AEP still water level, whereas Defence References 3 overtops downstream during the 10% AEP design run. As the boundary between these benefitting areas is complex, an approximate outline of the area benefitting from embankment 2 has been shown.

There are also two sections along Defence Reference 3 and Defence Reference 9 where the SoP is <10%AEP as shown in Figure 4.8.31 and Figure 4.8.32 respectively. Due the lower crest levels at these locations flood waters overtop the embankments here during the 10%AEP coastal dominated events. Therefore no defended areas were attributed to the areas behind these embankments.

3

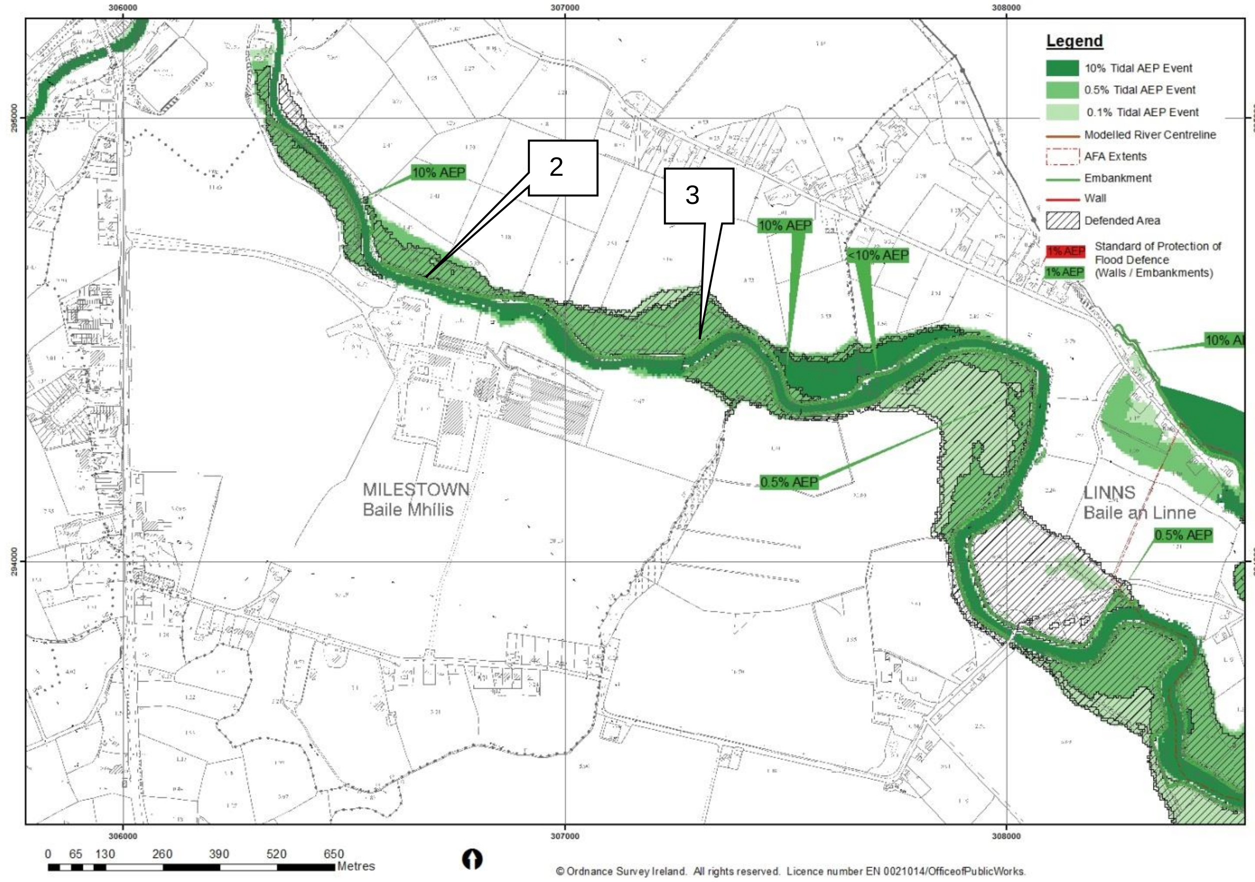


Figure 4.8.31: SoP and Defended Areas in the lower reach of the River Glyde

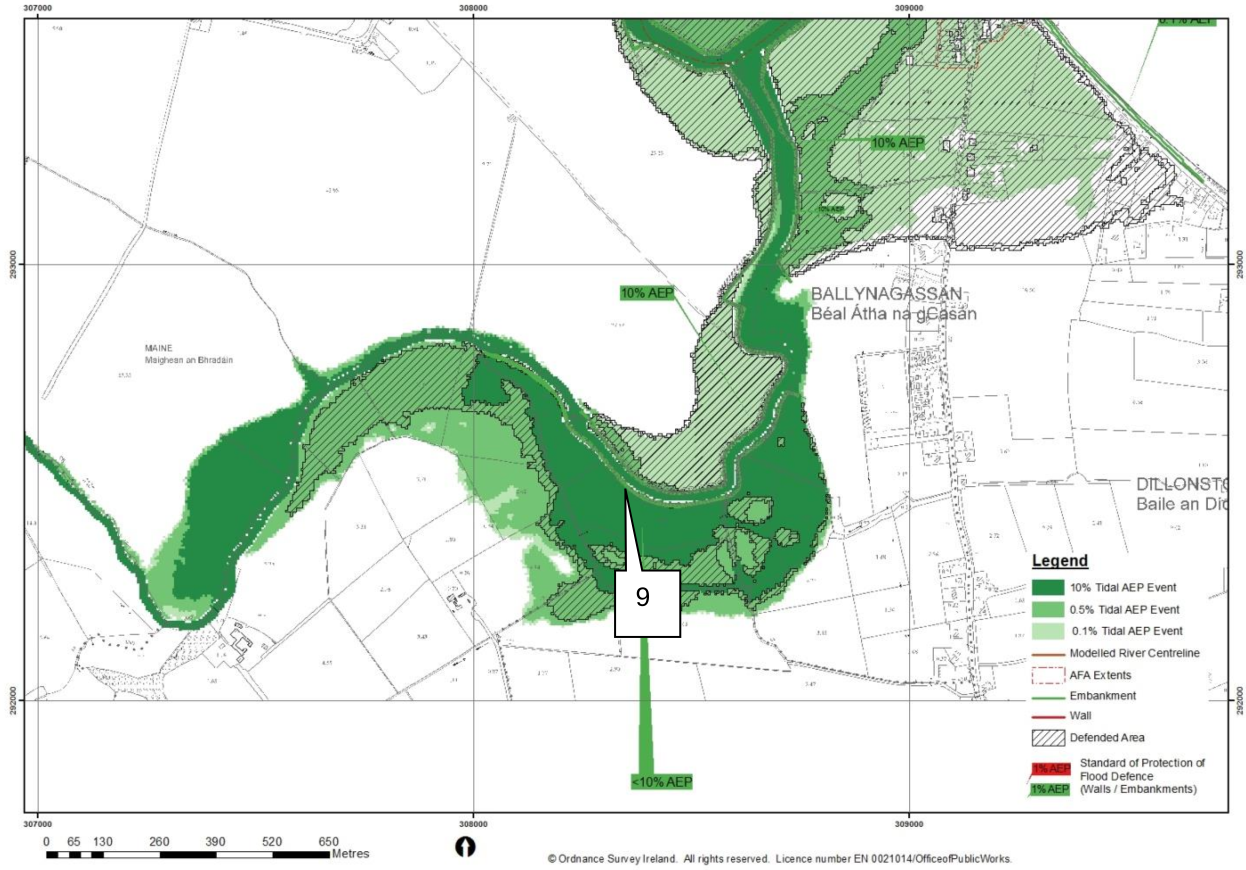


Figure 4.8.32: SoP and Defended Areas in the lower reach of the River Dee

(4) Gauging Stations:

There are three gauging stations within the model extent; Tallanstown (06014, River Glyde), Mansfieldstown (06021, River Glyde) and Castlebellingham (06052). Charleville Weir (06013, River Dee) is located approximately 2km upstream from the model upstream limit (refer to Ardee Model, Section 4.4)

Water level and flow data is available for stations 06013 and 06014 from October 1975 to the present, and similar data is available for station 06021 from August 1955 to the present. Stations 06014 is located the River Glyde MPW and denotes the upstream model extent. Station 06021 is also located on the River Glyde and was subject to a rating review, as detailed below.

Station 06052 is located on the River Glyde and is inactive with no data available. No staff gauge could be found at the location of station 06052. This station was not used for calibration. Instead it was used as in Intermediate HEP for anchoring of the model to hydrological estimates (refer to Appendix A.3).

(b) Tallanstown, River Glyde (06014)

The rating for this gauging station is excellent for flows in the range of 0.13-31.8 m^3/s, and therefore provides a good indication of the relationship for the estimated Q_{med} of 21.46 m^3/s. The survey provides a cross-section at the gauge location, although the centre of the channel could not be surveyed due to

dangerous fast flow. A point was added to the centre of the channel in order to deepen the bed by approximately 300mm and align the zero points of the existing rating curve and the model curve. This assumption was considered to be reasonable as the station is located on MPW, and the approach taken was approved by OPW on 28/02/2014.

Comparing the modelled Q-h relationship and the rating curve, as shown in Figure 4.8.33, there is good correlation between the curves, and they are always within 400mm of each other as required for MPWs. A Manning's n value of 0.013 at the weir was required in order to produce the Q-h relationship shown below. This value is within the normal range for a concrete lined section.

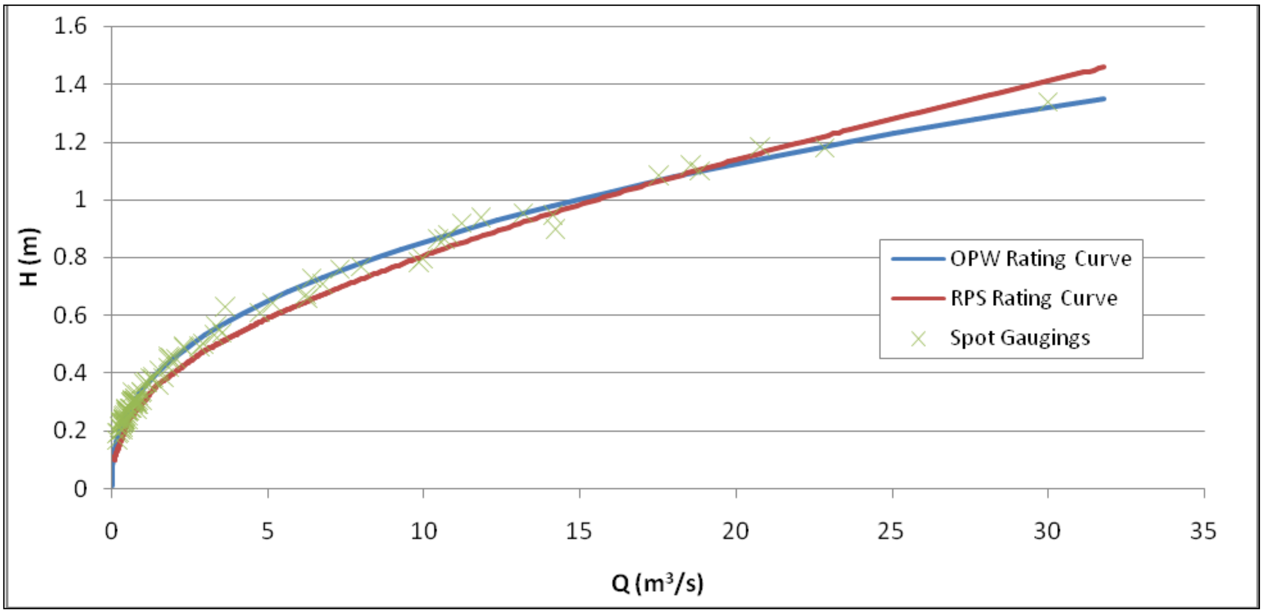


Figure 4.8.33: Tallanstown (06014) – Comparison of OPW Rating Curve with RPS Q-h Relationship

(c) Mansfieldstown (06021)

This gauging station is located on a segment of HPW as it is subject to a rating review, as detailed in IBE0700Rp0008_UoM 06 Hydrology Report_F01.

The rating for this gauging station is fair for flows in the range 0.30-1.0 m^3/s, poor for flows in the range 1.0-7.0 m^3/s, and good in the range 7.0-24.5m^3/s. The rating therefore encompasses the estimated Q_{med} of 23.69 m^3/s. The survey did not provide a cross-section at the location of the gauge board, but the detail of the staff gauge and hut was included in the cross-section of the bridge approximately 8m downstream of the gauge hut location. This detail was sufficient to enable calibration of the station.

Comparing the modelled Q-h relationship and the rating curve, as shown in Figure 4.8.34, there is good correlation between the curves up to the limit of the existing 'good' rating segment at a stage of 2.4m, and they are always within 200mm of each other as required for HPWs. There is a spot gauging recorded at a stage of 2.7m, however this doesn't appear to have been used during the calculation of the existing rating so the accuracy of this recording is uncertain. This spot gauging was therefore disregarded.

A Manning's n value of 0.03 at the cross-sections upstream of the bridge, and a Manning's n value of 0.02 at the bridge structure was required in order to produce the Q-h relationship shown below. These values are within the normal range for natural streams and masonry bridges.

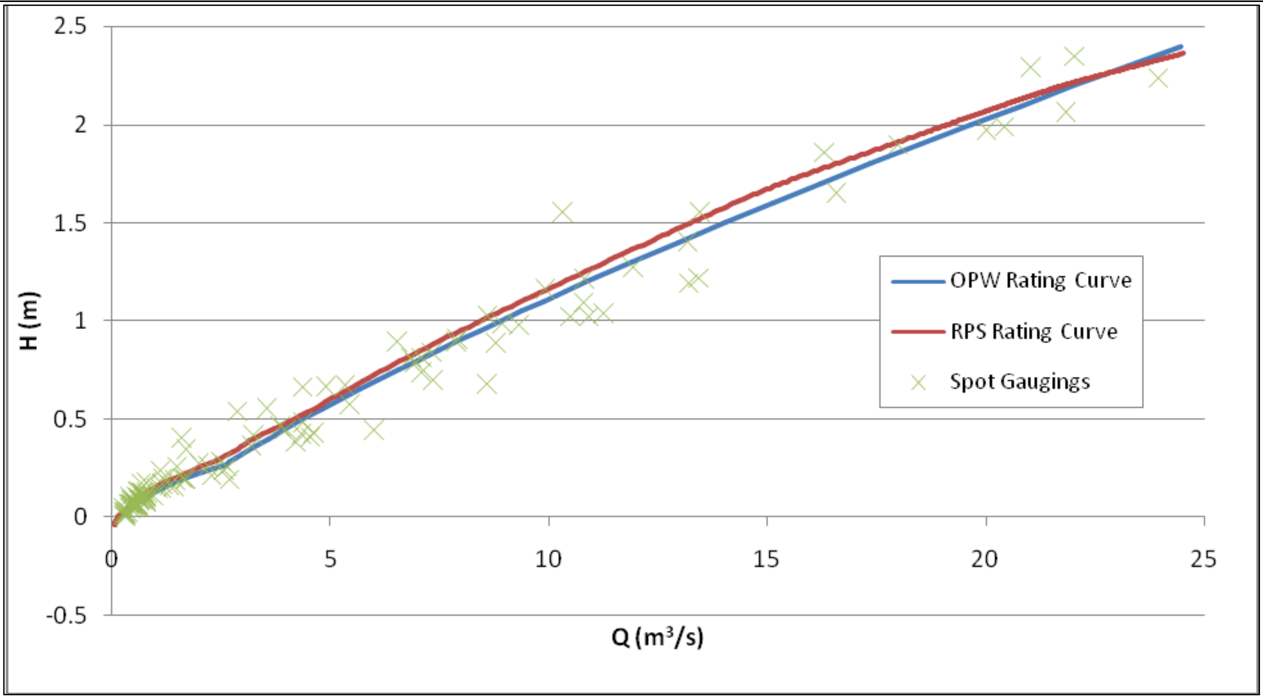


Figure 4.8.34: Mansfieldtown (06021) – Comparison of OPW Rating Curve with RPS Q-h Relationship

(5) Other Information:

(a) *Louth Area Engineer Meeting - Minutes (2005)* - Meeting with the Louth Area Engineer identifying areas which are prone to flooding.

'Annagassan Bridge. Glyde River burst its banks. House flooded.' - Flooding occurs in the model at the location described during coastal flood events of 0.5% AEP due to overtopping at the wall on the left bank of the River Glyde. This corresponds with the information in the report, as shown in Figure 4.8.35. (Please view Section 4.8.5(2) which discusses model updates for Final)

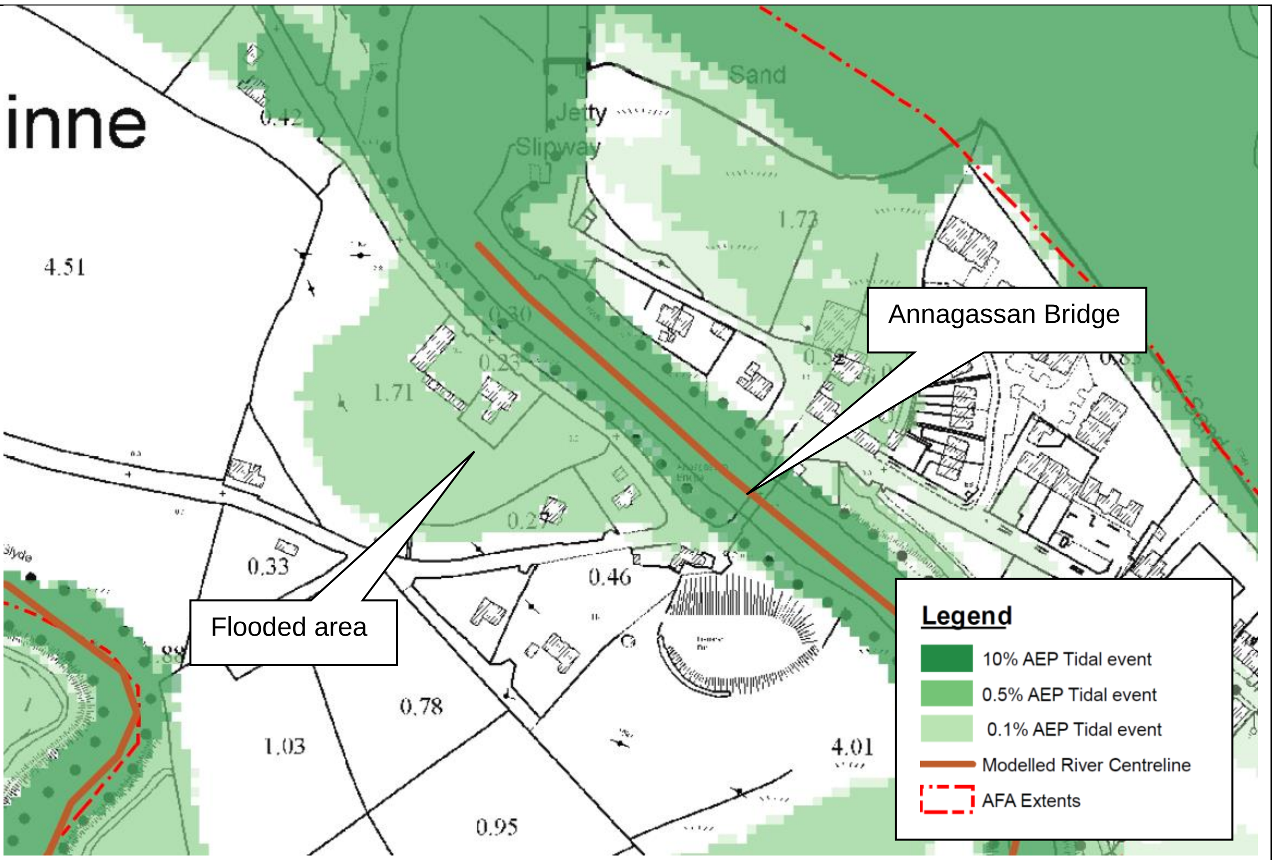


Figure 4.8.35: Flooding adjacent to Annagassan Bridge

All other comments within this report were outside the area of interest of this model.

4.8.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

(a) Edited timing of coastal boundary input so that surge peak corresponds roughly with fluvial peak and the fluvial timings in order to achieve agreement with calibration events.

(b) The in-channel roughness coefficients were selected based on normal bounds and have been reviewed during the calibration process using photos received from the channel and structure survey. Using CIRIA's (1997) Manning's n values for culverts as a reference it is considered that the final selected values are representative, see Chapter 3.5.1.

(c) The centre of the channels at gauging station 06014 could not be surveyed due to dangerous fast flow. In order to align the zero points of the rating curve, the assumed that the level in the centre of the channel is approximately 300mm lower. This is considered to be reasonable as the stations are located on MPW, and the approach taken was approved by OPW on 28/02/2014.

(d) The chainage of cross-sections on the River Glyde from the start of the model reach up to chainage 27752m had to be edited to make the sections relate to the correct spatial location. The chainage of each section had to be reduced by 57m.

(e) Modelled 0602M01020D and 0602M00815D as open sections as stability is improved and water level never reaches soffit of structure.

(f) The flood defence survey reported a gap in the embankment at Ballynagassan, see Figure 4.8.36. However no cross-sections were surveyed here. To represent this gap in the embankment within the model cross-sections 0602M00093 and 0602M00043 were copied to chainages 27650m and 28005m on the River Dee, respectively, with the crest level of the right bank taken from the flood defence survey.

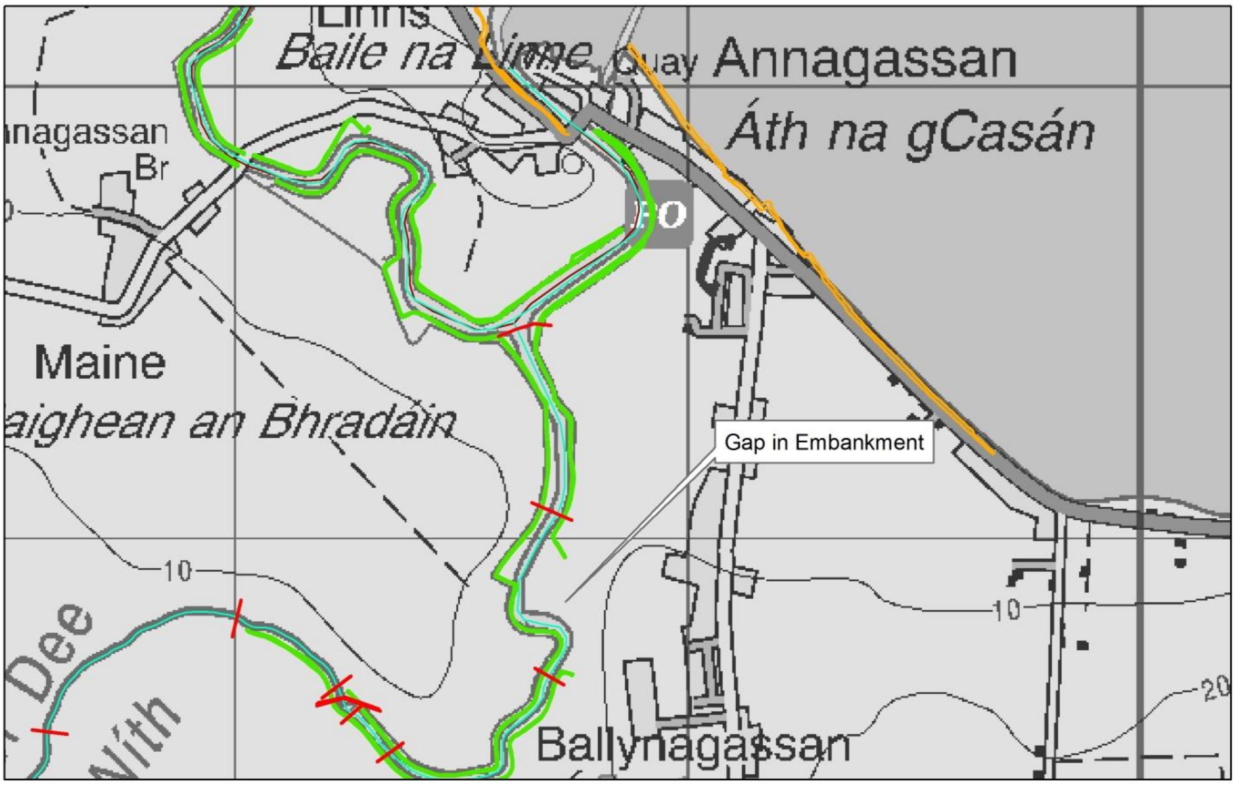


Figure 4.8.36: Gap in embankments on River Dee

(g) The low embankment on the seaward side of the sea wall at Harbour Road was not included in the model. The model was unstable when both defences were included as they are very close together, and as the wall offers a higher standard of protection it was decided that removing the embankment would not negatively impact the model hazard assessment or subsequent risk analysis. The location of these defences is shown in Figure 4.8.37. (Please view Section 4.8.5(2) which discusses model updates for Final)

Figure 4.8.37: Sea wall and embankment adjacent to Harbour Road (Please view Section 4.8.5(2) which discusses model updates for Final)

(h) Extra points were added to the end of the sea wall at Harbour Road to ensure flood water could not escape around the Eastern side of the wall. This assumption was based on imagery from Google which indicates there is no gap in the wall between Harbour Road and the bridge crossing the River Glyde, as shown in Figure 4.8.38. Levels were interpolated between data from the defence survey of the wall and level information available from cross-section 0601M00020D. (Please view Section 4.8.5(2) which discusses model updates for Final)

Figure 4.8.38: Google imagery of the location where the sea wall meets Annagassan Bridge

(i) For all simulations it has been assumed that all culverts and screens are free of debris and sediment.

(2) Hydraulic Model Limitations and Parameters:

(a) An overall timestep of 3 seconds has been selected for all model scenarios. The MIKE 21 model component is capable of dynamic timesteps in the range of 0.01-3 seconds.

(b) A maximum cell size of 25m^2 was used for all land adjacent to HPWs. A maximum cell size of 50m^2 was used for some land adjacent to MPW.

(c) A model instability occurs on the River Glyde at chainage 30893, as shown in Figure 4.8.39 and Figure 4.8.40. It is caused by the restriction in flow due to the presence of the bridge at this location, 0601M00881D and is situated 6.3km upstream of the HPW limit. This instability is at its most pronounced during the 1% AEP fluvial dominated event but is significantly reduced during the 1% AEP fluvial dominated event.

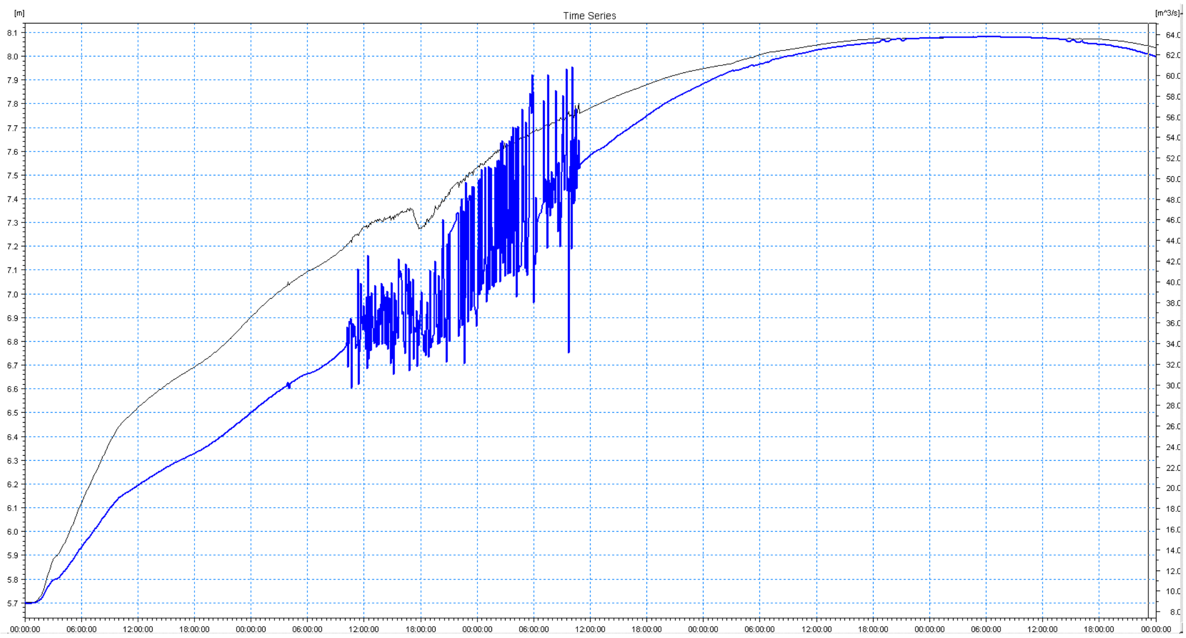


Figure 4.8.39:River Gylde, chainage 30893 Instability at Fluvial Dominated 0.1% AEP

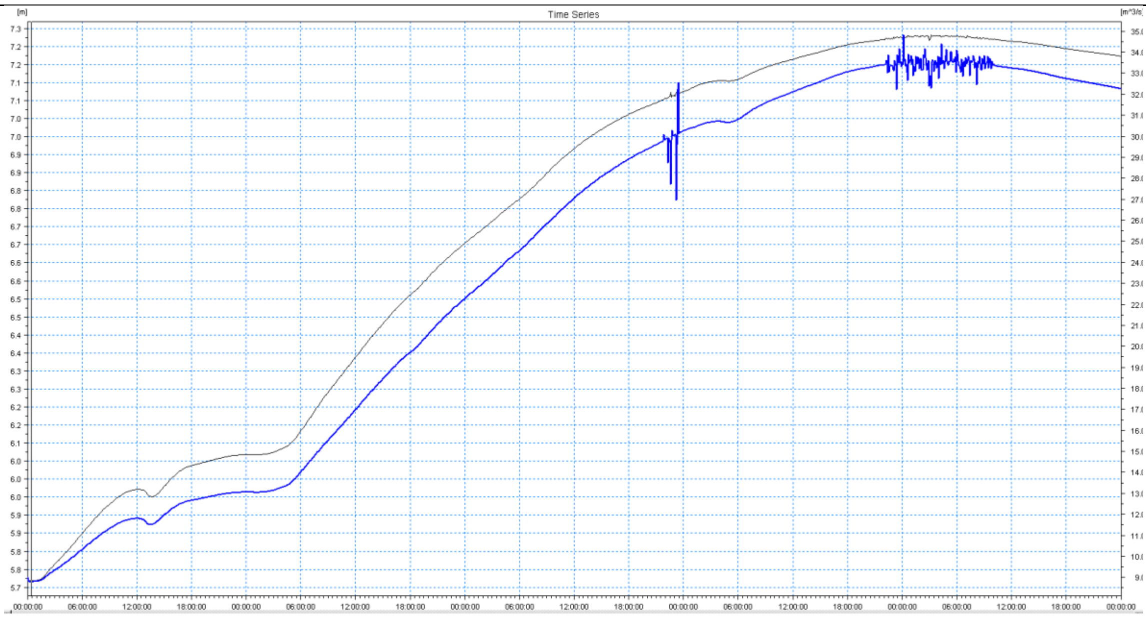


Figure 4.8.40: River Gylde, chainage 30893 Instability at Fluvial Dominated 1% AEP

This instability is largely associated extreme 0.1% AEP fluvial flows and causes a maximum decrease in water level of 0.1m which is within the 0.4m limit set for MPW reaches. Also the instability is far enough from the HPW and AFA that is does not affect discharge or water levels in these areas. Efforts were made to stabilise the model, however the instability could not be entirely removed.

(d) A model instability occurs on the River Dee at chainage 17920, as shown in Figure 4.8.41. It is caused by the restriction in flow due to the presence of the bridge at this location, 0602M01061D and is situated 8km upstream of the HPW limit. This instability behaves in the same way during all simulations. It causes minimal effects on the water levels in the location and does not cause out-of-bank flooding. Efforts were made to stabilise the model, however the instability could not be entirely removed.

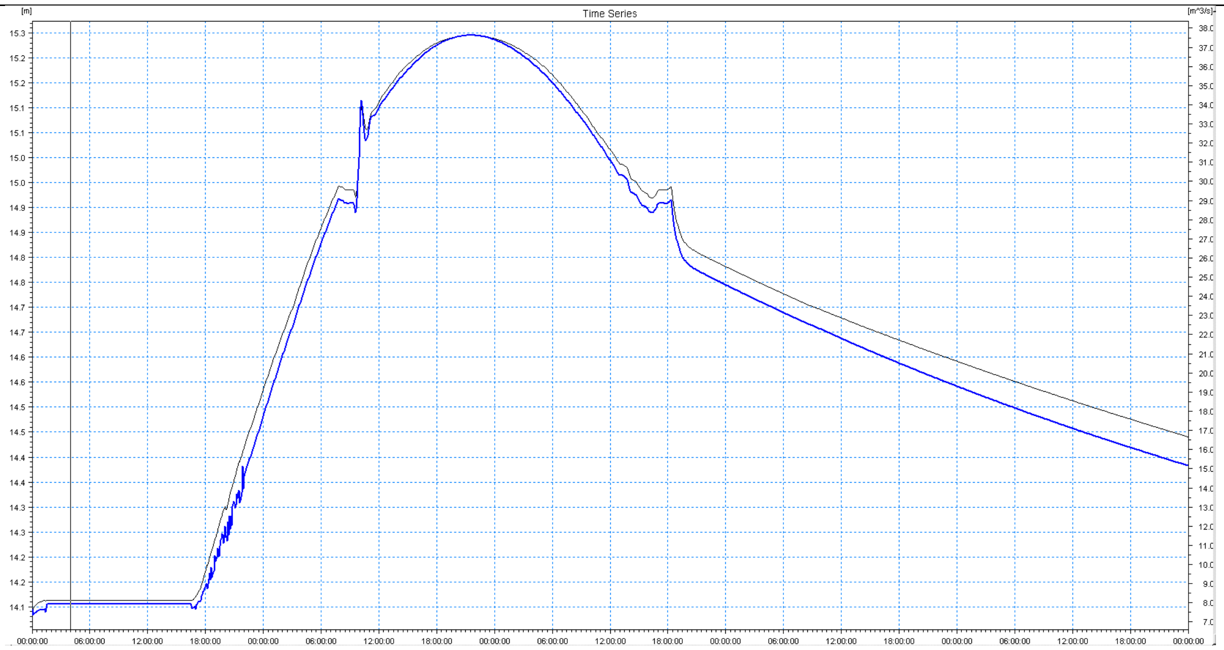


Figure 4.8.41:River Dee, chainage 17920 Instability at Fluvial Dominated 1% AEP

(e) A model instability occurs on the River Dee at chainage 18522, as shown in Figure 4.8.42. It is caused by the restriction in flow due to the presence of the bridge at this location, 0602M00999D and is situated 6.5km upstream of the HPW limit. This instability behaves in the same way during all simulations. It causes minimal effects on the water levels in the location and does not cause out-of-bank flooding. Efforts were made to stabilise the model, however the instability could not be entirely removed.

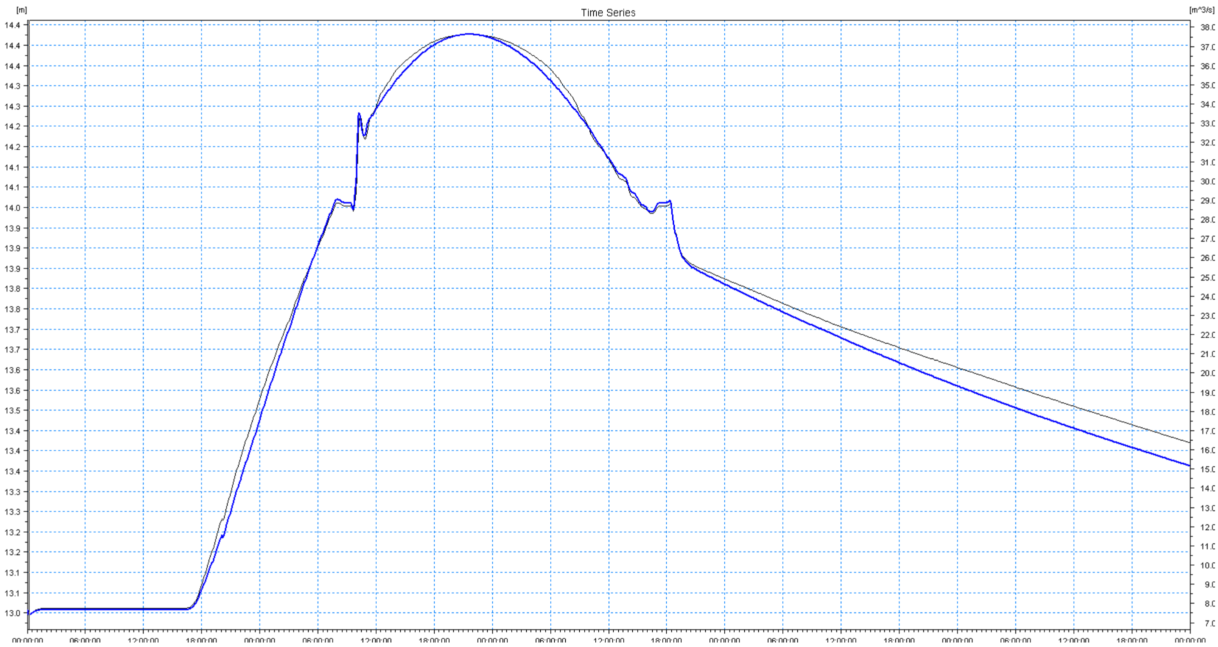


Figure 4.8.42: River Dee, chainage 18522 Instability at Fluvial Dominated 1% AEP

Hydraulic Model Parameters:	
MIKE 11	
Timestep (seconds)	3
Wave Approximation	Higher Order Fully Dynamic
Delta	0.7
Inter1Max factor	100
MIKE 21	
Timestep (seconds)	3
Drying / Flooding	0.001/0.02
Eddy Viscosity (and type)	Constant eddy formulation varying in space based on equation $0.02\Delta x^2/\Delta t$.
MIKE FLOOD	
Link Exponential Smoothing Factor (where non-default value used)	All default (1)

(3) Design Event Runs & Hydraulic Model Handover Notes:

(a) The coastal boundary total water level is based on tide levels at Soldiers Point and ICPSS point NE26.

(b) The parameters within the HD parameter file are identical for all design run scenarios.

(c) Steady state initial conditions have been used in the 1D model component during all design runs.

(d) Global surface elevation initial conditions of -0.43mOD Malin in the 2D domain have been used during all design runs. This value was chosen to match the water level of the first timestep at the coastal boundary to reduce instabilities during model start-up.

(e) The Link Exponential Smoothing Factor was lowered to 0.8 for the simulations with the flood defence structures removed.

(f)This model is influenced by both coastal and fluvial sources, as such a range of events were simulated with fluvial or tidal influences dominating flows. The 10% AEP, 1% AEP and 0.1% AEP fluvial return periods were simulated, all coinciding with the 50% AEP tidal event. The 10% AEP, 0.5% AEP and 0.1% AEP tidal events were also simulated, all coinciding with the 50% AEP fluvial event.

(g) The upper MPW sections within the model, i.e. the upper reaches of the River Dee and River Glyde, contained no constraining structures and the capacity of the channel was generally sufficient to convey flow up to at least 0.1% AEP.

(h) Fluvial flooding was found to occur upstream of the weir 0601M00300W at chainage 36696 on the River Glyde during events of 1% AEP and greater. The weir was 'drowned out' under high flows, but the

flow restriction still resulted in water backing up over a stretch approximately 2km upstream of the weir. This causes the embankments to be overtopped, and flooding of agricultural land and the water treatment works may occur as shown in Figure 4.8.43

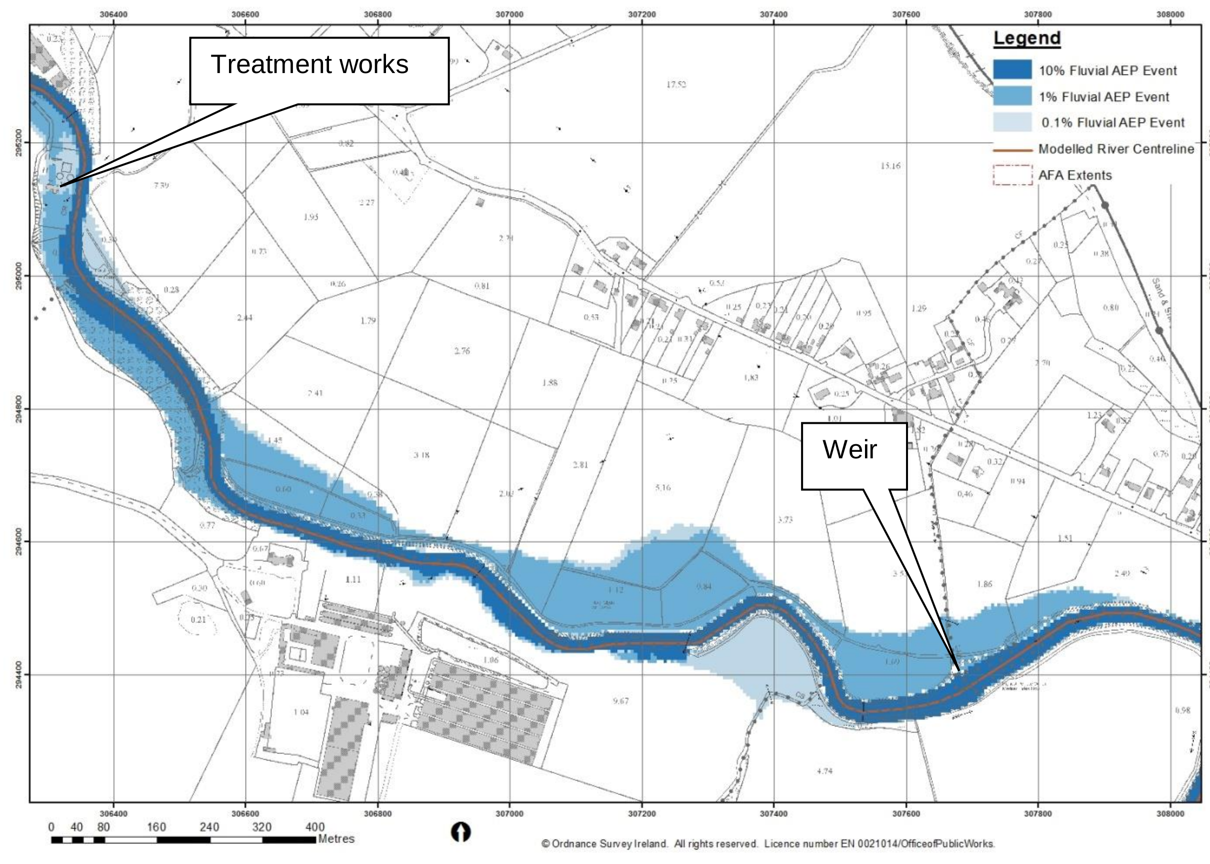


Figure 4.8.43: Fluvial flooding upstream of weir on River Glyde

(i) Coastal flooding was also found to occur upstream of the weir 0601M00300W at chainage 36696 on the River Glyde due to high water levels overtopping the embankments. This flooding occurred mainly during events of at least 0.5% AEP, but localised flooding to the left bank of the weir was found to occur at 10% AEP. Flooding was also found to occur on the inside of the meander immediately downstream of the weir during events of at least 0.5% AEP. This flooding is shown in Figure 4.8.44.

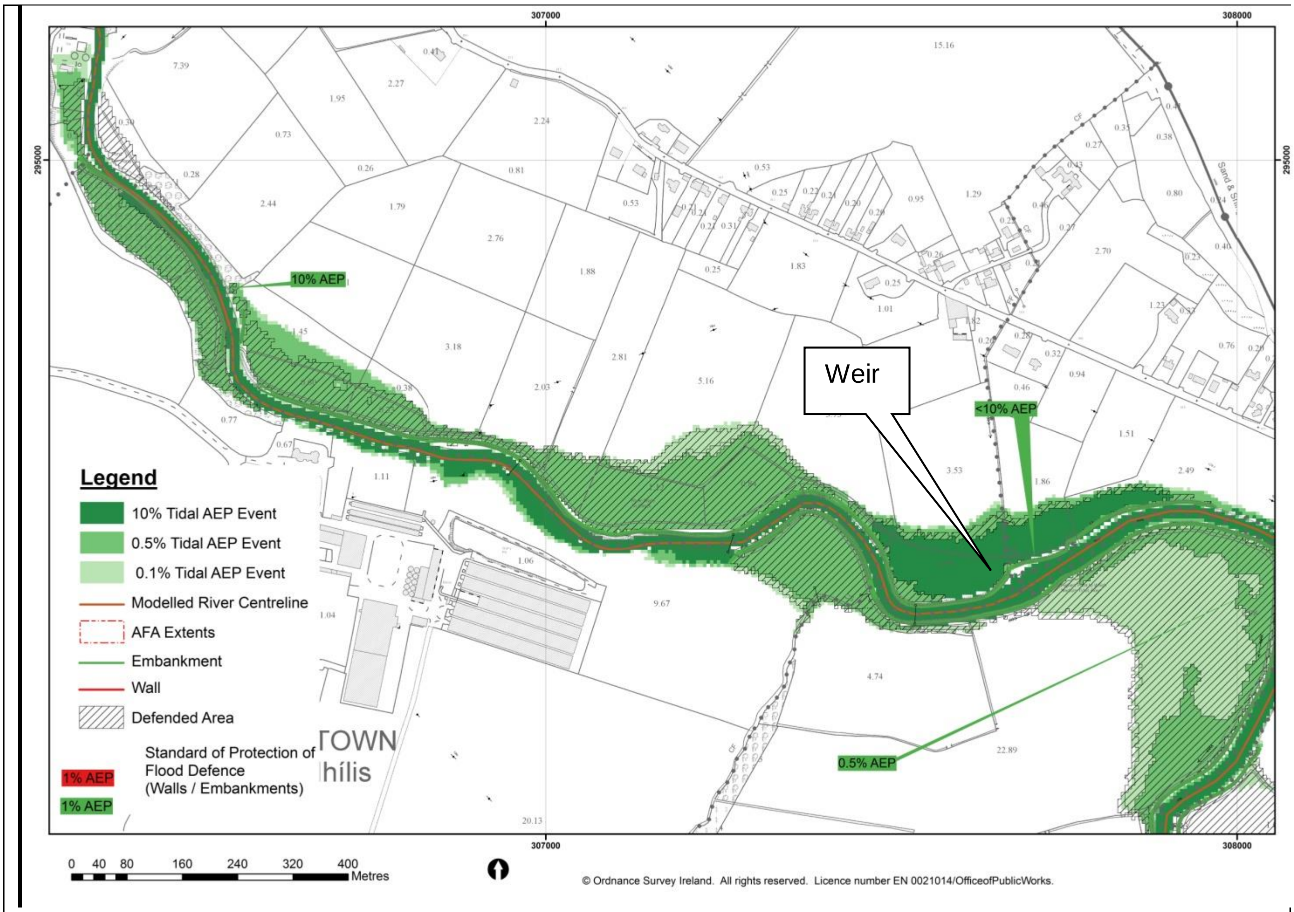


Figure 4.8.44: Coastal flooding around weir on River Glyde

(j) Fluvial and coastal flooding was found to occur upstream of the embankments of the River Dee. This occurred during fluvial and coastal events of 10% AEP at the left bank, and during the 0.1% AEP fluvial and 0.5% AEP or greater coastal events at the right bank as shown in Figure 4.8.45 and Figure 4.8.46. Fluvial flooding is due to the weir at chainage 26932m causing the flow to back up. Coastal flooding is due to the high water level.

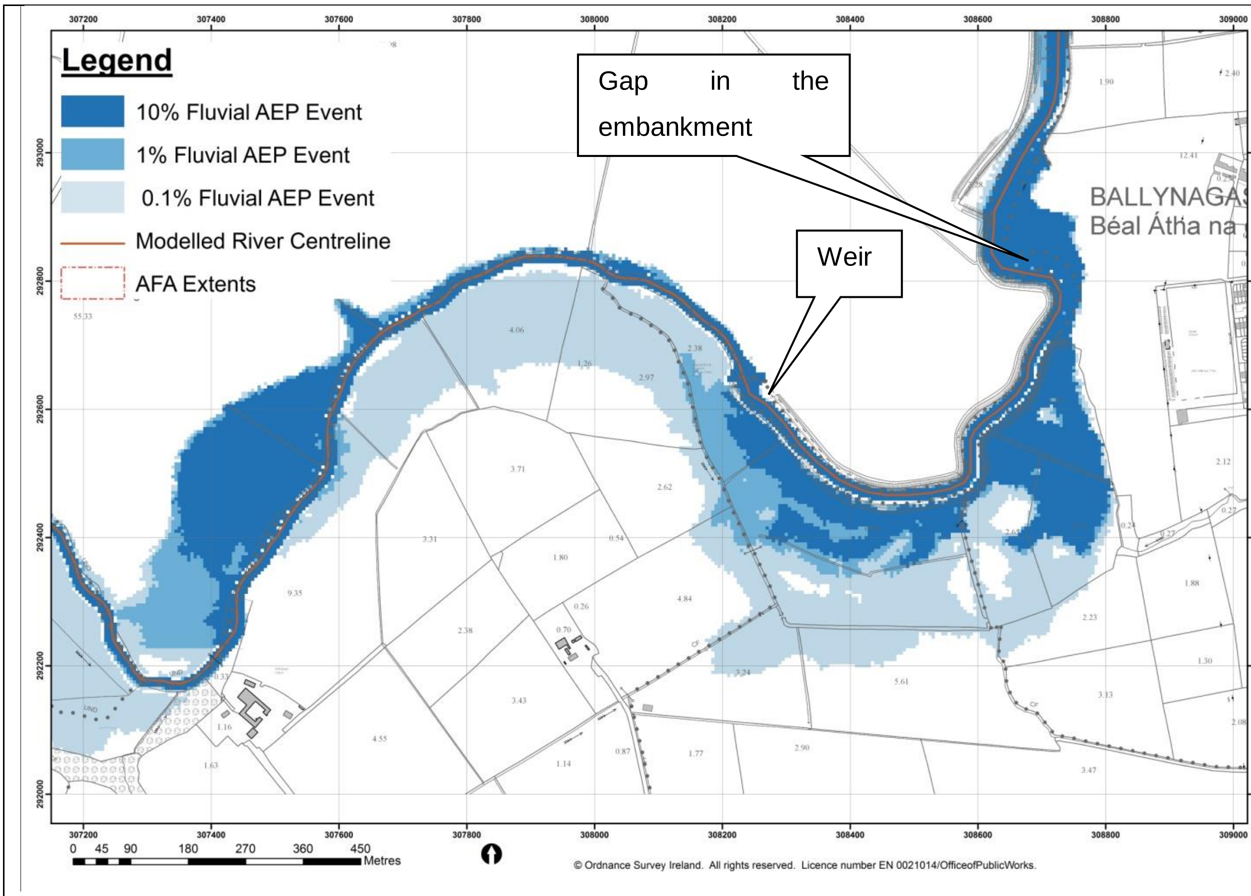


Figure 4.8.45: Fluvial flooding in vicinity of weir on River Dee

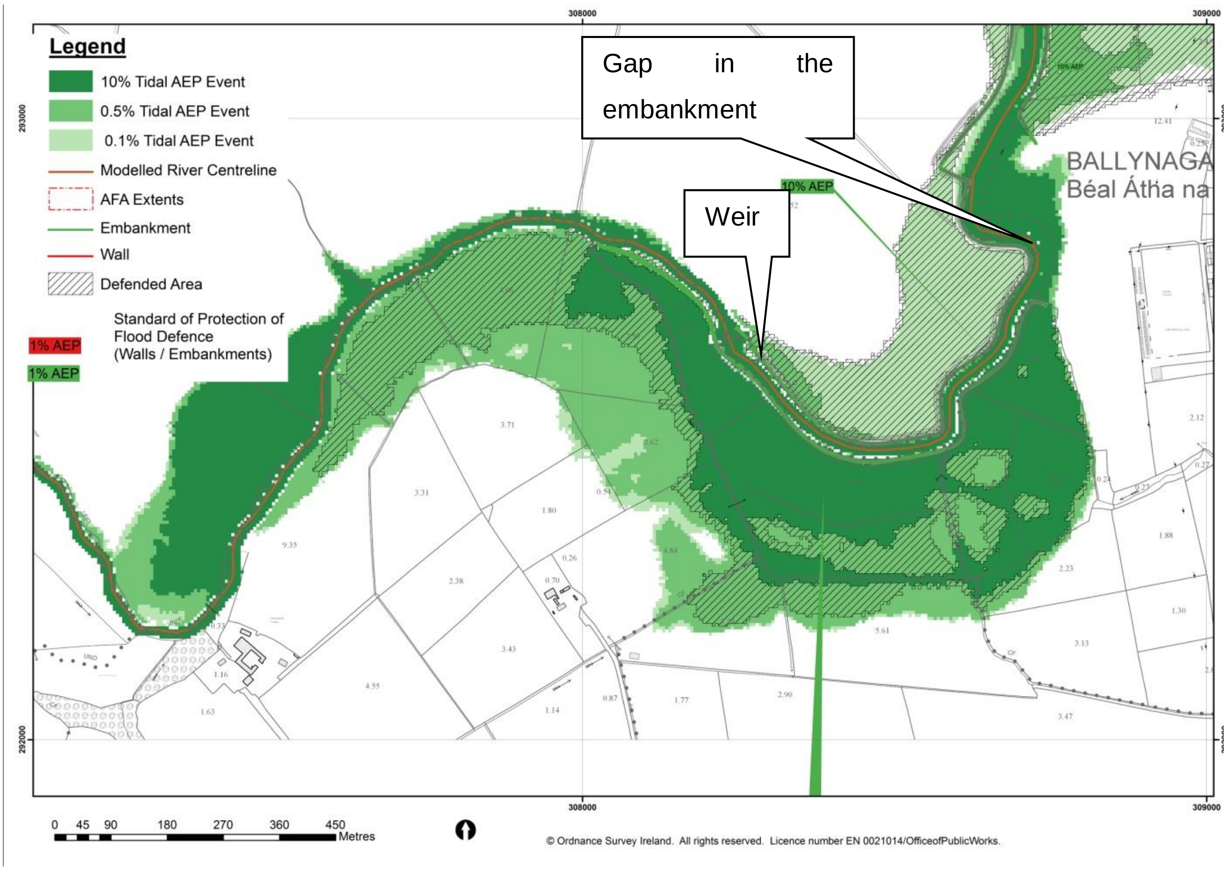


Figure 4.8.46: Coastal flooding in vicinity of weir on River Dee

(k) Fluvial and coastal flooding was found to occur due to the gap in the embankment at the right bank of the River Dee, as shown in Figure 4.8.45 and Figure 4.8.46. The edge of the Southern embankment was found to not extend far enough, allowing floodwater to bypass the embankment during all fluvial and coastal events. The edge of the northern embankment was found to be overtopped during coastal flood events of 0.5% AEP, leading to widespread flooding of the Ballynagassan area.

(l) Coastal flooding occurred in the lower 2km of the River Glyde due to high water levels overtopping the embankments at various locations. This was generally limited to events of 0.5% AEP or greater, and affected roads, properties and agricultural land. The worst affected areas were around Strand Road and Harbour Road. The capacity of Annagassan Bridge 0601M00020D, (chainage 39495m) is generally sufficient to convey flow. However the bridge does restrict flow upstream and downstream of the structure during 0.5% AEP and 0.1% AEP events. This causes a backup of water behind the bridge exacerbating the flooding in that location.

(m) There is a low point in the coastal defence where the River Glyde discharges to Dundalk Bay. This causes flooding of Harbour Road during 10% AEP events, as shown in Figure 4.8.47.

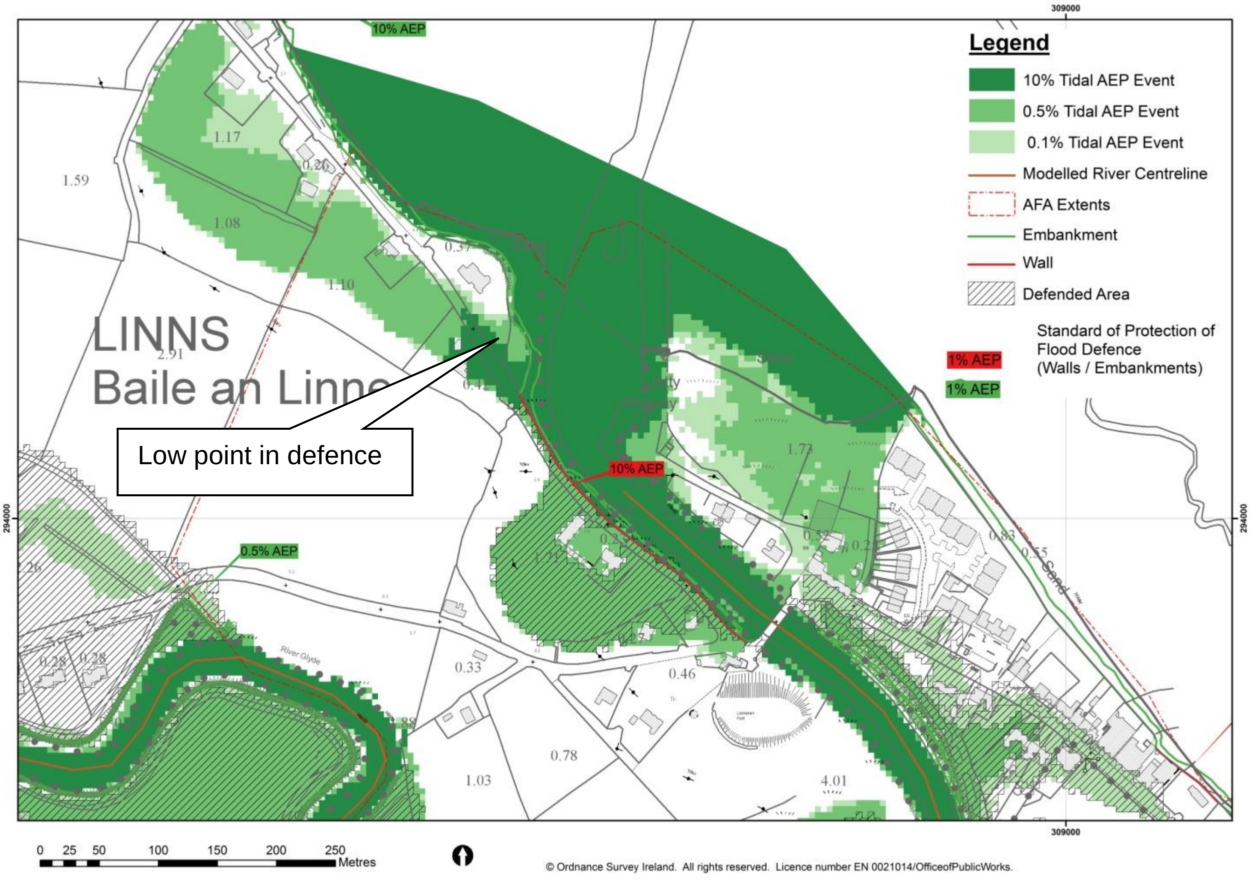


Figure 4.8.47: Low point in coastal defence adjacent to Harbour Road

(4) Hydraulic Model Deliverables:

Please see Appendix A.4 for a list of all model files provided with this report.

(5) Quality Assurance:	
Model Constructed by:	David Irwin/Emma Holland
Model Reviewed by:	Stephen Patterson
Model Approved by:	Malcolm Brian

APPENDIX A.1

Structure Details – Bridges and Culverts								
RIVER BRANCH	**CHAINAGE**	**ID****	**LENGTH (m)**	**OPENING SHAPE**	**HEIGHT (m)**	**WIDTH (m)**	**SPRING HEIGHT FROM INVERT (m)**	**MANNING'S n**
River Dee	22289.41	0602M00624D	7.4	Arch x 4	3.84, 3.74, 3.88, 2.86	4.14, 4.40, 4.39, 3.89	2.61, 2.48, 2.43, 1.26	0.013
River Dee	19029.57	0602M00950D	10.2	Arch x 3	7.66, 9.53, 9.56	6.03, 6.13, 6.07	4.94, 6.66, 6.74	0.013
River Dee	18532.49	0602M00999D	1.3	Arch	3.63	17.01	3.25	0.013
River Dee	17913.12	0602M01061D	7.5	Arch	3.93	11.98	2.09	0.013
River Dee	17549.88	0602M01099D	15.8	Irregular	3.88	16.59	N/A	0.03
River Glyde Tributary 1	163.834	0601A00031D	3	Irregular	3.84	7.08	N/A	0.02
River Glyde	39495.29	0601M00020D	7.7	Arch x 5	3.77, 5.14, 5.42, 4.95, 3.81	5.50, 6.91, 7.02, 6.93, 5.43	1.41, 2.63, 2.89, 2.41, 1.54	0.013
River Glyde	37840.29	0601M00184D	8.3	Arch	3.72	13.05	2.02	0.013
River Glyde	34240.56	0601M00543D	12	Arch x 3	2.72, 2.34, 2.65	3.24, 2.64, 3.02	1.42, 1.42, 1.45	0.02
River Glyde	33239.81	0601M00642D	4.7	Irregular	2.55	11.09	N/A	0.02
River Glyde	31675.71	0601M00799D	4.4	Irregular	3.35	9.24	N/A	0.035
River Glyde	30939	0601M00874D	9.5	Irregular	6.31	12.1	N/A	0.035
River Glyde	30872.81	0601M00881D	41	Irregular	4.20	15.04	N/A	0.035

Structure Details – Bridges and Culverts								
RIVER BRANCH	**CHAINAGE**	**ID****	**LENGTH (m)**	**OPENING SHAPE**	**HEIGHT (m)**	**WIDTH (m)**	**SPRING HEIGHT FROM INVERT (m)**	**MANNING'S n**
River Glyde	28884.69	0601M01078D	9.6	Arch	3.62	13.51	1.53	0.02
River Glyde	22808.46	0601M01686D	8.3	Arch	4.54	10.61	2.52	0.013
River Glyde	18061.31	0601M02161D	7.9	Arch x 2	2.92, 2.73	14.97, 14.97	1.73, 1.53	0.013

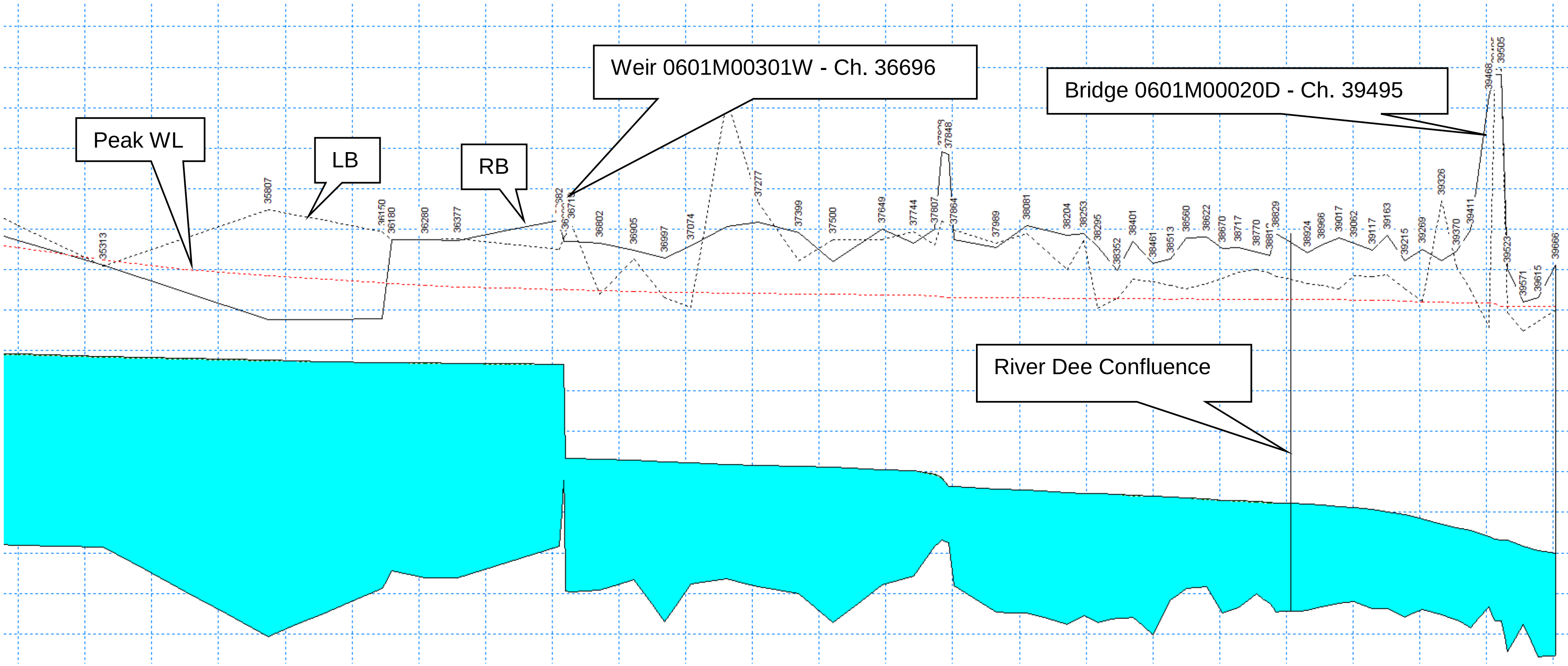
** Structure ID Key:

D – Bridge Upstream Face; E – Bridge Downstream Face; I – Culvert Upstream Face; J – Culvert Downstream Face

Structure Details - Weirs				
RIVER BRANCH	**CHAINAGE**	**ID**	**MANNINGS N**	**TYPE**
River Dee	26932.7	0602M00161W	0.013	Broad Crested Weir
River Dee	21833.55	0602M00665W	0.013	Broad Crested Weir
River Dee	19711.86	0602M00879W	0.013	Broad Crested Weir
River Glyde	36696.01	0601M00301W	0.013	Broad Crested Weir
River Glyde	17862.91	0601M02180W	0.013	Broad Crested Weir

APPENDIX A.2

Long section plots of calibration



River Glyde 1% AEP fluvial design run

APPENDIX A.3

	Peak Water Flows			
River Name & Chainage	**AEP**	**Check Flow (m^3/s)**	**Model Flow (m^3/s)**	**Diff (%)**
RIVER GLYDE 28167.1	10%	32.56	32.80	+0.72
06_603_Inter_1_RA	1%	45.02	45.79	+8.98
	0.1%	60.53	62.31	+2.94
RIVER GLYDE 28906.5	10%	32.57	32.88	+0.96
06021_RA	1%	45.03	45.92	+1.97
	0.1%	60.54	62.47	+3.19
RIVER GLYDE 31118.7	10%	32.59	33.79	+3.68
06_1097_2_RA	1%	45.05	47.43	+5.28
	0.1%	60.58	63.86	+5.42
RIVER GLYDE 34222.9	10%	32.59	34.29	+5.22
06052_RA	1%	45.05	47.88	+6.27
	0.1%	60.58	65.06	+7.39
RIVER GLYDE 39640.4	10%	64.36	97.17	+50.98
06_848_D_RARPS	1%	92.27	117.74	+27.60
	0.1%	128.62	145.59	+13.19
RIVER DEE 25952.2	10%	44.59	43.18	-3.16
06_1099_8_RA	1%	61.65	60.27	-2.24
	0.1%	82.89	79.99	-3.50
RIVER DEE 28184.4	10%	44.97	49.97	+11.13
06_1100_1_RA	1%	62.17	64.42	+3.62
	0.1%	83.59	83.77	+0.22

The table above provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

The model is very well anchored to gauged flow at Station 06021_RA on the River Glyde. Anchoring of the model to estimated fluvial flows has been satisfactorily achieved at most check points with differences under 10%. The only exceptions are discussed as follows.

HEP 06_848_D_RARPS at the downstream extent of the River Glyde is heavily influenced by tidal processes, so the discharge at this point could not be reliably checked against estimated flows.

HEP 06_1100_1_RA at the downstream extent of the River Dee is also influenced by tidal processes, which causes model flows during the 10% and 1% AEP design runs to be higher than the estimated flows. Fluvial processes begin to dominate at this point during the 0.1% AEP fluvial design run, which explains why the difference between estimated and modelled flows during this scenario is negligible.

APPENDIX A.4

MIKE FLOOD	MIKE 21	MIKE 21 - DFS0 FILE	MIKE 21 RESULTS
<u>Defended</u> HA06_ANNA8_MF_DES_2_Q10F HA06_ANNA8_MF_DES_2_Q100F HA06_ANNA8_MF_DES_2_Q1000F HA06_ANNA8_MF_DES_2_Q10S HA06_ANNA8_MF_DES_2_Q100S HA06_ANNA8_MF_DES_2_Q1000S <u>Undefended</u> HA06_ANNA8_MF_DEF_4_Q1000S HA06_ANNA8_MF_DEF_5_Q10S HA06_ANNA8_MF_DEF_5_Q200S	<u>Defended</u> HA06_ANNA8_M21_DES_2_Q10F HA06_ANNA8_M21_DES_2_Q100F HA06_ANNA8_M21_DES_2_Q1000F HA06_ANNA8_M21_DES_2_Q10S HA06_ANNA8_M21_DES_2_Q200S HA06_ANNA8_M21_DES_2_Q1000S HA06_ANNA8_MESH_26 <u>Undefended</u> HA06_ANNA8_M21_DEF_4_Q1000S HA06_ANNA8_M21_DEF_5_Q10S HA06_ANNA8_M21_DEF_5_Q200S HA06_ANNA8_mesh_26_DEF HA06_ANNA8_MESH_26_FPR HA06_ANNA8_MESH_26_EDDY	HA06_ANNA8_DFS0_Q2 HA06_ANNA8_DFS0_Q10 HA06_ANNA8_DFS0_Q200 HA06_ANNA8_DFS0_Q1000	<u>Defended</u> HA06_ANNA8_M21_DES_2_Q10F HA06_ANNA8_M21_DES_2_Q100F HA06_ANNA8_M21_DES_2_Q1000F HA06_ANNA8_M21_DES_2_Q10S HA06_ANNA8_M21_DES_2_Q200S HA06_ANNA8_M21_DES_2_Q1000S <u>Undefended</u> HA06_ANNA8_M21_DEF_4_Q1000S HA06_ANNA8_M21_DEF_5_Q10S HA06_ANNA8_M21_DEF_5_Q200S

MIKE 11 - SIM FILE & RESULTS FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
<u>Defended</u> HA06_ANNA8_M11_DES_2_Q10F HA06_ ANNA8_M11_DES_2_Q100F HA06_ ANNA8_M11_DES_2_Q1000F HA06_ ANNA8_M11_DES_2_Q10S HA06_ ANNA8_M11_DES_2_Q200S HA06_ ANNA8_M11_DES_2_Q1000S <u>Undefended</u> HA06_ANNA8_M11_DEF_4_Q1000S HA06_ANNA8_M11_DEF_5_Q10S HA06_ANNA8_M11_DEF_5_Q200S	<u>Defended</u> HA06_ANNA8_NWK_DES_1 <u>Undefended</u> HA06_ANNA8_NWK_DEF_4 HA06_ANNA8_NWK_DEF_5	<u>Defended</u> HA06_ANNA8_XNS_DES_1 <u>Undefended</u> HA06_ANNA8_XNS_DEF_4 HA06_ANNA8_XNS_DES_5	HA06_ANNA8_BND_ DES_1_Q2 HA06_ANNA8_BND_ DES_1_Q10 HA06_ANNA8_BND_ DES_1_Q100 HA06_ANNA8_BND_ DES_1_Q1000

MIKE 11 - DFS0 FILE		MIKE 11 - HD FILE & RESULTS FILE	
HA06_ANNA8_DFS0_Q2 HA06_ANNA8_DFS0_Q10 HA06_ANNA8_DFS0_Q100 HA06_ANNA8_DFS0_Q1000		Defended HA06_ANNA8_HD_DES_2_Q10F HA06_ANNA8_HD_DES_2_Q100F HA06_ANNA8_HD_DES_2_Q1000F HA06_ANNA8_HD_DES_2_Q10S HA06_ANNA8_HD_DES_2_Q200S HA06_ANNA8_HD_DES_2_Q1000S Undefended HA06_ANNA8_M11_DEF_4_Q1000S HA06_ANNA8_M11_DEF_5_Q10S HA06_ANNA8_M11_DEF_5_Q200S	

'Mechanism 2 Wave Overtopping' Model Files

MIKE 21	MIKE 21 - DFS0 FILE	MIKE 21 RESULTS
HA06_ANNA8_M21FM_WAV_11_Q10_A HA06_ANNA8_M21FM_WAV_11_Q10_B HA06_ANNA8_M21FM_WAV_11_Q200_A HA06_ANNA8_M21FM_WAV_11_Q200_B HA06_ANNA8_M21FM_WAV_11_Q1000_A HA06_ANNA8_M21FM_WAV_11_Q1000_B HA06_ANNA8_OT_MESH_10	HA06_ANNA8_DFS0_Q10 HA06_ANNA8_DFS0_Q200 HA06_ANNA8_DFS0_Q1000 HA06_ANNA8_DFS0_Outlet	HA06_ANNA8_M21FM_WAV_11_Q10_A HA06_ANNA8_M21FM_WAV_11_Q10_B HA06_ANNA8_M21FM_WAV_11_Q200_A HA06_ANNA8_M21FM_WAV_11_Q200_B HA06_ANNA8_M21FM_WAV_11_Q1000_A HA06_ANNA8_M21FM_WAV_11_Q1000_B

GIS Deliverables - Hazard		
Flood Extent Files (Shapefiles)	**Flood Depth Files (Raster)**	**Water Level and Flows (Shapefiles)**
Fluvial n01exfcd001F0 n01exfcd010F0 n01exfcd100F0 Coastal n01exccd001F0 n01exccd005F0 n01exccd100F0	Fluvial n01dpfcd001F0 n01dpfcd010F0 n01dpfcd100F0 Coastal n01dpccd001F0 n01dpccd005F0 n01dpccd100F0	Fluvial N01NFCDF0 Coastal N01NCCDF0
Flood Zone Files (Shapefiles)	**Flood Velocity Files (Raster)**	**Flood Defence Files (Shapefiles)**
N01ZNA_MCDF0 N01ZNB_MCDF0	Fluvial n01vlfcd001F0 n01vlfcd010F0 n01vlfcd100F0 Coastal n01vlccd001F0 n01vlccd005F0 n01vlccd100F0	Annagassan_Flood_Defences
GIS Deliverables - Risk		
Specific Risk - Inhabitants (Raster)	**General Risk - Economic (Shapefiles)**	**General Risk-Environmental (Shapefiles)**
Fluvial n01rifcd001F0 n01rifcd010F0 Coastal n01riccd001F0 n01riccd005F0 n01riccd100F0		