Kilkee Flood Relief Scheme – Hydraulic Modelling Report

Final Report

July 2024

COMHAIRLE CONTAE AN CHLÁIR
CLARE COUNTY COUNCIL

OPW
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Contract

This report describes work commissioned by Clare County Council (CCC) in partnership with the Office of Public Works (OPW). Chris Wason, Orla Hannon, Paul Browne and Caoimhe Downing of JBA Consulting carried out this work.

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Purpose

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Abbreviations

AEP	Annual Exceedance Probability
AFA	Area for Further Assessment
CCC	Clare County Council
CFRAM	Catchment Flood Risk Assessment and Management
DTM	Digital Terrain Model
EPA	Environment Protection Agency
FM	Flood Modeller
FRS	Flood Relief Scheme
HEFS	High End Future Scenario
LiDAR	Light Detection and Ranging
MBE	Mass Balance Error
MRFS	Medium Range Future Scenario
mOD	Metres above Ordnance Datum
OPW	Office of Public Works
OSI	Ordnance Survey Ireland
TOB	Top of Bank
TUFLOW	Two-Dimensional Unsteady Flow model

1 Introduction

This hydraulics report aims to provide technical details about the construction and schematisation of the hydraulic model of Kilkee and the surrounding area used within the development of the Kilkee Flood Relief Scheme (FRS). Two fluvial models were constructed as part of the Kilkee FRS to model the two main watercourses in the area, the Victoria Stream model and the Atlantic Stream model. As there is no hydraulic interaction between the two watercourses this was considered appropriate. A coastal model was also developed to represent the flood risk of wave overtopping. This is described in Appendix B.

1.1 Project aim

The overall purpose of the Kilkee FRS project is to design and build flood defences that will protect properties and critical infrastructure in future flood events. This is being done using hydraulic modelling to assess past events and the potential defence options.

1.2 Study area overview

Below is a summary of the study area and the catchment details:

- The Area for Further Assessment (AFA) boundary defined by the CFRAM has an approximate area of 3.6km², as shown in Figure 1-1,
- The Victoria Stream, to the south, and the Atlantic Stream, to the north are the main watercourses in the area.
- Additional tributaries and drainage channels also contribute flow through the AFA area.
- Elevation over the study area varies on average from 67mOD to 2mOD over approximately 2km.
- Agricultural lands, grasslands and pastural land are the main land use types within the model area. Kilkee town is the only urban area.

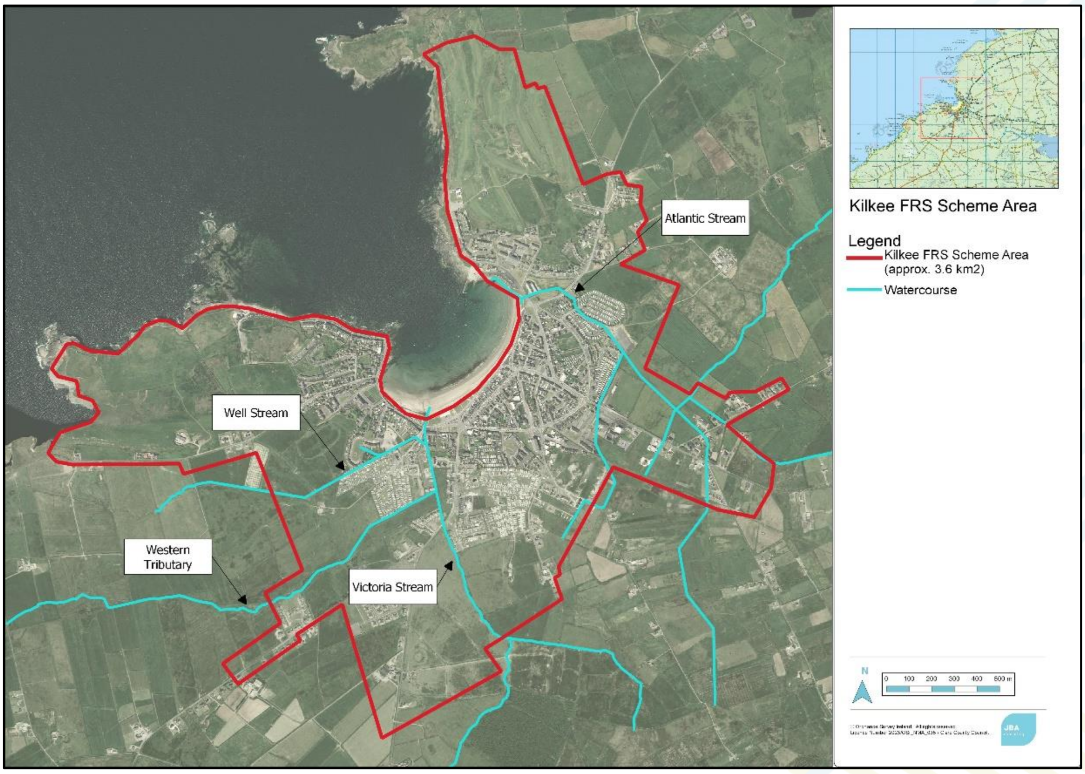


Figure 1-1: Kilkee AFA Overview

2 Available data

2.1 Existing models

The primary previous hydraulic model of Kilkee's watercourses was constructed under the OPW's CFRAM Study programme:

- **Shannon Catchment Flood Risk Assessment and Management Model S19:** This hydraulic model was developed under the national CFRAM study and is the most detailed hydraulic modelling study carried out in the area to date. CFRAM Model S19 includes Kilkee town (designated an Area for Further Assessment (AFA)) and the surrounding area, shown in Figure 2-1. The model used 1D-2D and was constructed using the ISIS-TUFLOW link based on the combination of the one-dimensional river modelling package ISIS and the two-dimensional modelling software TUFLOW, full details of the model schematisation and development can be found in the Shannon CFRAM hydraulic modelling report. The model has been examined to help inform the construction of the flood relief scheme model. The CFRAM modelled extents are shown in Figure 2-2 and Figure 2-3.

  The preferred option from the CFRAM is shown in Figure 2-4 and Table 2-1

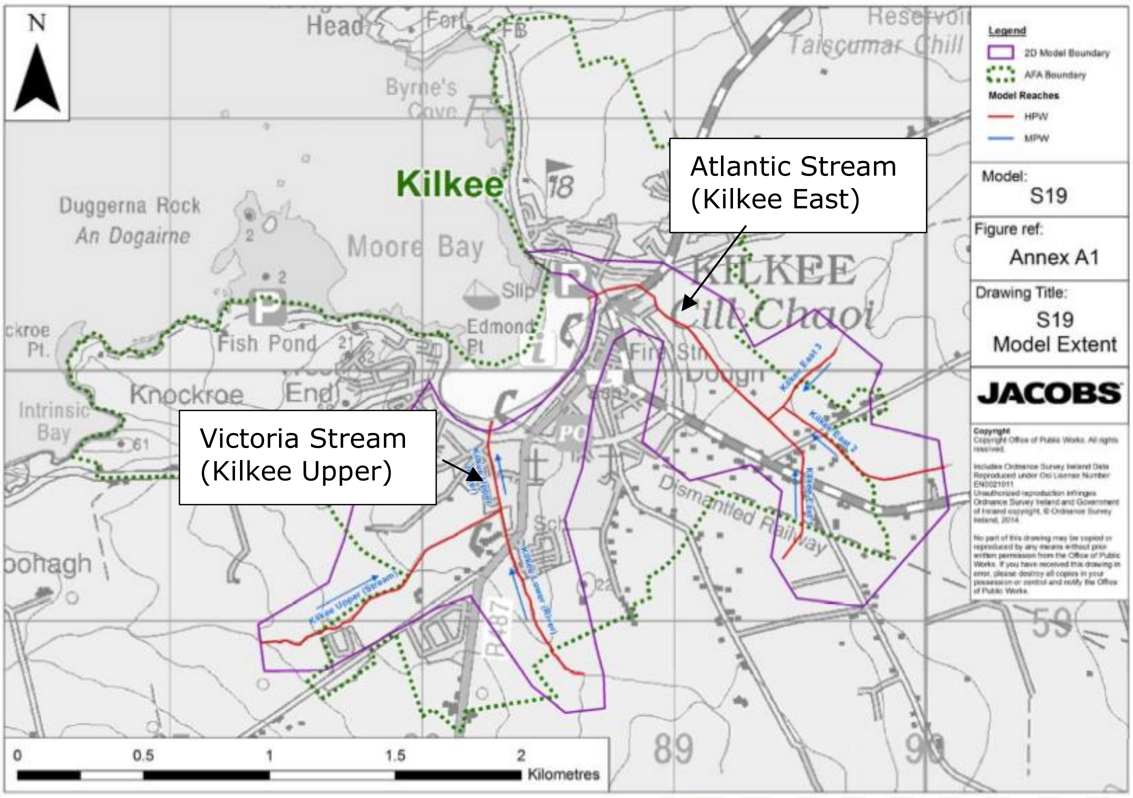
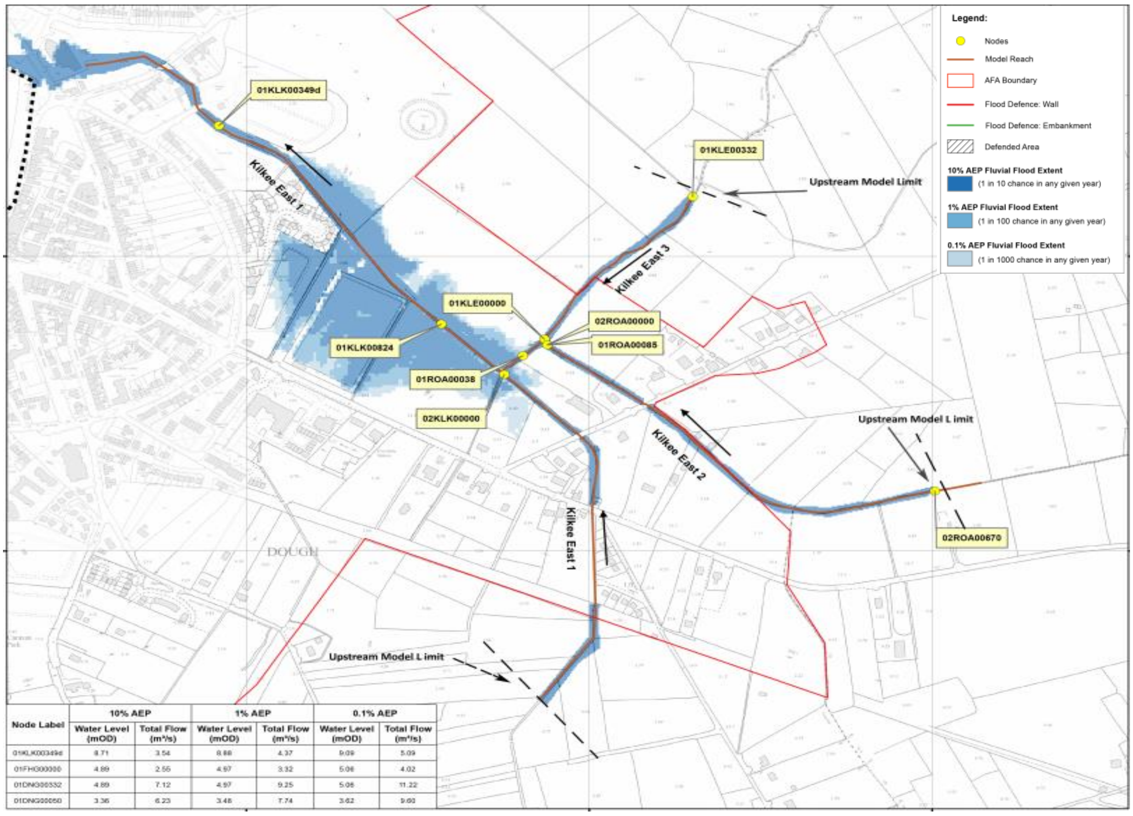
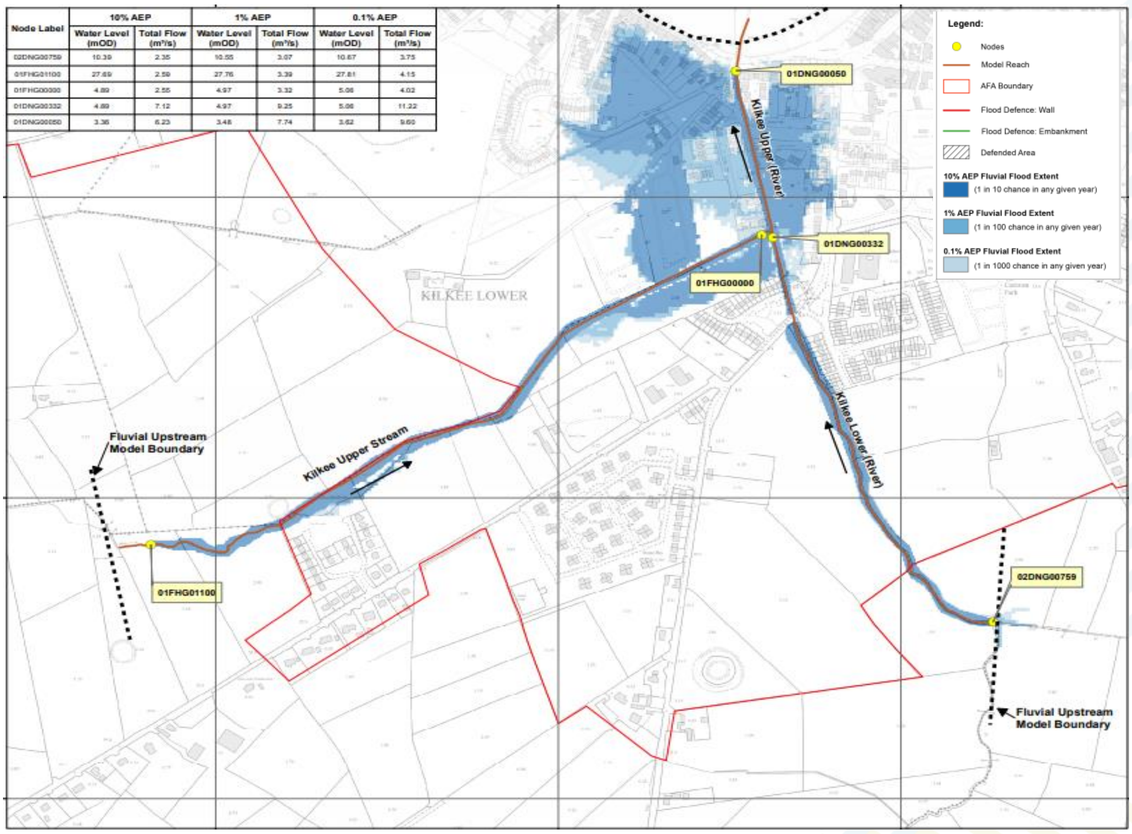


Figure 2-1: Shannon CFRAM Model S19 Extent (Source Shannon CFRAM Hydraulic Report UoM Annex A1)



Node Label	10% AEP		1% AEP		0.1% AEP	
	Water Level (mOD)	Total Flow (m^3/s)	Water Level (mOD)	Total Flow (m^3/s)	Water Level (mOD)	Total Flow (m^3/s)
01KLK00349d	8.71	3.54	8.88	4.37	9.09	5.09
01FHG00000	4.89	2.55	4.97	3.32	5.06	4.02
01DNG00332	4.89	7.12	4.97	9.25	5.06	11.22
01DNG00050	3.36	6.23	3.48	7.74	3.62	9.60

Figure 2-2: CFRAM Fluvial Flood Map Atlantic Stream (Source: floodinfo.ie)



Node Label	10% AEP		1% AEP		0.1% AEP	
	Water Level (mOD)	Total Flow (m^3/s)	Water Level (mOD)	Total Flow (m^3/s)	Water Level (mOD)	Total Flow (m^3/s)
02DNG00759	10.39	2.35	10.55	3.07	10.67	3.75
01FHG01100	27.69	2.59	27.76	3.39	27.81	4.15
01FHG00000	4.89	2.55	4.97	3.32	5.06	4.02
01DNG00332	4.89	7.12	4.97	9.25	5.06	11.22
01DNG00050	3.36	6.23	3.48	7.74	3.62	9.60

Figure 2-3: CFRAM Fluvial Flood Map Victoria Stream (Source: floodinfo.ie)

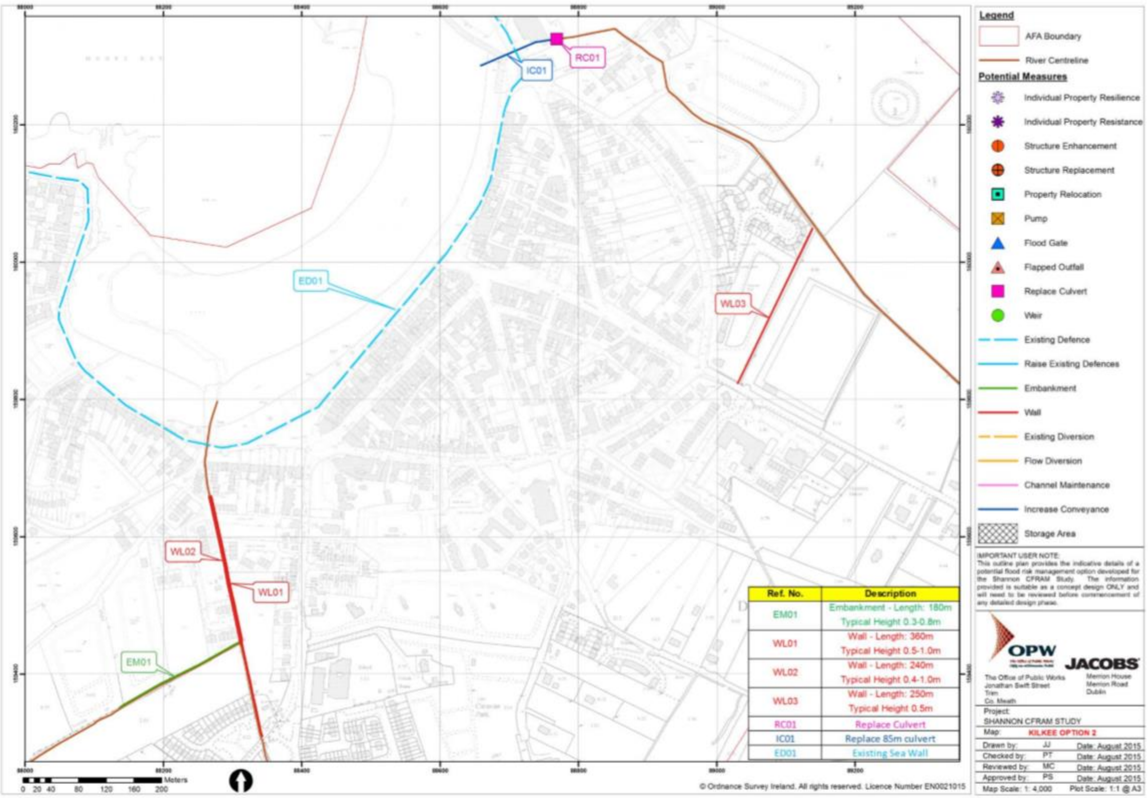


Figure 2-4: CFRAM preferred option

Table 2-1: CFRAM preferred option

Ref No	Description
EM01	Embankment - Length: 180m Typical Height: 0.3-0.8m
WL01	Wall: Length - 360m Typical Height: 0.5-1.0m
WL02	Wall: Length - 240m Typical Height: 0.4-1.0m
WL03	Wall: Length - 250m Typical Height: 0.5m
RC01	Replace Culvert
IC01	Replace 85m Culvert
ED01	Existing Sea Wall

2.2 DTM and Survey data

All survey has been provided in Irish National Grid coordinate system to Malin Head datum OSGM02 geoid.

2.2.1 Available DTM data

Table 2-2 summarises the DTM data that was made available for the study during the time of model construction and development. Figure 2-5 shows the coverage extents of the data sets.

Table 2-2: DTM data comparison

Source	OPW 5m	OPW 2m
Horizontal resolution	5.00m	2.00m
Quoted vertical accuracy bound	±500mm	±200mm
Date data received	14/01/2020	14/01/2020

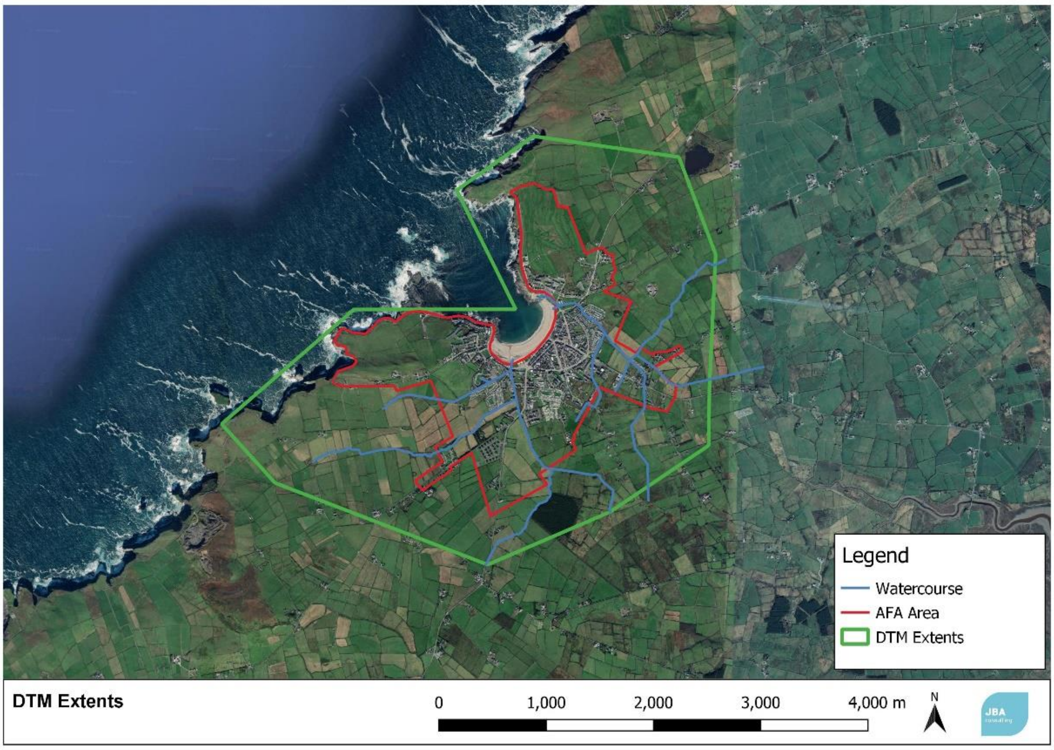


Figure 2-5: DTM Extents

2.2.2 Available Survey data

- **Shannon CFRAM river channel survey (Murphy Surveys 2012)[1]:** This cross section topographic survey data was collected for the Shannon CFRAM Model S19 and has been provided for this project. The survey cross sections cover the

1 N:\2019\Projects\2019s1431 - Clare County Council - Kilkee Flood Relief Scheme\02.Client_Shared\Incoming\OPW-20200218-CFRAM\CFRAM Survey\Watercourses\Kilkee\27S19_KILF_V0\27S19_KILF_V0

major watercourses considered. The data has been reviewed and there are no major alterations to the channels since the survey was taken. Figure 2-6 shows the locations of the CFRAM survey cross sections.

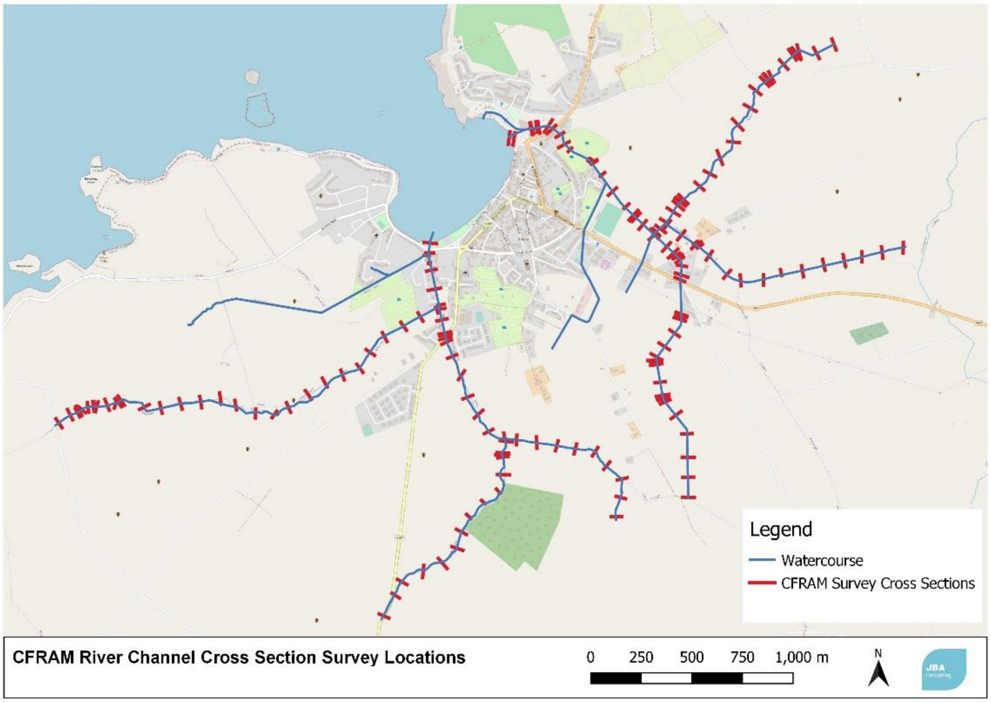


Figure 2-6: CFRAM Survey Cross Section Locations

- **Additional River Channel Survey (MCDS Surveys Ltd 2020)**[2]: Following review of the CFRAM model and survey additional survey was deemed necessary to meet the project requirements as general survey areas were lacking detail. This additional survey was collected in October 2020 targeting locations where there were gaps in information or new watercourses to be included. Additional cross sections surveyed are shown in Figure 2-7.

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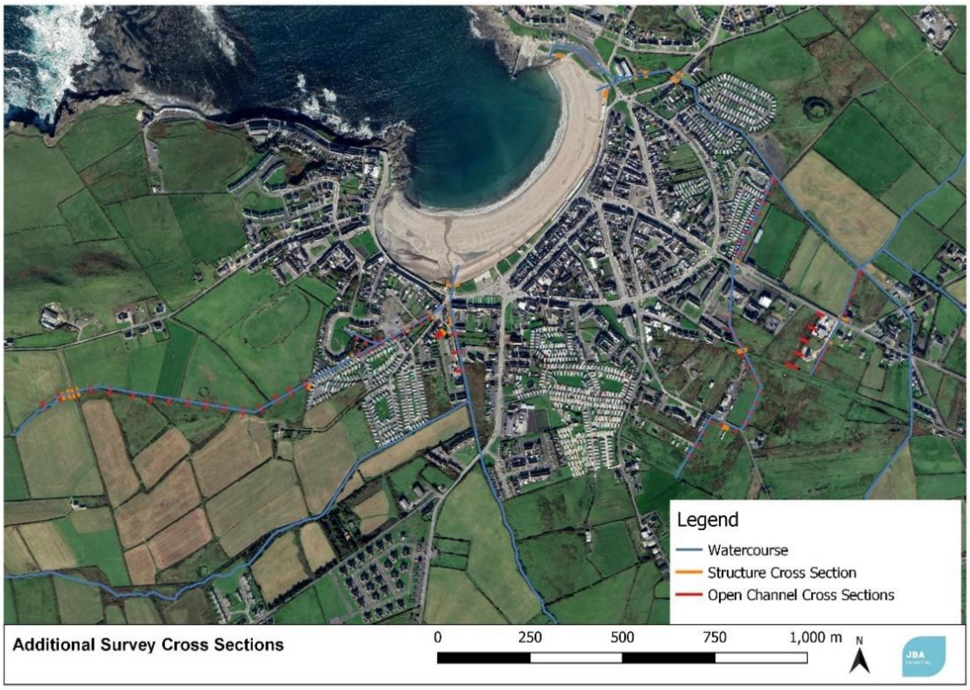


Figure 2-7: Additional Cross section locations

- **Culvert Surveys (Amelio Surveys 2020):** CCTV survey of a number of culverts was undertaken to determine details of culverts and manholes. In some cases, culverts details could not be confirmed. An overview of the CCTV Survey done by Amelio and the remaining gaps is shown in Figure 2-8.

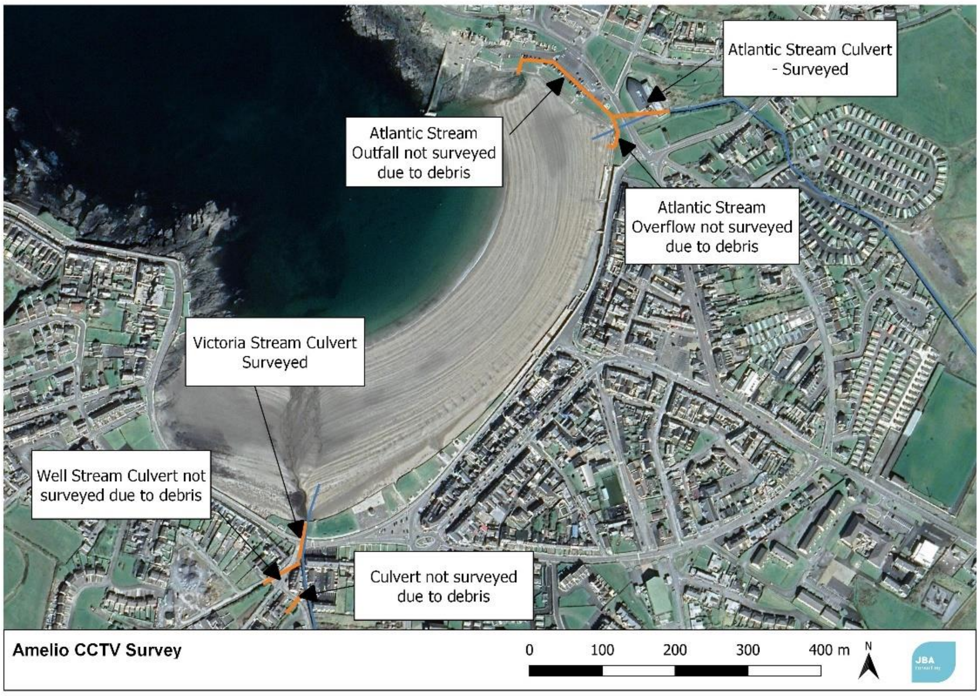


Figure 2-8: Overview of Amelio CCTV Survey

- **Additional Topographic Survey (MCDS Surveys Ltd 2020):** Topographical Survey of the downstream areas of the Victoria and Atlantic Streams was undertaken in 2020. This data was used in the model to represent changes in the topography since the DTM data was captured. The areas surveyed are highlighted in Figure 2-9**.**

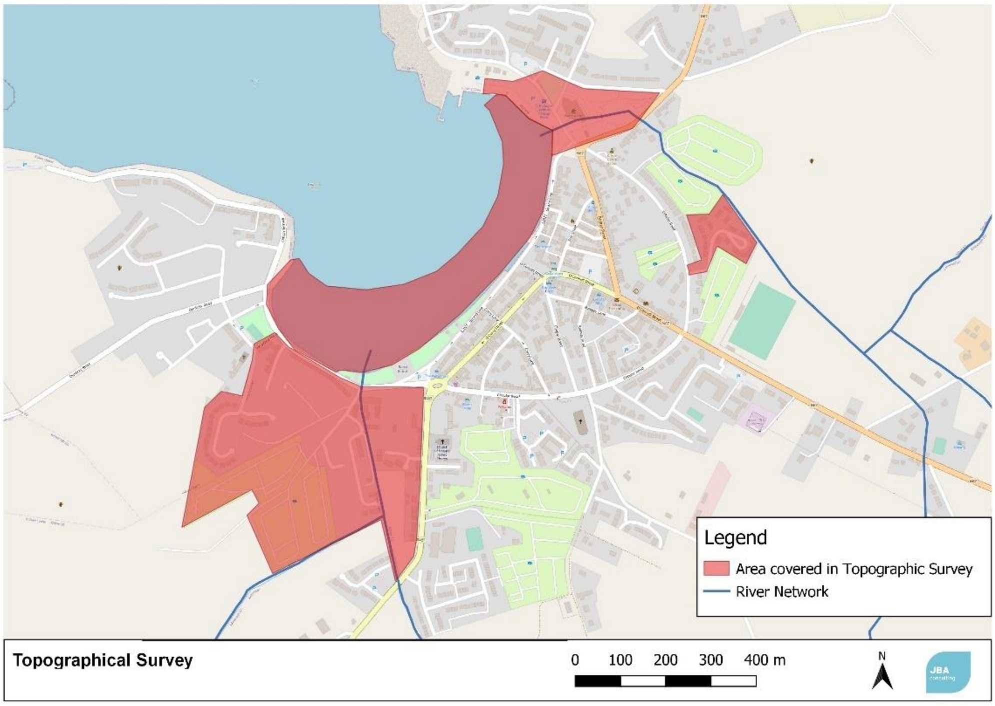


Figure 2-9: Areas covered by topographic survey

- **Culvert Surveys (Clare Drains 2021):** Additional CCTV survey of the Well Stream Culvert and the Atlantic Stream outfall was undertaken in an effort to fill in gaps in knowledge of the layout of these culverts, as shown in Figure 2-10.

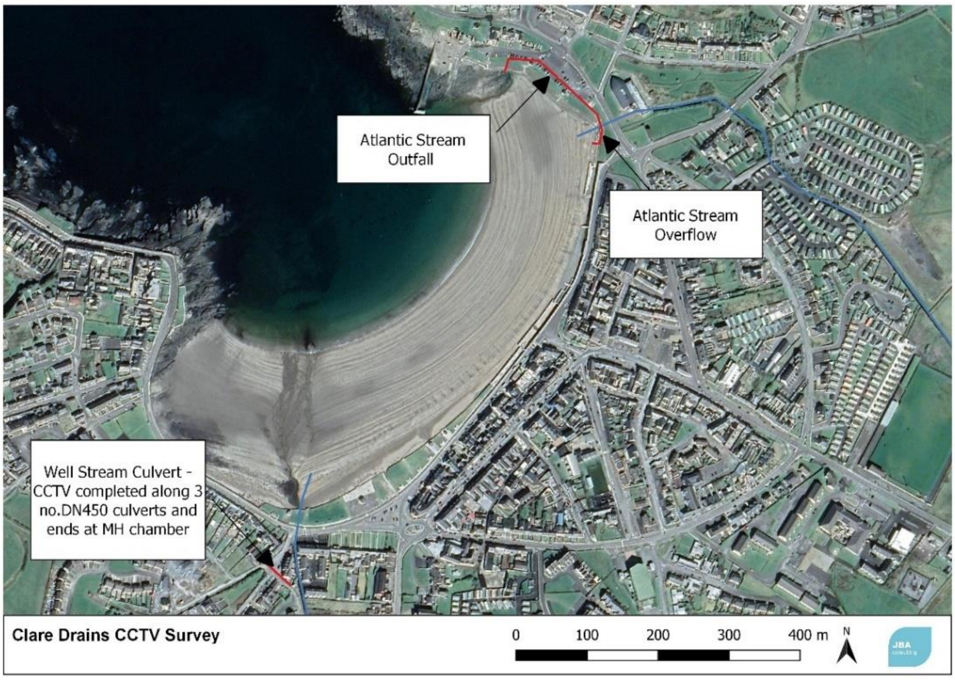


Figure 2-10: Clare Drains CCTV Survey

- **Laser Survey (MCDS Surveys Ltd 2020):** Laser survey of the sea wall was undertaken to determine details of the condition of the sea wall. Figure 2-11 gives an overview of the laser survey.

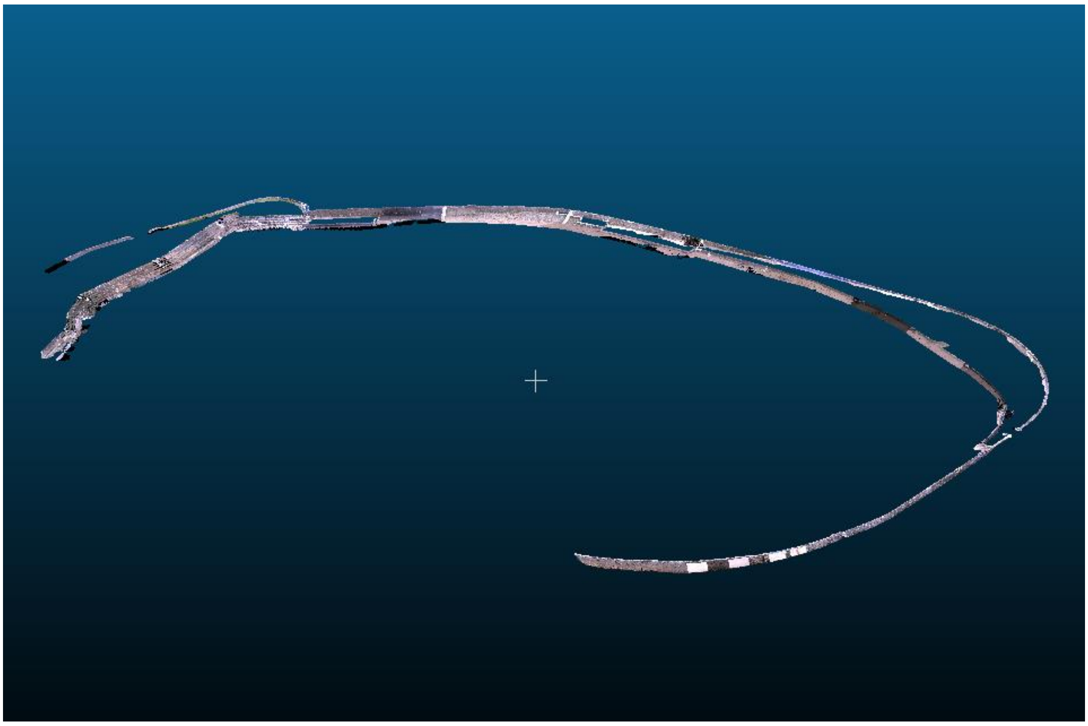


Figure 2-11: Laser Survey of Sea Wall

- **Culvert Surveys (Clare Drains 2021):** CCTV survey of the culvert at the Kilkee Bay hotel was undertaken by Clare Drains in September 2021. The culvert network surveyed is as shown in Figure 2-12Figure 2-10.

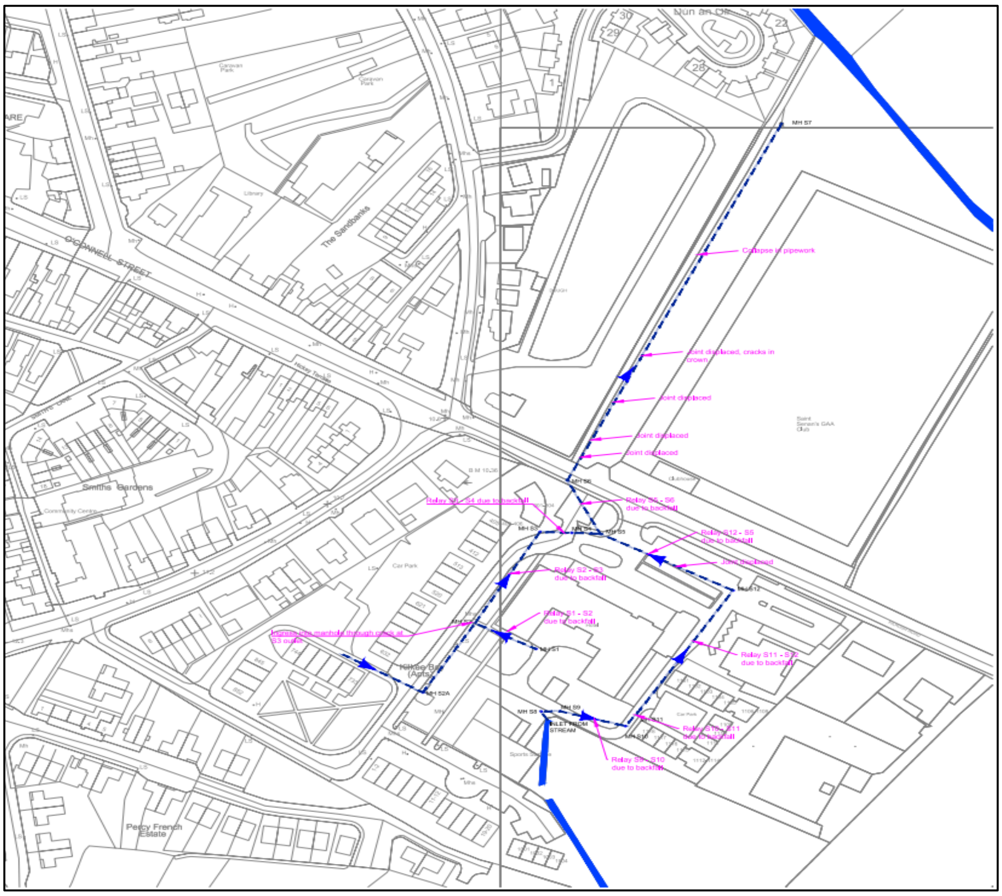


Figure 2-12: Clare Drains CCTV Survey of Kilkee Bay Hotel

- Stormwater Network Survey (Clare Drains). CCTV survey of the storm water pipe networks in the vicinity of Well Road and Victoria Court was completed in March 2022. The stormwater network identified in the survey is shown in Figure 2-13.

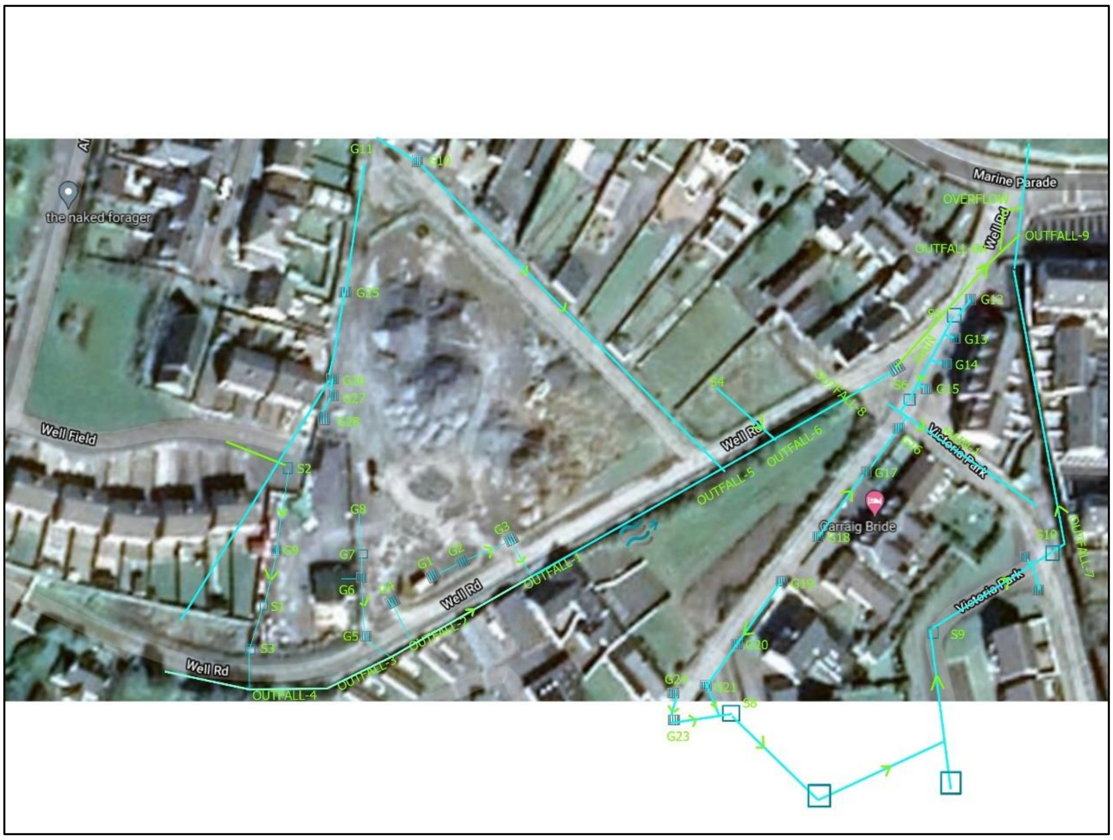


Figure 2-13: Clare Drains CCTV Survey of Victoria Stream

All survey data was reviewed to ensure it was fit for purpose, of an appropriate level of accuracy and gave sufficient information.

2.2.3 Comparison between DTM and Survey

2.2.3.1 DTM and CFRAM Survey Comparison

A comparison between the OPW 2m DTM data set and the CFRAM survey along the Atlantic and Victoria Streams was undertaken. The 2m DTM was chosen as it has a greater vertical accuracy than the 5m DTM. An example of the comparison between the DTM and the CFRAM survey is shown in Figure 2-14 for the Victoria Stream. The DTM and the survey were noted to match well with the largest differences identified at locations of wall, buildings, or heavy vegetation.

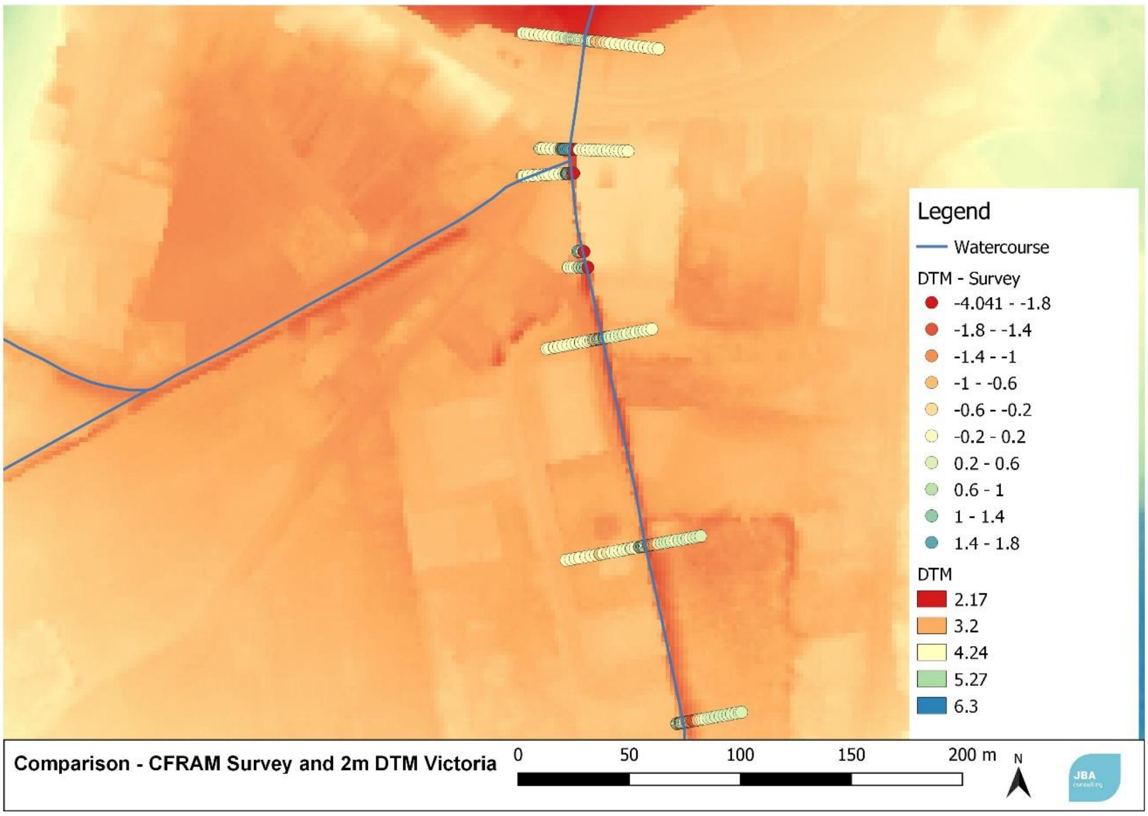


Figure 2-14: DTM and CFRAM Survey Comparison example at Victoria Stream

2.2.3.2 DTM and MCDS Survey Comparison

A comparison between the OPW 2m DTM data set and the topographic survey was undertaken. The survey and the LiDAR match relatively well, mostly within 1.50mm. However, there are some areas, as shown in which have heavy vegetation where there are large differences up to 900mm. Therefore, in these areas the topographic survey will be used over the LiDAR to represent the floodplain.

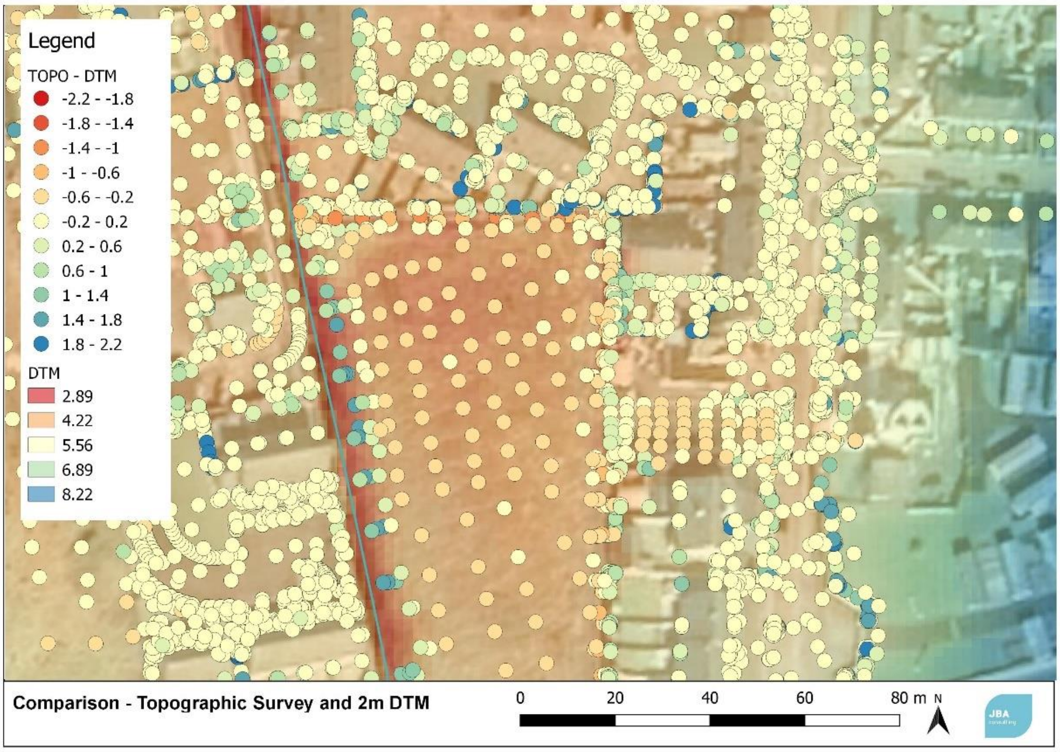


Figure 2-15: DTM and Topographic Survey Comparison example at Victoria Stream

2.2.4 Comparison between Surveys

The CFRAM Survey and the Additional River Channel Survey were also compared as shown below in Table 2-3. The comparison was completed at hardstanding points to allow for an accurate comparison. As can be seen below there is a difference between the CFRAM Survey and the additional survey.

A recheck was done by MCDS at the locations outlined below and little difference was noted between the recheck and the original MCDS survey, therefore the new channel survey was taken as the most up to date & accurate survey and used where possible. It should also be noted that the differences can be considered in the model output and how relevant they may be.

Table 2-3: Comparison between CFRAM and new survey

Location of Comparison	CFRAM Survey (mOD)	New Survey (mOD)	Difference (New-CFRAM) (m)
Victoria Stream			
Outfall Culvert Outlet Soffit	4.18	4.12	-0.06m
Outfall Culvert Inlet Soffit	4.11	4.04	-0.07m
Top of Wall at Outfall Culvert Outlet	4.35	4.29	-0.06m
Atlantic Stream			
Overflow Culvert Outlet Soffit	3.26	3.26	0
Soffit Outfall Culvert Inlet	6.28	6.33	0.05m
Soffit bridge N67	8.42	8.42	0
Footbridge Soffit	7.02	6.93	-0.09m

A combination of the CFRAM river channel survey and the updated river channel survey undertaken by MCDS was used in the model to represent the channel.

2.3 Hydrometric Gauges and Rain Gauges

Two water level gauges were temporarily installed in Kilkee in December 2020, one on the Victoria Stream and one on the Atlantic Stream. These were installed in December 2020 and will remain in place until December 2021.

Two temporary rain gauges were put in place from December 2020 to June 2021, one at the coast and the other in the upstream catchments of the Victoria Stream. A permanent rain gauge was installed by the OPW at the Irish Water pumping station. Figure 2-16 shows the location of the rain gauges and hydrometric gauges.

An example of an event recorded on the Victoria Stream is shown in Figure 2-17. It must be noted that no significant storms have been recorded in Kilkee since the installation of the gauges and therefore validation is limited.

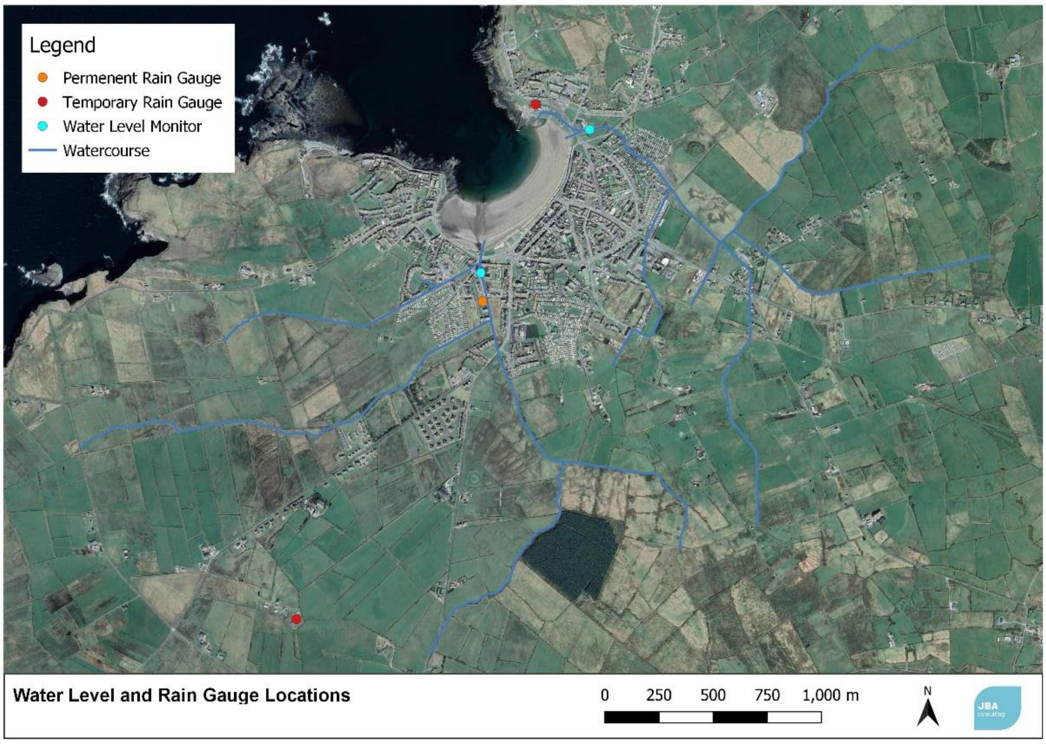


Figure 2-16: Rain gauge and water level gauge locations

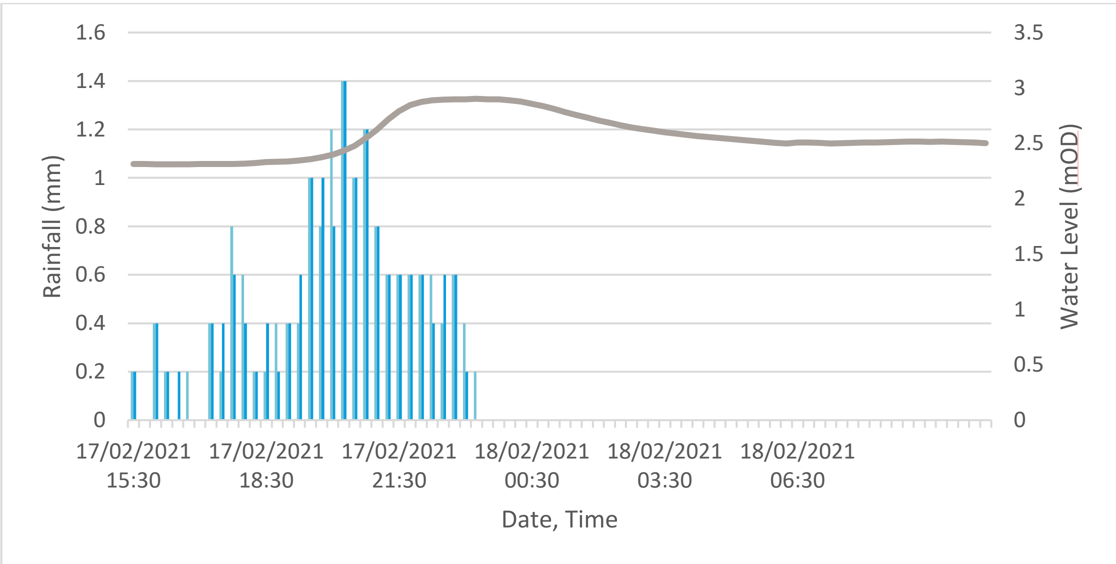


Figure 2-17: Example of event recorded on rain gauges and water level monitor on Victoria Stream (17/02/2021)

2.4 Flood history

A summary of the key events within Kilkee's flood history is provided. Refer to the Kilkee FRS Hydrology Report (19109-JBAI-XX-XX-RP-H-00344_Hydrology_Report_P03) for a full flood history review.

The following events in Kilkee were recorded:

- February 2020: Heavy rainfall as a result of Storm Ciara caused flooding along the Atlantic Stream, with most severe flooding in the vicinity of Waterworld and the sea wall. No properties were flooded during this event Possible blockage at the trash screens outfall was considered to contribute to the flooding.

- October 2019: Flooding along Well Road and Victoria Park as a result of heavy rainfall causing the Victoria Stream to overflow its banks. 3 no. houses were flooded internally and over 20 houses were close to being flooded. October 2019 flooding on Well Road is shown in Figure 2-18.

Figure 2-18: 2019 Flooding on Well Road

- February 2017: Flooding along Well Road and Victoria Park as a result of heavy rainfall from Storm Ewan, causing the Victoria Stream to overflow its banks
- April 2015: Flood event caused by heavy rainfall which resulted in the Atlantic Stream bursting its banks. 2015 flooding is shown in Figure 2-19.

Figure 2-19: 2015 Flood on Atlantic Stream

- January 2014: Significant coastal flood along the Clare coast. The coastal storm caused damage along the sea wall, resulting in the collapse of two sections.

Much of the beachside infrastructure was also destroyed, and properties along Marine Parade were flooded.

- February 1990: County Clare experienced serious tidal flooding with approximately 200 houses and many roads affected. Kilkee AFA was one of the most seriously affected areas.
- January 1965: Flood event caused by high tide and strong winds resulted in portions of the promenade wall being severely damaged and also affecting a house.
- October 1961: Flood event caused by torrential rainfall, damaging buildings along the seafront.
- December 1954: Event resulted in flooding to large areas of land and low-lying roads in Kilkee.
- October 1954: Flood Event was due to heavy rainfall and resulted in flooding to low lying roads and land in Kilkee.
- Recurring Events: Recurring flooding affects Church Street, Carrigaholt Road and Well Road car park when the Victoria Stream just north of the R487 road overflows its banks. This can affect a number of houses and is reported to happen approximately once a year. The flooding situation is said to be exacerbated by tides and winds.

2.5 Operation and management of the streams

2.5.1 Victoria Stream

The outfall of the Victoria Stream discharges onto the blue flag beach. As a result, during the summer months weir boards are put in place at the outfall to restrict flow onto the beach and maintain the blue flag status. Weir boards are shown in Figure 2-20. The flow from the Victoria Stream builds up behind the weir boards and is diverted through the Irish Water pumping station to Intrinsic Bay.

The weir board system can be defeated in times of heavy rain. This can temporarily close the beach for the blue flag status, and it can be difficult to remove the weir boards safely if not done in advance of the storm based on prior notification.

The gate in place downstream of the stop logs is only in place during the summer months when the stop logs are in place and is removed in the winter months when the stop logs are removed.

The pump has also been excluded from the model as the diverted flows are insignificant compared to flood flows and the pump is also turned off in significant events.

Figure 2-20: Victoria Outfall with weir board in place

2.5.2 Atlantic Stream

There are two screens located at the downstream end of the Atlantic Stream, a security screen at the inlet of the outfall culvert and a trash screen upstream of this, Figure 2-21. Both screens block with reeds/weeds which can result in flooding. There are safety issues related to the clearing of the screens particularly in times of high flow.

Figure 2-21: Trash Screen on Atlantic Stream

2.6 Stormwater System

There is an extensive combined sewer system in Kilkee. In conjunction with this there is also a stormwater system in Kilkee town. The trunk stormwater system is connected to the stream as shown in Figure 2-22, the network is connected to the main channel and the tributaries. Along the Atlantic Stream the storm water network connects to the culvert alongside the GAA field and at two points along the main channel. Along the Victoria Stream the storm water network connects at three points along the main channel. The storm water system has been accounted for in the catchment delineation and therefore included in the inflows. Any smaller storm water networks or combined sewer networks have not been included in the figure below.

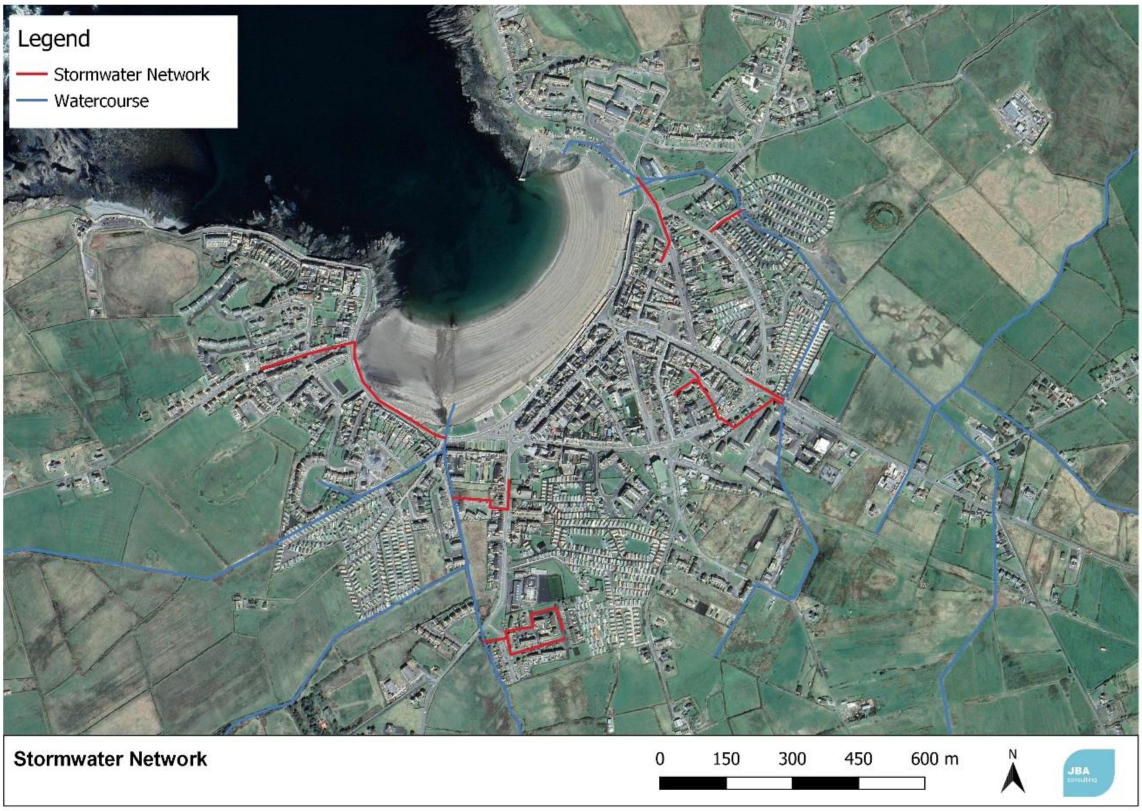


Figure 2-22: Kilkee Stormwater Network

2.7 Hydrological assessment

Refer to the Kilkee FRS Hydrology Report (19109-JBAI-XX-XX-RP-H-00344_Hydrology_Report) issued in November 2020 (or any further updates) for this project for a full description of the methods used to estimate inflows. In summary:

- Inflow hydrographs were developed using FM FSSR 16 inflow units for the upstream inflow points for all watercourses with considerable upstream catchments.
- Lateral flow hydrographs were also developed using FM FSSR 16 units to represent the overland flow entering the watercourses along the reaches
- The inflow hydrographs were generated using the necessary catchment descriptors for each individual catchment.

3 Model development

3.1 Software

The model was developed using Flood Modeller and TUFLOW software packages creating a linked 1D-2D model. The outfall to the Atlantic Stream was modelled using Causeway FLOW software package (a similar package to WINDES/MicroDrainage used in a ID format) due to the complexity of the culvert arrangement and the difficulties of modelling this element in Flood Modeller. Hydrographs were generated for the Atlantic Stream from Flood Modeller for input into FLOW and the level output files for a given storm return period from FLOW input into Flood modeller to create a boundary.

3.2 Overview

Table 3-1 shows a summary of general model details. For efficiency two separate models were developed, the Victoria model and the Atlantic model. The Victoria model includes all the watercourses that flow within the Victoria Stream system and the Atlantic model includes all those watercourses within the Atlantic Stream system. This division of the watercourses into two separate models could be done because there is no hydraulic connection between the two. The schematisation of the 1D and 2D model components are shown in Figure 3-1 and Figure 3-2

A 2m grid cell size was selected for the 2D domain. This resolution was chosen due to the size of the channels within the system.

The model has been completed to geoid OSGM02, to keep in line with survey completed as part of the CFRAM study.

Table 3-1: Hydraulic model summary

1D model	Value – Victoria Model	Value – Atlantic Model
Total 1D modelled length	2.232km	4.59km
Number of modelled structures	8	19
1D timestep	0.5	1
Number of inflows	13	11
Number of outflows	1	1
2D model		
Total model area	3.52km2	3.41km2
Model orientation	South to North	South to North
2D grid cell size	2m	2m
2D timestep	0.5	0.5
1D-2D model linkage	HX & CN lines	HX & CN lines
Average model run time	3hrs	4hrs
FLOW model		
Outfall culvert length		106m
Number of inflows		1 – from Atlantic Stream
Number of outflows		2 – outfall and overflow

3.3 Schematisation

This section discusses the schematisation of the model and how the different aspects of the model including boundaries have been applied. Figure 3-1 and Figure 3-2 show the schematisation of the model in the 1D for the Victoria and Atlantic Streams respectively. Figure 3-3 and Figure 3-4 show schematisation of the models in 2D.

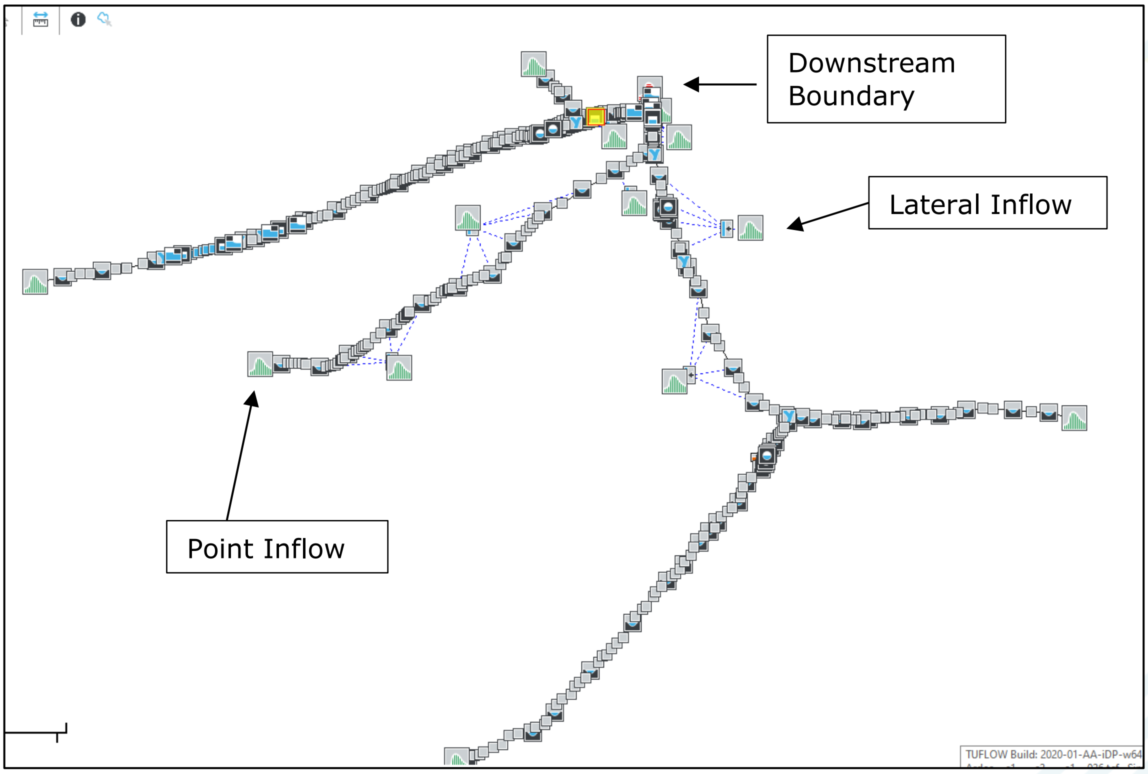


Figure 3-1: 1D Schematisation of Victoria Model

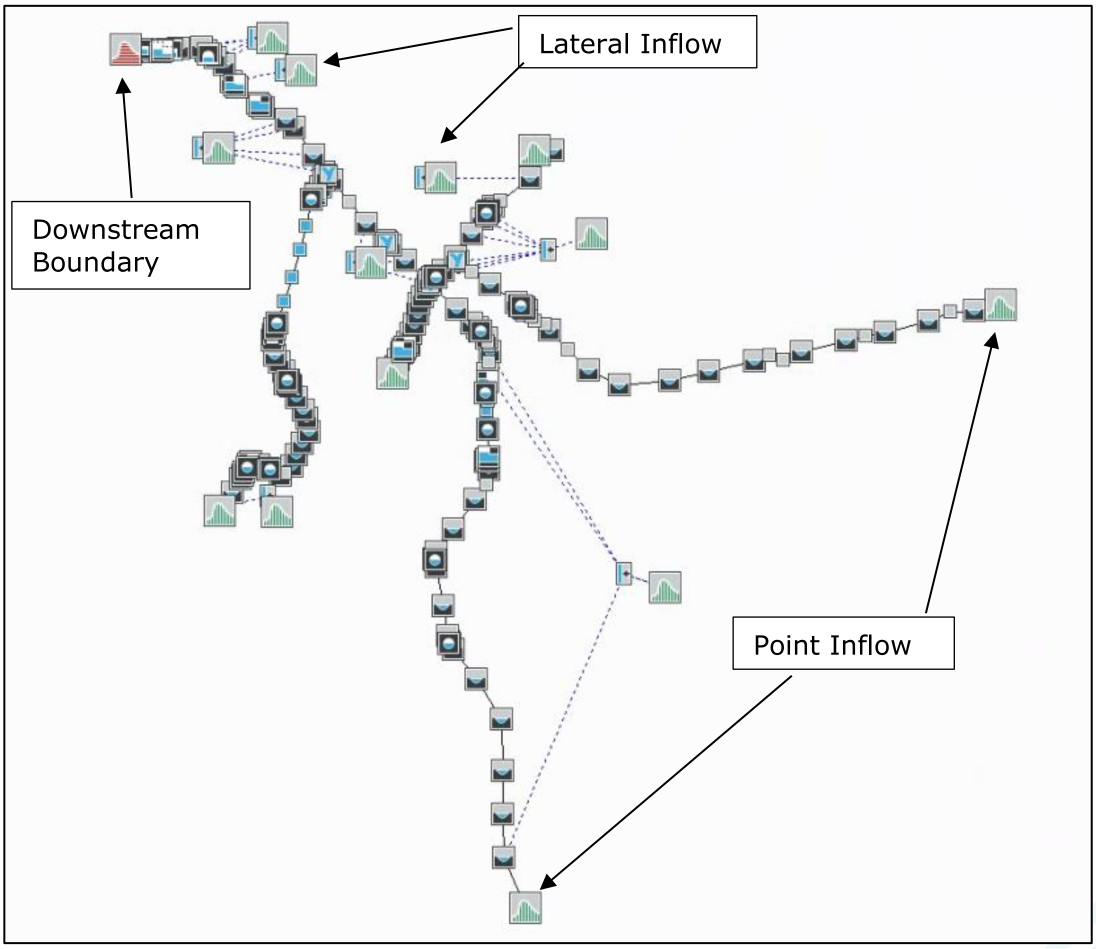


Figure 3-2: 1D Schematisation of Atlantic Model

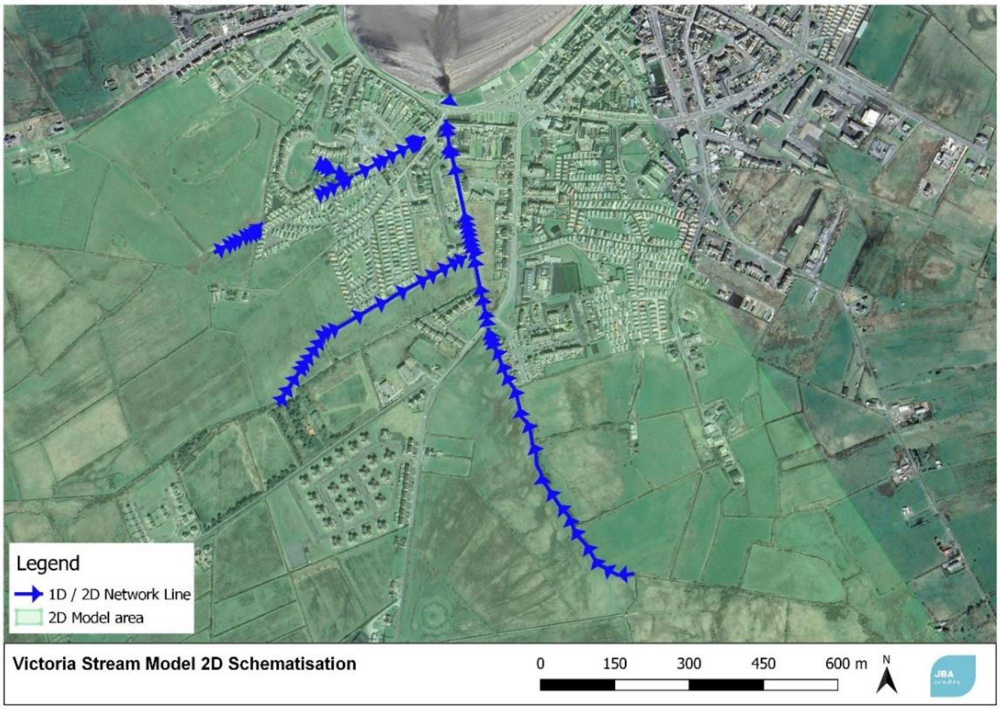


Figure 3-3: 2D Schematisation of Victoria Model

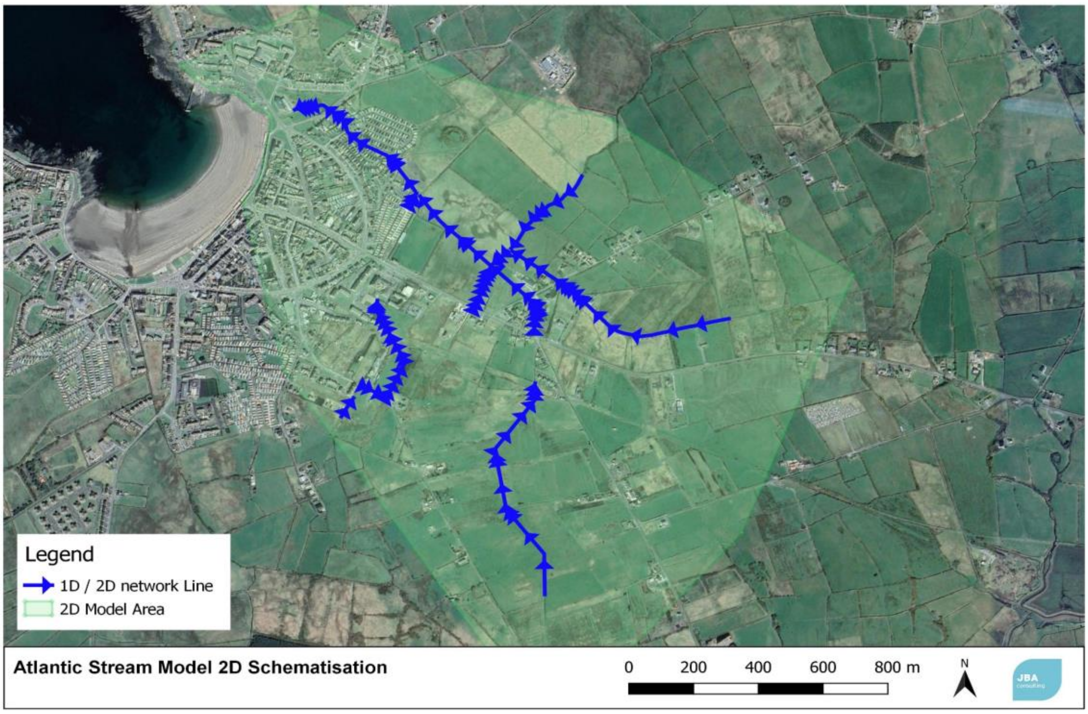


Figure 3-4: 2D Schematisation of Atlantic Model

3.3.1 Inflow boundaries

All inflow hydrographs have been generated using the FSSR16 FM units and then applied to the models. Refer to the Kilkee FRS Hydrology Report (19109-JBAI-XX-XX-RP-H-00344_Hydrology_Report_P03) for details on the estimation of the hydrological inflows for this study.

The hydrographs and flows calculated for this study were applied in the 1D component of the models. Figure 3-1 and Figure 3-2 shows the location of the point inflows within the models. There are 8 lateral inflows in the Victoria Model and 7 lateral inflows in the Atlantic Model.

3.3.2 Downstream boundaries

The Victoria outflow boundary is based on the calculated tidal curves. The peak of the tidal curves are based on the IBE1781_CWWS_Kilkee_Rp01_D01 report issued in August 2021 and includes for wave set up. Wave set up can occur on wide relatively flat beaches where large storm waves break and reform resulting in a lowering of the mean sea level under the first line of breaking waves and a subsequent increase in the mean sea levels closer to the beach. This increase or set-up of the mean water levels close to the beach means that reformed waves approaching a coastal structure at the back of the beach may be slightly larger due to the locally increased water depth at the toe of the structure. The height of the waves that can approach Kilkee is strongly influenced by the water depth in the area, therefore wave set-up could potentially have a significant impact on the inshore wave climate in Kilkee.

An example of the tidal curve for the 0.5% AEP and the 50% AEP are shown in Figure 3-5. As can be seen from the curves the surge event occurs only over a defined period and the curve then reverts to mean hight water surface.

Refer to Appendix B for details on the tide curve generation and wave overtopping analysis.

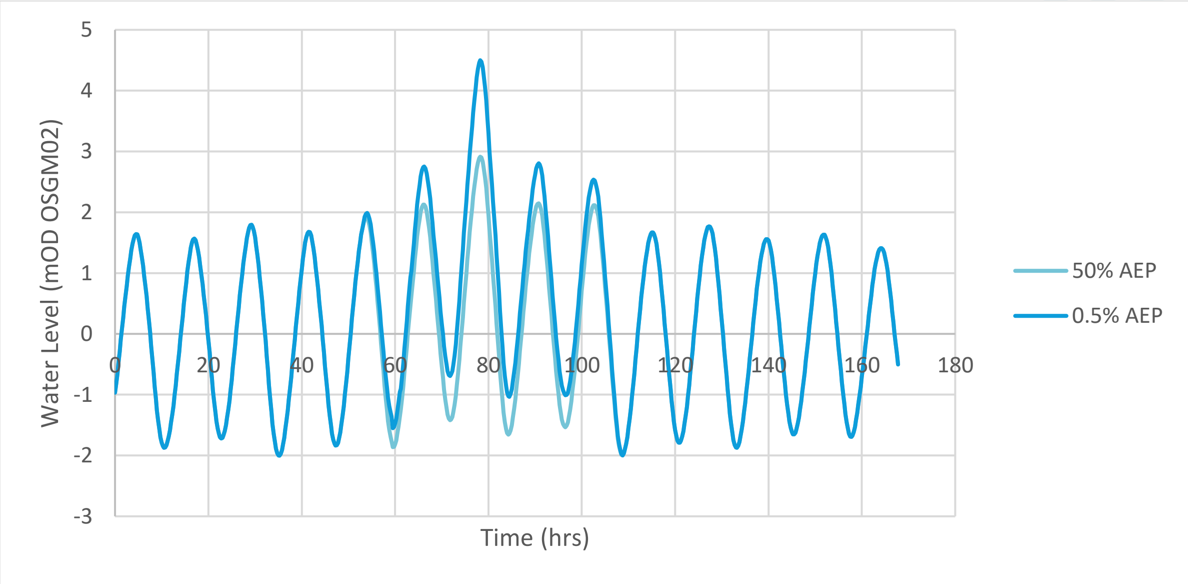


Figure 3-5: Tidal curves for downstream boundary

The Atlantic outflow boundary is based on the water level at the end of the model i.e., the upstream of the outfall culvert as exported from FLOW software for a given return period.

A 2D outflow HQ boundary has also been included in the TUFLOW model along the sea wall. A HQ boundary assigns a water level to the cell based on a water level versus flow curve. This boundary allows any flow to exit the 2D model along the sea wall, however the model runs show that there is no flow in the fluvial – tidal model that passes this boundary.

3.3.3 1D - 2D boundaries

The 1D is connected to the 2D using connection lines called HX and CN lines. Water level in the 2D boundary cells is determined based on the flow from the 1D node and, conversely, water level in the 1D node is determined based on the average water level along the 2D boundary cells. Flow is proportioned via depth because multiple cells are connected to a single 1D node. The HX and CN lines connect the 1D node water level to the 2D boundary cells and vice versa.

HX lines have been applied at the 1D-2D boundaries within the models, namely at the top of banks (TOB) of the watercourses. The HX line elevations (TOB) were sourced from surveyed data and intermediate points from the underlying 2m LIDAR data, see Section 2.2 for detail of the LiDAR. This ensured that the crest levels of the channel in the 1D model were being read into the 2D models. The TOB levels sourced from the LIDAR were reviewed to ensure that they were appropriate and consistent with the surveyed levels. A 2D inactive area was applied to remove the 1D modelled areas from the 2D domain. Figure 3-6 gives an example of HX and CN lines in the model.

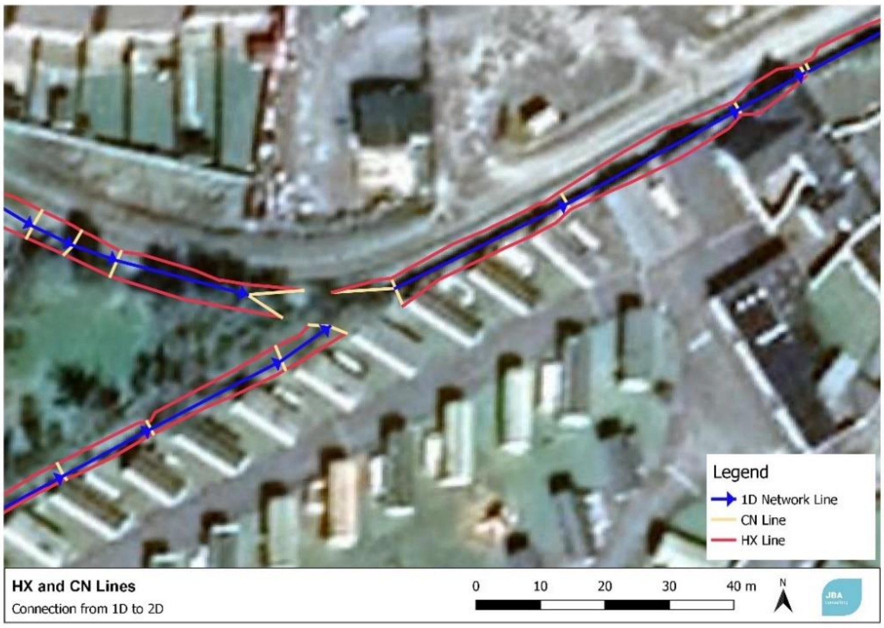


Figure 3-6: Example of HX and CN boundaries

3.4 Coefficients used

3.4.1 Culvert coefficients

In general, the default culvert inlet and outlet coefficients provided with the FM software to the 1D culvert structures within the model were used. This was considered appropriate as there was no evidence provided to warrant a deviation from the default values such as significant wear and tear on a structure or extreme scouring of channel bed. Any deviations from the generic values or approach for specific structures are recorded in Section 4.

3.4.2 Weir coefficients

Weir coefficient values were applied to all spill units used to represent overtopping of structures in the 1D model or for the connection of tributaries to main channels where a drop in bed level was observed. The coefficients used were reviewed and chosen based on the condition of the surface above the structure and to ensure the value was representative of reality. Refer to Section 4 for values used for individual structures.

3.4.3 Screen losses

A number of trash screens were recorded at the inlets to culverts along the modelled watercourses. To ensure the hydraulic effect of the screens is accounted for in the model screen loss coefficients were included in the FM Culvert Inlet units. The losses at each screen were estimated using the following equation:

$$K_t{}^* = 2.45A_r - A_r^2 \ *$$

Where:

$K_t{}^*$ *= Head loss coefficient*

A_r *= Area Ratio =* A_b/A

A_b *= Blockage surface area of the screen bars*

A *= Gross flow area at the screen*

**Equation 7.26 from the Queensland Urban Drainage Manual Professional Edition 2013*

3.4.4 Manning's N – Roughness

3.4.4.1 1D channel roughness

Different Manning's N values have been used to represent roughness within the model. The Manning's N values applied have been sourced from a number of sources including Chow 1959, general values applied in hydraulic modelling, site walkover and consultation of photographs and survey notes.

Table 3-2: Examples of Manning's roughness values for Open Channels

Channel description	Chow 1959 channel no.	Manning's N value	Example cross section photograph
Clear wide channel with increased weeds	1.b	0.040	 Model Ref: 27ATLA00015
Wide channel but with increased stones and weeds	1.f	0.050	

Channel description	Chow 1959 channel no.	Manning's N value	Example cross section photograph
			Model Ref: 02DNG00167
Unmaintained channel, dense weeds and slow-moving water	4.e.1	0.080	 Model Ref: 01KILK00650
Unmaintained channel with weeds on banks but clean bottom	4.e.2	0.050	 Model Ref: 02DNG00000
Unmaintained channel with weeds on banks but clean bottom	4.e.2(High end)	0.08	 Model Ref: 02DNG00668
Unmaintained channel, dense brush and slow-moving water	4.e.4	0.1	 Model Ref: 27KILF00198

Channel description	Chow 1959 channel no.	Manning's N value	Example cross section photograph
Earth drainage channel with sluggish water, stony bottom and weedy banks	4.b.5	0.035	 Model Ref: 01DNG00103
Gravel bottom with sides of random stone mortar	5.e.2(high end)	0.026	 Model Ref: 01DNG00060

Table 3-3: Examples of Manning's roughness values for Conduits

Conduit Description	Manning's N value	Example Conduit photograph
PE Pipe – Corrugated inner walls	0.02	 Model Ref: 01KIL00476
Concrete Culvert	0.02	 Model Ref: 02DNG00147
Rubble Masonry	0.025	 Model Ref: 02KLK00459

3.4.4.2 2D floodplain roughness

The surface roughness, including buildings and various land uses within the 2D model, has been applied using a 2D materials layer. The different Manning’s n roughness values given to each land-use have been based on values from site visits, consultations of photographs, Chow 1959 and general values applied in hydrological modelling. Refer to Figure 3-7 for the modelled land use types and Table 3-4 for the corresponding Manning`s roughness values applied.

Buildings and caravans have been modelled by applying a high roughness value to them (n=0.3) in order to ensure that water preferentially flows around buildings/caravans before moving through them.

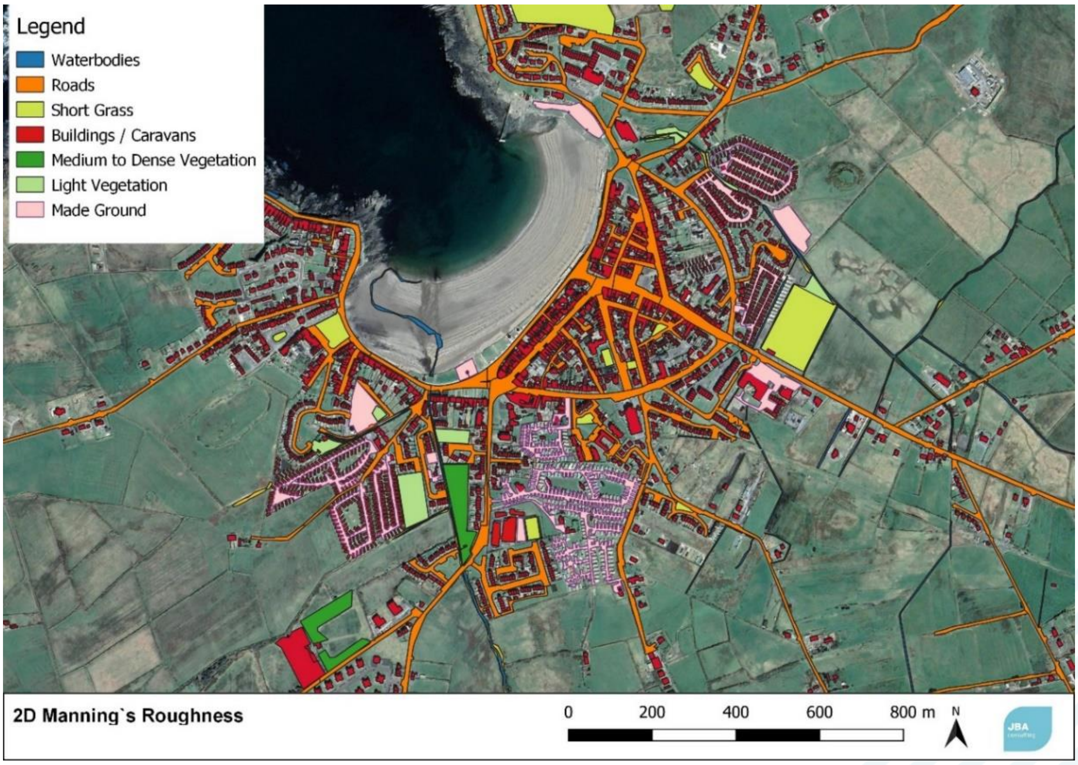
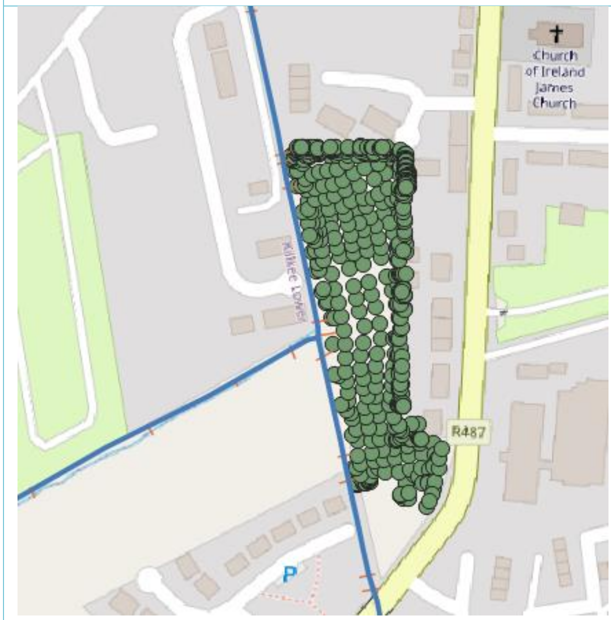


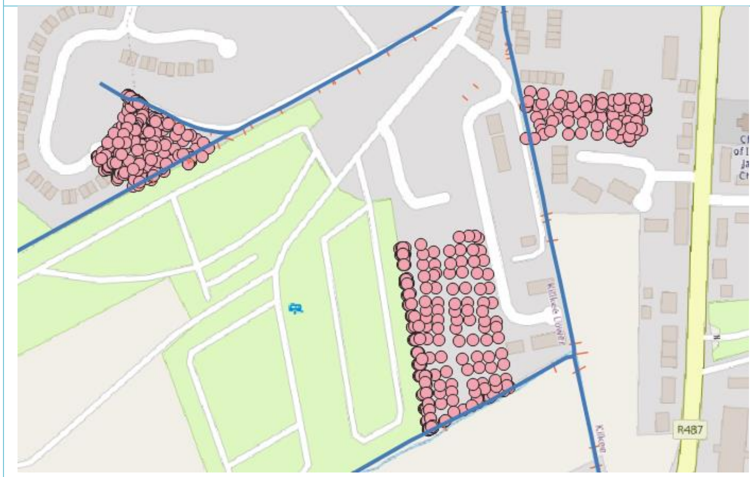
Figure 3-7: 2D Manning`s Roughness

Table 3-4: Manning’s roughness values applied to the 2D floodplain

Surface	Manning’s N value applied
Buildings/Caravans	0.300
Made ground	0.035
Roads	0.025
Short Grass	0.035
Waterbodies	0.020
Forested areas	0.100
Dense Vegetation	0.080
Medium to dense brush	0.070
Light Scattered Brush	0.050
General Rural (baseline layer)	0.045
General Urban	0.060

3.5 Topographic features

Modification	Shapefile
The elevation at the points of the 2D spills along the HX lines for both the Atlantic and the Victoria Streams has been adjusted using 2D zsh points and lines to represent the levels of the road.	2D_zsh_ATL_P_001 / 2D_zsh_ATL_L_001 2D_zsh_VIC_P_001 / 2D_zsh_VIC_L_001
 The LiDAR at the location of the CCC compound area was not deemed representative as it is known that some embankments have been constructed and road levels raised due to recent flooding. To represent this the area was replaced with recent topographic survey using Z tin points and polygon	2D_ztin_Compound_P_001 / 2D_ztin_Compound_R_001
 The LiDAR at the marshy field upstream of the pumping station was noted to differ from the topographic survey due to the vegetation. The topographic survey was therefore used using z TIN points and polygon. This was separated from other	2D_ztin_VIC_Field_P_001 / 2D_ztin_VIC_Field_R_001

areas of heavy vegetation as this field may be of interest for scheme works	
 The LiDAR at three areas along the Victoria Stream have been noted to differ from the topographic survey due to heavy vegetation. The topographic survey was therefore used using z shape points.	2D_ztin_VIC_HeavyVegetation_P_001 / 2D_ztin_VIC_HeavyVegetation_R_001
 To be more representative of the area at the Atlantic Stream outfall, the topographic survey data was used using z shape points.	2D_ztin_ATL_Outfall_P_001 / 2D_ztin_ATL_Outfall_R_001

3.5.1 Retained Walls and Embankments

3.5.1.1 Victoria Stream Model

A number of walls and embankments have been purposefully left in along the Victoria Stream. There are currently no formal defences in place along the Victoria Stream, but a number of informal embankments exist. These were included in the model as they have an impact on the flood mechanism. A number of boundary walls which have an impact on the flood mechanism were also included. Some boundary walls which have no impact on the flood mechanism at present were also included as they may become important as the scheme progresses. See Figure 3-8 for details of the defences along the Victoria Stream

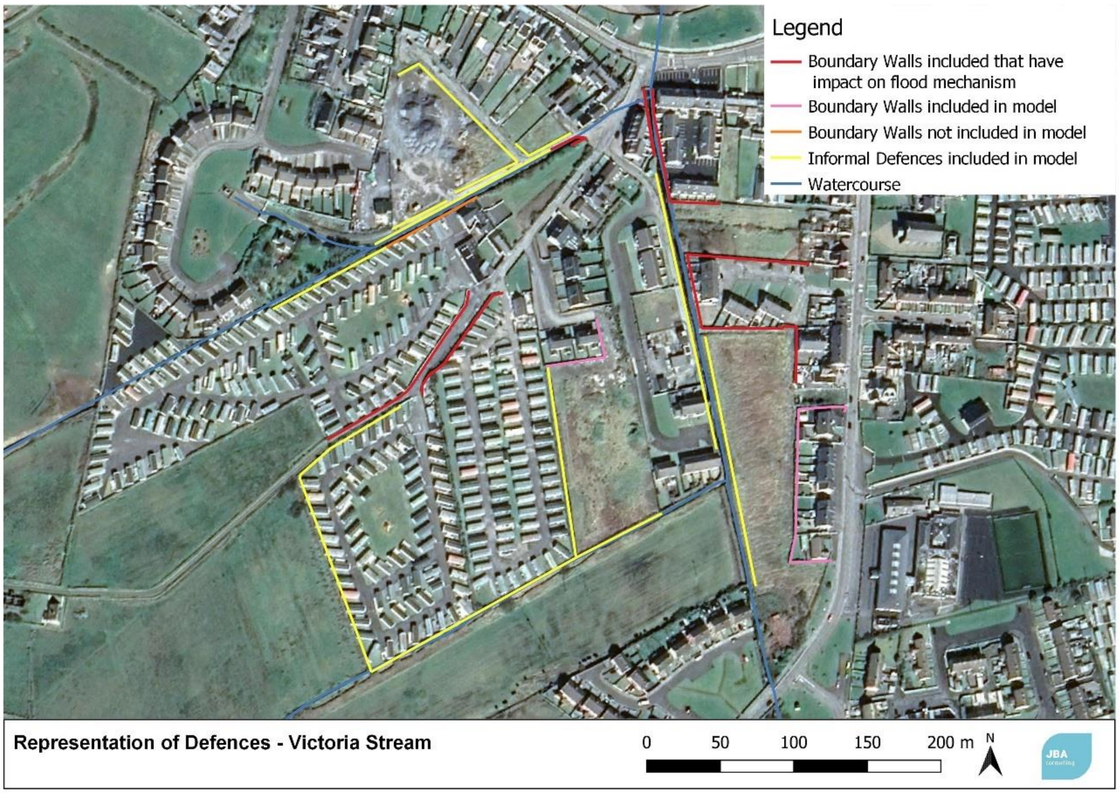


Figure 3-8: Walls retained in Model along Victoria Stream

3.5.1.2 Atlantic Stream Model

A number of walls and embankments have been purposefully left in along the Atlantic Stream. The boundary wall along the caravan park was included in the model as a formal defence wall. The GAA Field boundary wall, although not a formal defence, was also included in the model as it impacts the flood mechanism. Gaps which exist in the GAA Field wall due to fences / gates were included in the model to be representative of on-site conditions. The retaining wall along the housing estate was also included in the model, this wall does not provide any flood protection and is overtopped in the 1% AEP event, however it was included in the model as it may become important to the scheme as it progresses.

See Figure 3-9 or details of the defences along the Atlantic Stream.

Figure 3-9: Walls retained in Atlantic Stream Model

3.5.2 Representation of additional reaches

The scope of this project specified an examination of four additional small reaches which were not included in the CFRAM study:

- Additional Reach 1: Well Stream
- Additional Reach 2: Drain/Stream tributary of the Atlantic Stream
- Additional Reach 3: Drain tributary of the Atlantic Stream
- Additional Reach 4: Drain south of the Atlantic Stream

Figure 3-10 shows their locations. Reaches 1 and 2 are included as 1D - 2D linked channels in the model while the upstream of reach 3 is included in the 1D model only and the downstream section is included as 1D -2D linked in the model. Reach 4 was not included in the model as investigation in the channel survey showed the drain did not connect into any other drain or channel. The drain serves a small catchment area, calculated to be approximately 0.018km^2. It was therefore decided that the drain was not necessary to be included in the model and the catchment area would be included in the Atlantic Stream catchment.

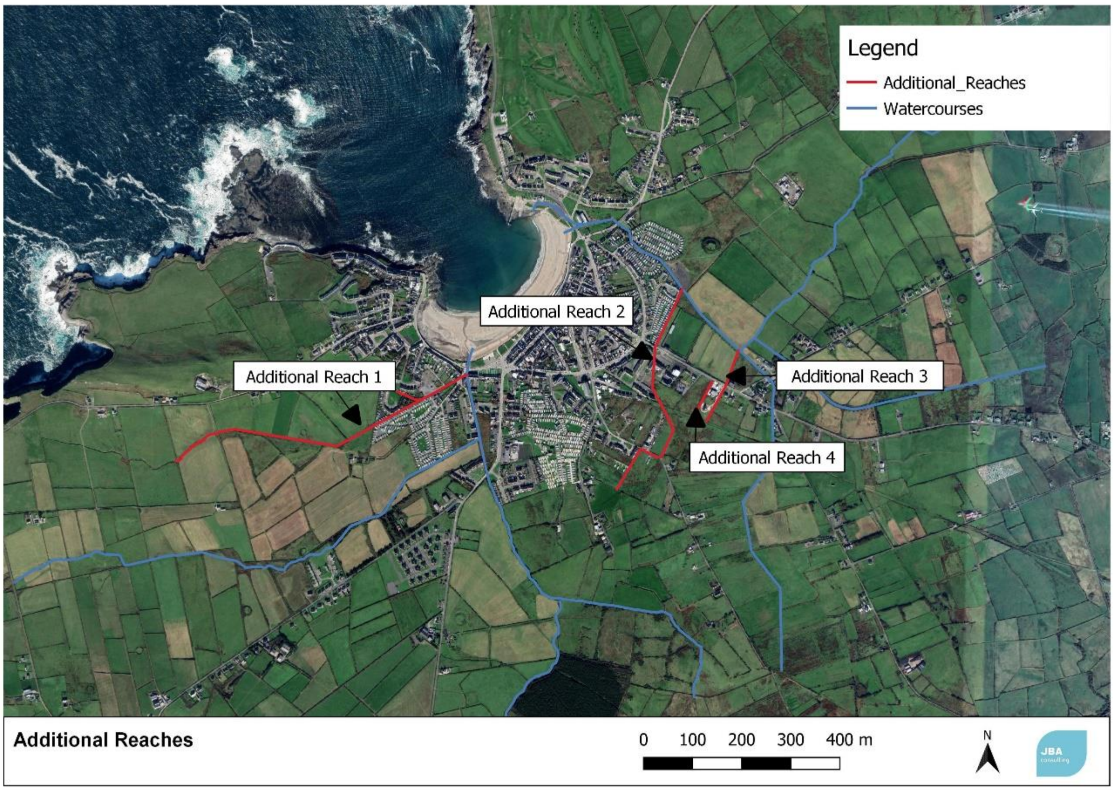


Figure 3-10: Location of Additional Reaches

3.6 Extents of Model

In addition to the additional reaches included in the model described above, the upstream extents of the CFRAM model were extended for the purposes of this study.

3.7 2D Stability Patches

Roughness stability patches have been applied to 2D areas within both the Victoria and Atlantic Stream models.

The patches have been included to provide stability to junctions and areas which are drowned in high flow events. This is to help stability and with the transfer of water between the 1D and 2D solution schemes as the water moves over the area drowning the 1D channels. Refer to Figure 3-11 for location of stability patches.

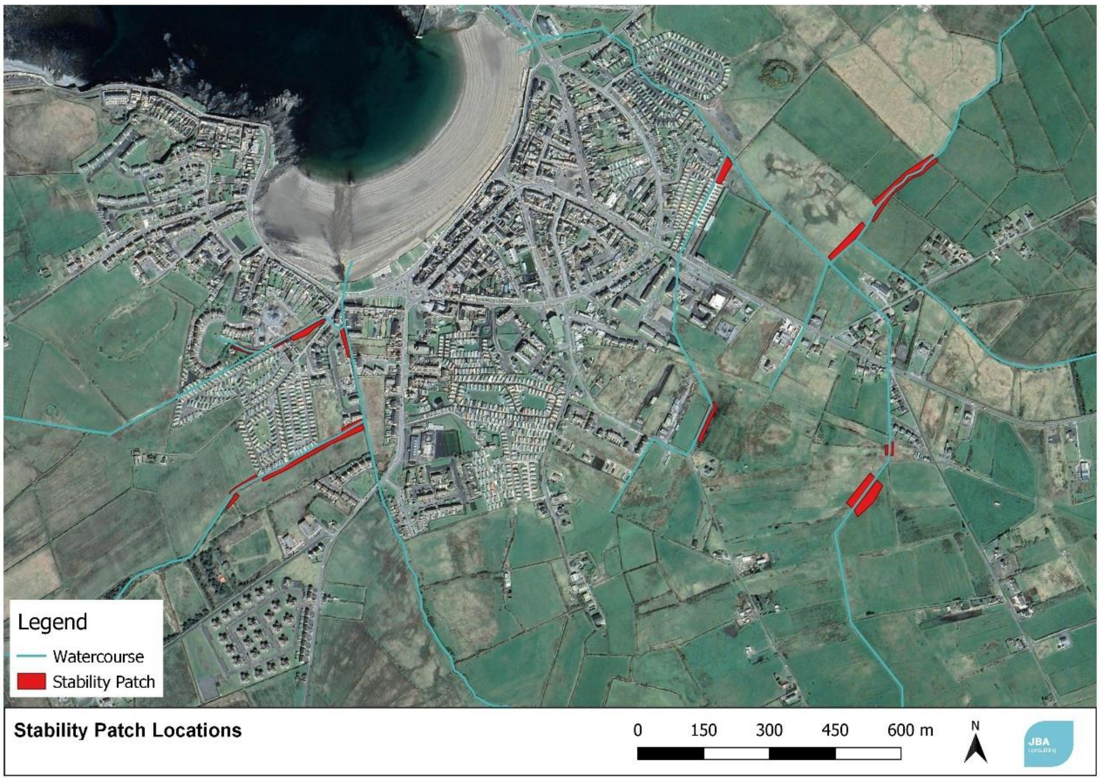


Figure 3-11: Location of Stability Patches in Victoria and Atlantic Stream Models

3.8 Limitations of modelling method used

Within the modelling process various limitations were identified. This section describes the limitations found and how they were accounted for within the model.

- **Channels modelled in 1D only:** A number of channels have been included as 1D only. This limits the model capability as flow can only be perpendicular to the channel cross section. This has only been done along reaches where channels are steep, and flow is expected to stay within banks, so it will therefore not affect the overall model performance.
- **Modelling of channels with single HX lines:** A section of the Well Stream and the Atlantic Stream tributaries were modelled using a single HX line to connect the 1D and 2D domains, i.e., instead of a HX line representing both banks only one bank was represented with HX line. This was done as the flooding at the points of the single HX lines are dominated by the main channels flooding. The single HX line allows for greater stability and allows the out of bank flow from the main channels to easily pass through the tributary channels stably. The bank with the lower top of banks levels was represented by the single HX lines as this bank level would dominate out of bank flow from the tributaries. Refer to Figure 3-12 for locations of channels where single HX line is used. The use of single HX line will not impact the overall model performance.

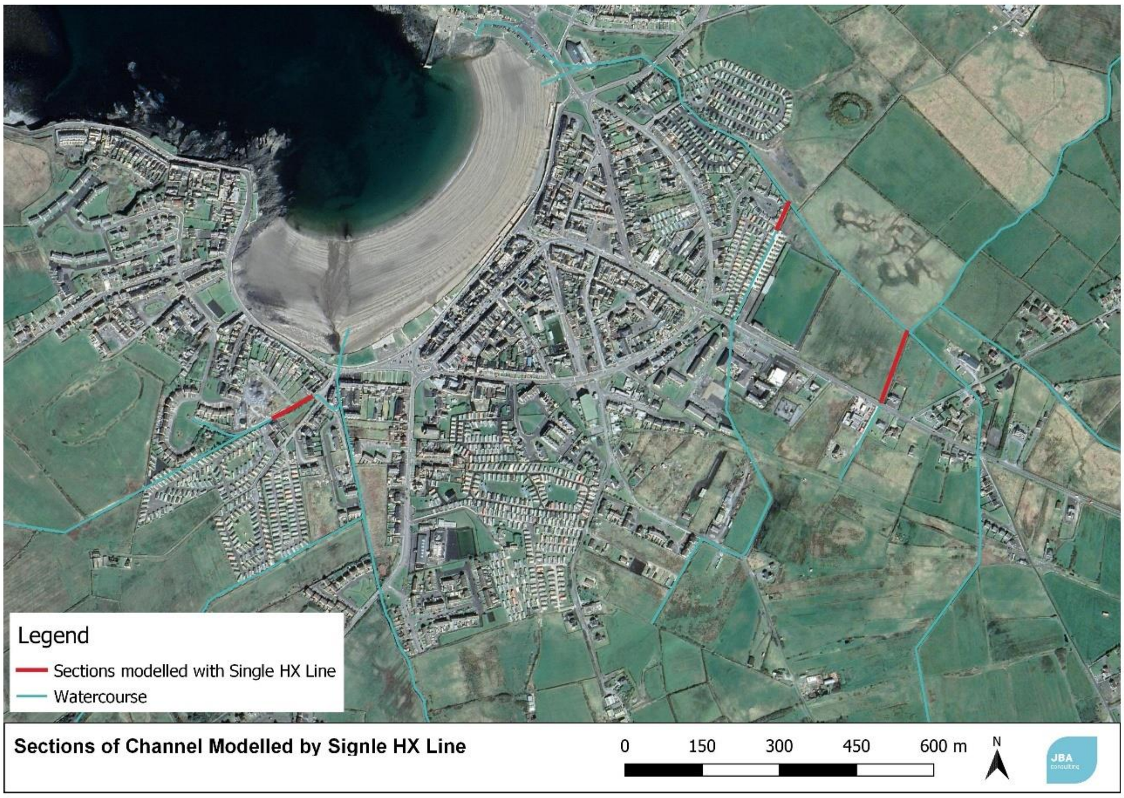


Figure 3-12: Sections of Channel Modelled with single HX line

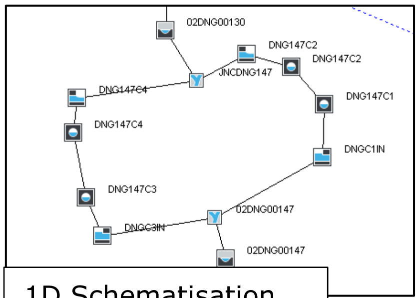
4 1D Structures

Details of all structures are provided in this Section. Photographs, survey data and modelled outputs of flow and stage are provided to give general overall indication of structure and performance.

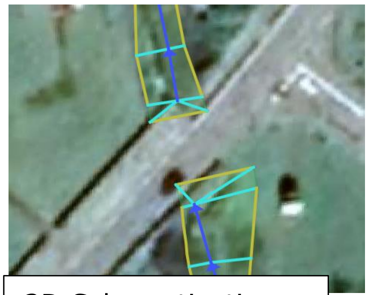
4.1 Victoria Stream Structures

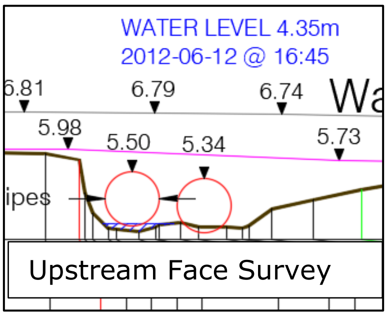
4.1.1 Culverts

DNG147C1/ DNG147C3– 2 no. Circular culverts parallel			
Width /Diameter	1.2m	**Length**	16.8m
Height	N/A	**Manning's**	0.02
US IL	4.3mOD/ 4.14mOD	**DS IL**	4.13mOD/4.030mOD
Present in Model?	Yes – Circular Conduit		
Notes	Spill modelled in 2D using HX lines and CN lines set at road levels.		



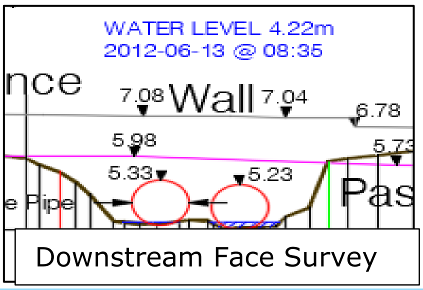
1D Schematisation


2D Schematisation



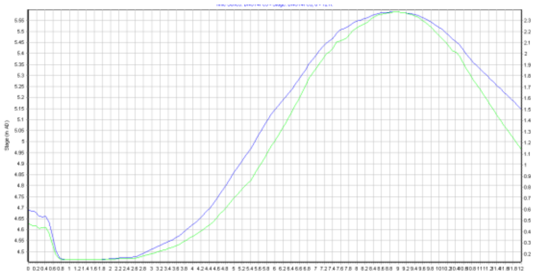
Upstream Face Survey

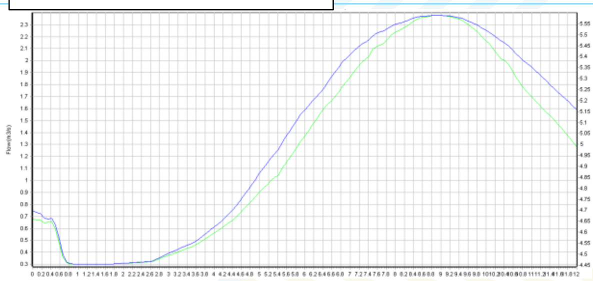

Upstream Face



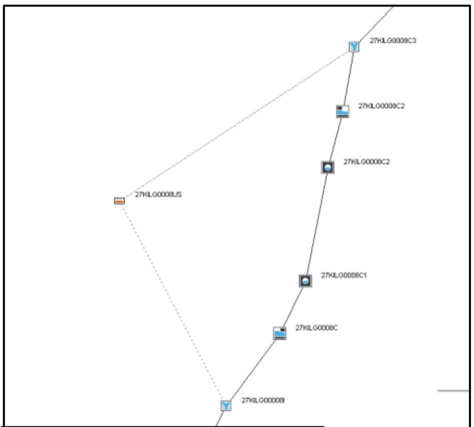
Downstream Face Survey


Downstream Face


Flow (green) and Stage(blue) through culvert DNG147C3

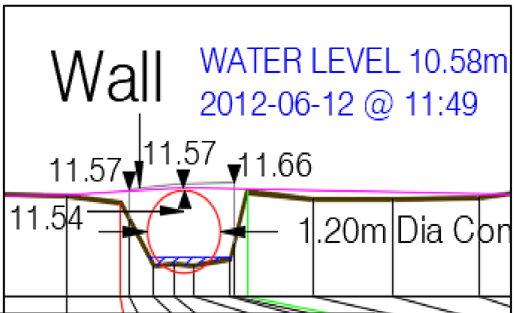

Flow (green) and Stage(blue) through culvert DNG147C1

27KILG00008C1 – Circular Culvert			
Width /Diameter	1.2m	**Length**	5.183m
Height	N/A	**Manning's**	0.02
US IL	10.340mOD	**DS IL**	10.26mOD
Present in Model?	Yes – Circular Conduit		
Notes	Spill modelled in 1D using spill unit with levels set to height of headwall.		

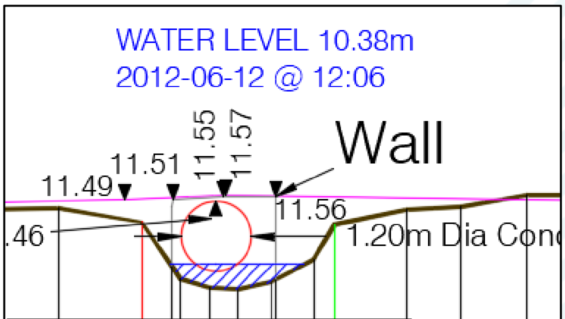


1D Schematisation

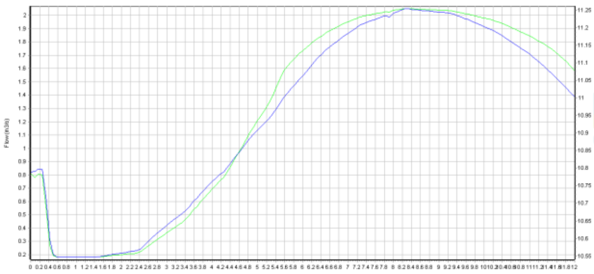
Upstream Face



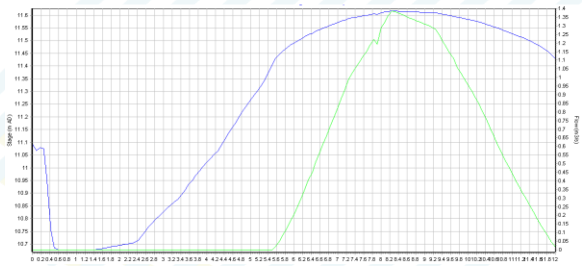
Upstream Face Survey



Downstream Face Survey

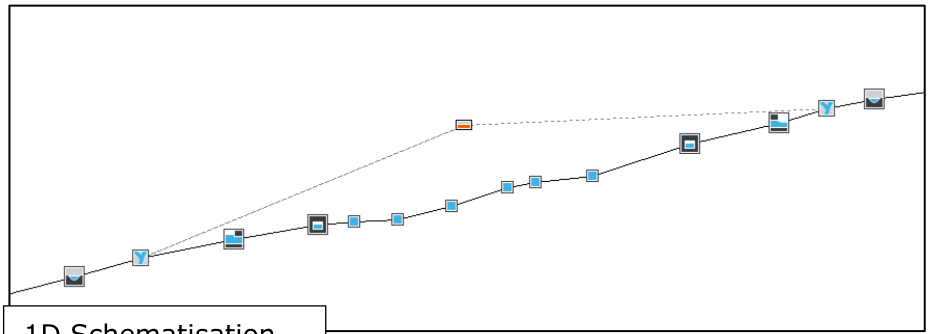


Flow (green) and Stage(blue) through culvert

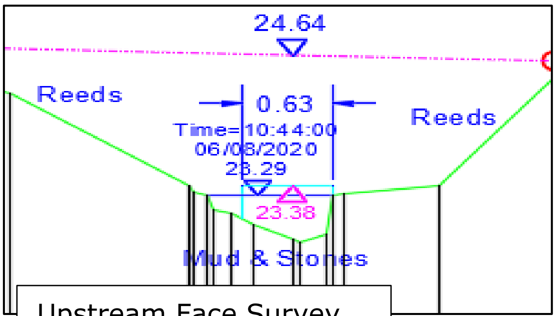


Flow (green) and Stage(blue) through spill

27VTB1_114C1 – Irregular Culvert			
Width /Diameter	0.63m	**Length**	16.75m
Height	0.429m	**Manning's**	0.025
US IL	22.846mOD	**DS IL**	22.577mOD
Present in model?	Yes – Rectangular Conduit		
Notes	Culvert US and DS faces differ. US face has smaller conveyance area Therefore modelled as rectangular culvert with same area as US Face. Invert level and soffit level of 22.846mOD and 23.38mOD. Spill modelled in 1D using FM spill unit with levels set to road levels.		

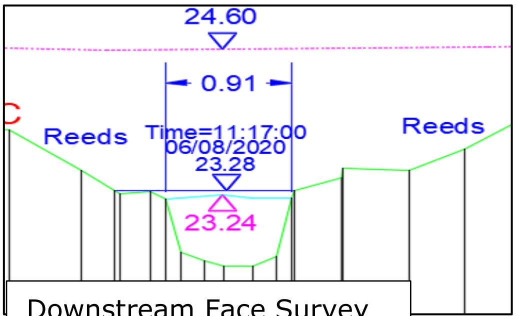


1D Schematisation



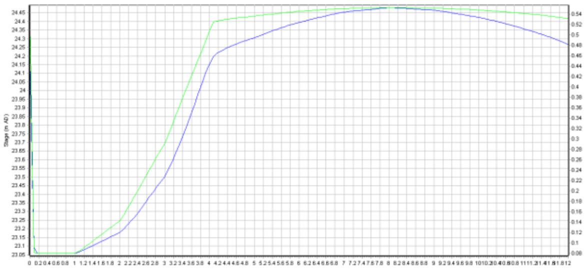
Upstream Face Survey

Upstream Face

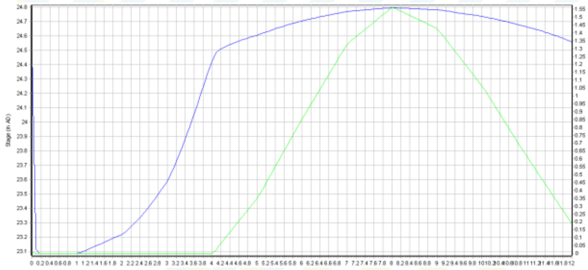


Downstream Face Survey

Downstream Face

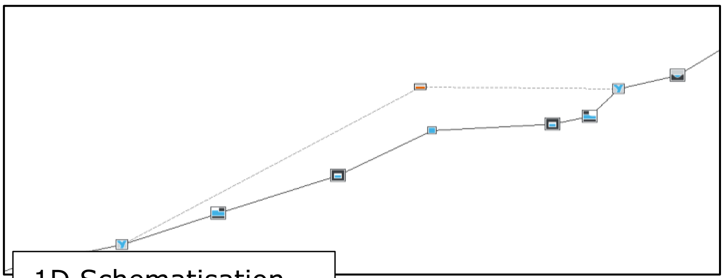


Stage (blue) and flow (green) through the culvert



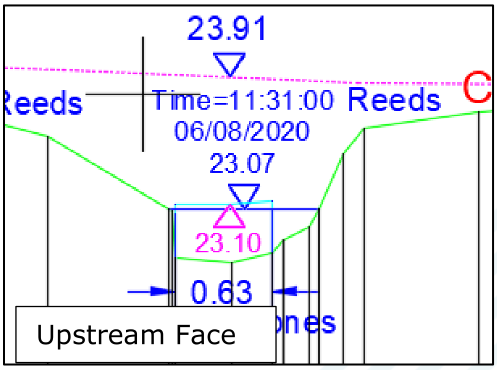
Stage (blue) and flow (green) through the spill

27VTB1_111C – Rectangular Culvert			
Width /Diameter	0.63m	**Length**	4.12m
Height	0.4342m	**Manning's**	0.025
US IL	22.760mOD	**DS IL**	22.620mOD
Present in Model?	Yes – Rectangular Conduit		
Notes	Culvert US and DS faces differ. US face has smaller conveyance area. Therefore, modelled as rectangular culvert with same area as US Face. Spill modelled in 1D using FM spill unit with levels set to road levels.		



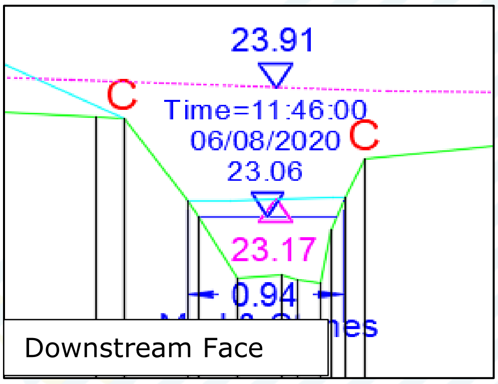
1D Schematisation

Upstream Face

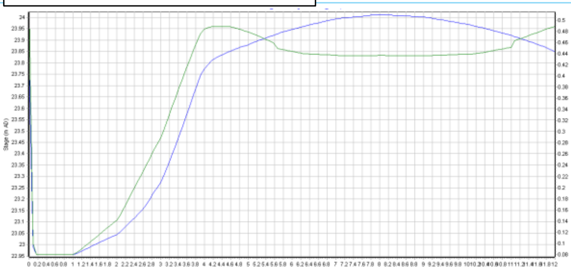


Upstream Face

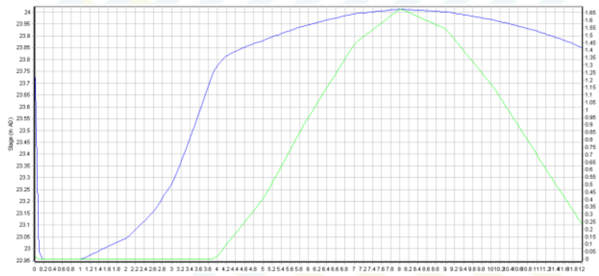
Downstream Face



Downstream Face

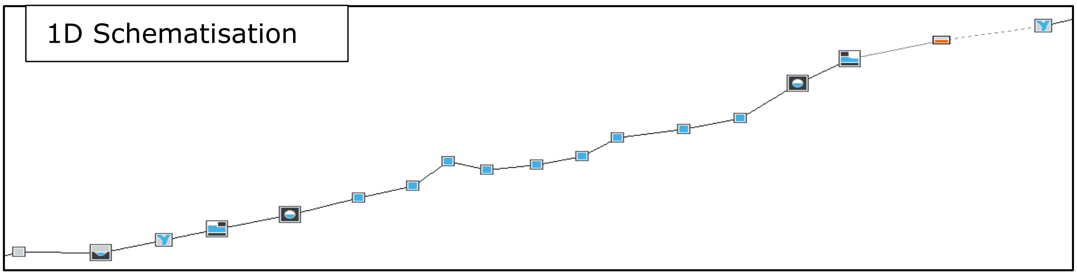


Stage (blue) and flow(green) through culvert



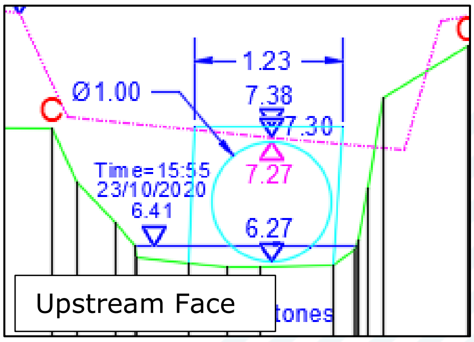
Stage (blue) and flow(green) through spill

27VTB1_44C– Circular Culvert			
Width / Diameter	1m	**Length**	132.55m
Height	N/A	**Manning's**	0.02
US IL	6.27mOD	**DS IL**	3.94mOD
Present in Model?	Yes – Circular Conduit		
Notes	Spill was modelled in the 2D using HX and CN lines set to road levels. A drop from the DS invert level to the bed level was noted, therefore this difference in level was modelled as a weir. Trash screen shown in the figure was included in the model culvert inlet unit. The loss coefficient was estimated at 0.335.		



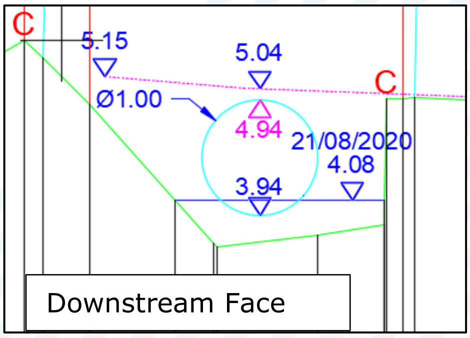
1D Schematisation

Upstream Face

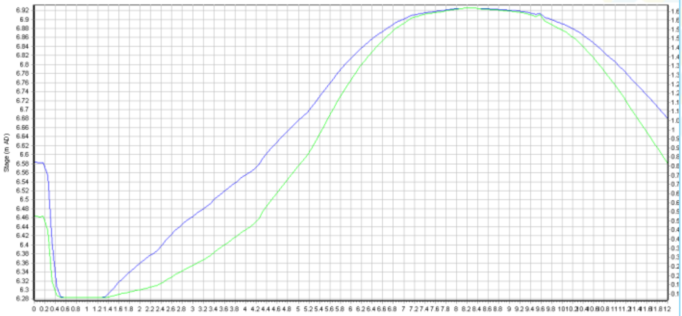


Upstream Face

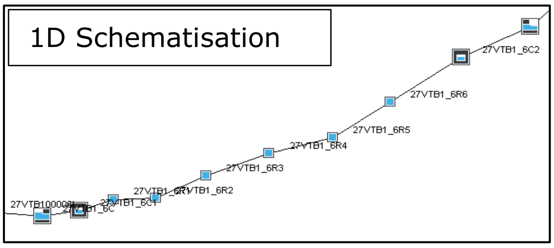
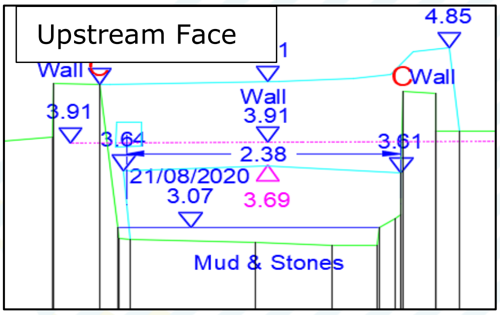
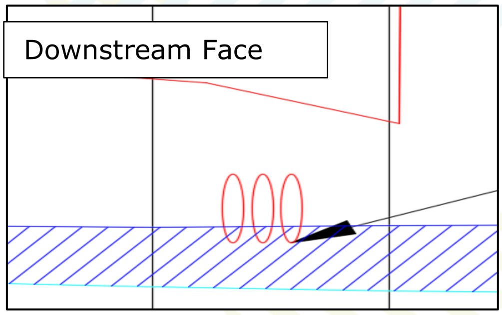
Downstream Face

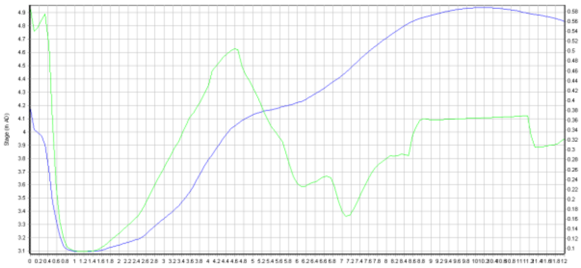


Downstream Face

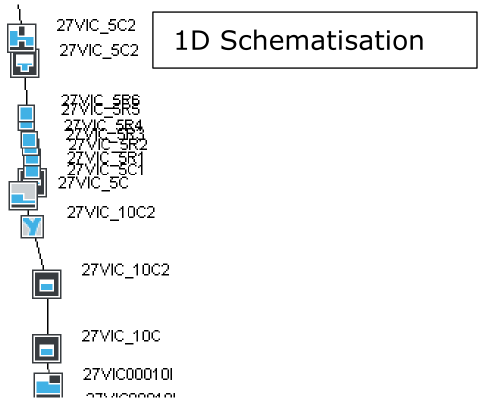


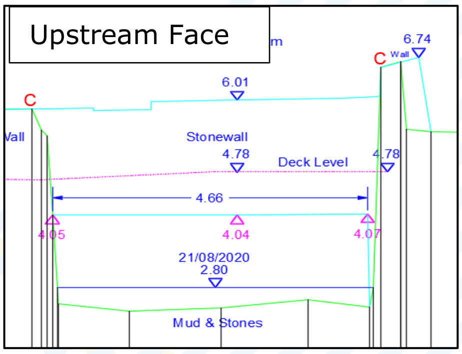
Stage (blue) Flow(green) through culvert

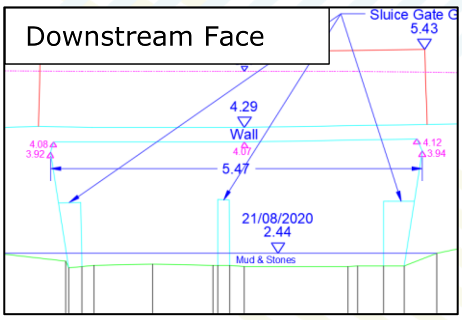
27VTB1_6C – Rectangular Culvert			
Width /Diameter	0.668m	**Length**	70m
Height	0.714m	**Manning's**	0.02
US IL	2.896mOD	**DS IL**	2.530mOD
Present in Model?	Yes – Rectangular Conduit		
Notes	Spill is not modelled as flow spill over banks which are at a lower level that the wall at the inlet to the culvert. The main route of the Well Stream culvert follows from the 2.8m x 0.714m rectangular culvert to 3 no. DN450 culverts and outflows into the Victoria Stream via 3. No DN600 diameter pipes upstream of the rear garden walls in Victoria Park. This diversion was done by CCC as the original route of the Well Stream culvert was subject to blockage. The original outfall into the Victoria Stream was within the Victoria Stream Outfall Culvert and was subject to blockage due to debris from tidal inflow. Inspection of this outfall point concluded that there is very little flow coming along this direction. The culvert was therefore modelled as 1 rectangular culvert with the same conveyance area as the 3 no. DN450 pipes. This was chosen as this section of the culvert has the smallest conveyance area and is therefore limiting factor. As the culvert is drowned in flow from the Victoria Stream, it was chosen to model the 3 circular culverts as 1 rectangular culvert with the same conveyance area instead of three separate circular culverts.		
			
			
			

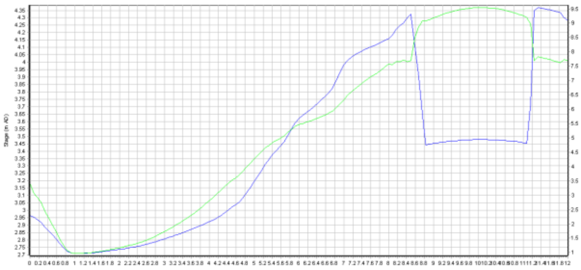
 **Stage (blue) Flow(green) through culvert**	

27VIC_10C – Rectangular Culvert // Symmetrical Culvert			
Width /Diameter	4.66m // 5.45m	**Length**	132.55m
Height	1.64m // 1.85m	**Manning's**	0.02 (for sides) / 0.04 (for invert)
US IL	2.4mOD // 2.370mOD	**DS IL**	2.370mOD//2.229mOD
Present in Model?	Yes – Rectangular Conduit	**Stop Log**	Throat Level:3.22 Throat Soiift:4.07 US & DS Sill Level:2.229 Bore Area: 4.56
Notes	The Outfall culvert upstream and downstream face differs. The culvert was therefore modelled as two culverts in series separated by a junction. The US culvert is a rectangular culvert and is based off the upstream face, the downstream culvert is modelled as symmetrical culvert and is based on the downstream face. To represent the losses within the culvert a general loss unit was used downstream of the junction. The general loss coefficient of 0.5 was used. A symmetrical culvert requires roughness to be entered as Colebrook White Friction. The Colebrook White Friction value for a concrete culvert is 1.65mm. Manning's value of 0.02 was used for the upstream culvert. Spill was modelled in the 2D using HX and CN lines set to level of inlet wall. At the downstream end of the structure a spill unit is included to represent the Stop Logs.		

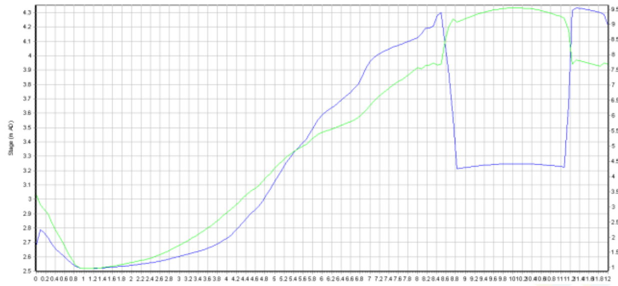




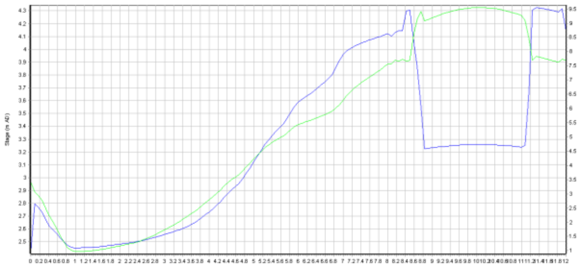




Stage (blue) and flow (green) through upstream culvert

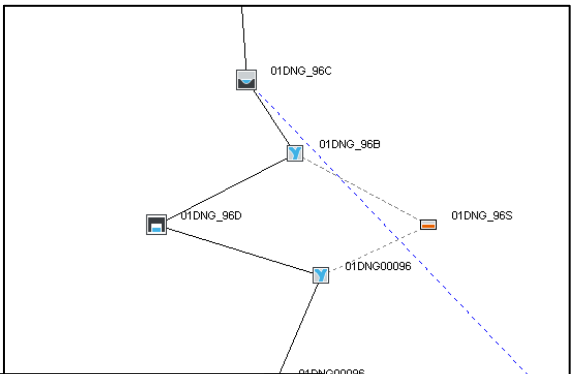
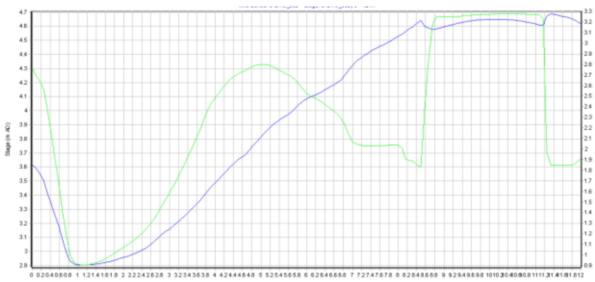
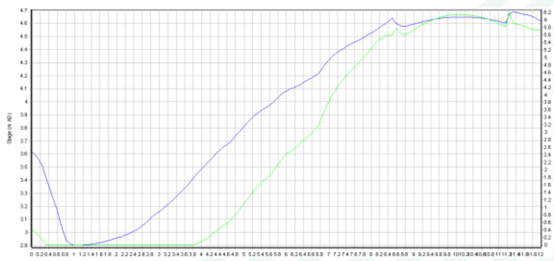


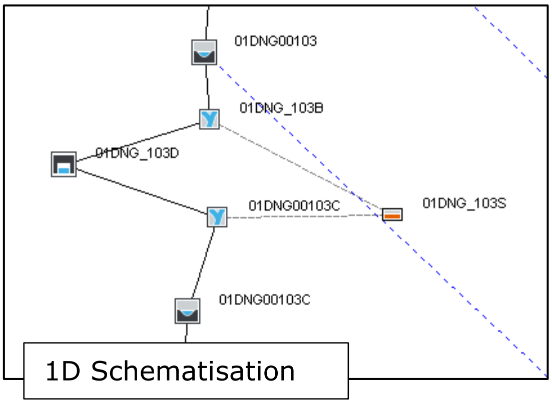
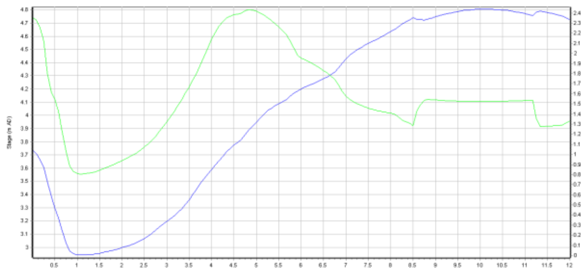
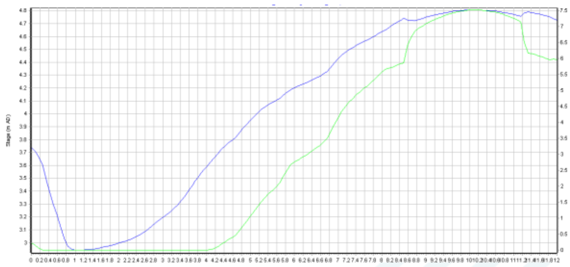
Stage (blue) and flow (green) through downstream culvert



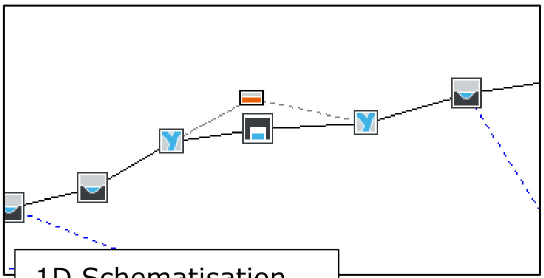
Stage (blue) and flow (green) through spill unit

4.1.2 Bridges

01DNG_96D – Pipe Crossing modelled as Bridge			
Width	2.498m	**Length**	0.4m
Soffit	3.010mOD	**Springing height**	3.010mOD
Present in Model?	Yes - USBPR		
Orifice equation used	Yes	**Overtopping**	1D Spill unit
Notes	400mm diameter pipe modelled as bridge with soffit of bridge set to invert of pipe and 1D spill set to soffit of pipe.		
 1D Schematisation		 Image of Pipe Crossing	
 **Stage (blue) Flow(green) through bridge**		 Stage (blue) Flow(green) through spill	

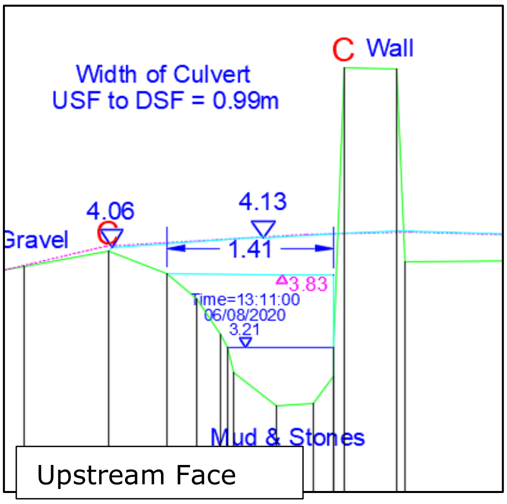
01DNG_103D – Pipe Crossing modelled as Bridge			
Width	2.653m	**Length**	0.25m
Soffit	3.36mOD	**Springing height**	3.36mOD
Skew	Yes - USBPR		
Orifice equation used	Yes	**Overtopping**	1D Spill unit
Notes	250mm diameter pipe modelled as bridge with 1D spill with soffit of bridge set to invert of pipe and 1D spill set to soffit of pipe.		
 1D Schematisation		 Image of Pipe Crossing	
 **Stage (blue) Flow(green) through bridge**		 Stage (blue) and flow (green) through spill	

27VTB1_15B – Rectangular Bridge			
Width	1.949m	**Length**	0.99m
Soffit	3.835mOD	**Springing height**	3.835mOD
Present in Model?	Yes – USBPR Unit		
Orifice equation used	Yes	**Overtopping**	1D Spill unit with level set to the top of bridge level
Notes			

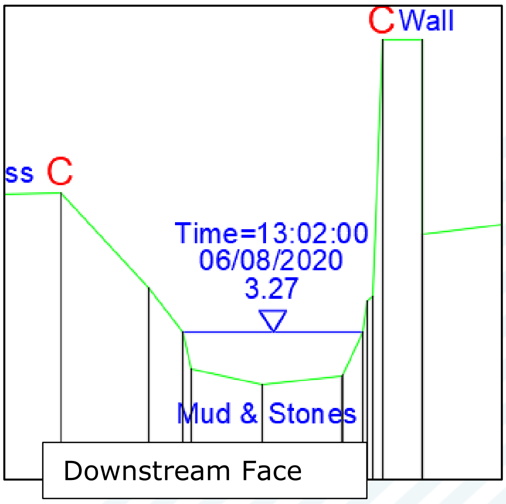


1D Schematisation

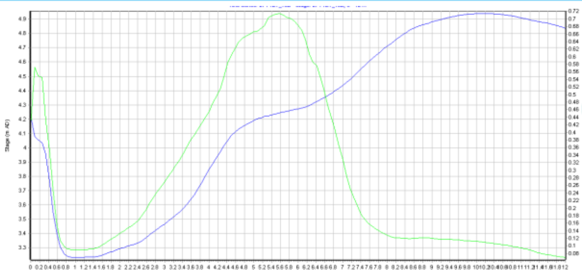
Image of Structure



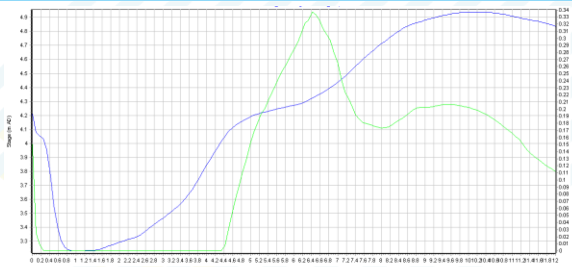
Upstream Face



Downstream Face



Stage (blue) Flow(green) through structure

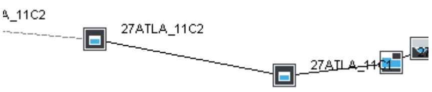


Stage (blue) Flow(green) through spill

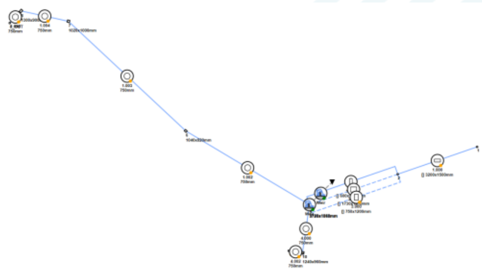
4.2 Atlantic Stream Structures

4.2.1 Culverts

27ATLA_11C1 – Atlantic Outfall Culvert			
Width /Diameter	3.20m	**Length**	263m
Height	1.5m	**Manning's**	0.035
US IL	4.719mOD	**DS IL**	1.484mOD
Present in Model?	Yes – Rectangular Culvert		
Notes	The outfall culvert has been modelled using a combination of the Flood Modeller – TUFLOW 1D -2D linked model and a FLOW model. FLOW is a hydraulic modelling package for the design/analysis of drainage networks. It was decided to model the outfall culvert in FLOW as this provides a better representation of the culvert complexity. FLOW software provides better analysis of surcharging pipes and complex networks than Flood Modeller. The outfall culvert splits into 3 under the road, approximately 37m from the inlet. This flows into a manhole chamber and outfalls via a DN750 culvert. In the manhole chamber there is a weir which overtops in times of high flow and leads to a DN750 overflow pipe which outfalls onto the beach. The FLOW model begins at the inlet to the culvert and the FM – TUFLOW model finishes at the point where the culvert splits into 3 separate culverts. Blockage of 80% is considered the baseline flood condition at the trash screen. Severe blockage has been reported during recorded flood events therefore a large degree of blockage was considered to be necessary to represent the existing conditions reported on site. Refer to Section 4.2.3 for more detail of the FLOW model.		

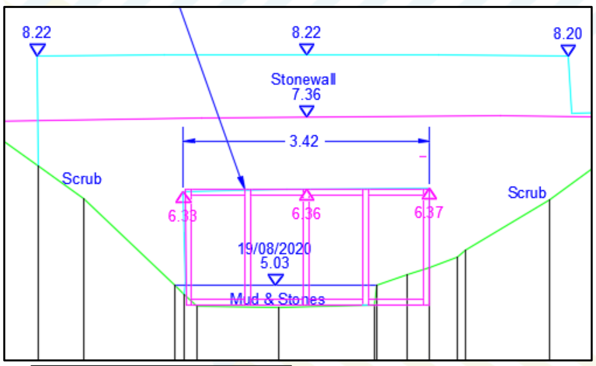


1D Schematisation -FM



1D Schematisation -FLOW

Upstream Face



Upstream Face

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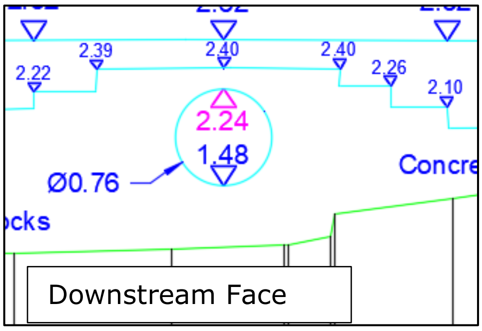
</td><td>
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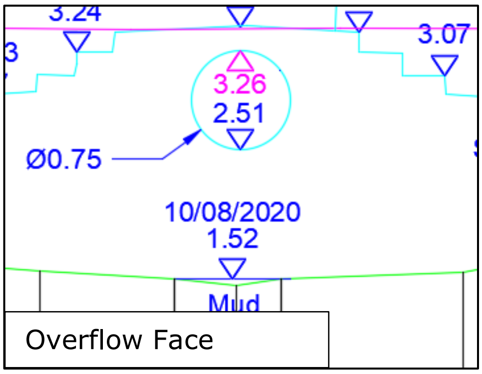
</td><td>
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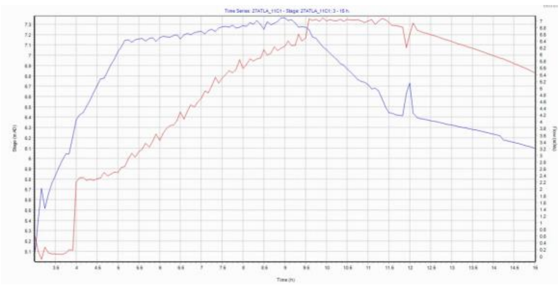
<tr><td>
Flow (red) and Stage(blue) through culvert</td><td></td></tr>
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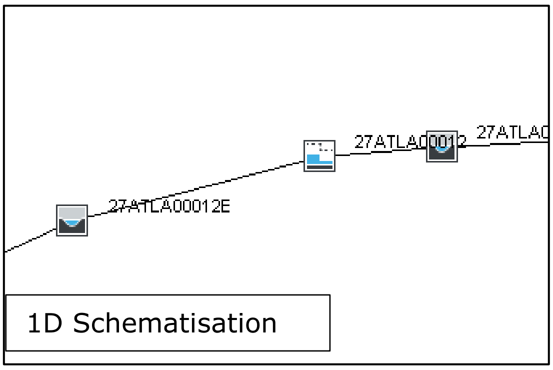
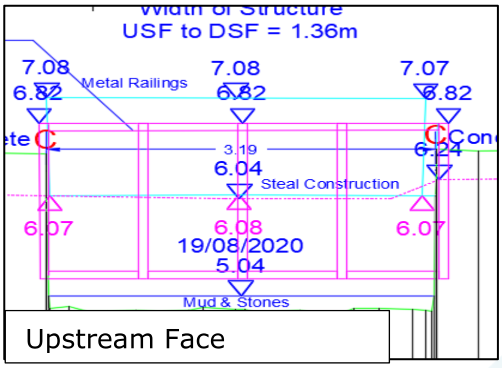
27ATLA00012 - Footbridge			
Width /Diameter	3.190m	**Length**	1.360m
Height	1.751m	**Manning's**	N/A
US IL	4.849mOD	**DS IL**	4.811mOD
Present in Model?	Yes – Bernoulli Loss		
Notes	The culvert and trash screen has been represented as a Bernoulli Loss unit. A Bernoulli loss unit was chosen to represent the reduction in flow area due to the footbridge and trash screen. A blockage of 100% to the deck of the bridge has been included in the baseline model as severe blockage has been recorded during previous flood events.		



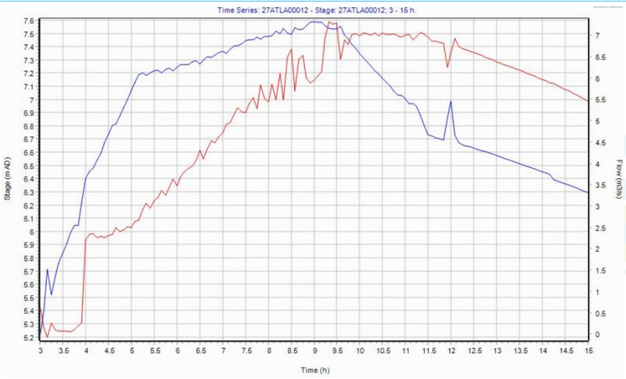
1D Schematisation



Upstream Face

Upstream Face

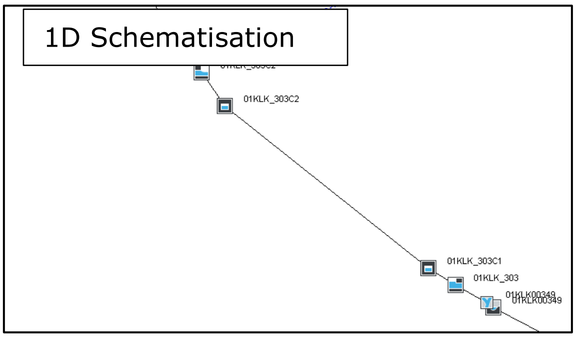

Downstream Face



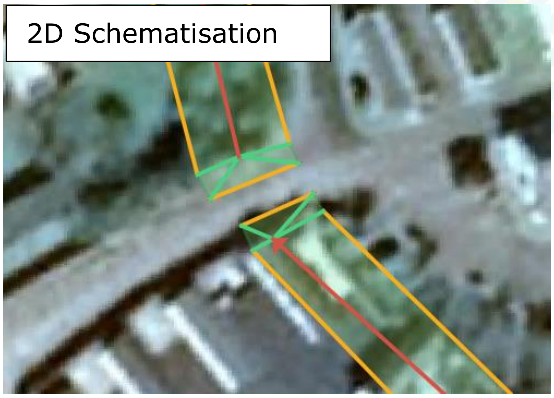
Flow (red) and Stage(blue) through culvert

01KLK_303 – Road Bridge			
Width	2.50m	**Length**	7.81m
Height	1.8m	**Manning's**	0.025
US IL	4.849mOD	**DS IL**	4.811mOD
Present in Model?	Yes – Rectangular Culvert		
Notes			

1D Schematisation

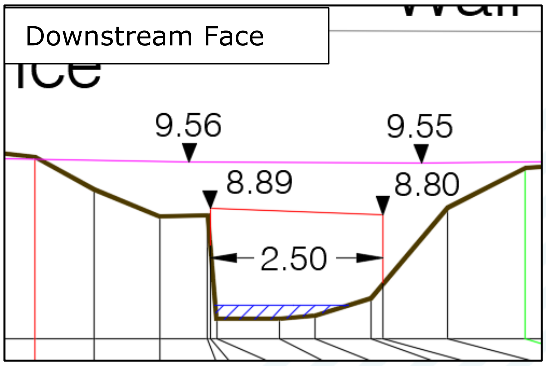


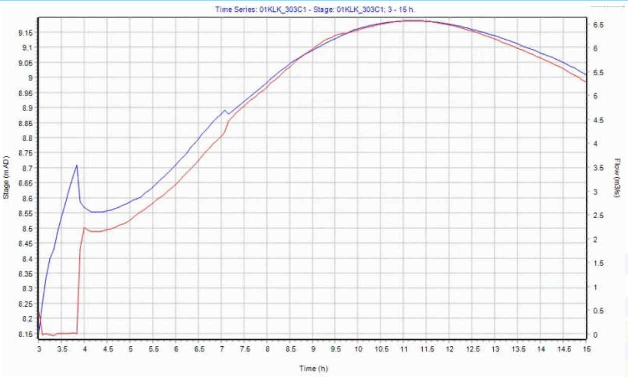
2D Schematisation



Upstream Face

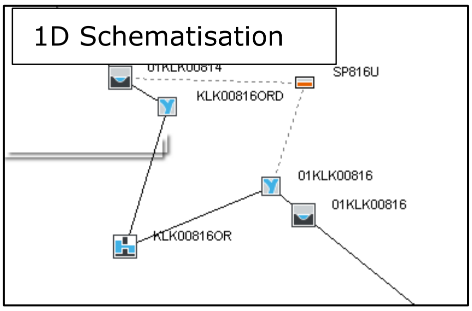
Downstream Face

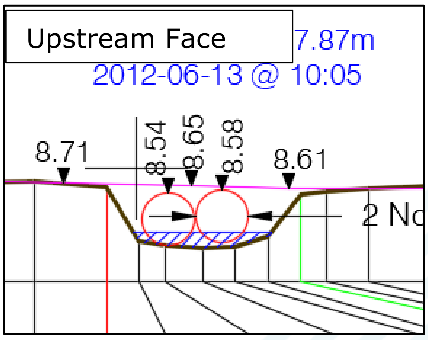


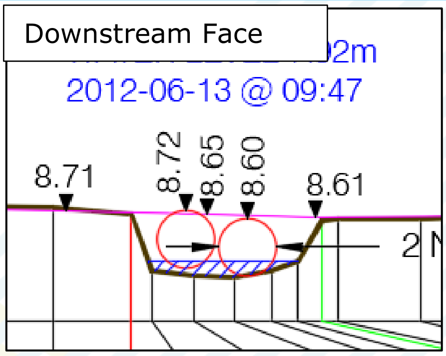


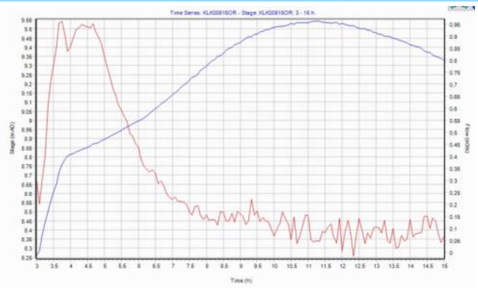
Flow (red) and Stage(blue) through structure

KLK00816OR – Double Circular Culvert			
Width /Diameter	0.9m	**Length**	2.11m
Height	N/A	**Manning's**	N/A
US IL	7.7mOD	**DS IL**	7.595mOD
Present in Model?	Yes – Orifice		
Notes	Due to short length of pipes orifice was found to be more stable so was used to represent opening. Bore area is the conveyance area of the two culverts - 1.27m^2 Spill modelled in 1D using spill unit with levels set to the ground levels over the culverts.		

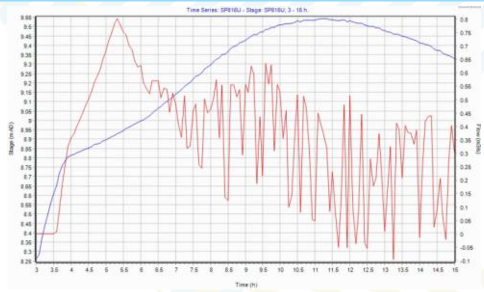




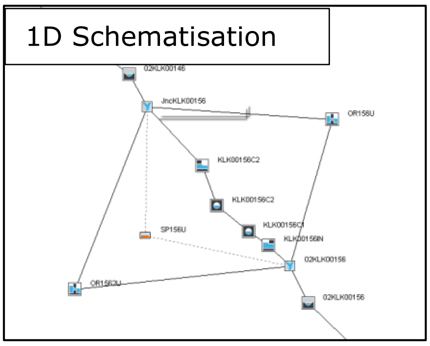
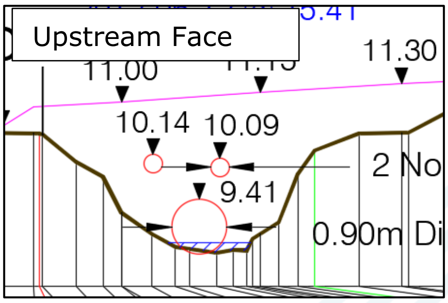
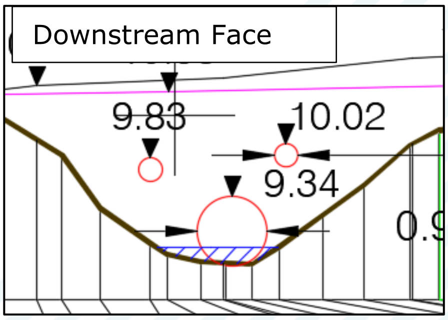
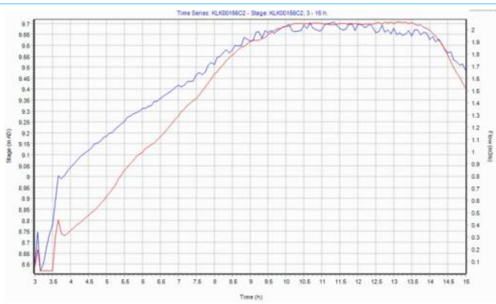
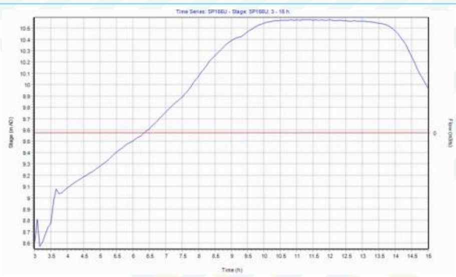




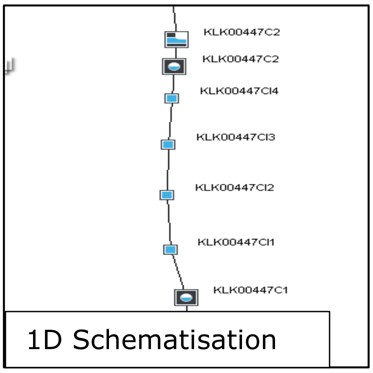
Flow (red) and Stage(blue) through culvert



Flow (red) and Stage(blue) through spill

KLK00156C1 – Circular Culvert			
Width /Diameter	0.9m	**Length**	10.0m
Height	N/A	**Manning's**	0.02
US IL	8.510mOD	**DS IL**	8.44mOD
Present in Model?	Yes – Circular Culvert		
Notes	Two smaller pipes of 0.3m diameter located just above the main culvert (soffit levels of 10.14mOD and 10.09mOD) have been represented as orifices. Spill modelled in 1D using a FM Spill unit with levels set to levels of track over culverts.		
			
			
			
 **Flow (red) and Stage(blue) through culvert**		 Flow (red) and Stage(blue) through spill	

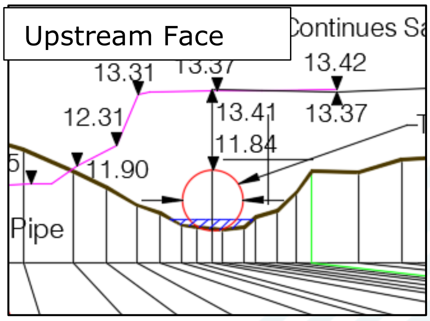
KLK00447C1 – Circular Culvert			
Width /Diameter	1.2m	**Length**	170m
Height	N/A	**Manning's**	0.02
US IL	10.764mOD	**DS IL**	9.84mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to levels of ground over culvert. Trash screen included in mode – loss coefficient of 0.431 used.		



1D Schematisation

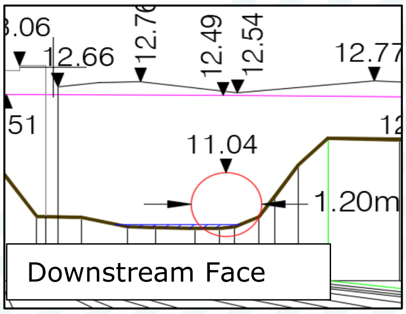

2D Schematisation


Upstream Face

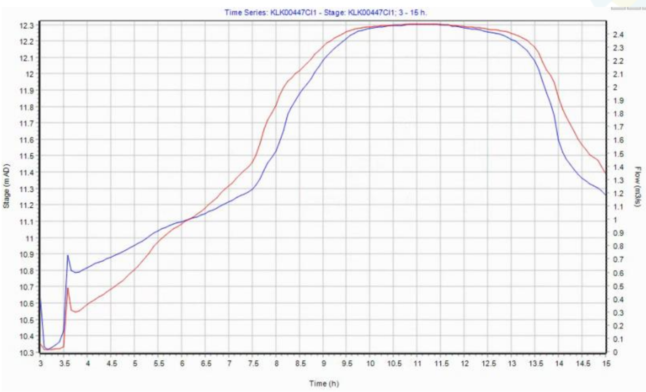


Upstream Face

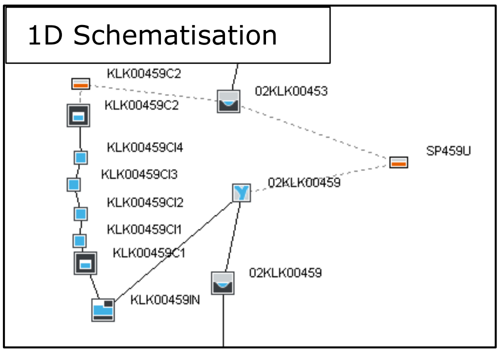

Downstream Face

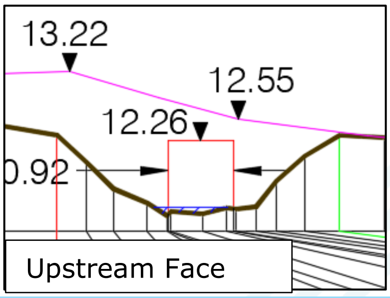


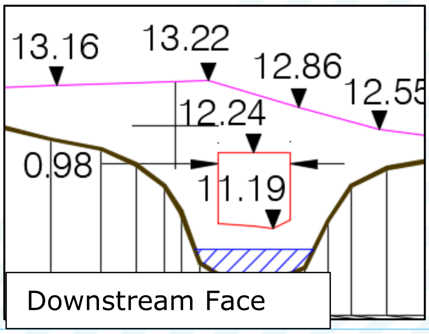
Downstream Face

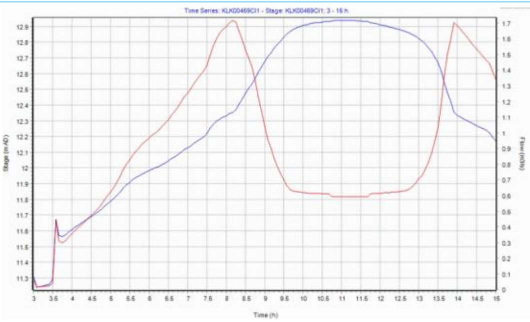

Flow (red) and Stage(blue) through culvert

KLK00459C1 – Rectangular Culvert			
Width / Diameter	0.920m	**Length**	6.91m
Height	1.050m	**Manning's**	0.025
US IL	11.205mOD	**DS IL**	11.190mOD
Present in Model?	Yes – Rectangular Culvert		
Notes	Spill modelled in 1D using Spill unit with levels set to road level over culvert. Spill included at downstream end of culvert to represent drop in level between invert level and bed level. A weir coefficient of 1.67 was used.		

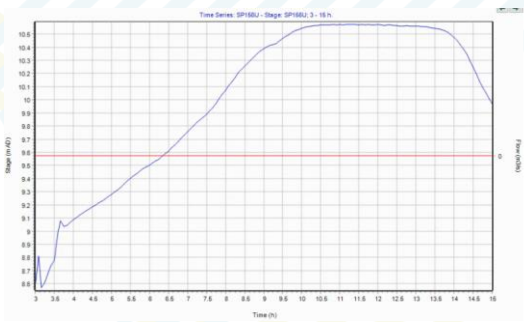






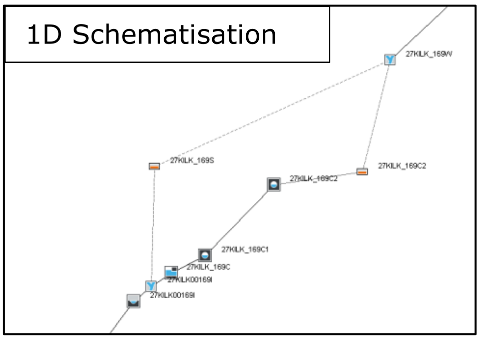


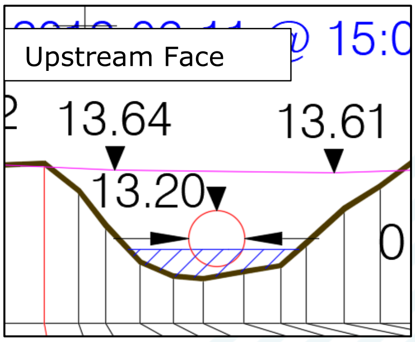
Flow (red) and Stage(blue) through culvert

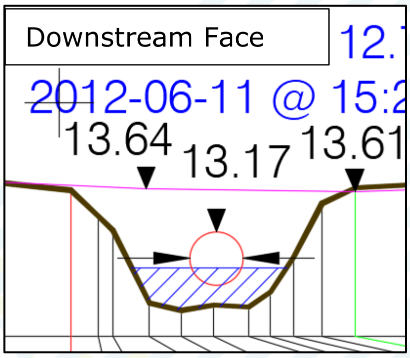


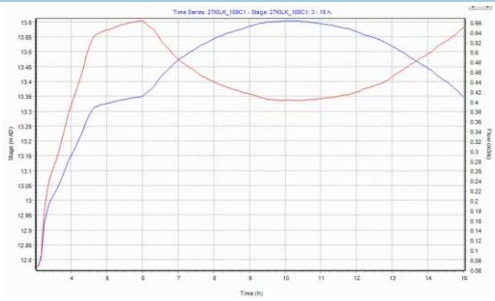
Flow (red) and Stage(blue) through spill

27KILK_169C1 – Circular Culvert			
Width /Diameter	0.6m	**Length**	4.82m
Height	N/A	**Manning's**	0.02
US IL	12.60mOD	**DS IL**	12.57mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 1D using Spill unit with levels set to road level over culvert. Drop in bed level at downstream end represented by spill unit. Culvert outlet loss unit not required as spill takes loss into consideration. Weir coefficient of 1.67 was used.		

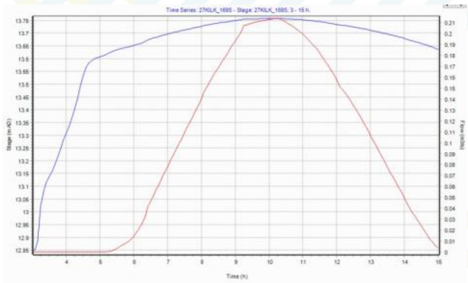






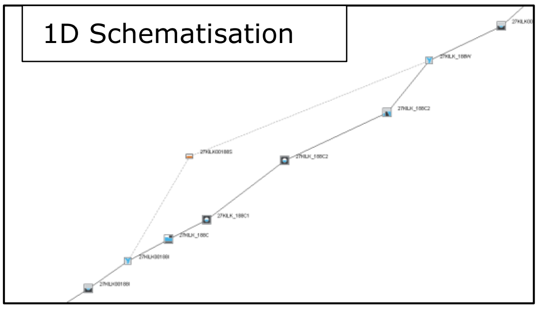


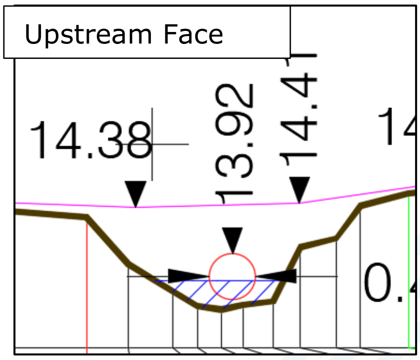
Flow (red) and Stage(blue) through culvert

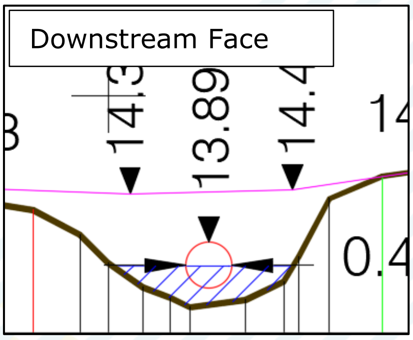


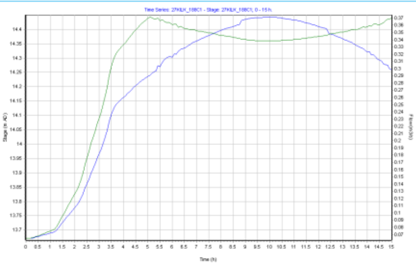
Flow (red) and Stage(blue) through spill

27KILK_188C1 – Circular Culvert			
Width / Diameter	0.45m	**Length**	4.24m
Height	N/A	**Manning's**	0.02
US IL	13.47mOD	**DS IL**	13.44mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 1D using Spill Unit with levels set to road level over culvert. Drop in bed level at downstream end represented by weir. Outlet loss unit not needed as weir will take headloss into account.		

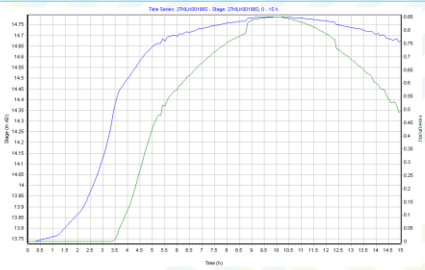






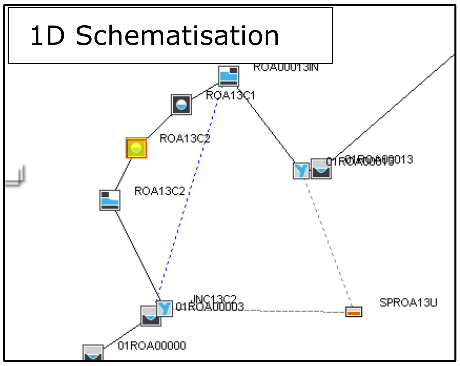


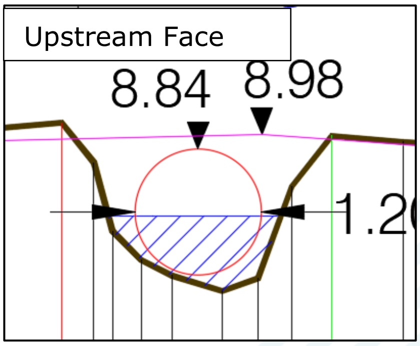
Flow (red) and Stage(blue) through culvert

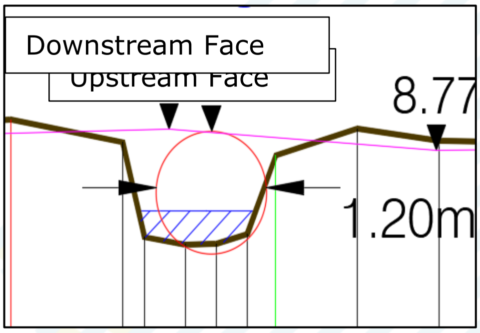


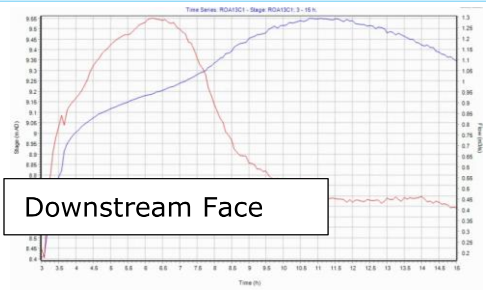
Flow (red) and Stage(blue) through spill

ROA13C1 – Circular Culvert			
Width /Diameter	1.2m	**Length**	9.580m
Height	N/A	**Manning's**	0.02
US IL	7.64mOD	**DS IL**	7.76mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 1D using Spill unit with levels set to ground level over culvert		

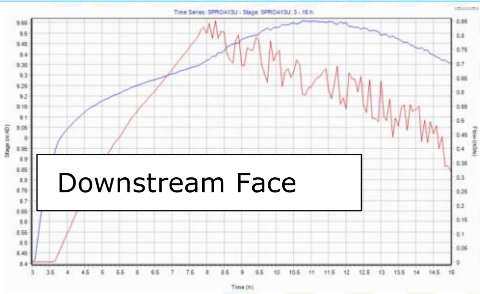






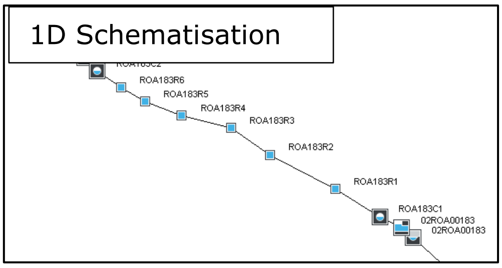


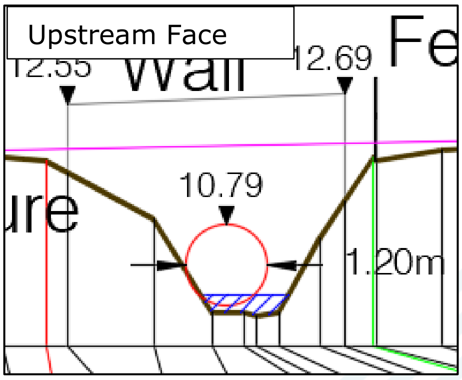
Flow (red) and Stage(blue) through culvert

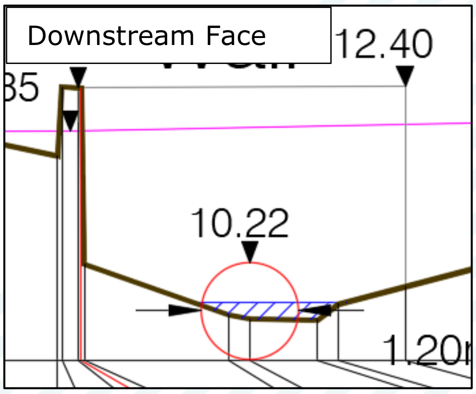


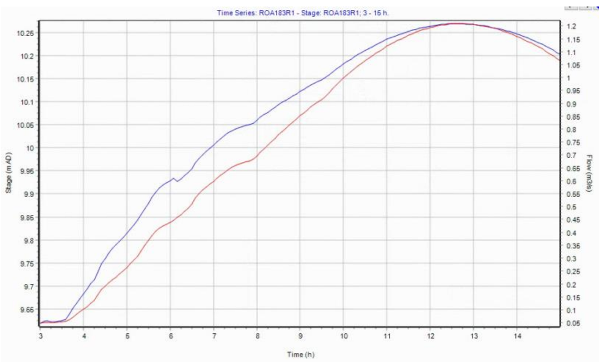
Flow (red) and Stage(blue) through spill

ROA00183C1 – Circular Culvert			
Width /Diameter	1.2m	**Length**	10.95m
Height	N/A	**Manning's**	0.02
US IL	9.59mOD	**DS IL**	9.02mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to road level over culvert		



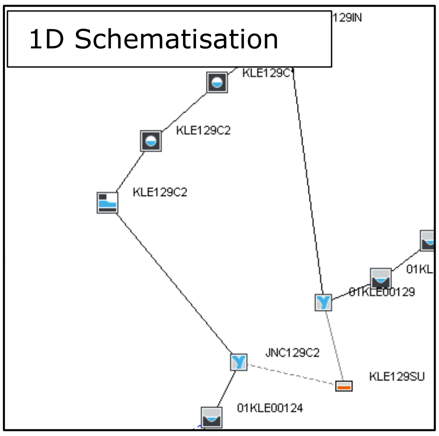


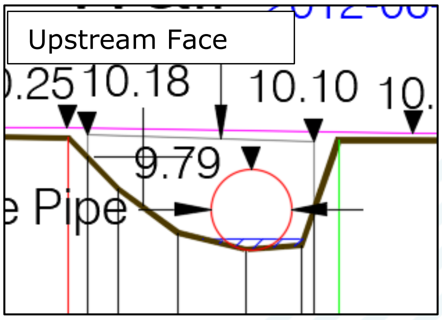


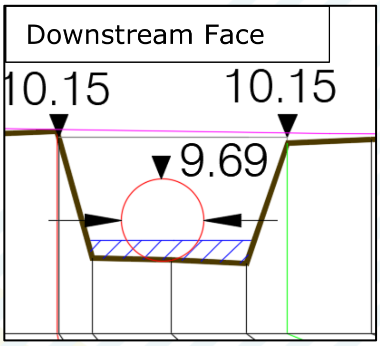


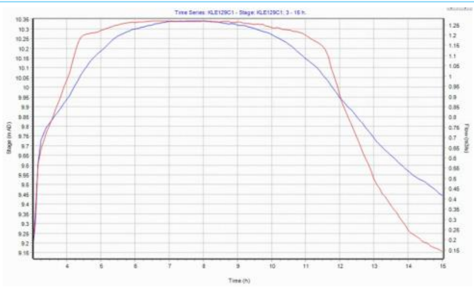
Flow (red) and Stage(blue) through culvert

KLE129C -Circular Culvert			
Width /Diameter	0.9m	**Length**	4.61m
Height	N/A	**Manning's**	0.02
US IL	8.89mOD	**DS IL**	8.79mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 1D using spill unit with levels set to road level		

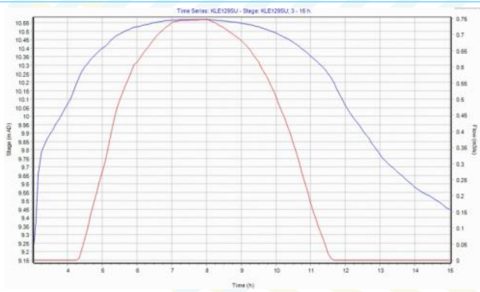






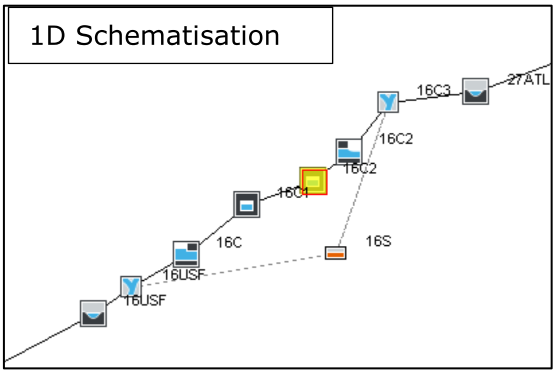


Flow (red) and Stage(blue) through culvert



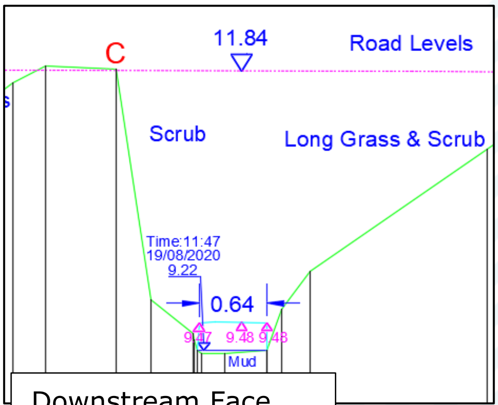
Flow (red) and Stage(blue) through spill

27ATLAT300016 (16C1)- Rectangular Culvert			
Width /Diameter	0.64m	**Length**	68.52m
Height	0.262m	**Manning's**	0.025
US IL	9.467mOD	**DS IL**	9.193mOD
Present in Model?	Yes – Rectangular Culvert		
Notes	Spill modelled in 1D using spill unit in FM with levels set to road level. Upstream Face was not captured in survey due to vegetation therefore levels at upstream were based on slope between downstream face and closet upstream cross section.		

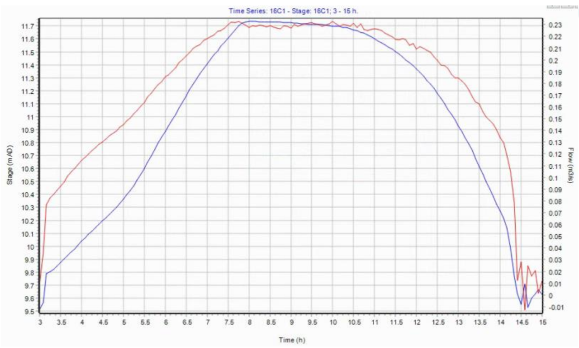


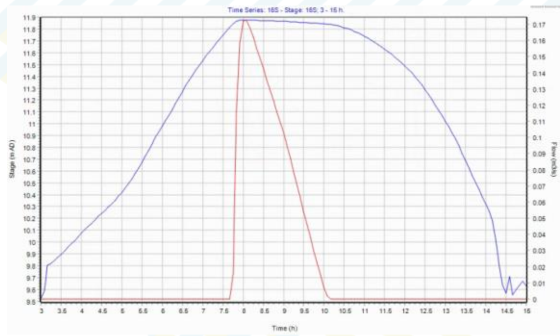
1D Schematisation


Downstream Face

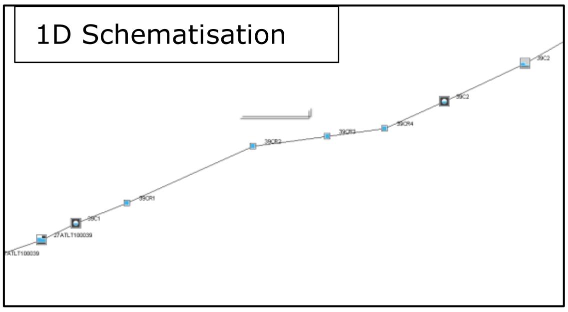


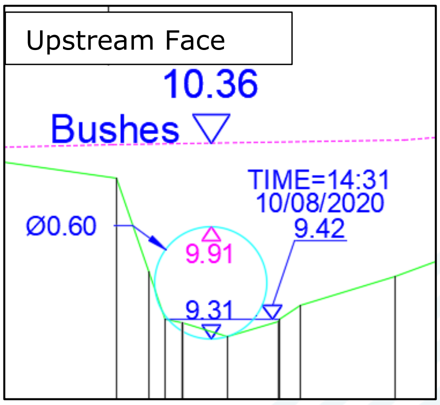
Downstream Face

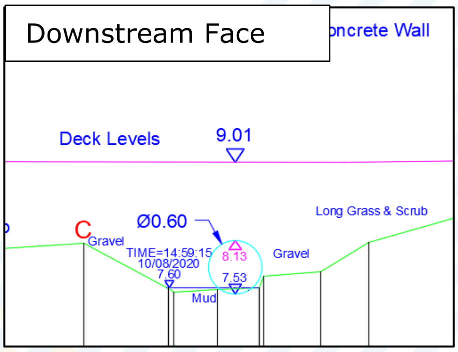

Flow (red) and Stage(blue) through culvert

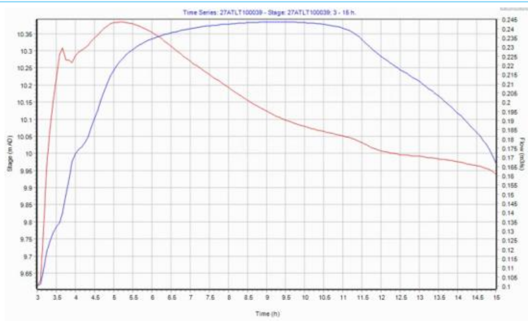

Flow (red) and Stage(blue) through spill

27ATLT100039 (39CR1) – Circular Culverts			
Width /Diameter	0.6m	**Length**	337.55m
Height	N/A	**Manning's**	0.025
US IL	9.31mOD	**DS IL**	7.53mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to ground levels above culverts		



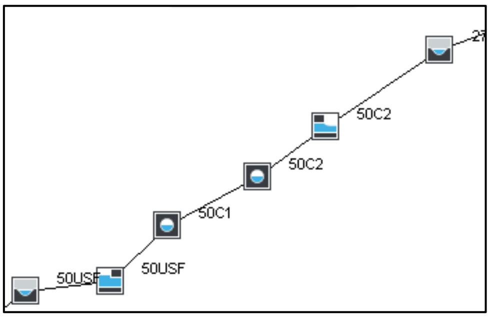




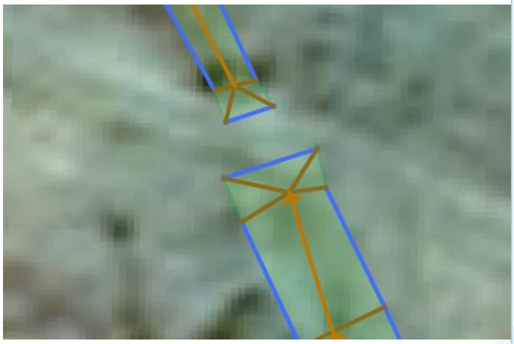


Flow (red) and Stage(blue) through culvert

27ATLT100050 (50C1)-Circular Culvert			
Width /Diameter	0.75m	**Length**	6.170m
Height	N/A	**Manning's**	0.025
US IL	9.598mOD	**DS IL**	9.47mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to the road levels The upstream face was not captured in survey due to heavy vegetation therefore levels at upstream were based on slope between downstream face and closet upstream cross section.		

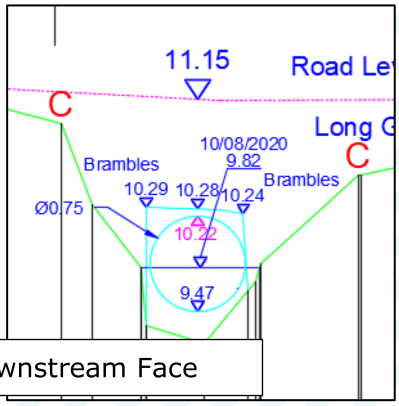


1D Schematisation

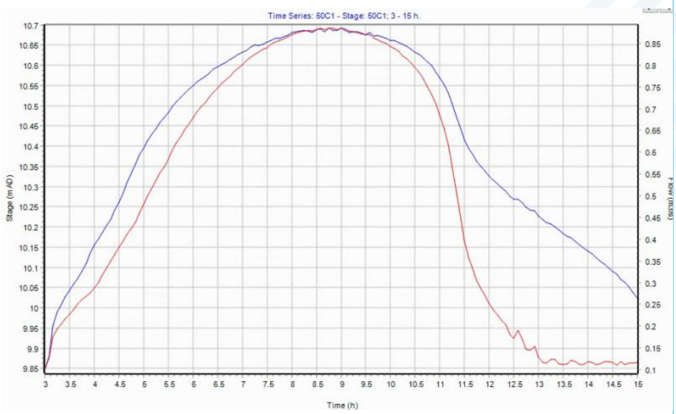


2D Schematisation

Downstream Face

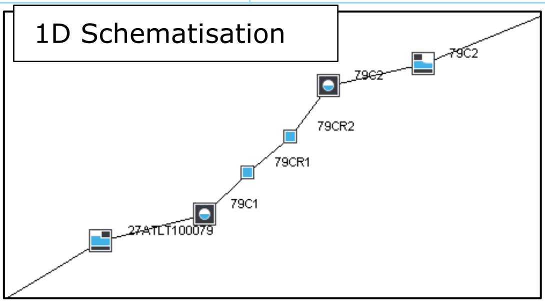


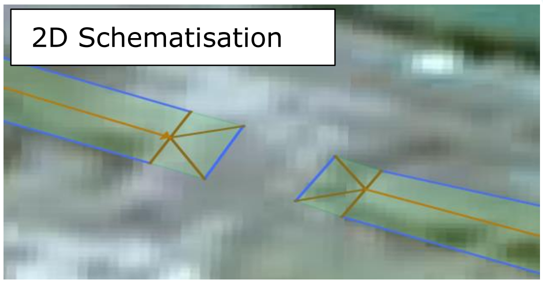
Downstream Face

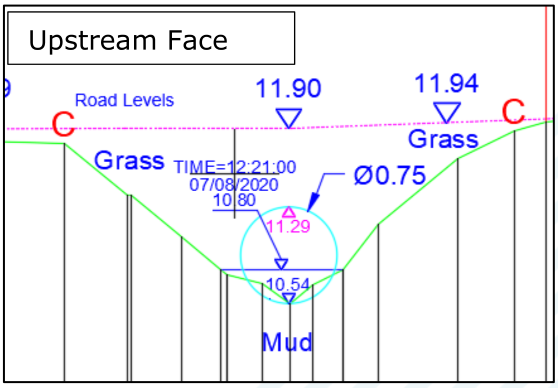


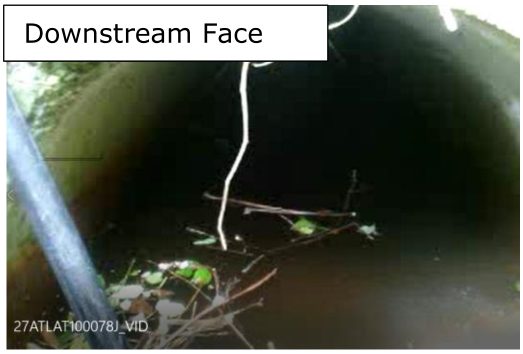
Flow (red) and Stage(blue) through culvert

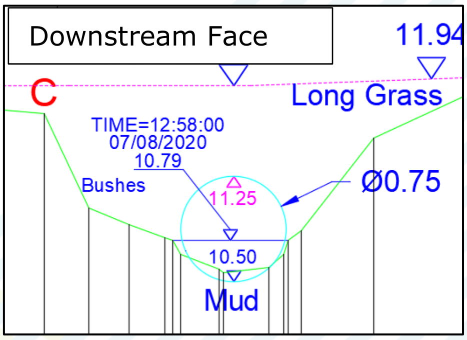
27ATLT100079 (79C1) – Circular Culvert			
Width /Diameter	0.75m	**Length**	7.5m
Height	N/A	**Manning's**	0.02
US IL	10.54mOD	**DS IL**	10.5mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to the road levels		

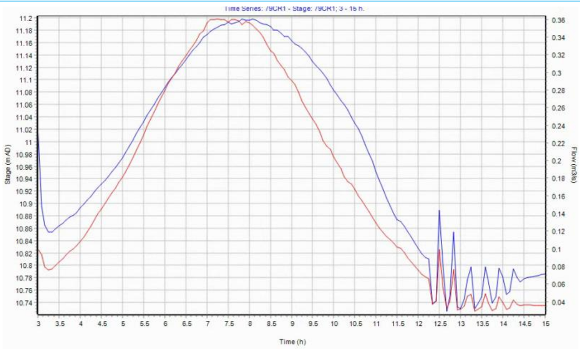




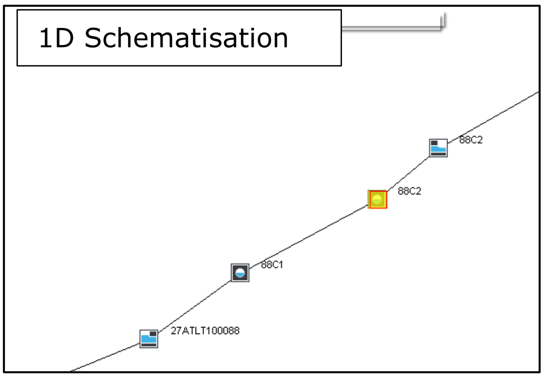
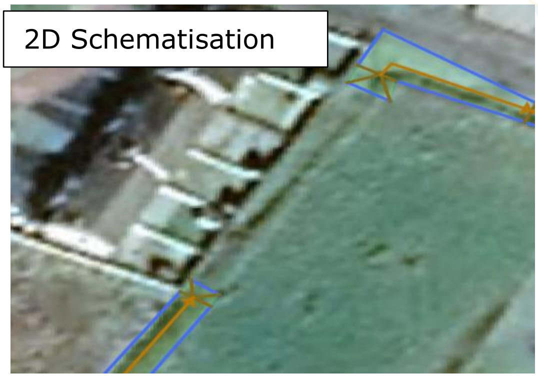
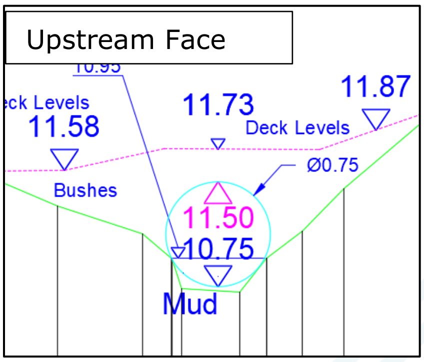
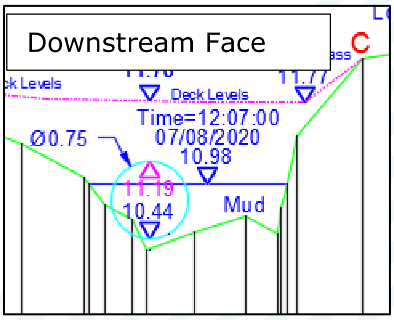
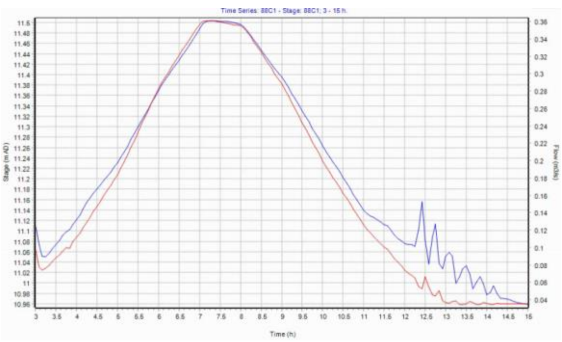






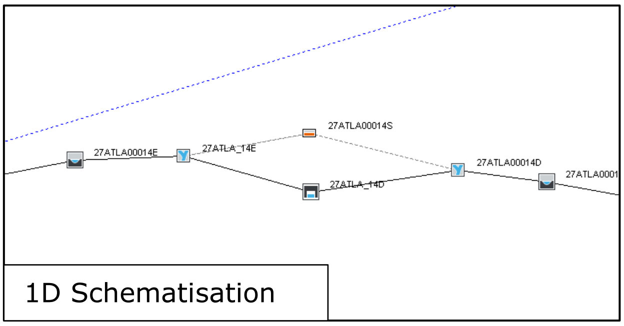


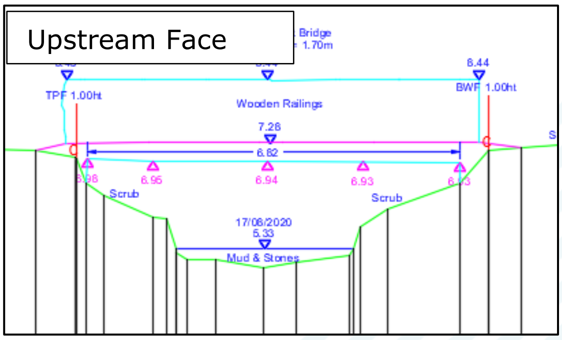
Flow (red) and Stage(blue) through culvert

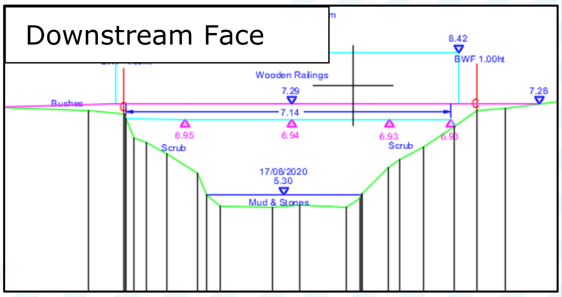
27ATLT100088 (88C1) – Circular Culvert			
Width /Diameter	0.75m	**Length**	45.10m
Height	N/A	**Manning's**	0.02
US IL	10.75mOD	**DS IL**	10.44mOD
Present in Model?	Yes – Circular Culvert		
Notes	Spill modelled in 2D using HX and CN lines with levels set to the deck levels		
			
			
			
 **Flow (red) and Stage(blue) through culvert**			

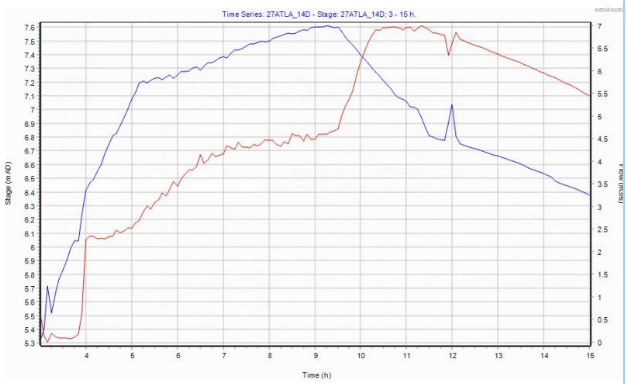
4.2.2 Bridges

27ATLA_14D - Footbridge			
Width	7.14m	**Length**	1.70m
Soffit	6.94mOD	**Springing height**	6.94mOD
Present in Model?	Yes – USBPR unit		
Orifice equation used	No	**Overtopping**	1D spill with levels set to the deck levels
Notes			



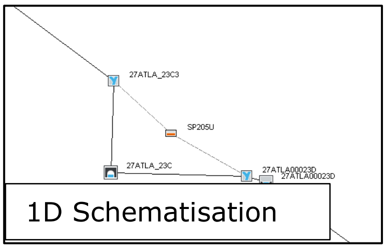






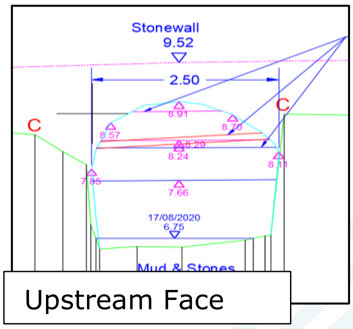
Flow (red) and Stage(blue) through structure

27ATLA_23C1 - Bridge			
Width	2.5m	**Length**	11.63m
Soffit	8.91mOD	**Springing height**	8.11mOD
Present in Model?	Yes – Arch Bridge		
Orifice equation used	No	**Overtopping**	1D spill with levels set to the deck levels
Notes	Spill modelled in 1D using FM Spill unit with levels set to road levels. . Structure upstream and downstream faces differ, modelled as upstream face as it has smaller conveyance area. The pipe crossing at the upstream face shown in the survey drawing below was not included in the model as the 1% and the 0.1% AEP maximum water level in the bridge is below the soffit of the pipes. The pipe therefore does not affect the conveyance area of the bridge.		



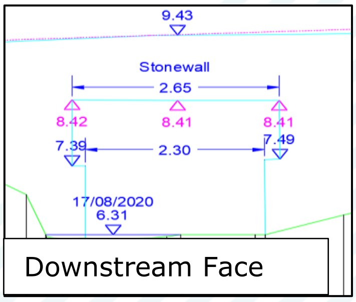
1D Schematisation

Upstream Face

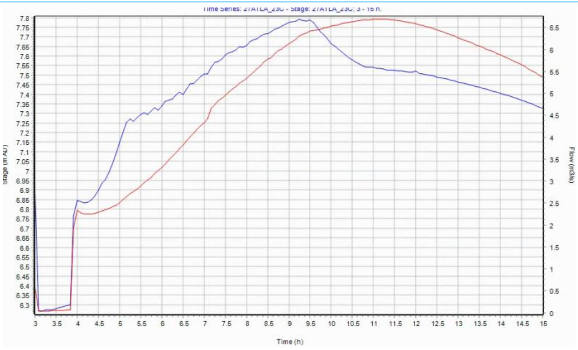


Upstream Face

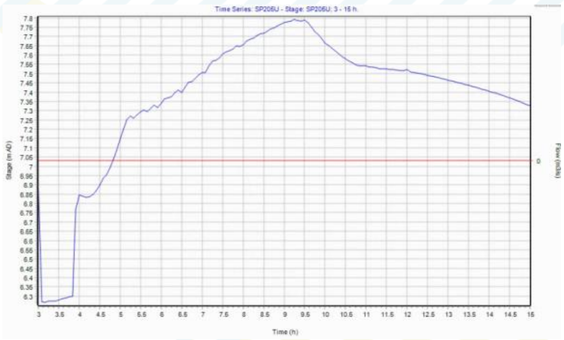
Downstream Face



Downstream Face



Flow (red) and Stage (blue) through culvert



Flow (red) and Stage(blue) through spill

4.2.3 Outfall Culvert

The Atlantic Stream outfall culvert was modelled using Causeway FLOW software. FLOW is a hydraulic modelling package for the design/analysis of drainage networks and is particularly useful in the representation of surcharging culverts and complex pipe networks as is the case for the Atlantic Stream outfall.

Downstream from the Atlantic Stream open channel, just at the Waterworld boundary, the stream enters a box culvert (3.2m wide X 1.5m high) which then splits into three separate culverts approx. 37m downstream of the inlet (under the road) before each culvert connects to an overflow chamber (referenced as CUL_A_MH1 on the CCTV Survey output shown in Figure 4-3 and Figure 4-5 below, MH5 in the FLOW model), with an outfall pipe and overflow pipe. The overflow chamber is located on the promenade steps with a cover level of 4.617m (from MCDS laser scan survey). The overflow chamber separates the DN750 outfall from the DN750 overflow pipe by a 3.72m wide weir at level of 3.522m (Node 5 leading to the outfall pipe, Node 9 leading to the overflow pipe). The main DN750 outfall pipe discharges to Moore Bay approx. 163m downstream. The DN750 overflow pipe discharges to the beach approx. 28m downstream of the overflow chamber above the high water mark. The weir wall is a concrete wall set approx. at the soffit level of the overflow pipe.

At the time of the CCTV survey the two larger culvert chambers were blocked with stone debris to approximately 25% of their depth and flow all went down the smaller culvert opening. However, the debris blockage will be overtopped in times of high flow and should be removed as part of general maintenance. The partial blockage has not been modelled as it does not affect the flooding scenario upstream of the culvert. The plan layout of the FLOW model is shown in Figure 4-1 below. Reference Node 5 is the overflow manhole.

The detail of the three culverts and overflow arrangement are also shown in Figure 4-2 for more clarity.

The CCTV survey revealed that the DN750 outfall pipe had numerous cracks and fractures.

The FLOW model begins at the upstream of the outfall culvert and ends at the two outfalls onto the beach. The upstream boundary of the FLOW model (at Node 1 – the 3.2m wide X 1.5m high inlet culvert) is a flow hydrograph extracted from the Flood Modeller model and the downstream boundaries are tidal boundaries. The changeover node is Node 2, where the culvert splits into three separate culverts. The downstream boundary for the Flood Modeller model is extracted from this node.

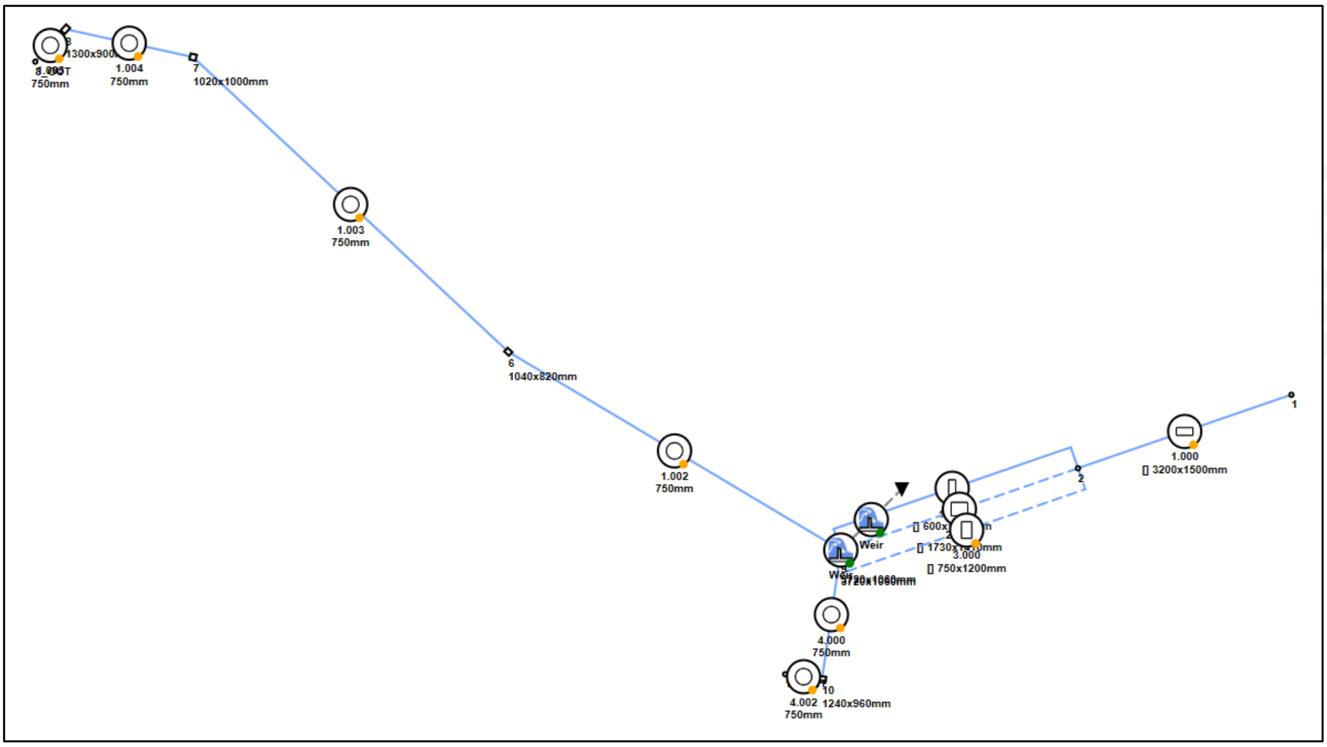


Figure 4-1 - FLOW Model Culvert Layout

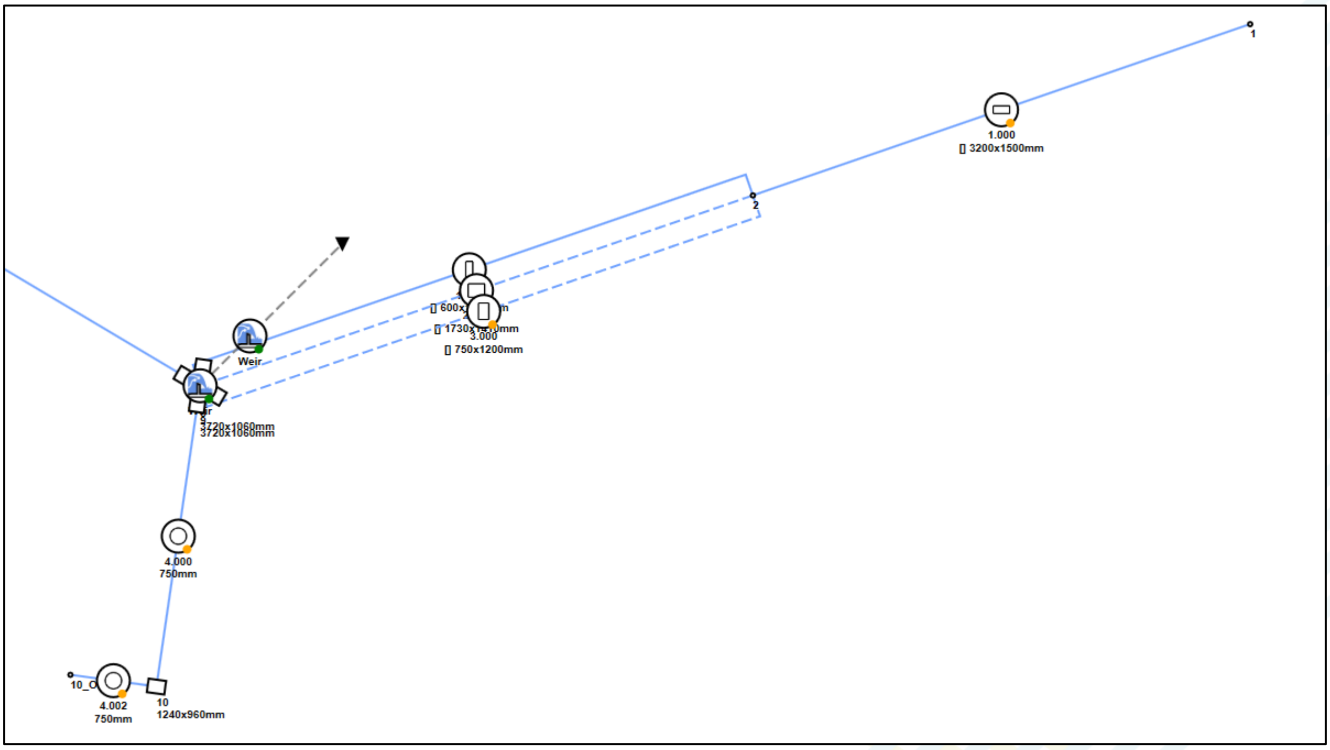


Figure 4-2 - FLOW model - detail of culvert and overflow

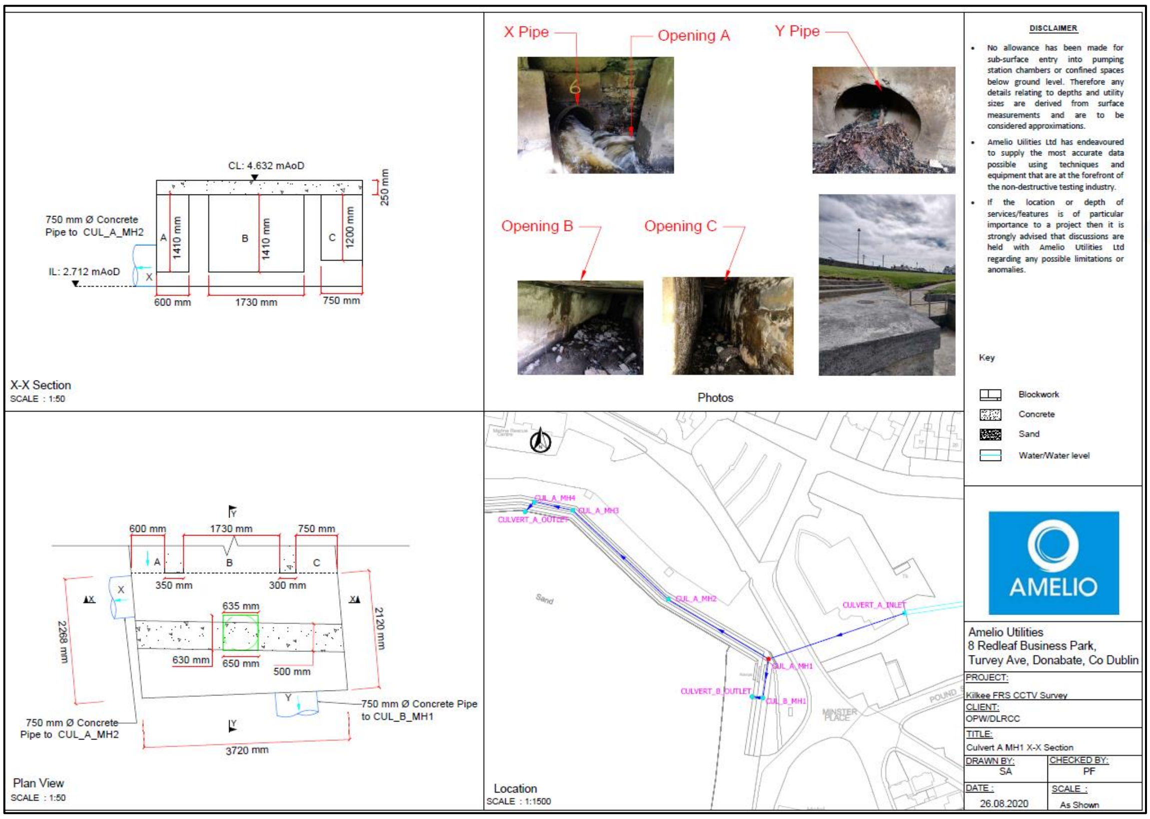


Figure 4-3 - Atlantic Stream Culvert from CCTV Survey

4.2.3.1 FLOW Model Results

The results of the 1% AEP storm are indicated in the profile below and indicates flooding at Node 5, the overflow manhole. A stage-discharge relationship at the inflow to the culvert which was generated from the Flood Modeller model at this point and input into the FLOW model as the inflow hydrograph. The profile, shown in Figure 4-4, indicates that the DN750 outfall pipe (and overflow pipes) are not capable of taking the flow and are surcharged.

Light flooding at Node 5 is also indicated for the 20% AEP event.

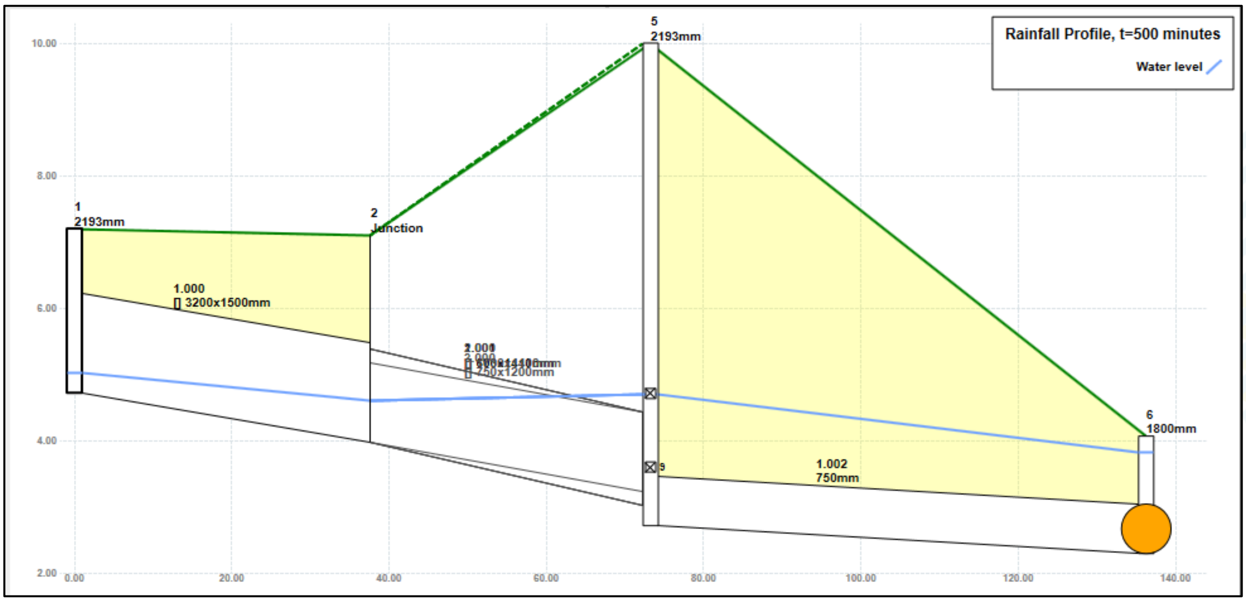


Figure 4-4: FLOW Profile for 1% AEP event

The IL of the inlet culvert is at 4.719mOD and the cover level of the downstream overflow manhole, Node 5, is 4.617mOD. Flood relief of the system occurs at Node 5, the overflow manhole, with flood waters flowing down the two steps of the promenade and onto the beach. However, the release at this point is not sufficient and the insufficient capacity of the pipes still results in a pressurised pipe system upstream. The outfall and overflow pipes therefore result in backing up of flow upstream of the culvert network and flooding of Waterworld.

Providing a sealed manhole at Node 5 will allow the system to surcharge with the next low point being gullies in the adjacent car park with typical ground level of 6.2mOD. Flood flow would be directed over the promenade steps and onto the beach.

The ground level at the culvert inlet upstream from Waterworld is 7.2mOD.

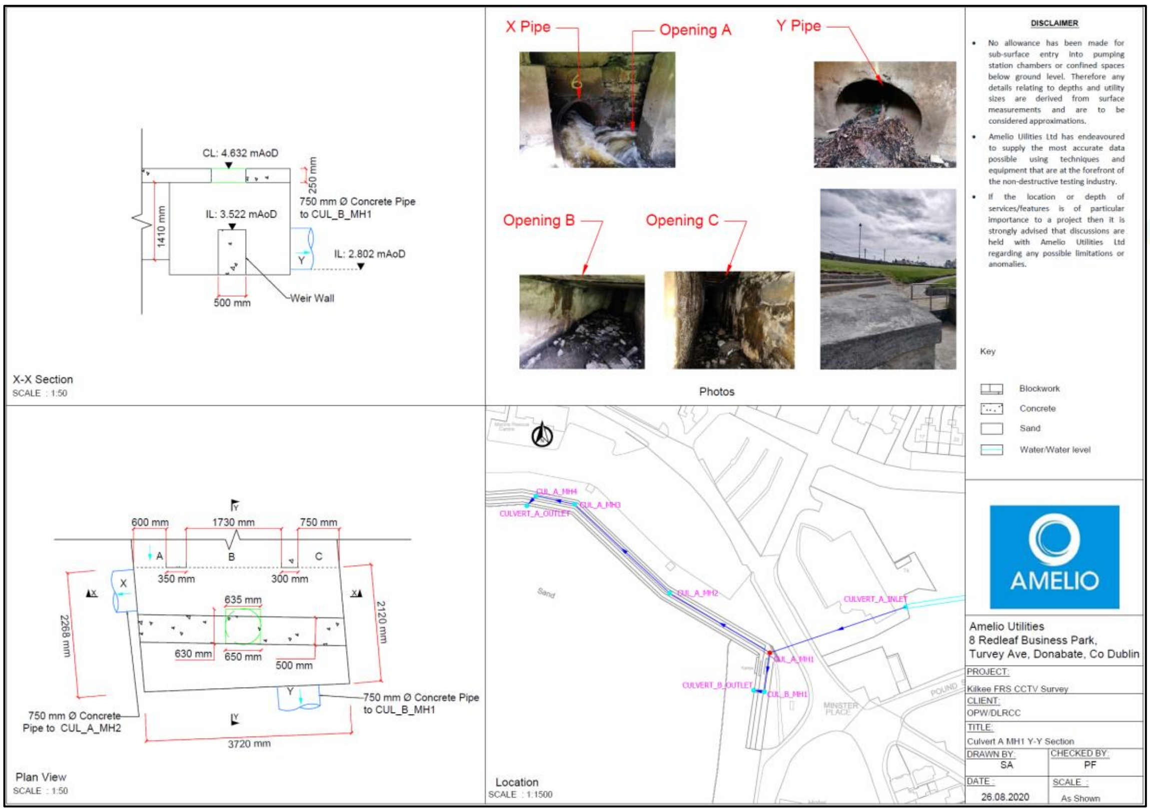


Figure 4-5: Atlantic Stream Culvert - CCTV Survey

The stage – discharge curve which connects the FLOW model with the Flood Modeller model is shown in Figure 4-6. This stage discharge takes the flow and water level from the Flood modeller model at the upstream point of the outfall culvert and inputs into the FLOW model at the upstream boundary.

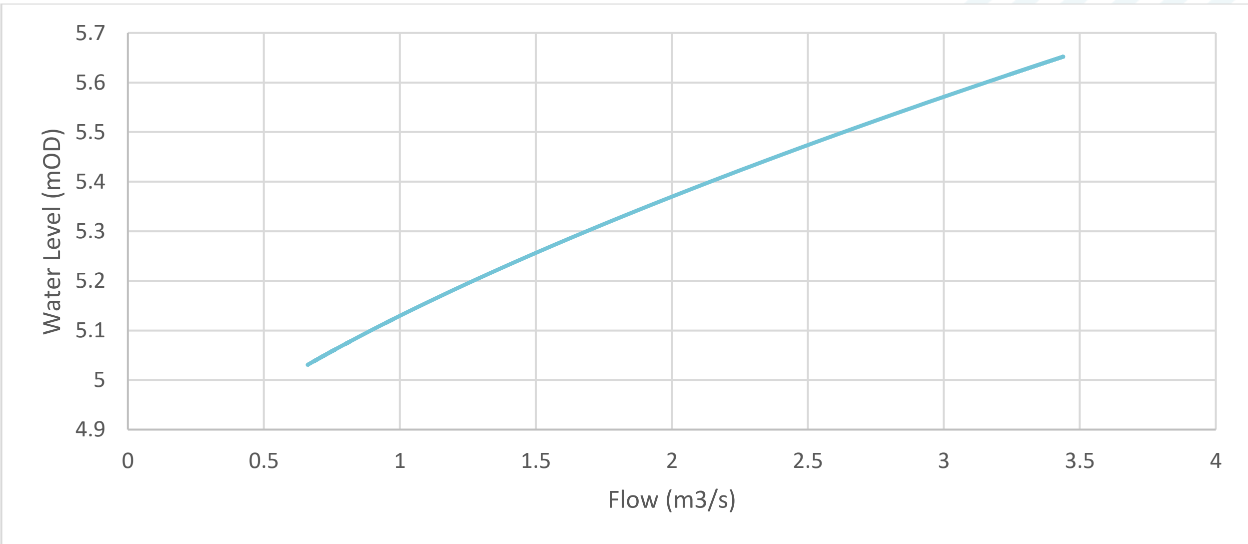


Figure 4-6: Stage – Discharge Relationship

5 Model Scenarios

A series of model runs were undertaken to meet the objectives of the project.

The key model files for Flood Modeller and TUFLOW are outlined in Table 5-1 and the FLOW model files are outlined in Table 5-2.

Table 5-1: Key FM – TUFLOW model files – Flow events

Key model Files			
Flood Modeller Files	IEF	27S19_KILK_VIC_SL_Q100_052	27S19_KILK_ATL_Current_Q100_061
	DAT	27KILK_VIC_SL_027	27KILK_ATL_NOBLK_060
	IIC	27S19_KILK_VIC_Q100_035	In dat
	IED	VIC_W_Q100_002 / VIC_T2_003	ATL_W_Q100_060 / ATL DS Q100_NOBLK_001
	Results	27S19_KILK_VIC_SL_Q100T2_052	27S19_KILK_ATL_Current_Q100_061
Tuflow Files	Tcf	VIC_~s1~_~s2~_~e1~_052	ATL_~s1~_~s2~_~e1~_061
	Tgc	27KILK_VIC_052	27KILK_ATL_058
	Tbc	27KILK_VIC_052	27KILK_ATL_061
	Tmf	landuse_002	landuse_002
	Trd	VIC_Common_Commands_052	ATL_Common_Commands_061

Table 5-2: FLOW model files

Key FLOW model Files		
FLOW Files	*.pfd	19109-JBAI-XX-XX-CA-C-00846_FLOW_Q100_720_mins
	*.pfd	19109-JBAI-XX-XX-CA-C-00845_FLOW_Q5_900_mins
	*.pfd	19109-JBAI-XX-XX-CA-C-00844_FLOW_Q2_720_mins

6 Model Performance

This section summarises the general performance of the hydraulic model.

6.1 Timestep and Model Run Time

Model runs were run in double precision with the following timesteps:

- Victoria Model – 0.5 second 1D FM timestep and 0.5 second 2D timestep
- Atlantic Model - 1 second 1D FM timestep and 0.5 second 2D timestep

6.2 1D Flood modeller stability

The plots in Figure 6-1 and Figure 6-2 below show the 1D FM convergence plots for the 1% AEP event. The 1D models are stable with minor points of non-convergence and the convergence of flow through the model is good. FM 1D mass balance is reported as negative due to spilling of water into the 2D domain and so not reflective of the overall model health.

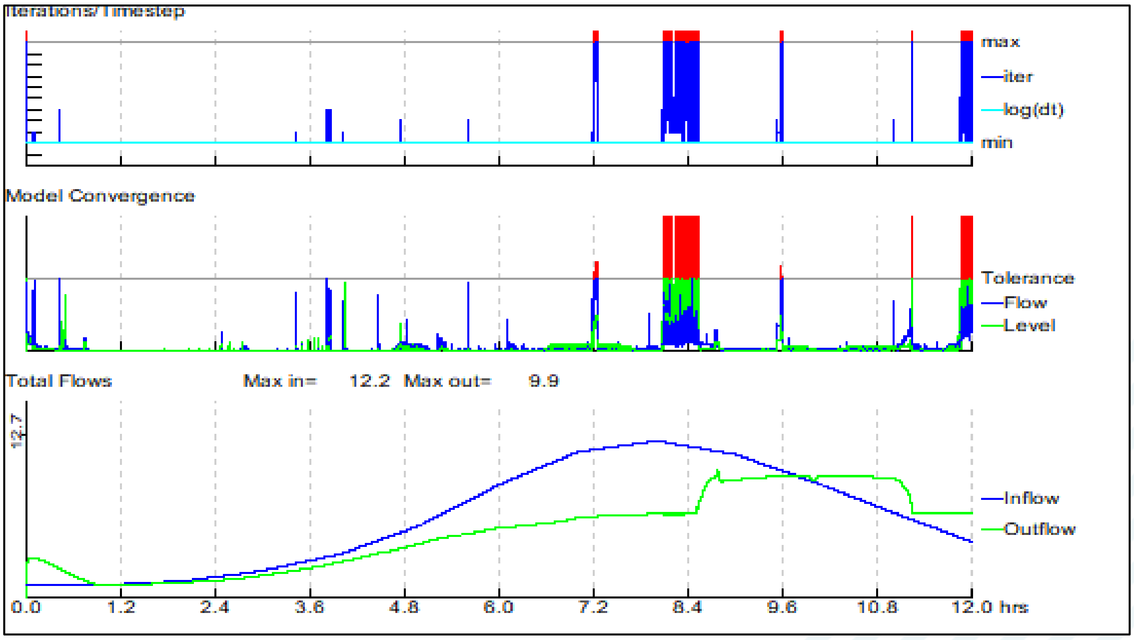


Figure 6-1: FM convergence plots for the 1% AEP for the Victoria Model

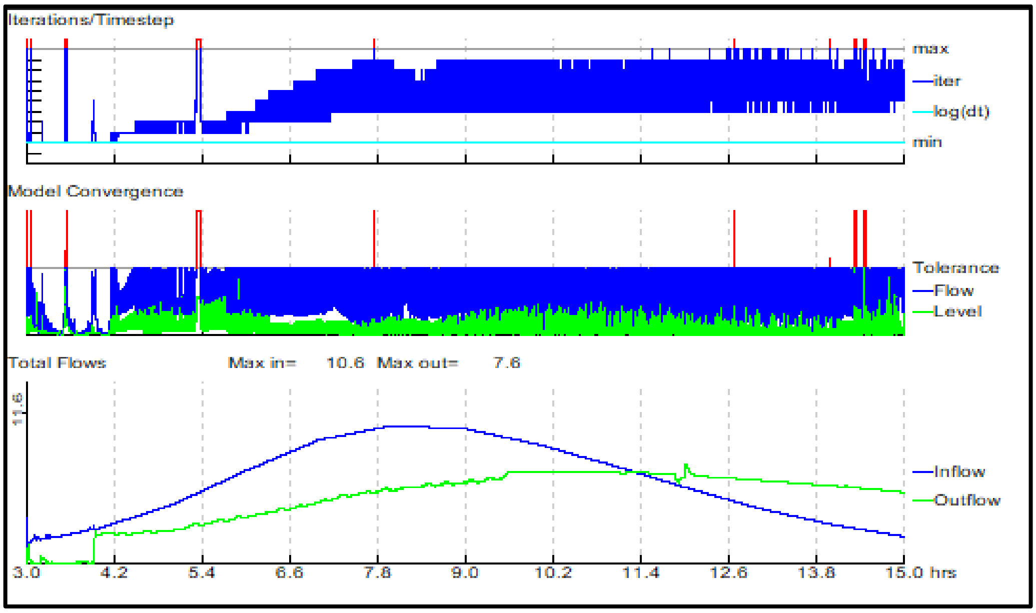


Figure 6-2: FM convergence plots for the 1% AEP for the Atlantic Model

6.3 2D TUFLOW stability

Within the 2D domain to ensure that the model is stable and performing adequately three main factors are examined:

- Checks and warnings recorded,
- Number of negative depths,
- Mass balance error (MBE).

6.3.1 Checks and warnings

The following checks and warnings occurred prior the start of the model run:

- WARNING 2073 – Object ignored. Only Points, Lines, Polylines, Regions & Region Centres used.

  This WARNING occurs when there is a null shape in the GIS data. This WARNING does not affect model performance.
- CHECK 2370 - Ignoring coincident point found in Create TIN layer.

  This CHECK occurs where there is overlapping points in the TIN layer. This CHECK does not affect model performance.
- CHECK 2231 - No ZP points snapped to HX line. HX line not used to modify Zpts.

  All points where this CHECK occurs have been checked. This CHECK does not affect model performance.

6.3.2 Negative depths

No negative depths are reported in the model.

6.3.3 Mass balance error (MBE)

The tolerance limit for MBE is +/- 1.0%. The highest MBE value recorded for the 1%AEP is -0.79% and -0.59% for the Victoria and the Atlantic Stream respectively. The mass balance error is within the tolerance of +/-1%. The final baseline models were re-run with differing start times and initial conditions and there is a higher MBE at the start time of these runs as a result of this.

7 Sensitivity testing

This section describes the various sensitivity tests carried out to examine the performance calibrated model. All sensitivity tests have been carried out using the 1% AEP event. Results for both models are presented in the same figure unless stated otherwise.

7.1 Joint probability screening

Joint Probability analysis predicts the probability of occurrence of events in which two or more partially dependent variables simultaneously take high or extreme values. Joint probability between coastal and fluvial events, is considered to be an issue on the Victoria Stream. It is not considered on the Atlantic Stream as the as the area of flood risk is above the tidal levels, even in climate change.

Refer to 19109-JBAI-XX-XX-RP-H-00344_Hydrology_Report_C06 for details on how the joint probability combinations were determined. Before looking at the complete suite of JP combinations throughout the Victoria system it is usual to undertake a screening assessment. The extreme bounds of the JP combinations were tested in the hydraulic model i.e., the 0.5% Tidal AEP combined with 50% Fluvial AEP, the 0.5% Fluvial AEP combined with the 50% Tidal AEP and the 0.5% Fluvial AEP with the 0.5% Tidal AEP. The maximum water level along the Victoria Stream is shown in Figure 7-1 for each of these combinations. The fluvial dominant zone and the tidal dominant zone are identified, upstream of the Victoria Stream boundary walls is dominated by the fluvial flow, and downstream of this point the tidal element is dominant. There is no section of river reach where the tidal and fluvial are equally dominant . The baseline Well Stream connection to the Victoria occurs within the fluvially dominant zone, therefore there is no intertidal zone along the Well Stream and it is also fluvially dominated. Therefore, a detailed joint probability analysis is not needed, and upstream of the boundary walls the fluvial levels will dictate levels and extents and downstream of the boundary walls the downstream levels will dictate levels and extents when it comes to mapping and design.

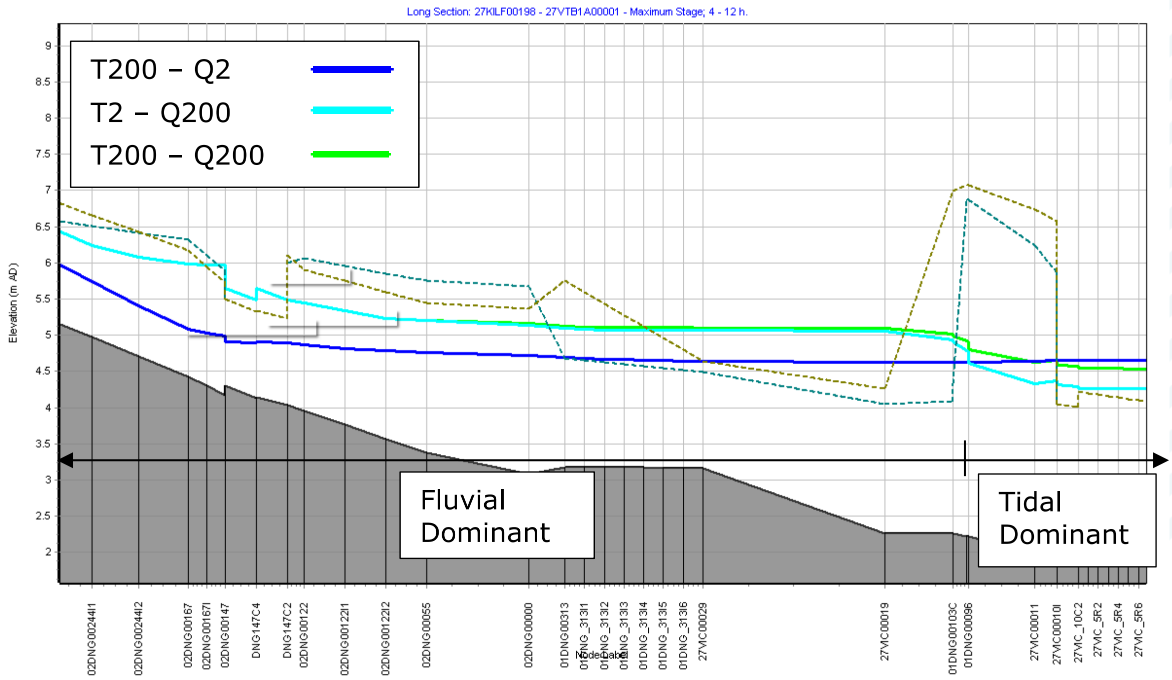


Figure 7-1: Joint Probability Screening on the Victoria Stream

7.2 Cell size sensitivity – 2m 2D grid cell size

All runs have been completed using a 2m resolution 2D grid as it was felt to appropriately capture all the critical flow paths in the key risk receptor areas. Figure 7-2 compares the modelled flood extents between the 2m and 1m cell size runs. The total run time for the 1m grid cell size 1%AEP event was 20 hours which is significantly longer than the 5 hours it takes to run the 2m grid for the same event.

Overall, the 2m grid cell run shows greater flood extents compared to the 1m cell run. This is likely due to increased detail in the 1m grid restricting flow paths with more water being retained on flood plains and not flowing out across areas.

Given that there is little difference in flood level reported and the 2m cell run produces more conservative extents and has a significantly shorter run time the 2m grid cell size is considered sufficient for the purpose of the model and overall project aim.

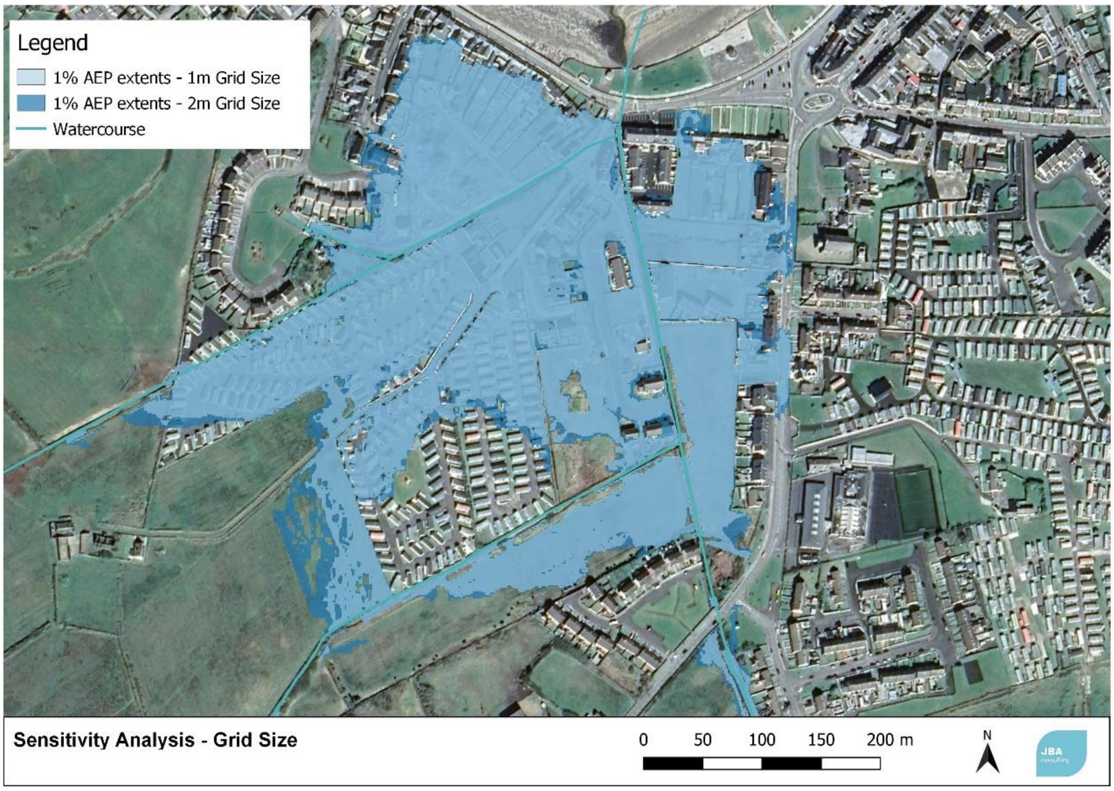


Figure 7-2: Sensitivity Analysis of Grid Size on Victoria Stream

7.3 Increase/decrease in roughness

To test sensitivity, Manning's N roughness of the 1D channel and the 2D floodplain were increased and decreased by 20% to assess the impacts. Figure 7-3 compares the flood extents generated for the 1% AEP event with different roughness values.

It is noted that the model extents change with variation in roughness with slightly larger flood extents observed when roughness is increased. This is expected as increased roughness results in build-up of water within the channel and floodplain. However, the increases in extents are not great, highlighting that seasonality of vegetation growth does not significantly impact potential flood extents.

Overall, it is considered that the in channel and flood plain roughness values selected are appropriate and represent the average catchment condition in relation to roughness.

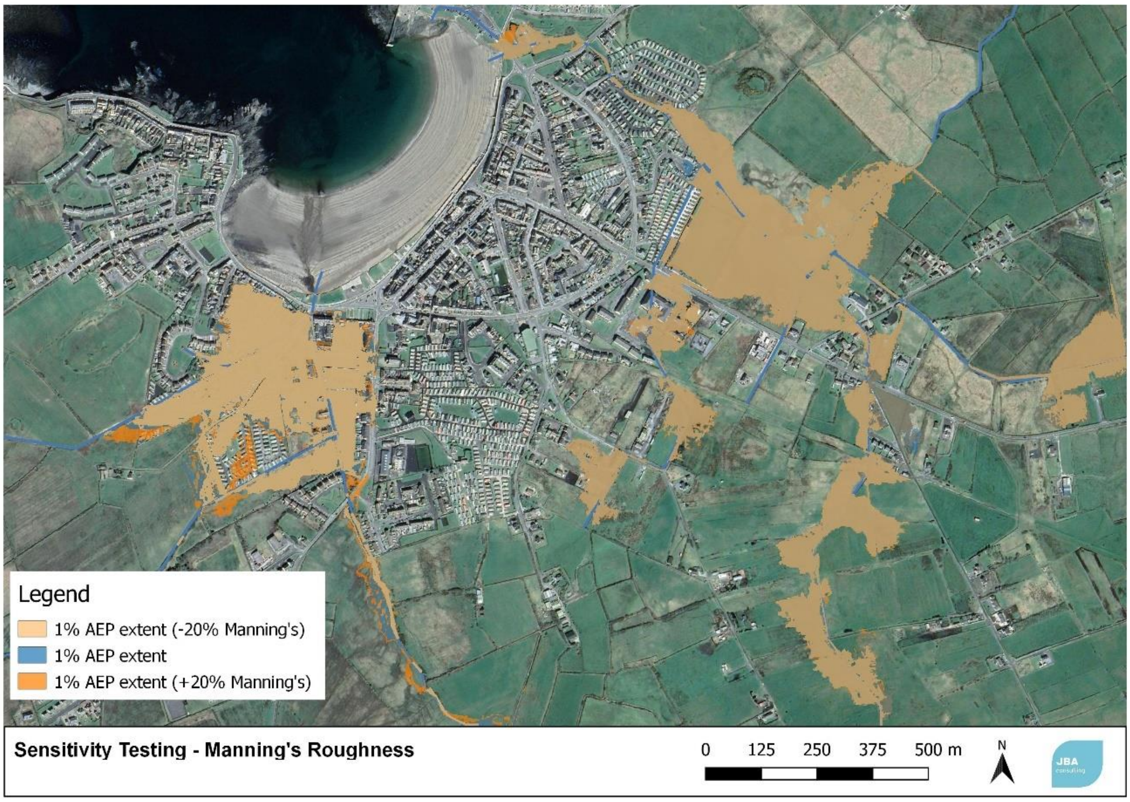


Figure 7-3: Sensitivity Analysis of Manning`s Roughness

7.4 Summer and Winter Storm

As flooding in Kilkee has occurred in both winter and summer months the 75% winter storm hydrograph profiles and the 50% summer storm hydrograph profiles were both compared. The summer profile has a higher peak flow; however, the winter profile has a larger volume of flow. The winter storm profile produces greater flood extents from the 1% AEP than the summer storm profile, as shown in Figure 7-4 below. It is also noted that most flooding is reported in the winter months. The winter storm profile is therefore used for all runs.

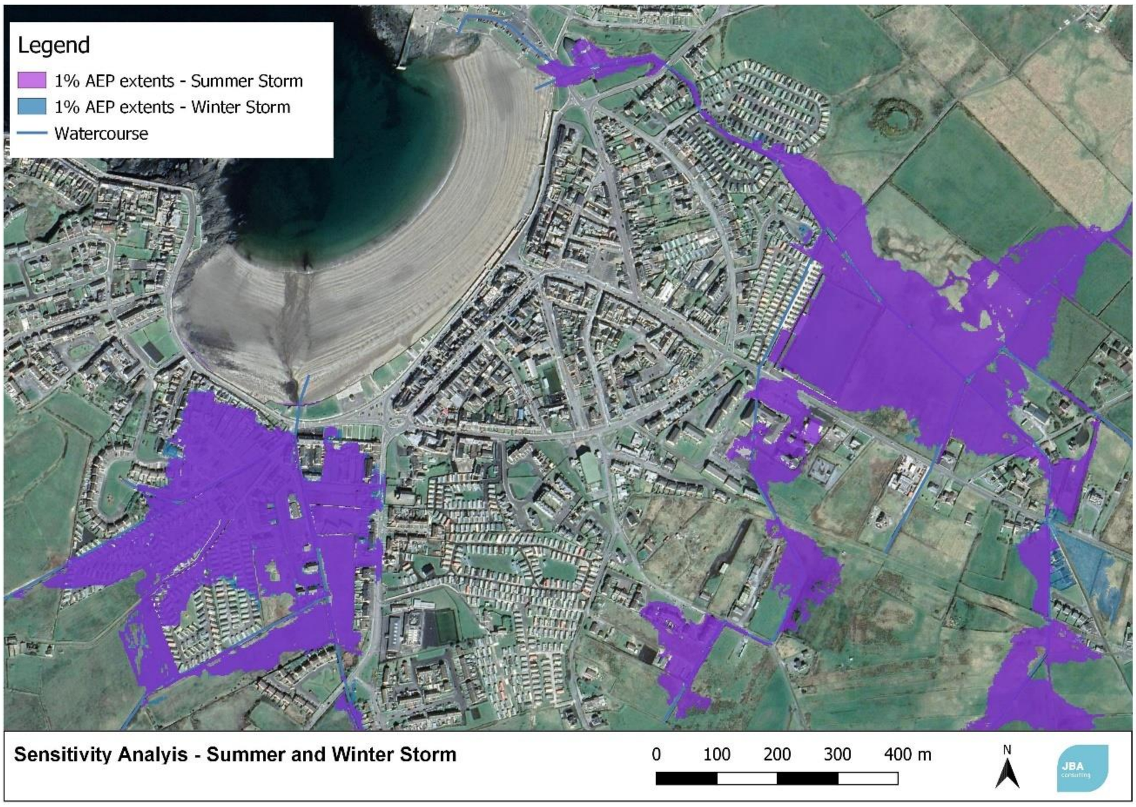


Figure 7-4: Comparison of Summer and Winter Storm 1% AEP flood extents

7.5 Blockage

Blockage along a number of key structures was tested as part of this analysis.

7.5.1 Atlantic Stream

7.5.1.1 Kilkee Bay Hotel culvert

67% blockage was tested at the culvert by the Kilkee Bay Hotel. This channel is heavily vegetated, and the CCTV of the culvert also shows significant silting and damage, which indicates blockage of the culvert is likely. As can be seen in Figure 7-5 blockage at this culvert results in a minor increase in flood extents along the tributary and also due to the increased flow across the road there is a minor increase in flood extents at the GAA field. The flood depths also increase by less than 100mm as a result of the blockage at the culvert.

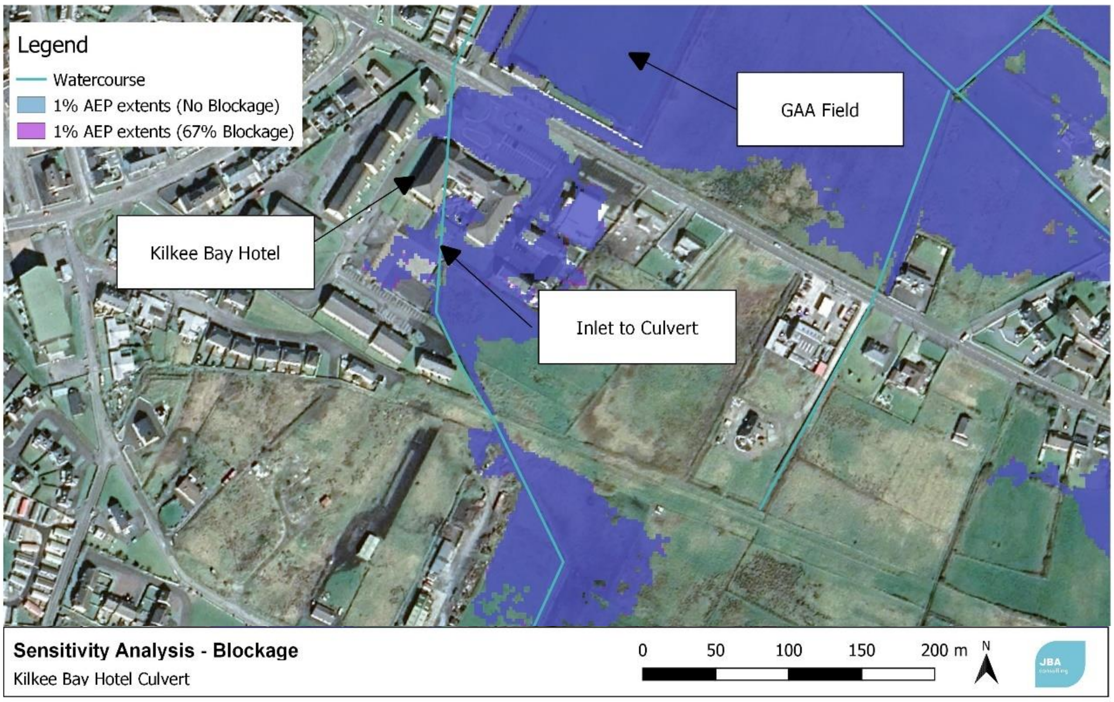


Figure 7-5: Sensitivity Analysis of blockage at the Kilkee Bay Hotel Culvert

7.5.1.2 Carrigaholt Road Culvert

The result of testing 67% blockage at the Carrigaholt Road culvert is shown in Figure 7-6. Flow backs up behind the culverts and flows over the road. A maximum depth of 230mm is modelled on the road.

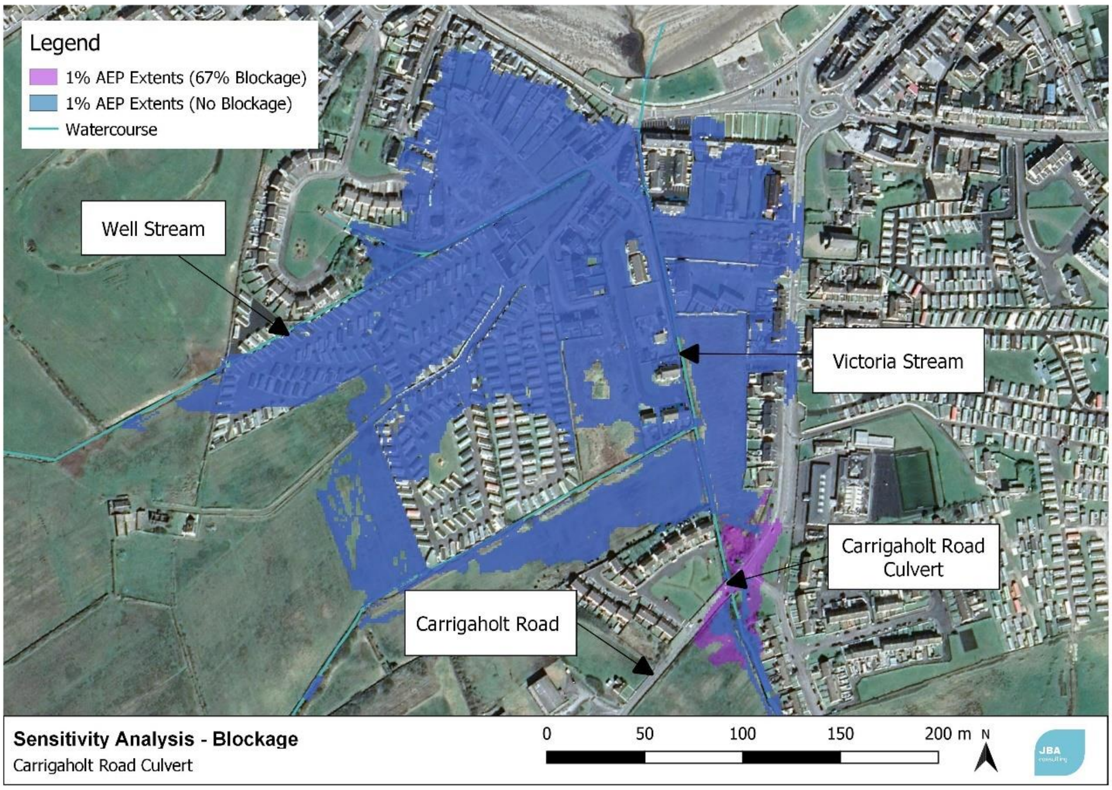


Figure 7-6: Sensitivity Analysis of blockage at the Carrigaholt Road Culvert

7.5.1.3 Trash Screen and Security Screen at Outfall

A blockage of 80% was tested on the Atlantic Stream security and trash screens at the outfall. Significant blockage has been reported at both screens in the past during flood events, therefore a consideration of blockage at the screens is important. As can be seen in Figure 7-7, blockage results in a significant increase in flood extents. This blockage results in flooding of the Waterworld building which is representative of reported historic flood events. Significant blockage is therefore considered representative of the baseline event and 80% blockage on the security screen and the trash screen will be used.

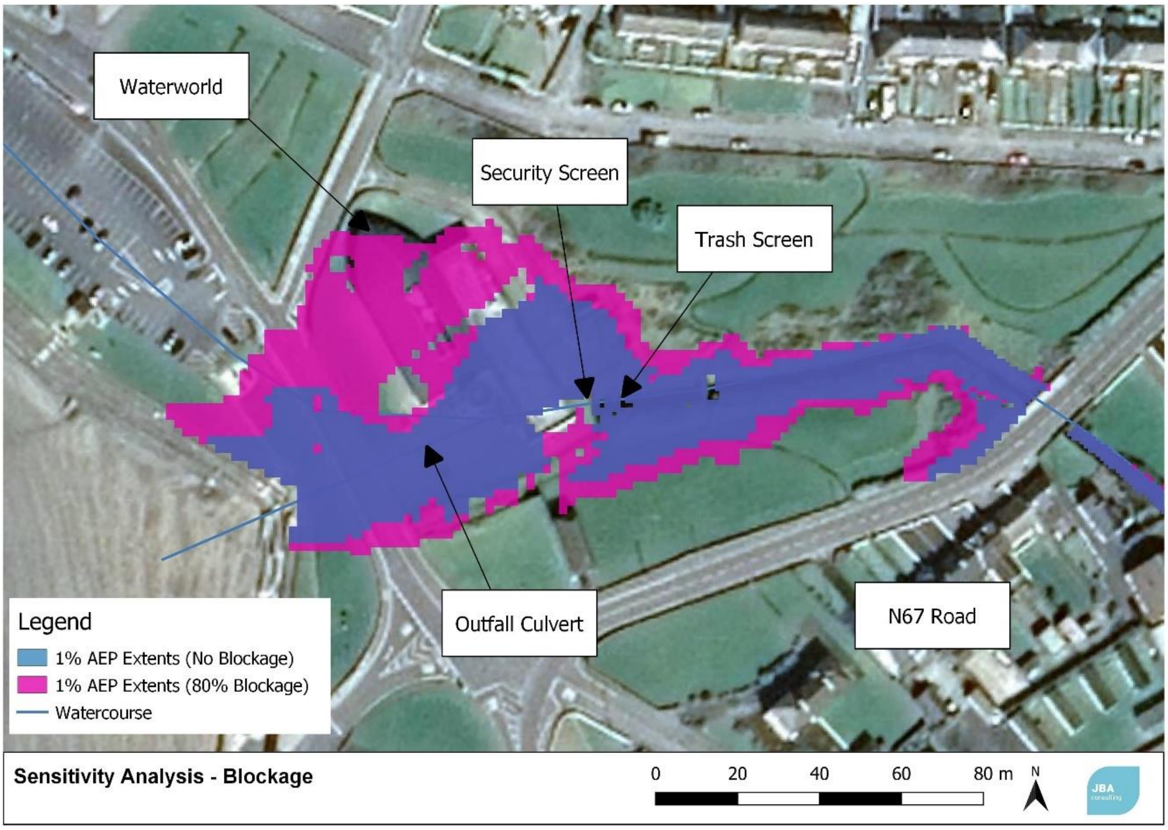


Figure 7-7: Sensitivity Analysis of blockage at Atlantic Stream trash Screen and Security Screen

7.5.2 Victoria Stream

7.5.2.1 Well Stream Culvert

A blockage of 67% was tested on the Well Stream Culvert. Due to the skew face of the inlet to the culvert, blockage was considered an important factor to consider. As can be seen in Figure 7-8 there is a minor increase in flood extents and an increase in flood depth of up to 100mm as a result of blockage at the Well Stream Culvert.

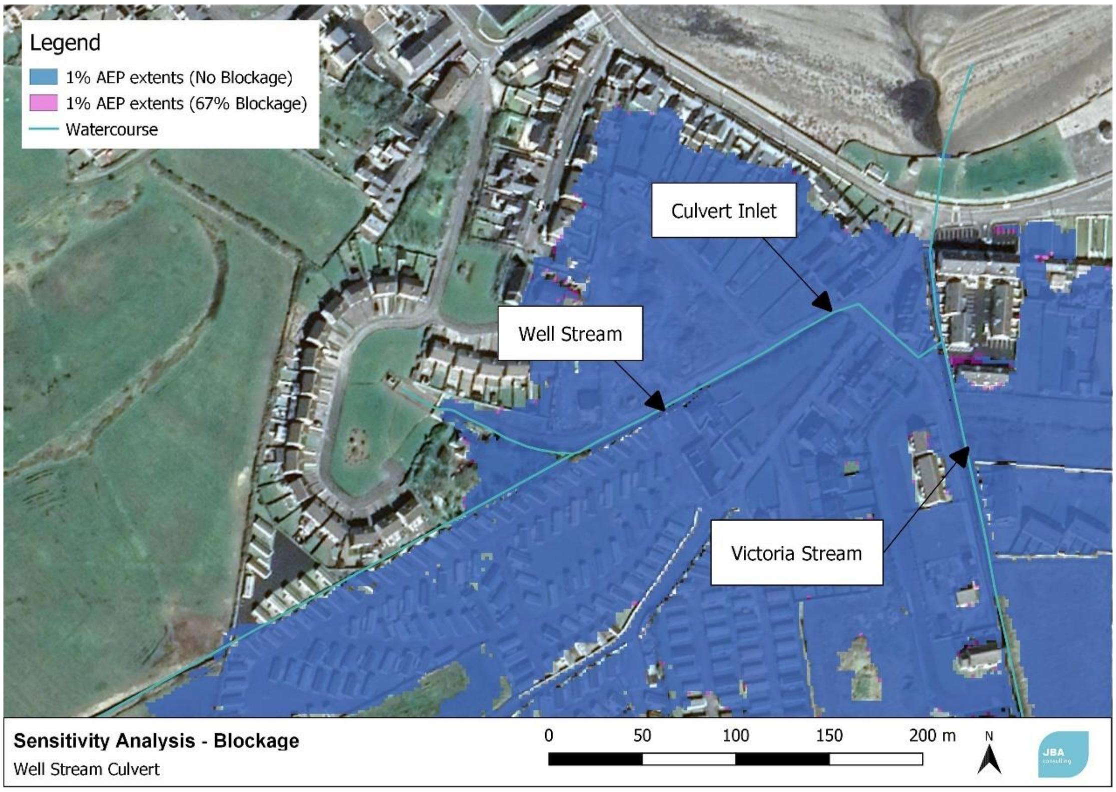


Figure 7-8: Sensitivity Analysis of blockage at the Well Stream Culvert

7.5.2.2 Victoria Stream Outfall Culvert

A blockage of 67% was tested on the Victoria Stream outfall Culvert. As can be seen in Figure 7-9 there is a minor increase in flood extents and an increase in flood depths of up to 40mm as a result of blockage at the outfall culvert.

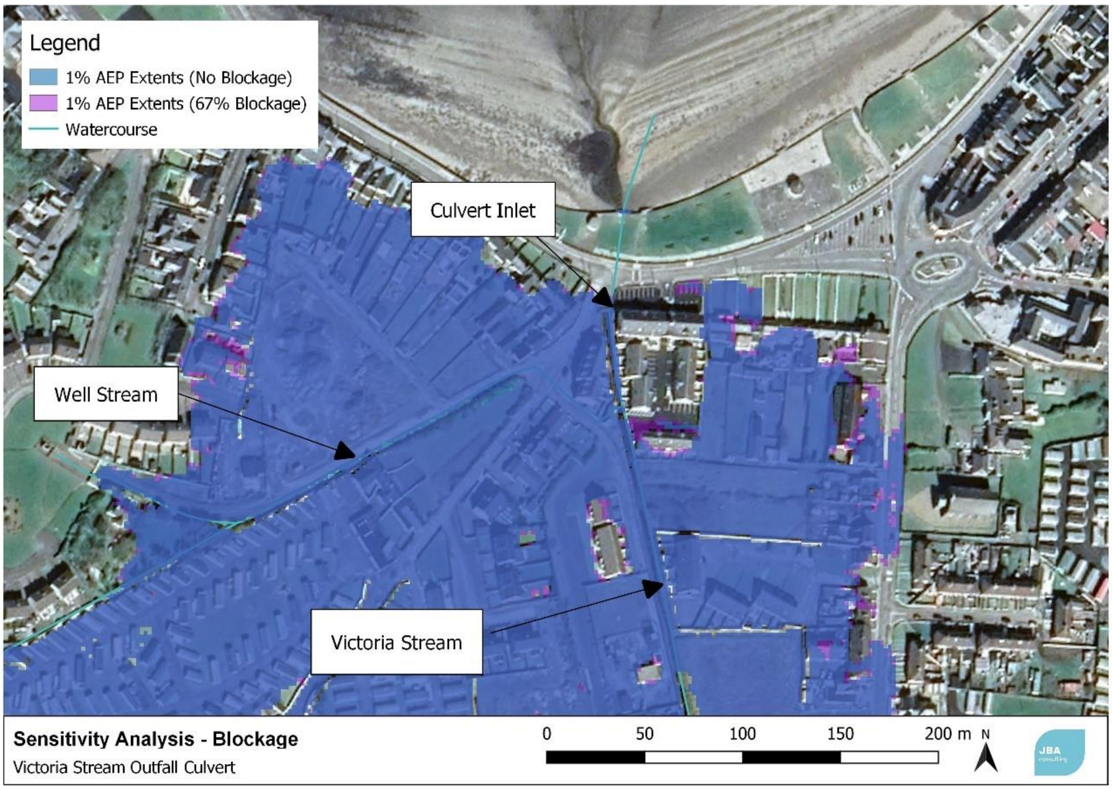


Figure 7-9: Sensitivity Analysis of blockage at the Victoria Stream Culvert

7.5.2.3 Victoria Stream Pipe Crossings

A blockage of 67% was tested on both the 400mm diameter pipe crossing and the 250mm diameter pipe crossing on the Victoria Stream. As can be seen in Figure 7-10 blockage of the 400mm diameter pipe crossing results in a significant increase in flood extents. Flood depths also increase by approximately 100mm. Blockage of the 250mm pipe crossing results in minor increase in flood extents and an increase in flood depths of a maximum of 20mm, as shown in Figure 7-11.

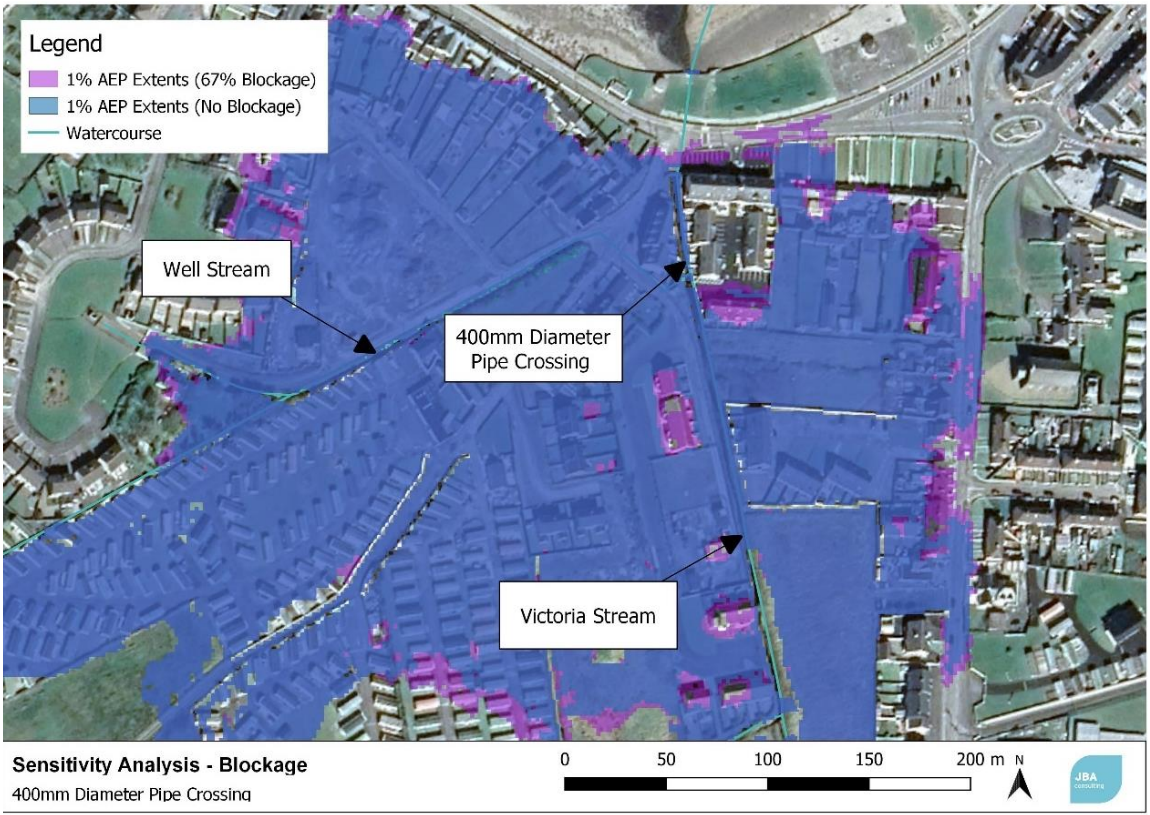


Figure 7-10: Sensitivity Analysis of blockage at the 400mm diameter Pipe Crossing

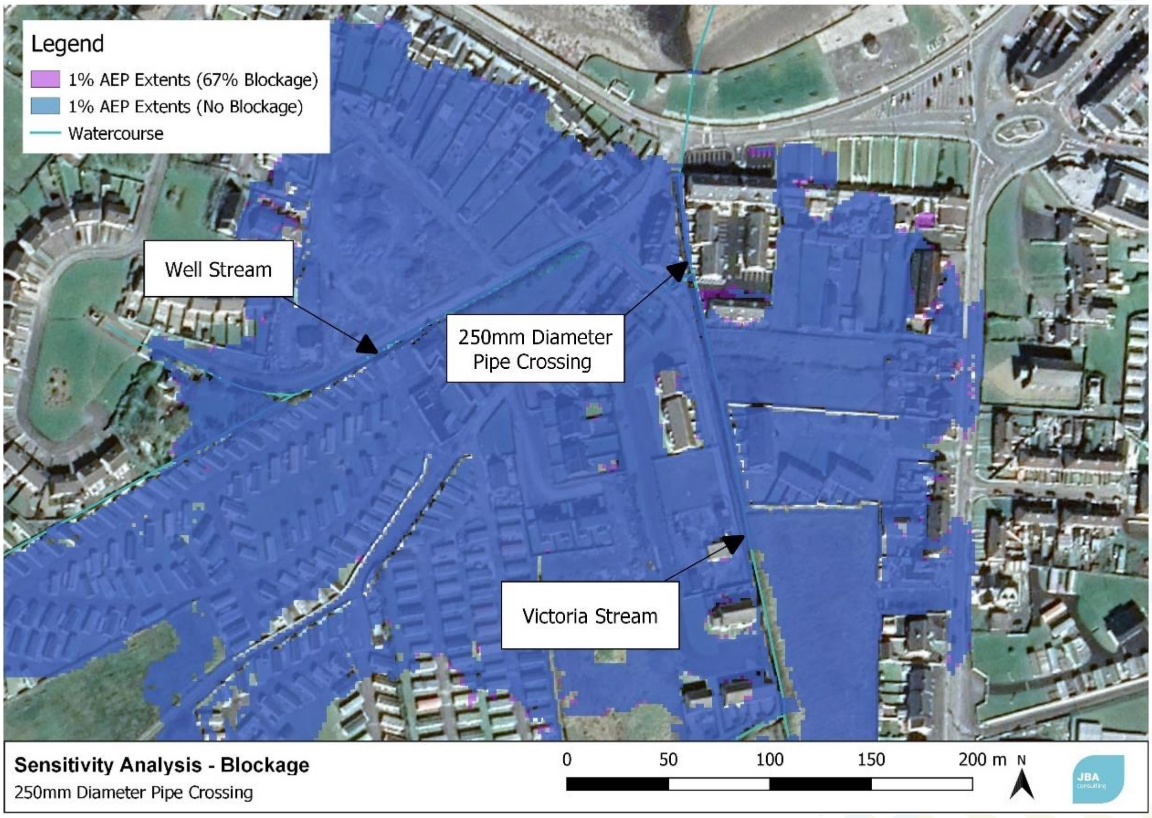


Figure 7-11: Sensitivity Analysis of blockage at the 250mm diameter Pipe Crossing

7.6 Structure Units

The units used to represent the structures in the model have been tested for a number of structures.

7.6.1 Informal Bridge on Well Stream

The crossing on the Well Stream leading to a private dwelling is represented in the model using a Flood Modeller USBPR unit. To test the sensitivity to the unit type chosen the USBPR unit was changed to an ARCH unit. There is a no increase in flood extents as shown in Figure 7-12, with water level increasing by less than 10mm at the bridge. As the change in flood levels is minor the model is not sensitive to the type of unit chosen to represent the bridge

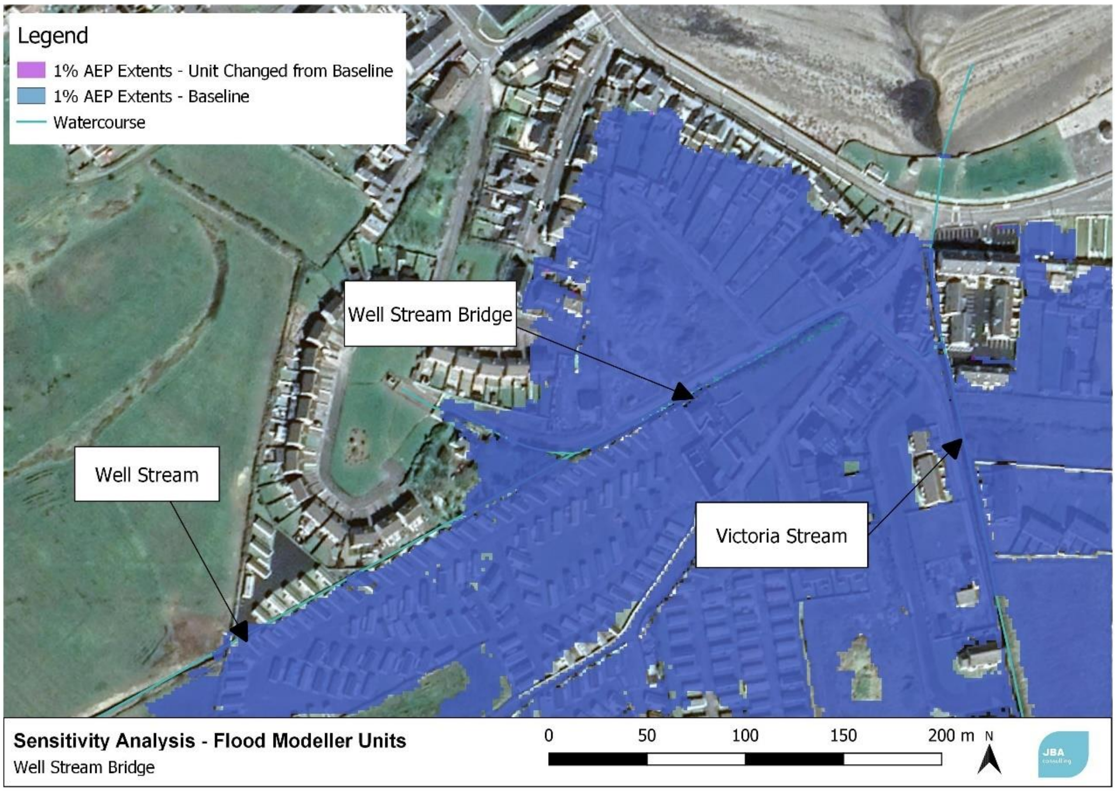


Figure 7-12: Sensitivity Testing of Structure Unit of Well Stream Bridge

8 Hydraulic system summary

From examining the modelled outputs for a number of key hydraulic features a number of factors are identified as having an impact on overall flood risk to Kilkee town and are described in the following sections

8.1 Victoria Stream

The 1% AEP fluvial combined with the 50% AEP tidal flood (i.e. Q100/T2) extents for the Victoria Stream and the 10% AEP fluvial combined with the 0.5% AEP tidal flood (i.e. Q10/T200) extents are shown in Figure 8-1 and a long section showing the maximum water level for the Q100/T2 event is shown in Figure 8-2. A number of areas of interest are highlighted and described in the sections below.

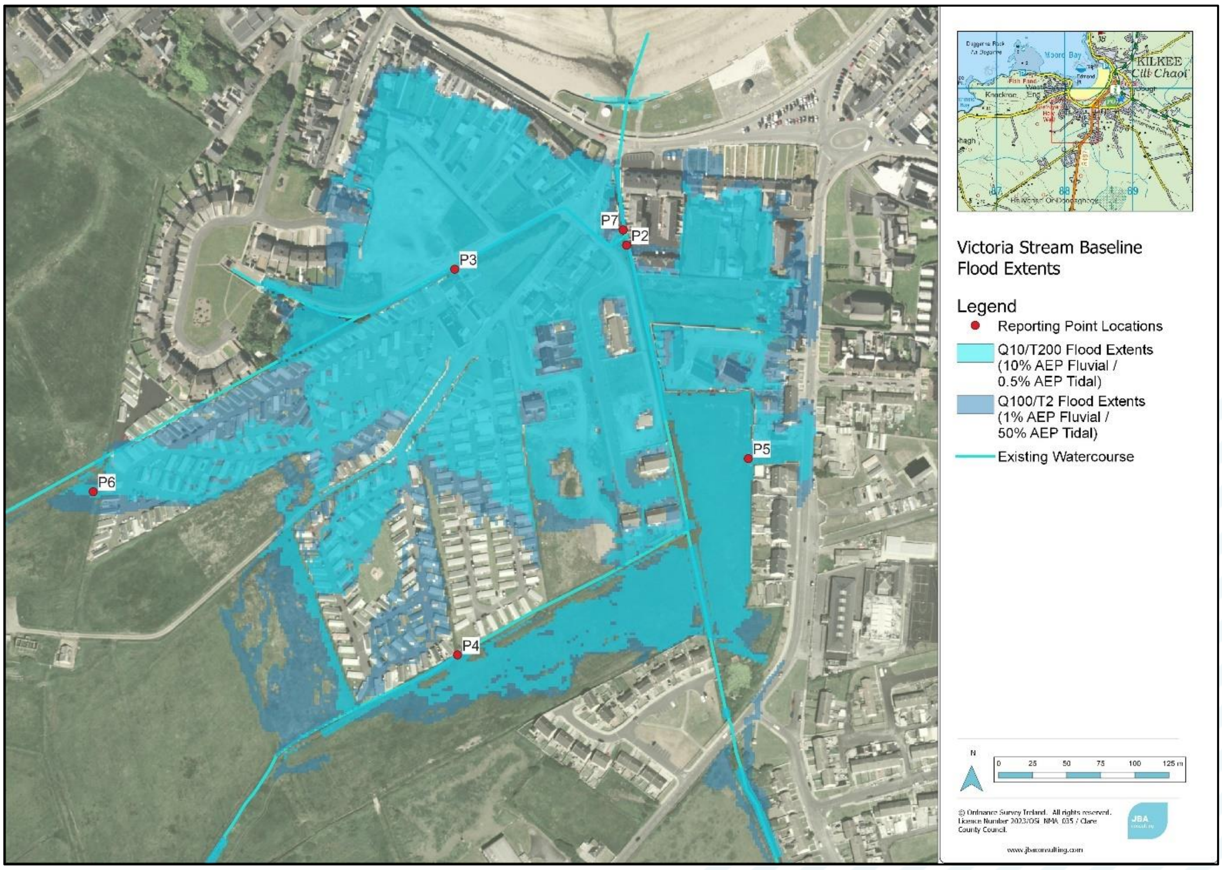


Figure 8-1: Victoria Stream Flood Extents

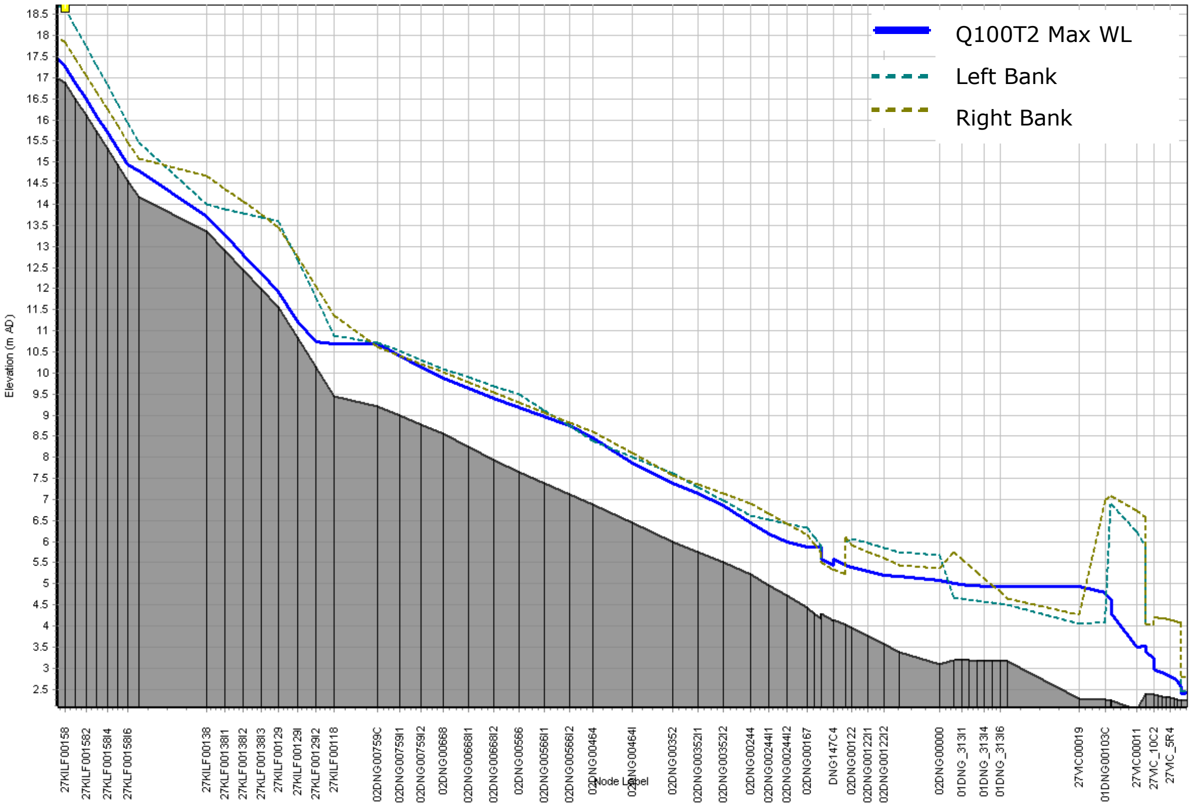


Figure 8-2: Victoria Stream 1% AEP Fluvial / 50% AEP Tidal Maximum Water Level

8.1.1 Well Stream Culvert

The main route of the Well Stream culvert follows from the 2.8m x 0.714m rectangular culvert to 3 no. DN450 culverts into a manhole chamber on Well Road and then outflows into the Victoria Stream via 3 no. DN600 culverts. This diversion through the 3 no. DN450 culverts and 3. No DN600 culverts was done by CCC as the original route of the Well Stream culvert was subject to blockage. The original Well Stream Culvert outfall into the Victoria Stream was within the Victoria Stream Outfall Culvert and was subject to blockage due to debris from tidal inflow. Inspection of this outfall point concluded that there is very little flow coming along this direction.

Flow in the Victoria Stream can back-up along the Well Stream culvert and results in a backing up of flow in the channel upstream of the culvert. The consequences of this is out-of-bank flow along the Well Stream and flooding of the Clare County Council Compound area and surrounds, Marine Road properties are also at risk and a makeshift bund has been created at the rear of the gardens to mitigate against flooding. Well Road floods and makes it difficult for access to properties in the higher ground. Properties at the junction of Well Road and Victoria Park are subject to flooding or at high risk of flooding. Refer to Figure 8-3.

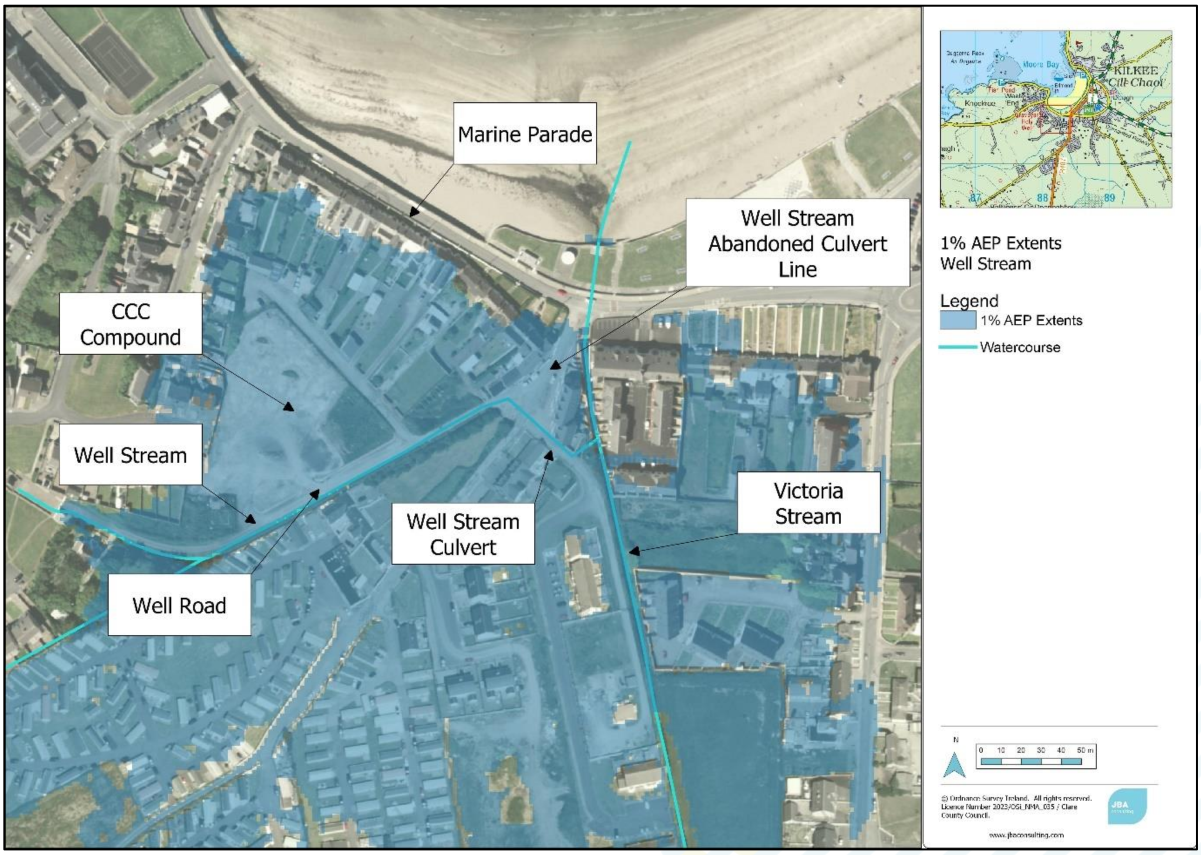


Figure 8-3: Well Stream Culvert 1% AEP Fluvial Flood Extents

8.1.2 Main Channel Flooding

Out-of-bank flow occurs on both the left and right bank of the Victoria Stream channel through the town. The out-of-bank flow on the left bank flows into the fields adjacent to the river and then onto the Carrigaholt road. The right bank flow floods the Pumping Station lands, Victoria Park and flows towards Well Road. Refer to Figure 8-4.

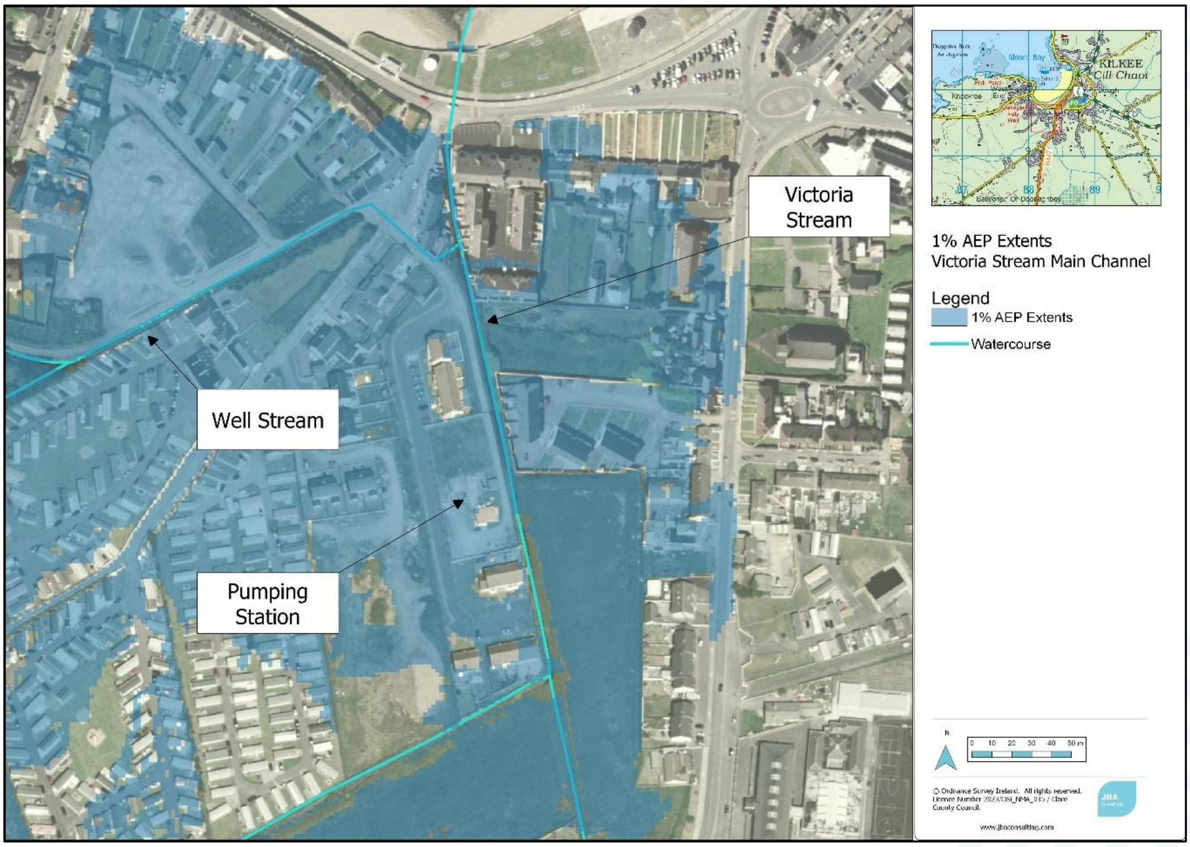


Figure 8-4: Victoria Stream Main Channel 1% AEP Fluvial Flood Extents

8.1.3 Caravan Park Area

Flow is directed along the western boundary of the caravan park onto the road due to the presence of embankments along the southern and western boundaries. The caravan park boundary walls direct flow down the road, with flow then entering the caravan parks through the openings in the walls i.e., gates. As well as flowing down the road the flow also crosses the road and enters the caravan park to the north. Refer to Figure 8-5.

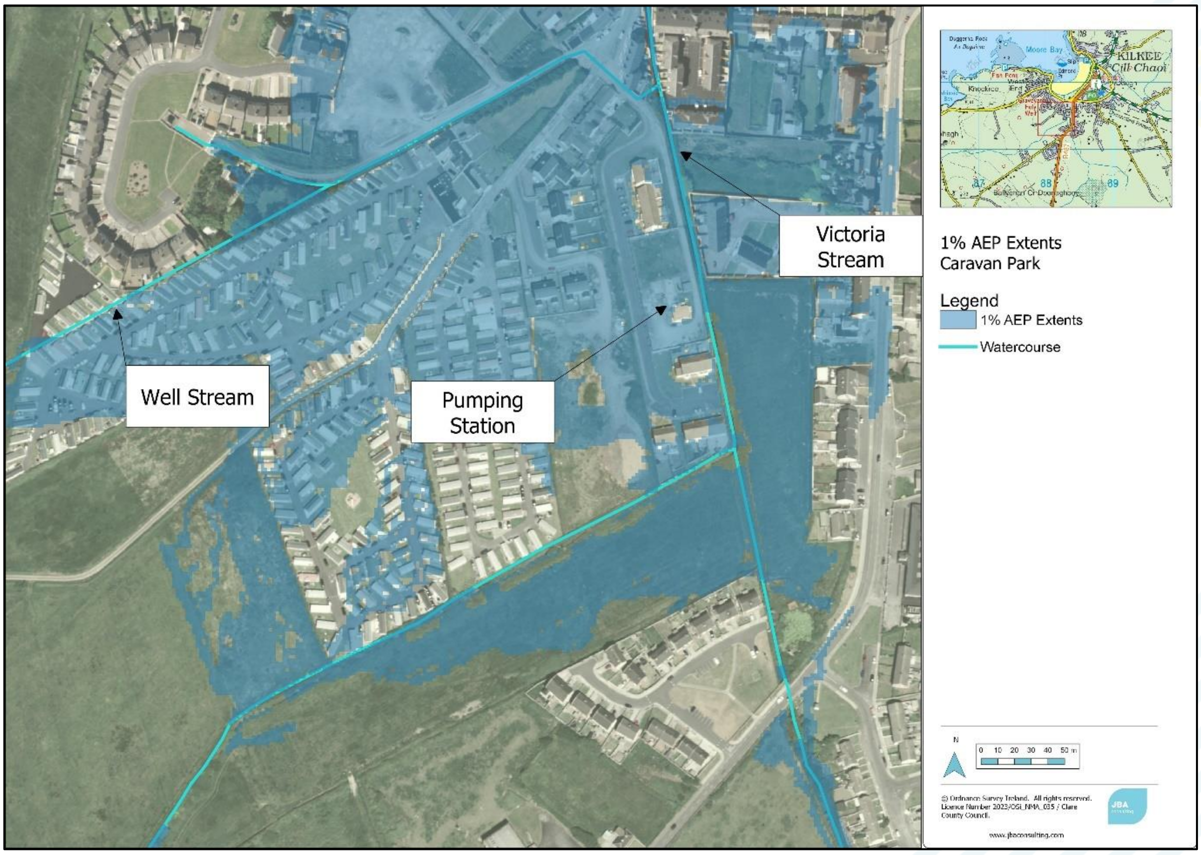


Figure 8-5: Victoria Stream Caravan Park 1% AEP Fluvial Flood Extents

8.1.4 Stop Logs

The stop logs are put in place at the downstream end of the Victoria Stream outfall culvert during the summer months (May to August) to stop flow from the stream flowing onto the beach during the bathing season and protect the Blue Flag status of the beach. Flow is allowed to backup and enters the pumping station wet well chamber and is pumped to Intrinsic Bay. At times of heavy rain, the stop logs are removed, if enough notice is given to CCC maintenance, or the stop logs are overtopped and flow discharges over the beach and into the bay.

The stop logs in place do not result in an increase in flood extents. However, it does result in an increase in water depth of approximately 196mm along the boundary walls. The long section profile in Figure 8-6 shows that the greatest effect of the stop logs is within the culvert.

Due to the impacts of the stop logs and the potential for them to not be removed, the stop logs have been included in the baseline scenario.

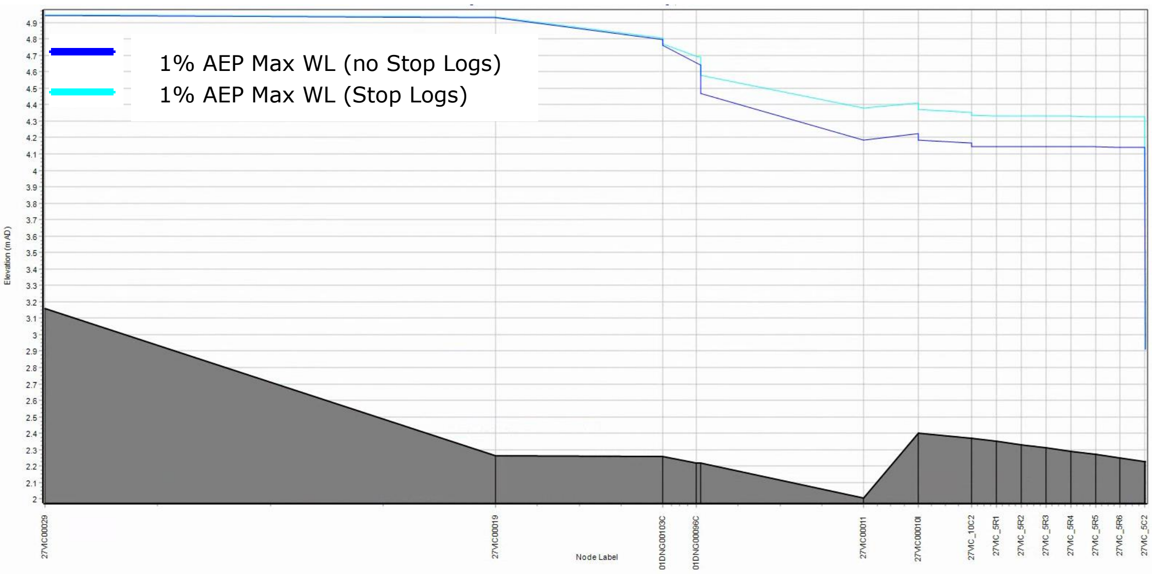


Figure 8-6: Long Section Profile of 1% AEP maximum water level with and without stop logs in place

8.2 Atlantic Stream

The 1% AEP fluvial combined with the 50% AEP tidal flood (i.e. Q100/T2) extents for the Atlantic Stream are shown in Figure 8-7 and maximum water level long section is shown in Figure 8-8. A number of areas of interest are highlighted and described in the sections below.

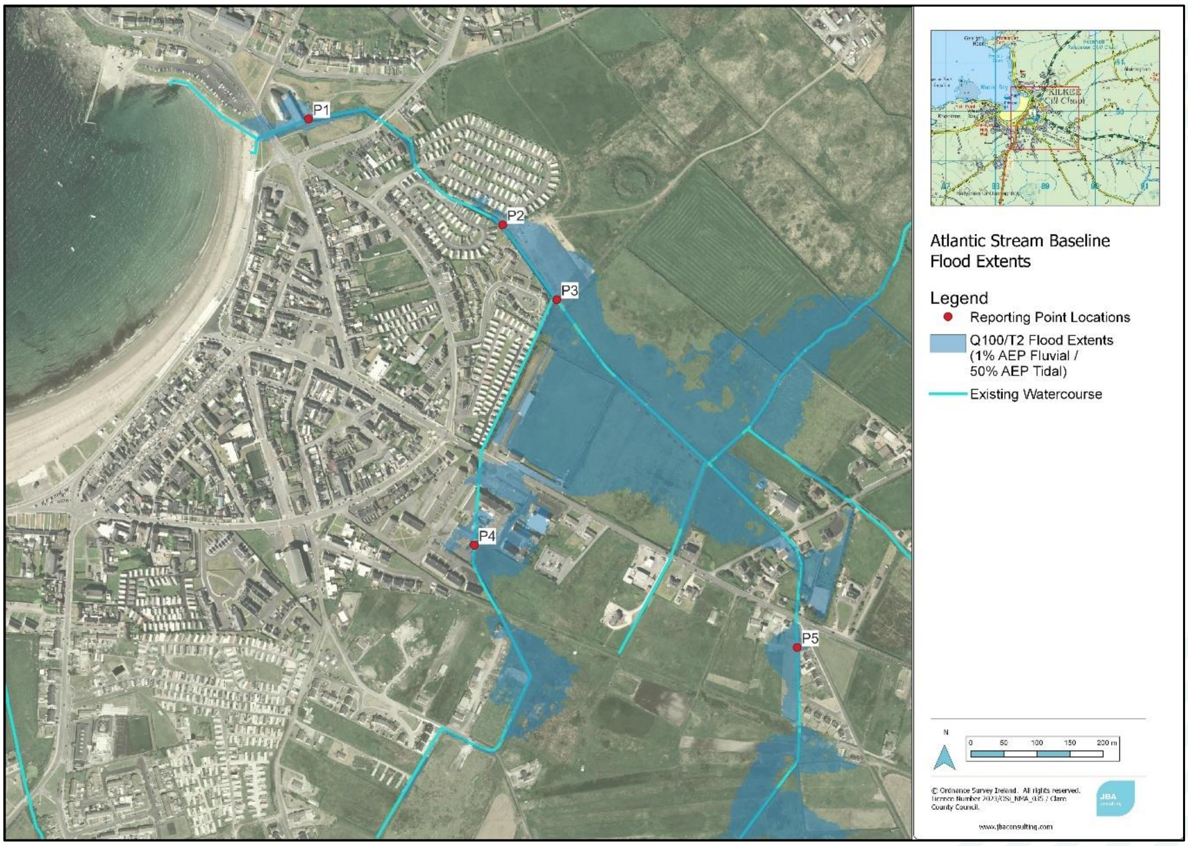


Figure 8-7: Atlantic Stream Flood Extents

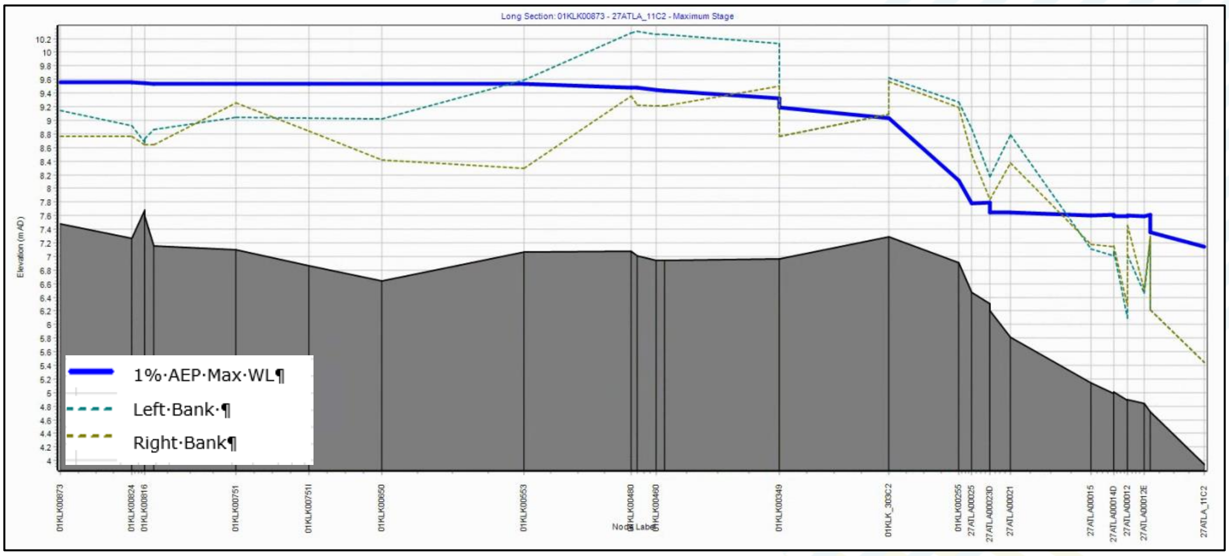


Figure 8-8: Atlantic Stream 1% AEP Maximum water level

8.2.1 Outfall Culvert

The outfall culvert has been modelled with significant blockage of both the security screen and trash screen at the culvert entrance as this has been noted by CCC as the main reason for flooding occurring. The significant blockage results in constriction of flow resulting in overland flow in and around Waterworld and flow onto the main road and cascading down the promenade steps onto the beach as has been seen in recorded events. Refer to Figure 8-11.

A FLOW model has been built of the culverted section downstream of the Atlantic Stream open channel. The Flood Modeller model ends within the outfall culvert at the point where the culvert splits into the three separate culverts. The FLOW model begins at the inlet to the culvert. There is therefore an overlap between the FLOW model and the Flood Modeller model. A flow hydrograph from the flood modeller model has been input as the upstream boundary of the FLOW model. The FLOW model then generated a stage-time hydrograph at the trifurcation point that could be input into the Flood Modeller model as the downstream boundary.

The culvert splits into three separate culverts under the road, which are all connected to an overflow chamber in the promenade from which there is a DN750 pipe outfall to Moore Bay and a DN750 overflow pipe discharging to the beach. The overflow mechanism is a simple low weir wall, as shown in Figure 8-9.

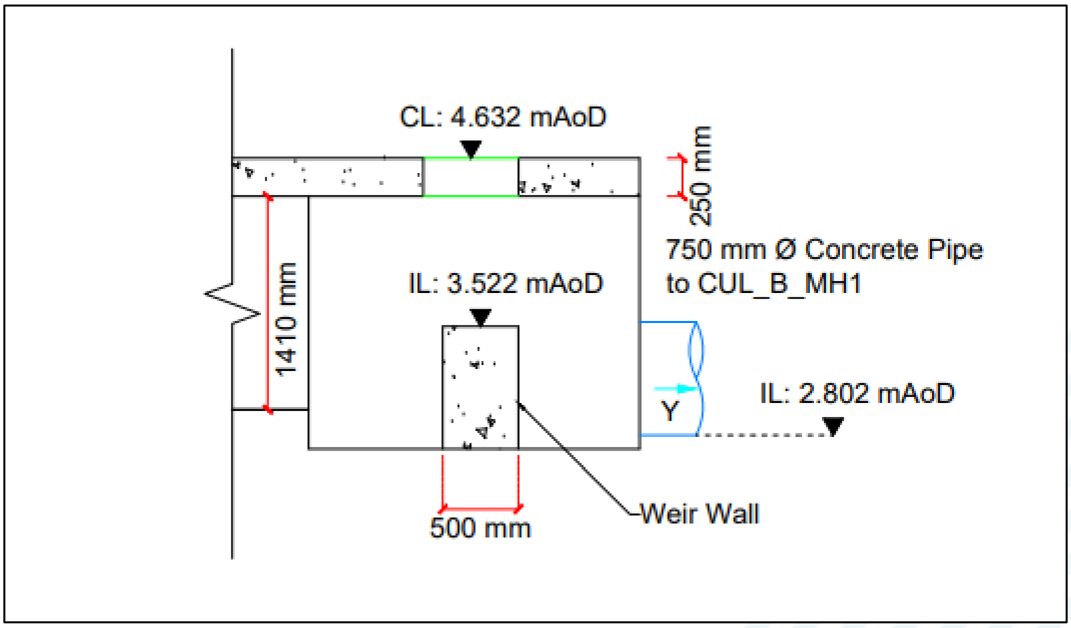


Figure 8-9: Survey of overflow chamber, weir wall and overflow pipe.

It is noted that CCC maintenance staff have created holes in the western road boundary wall near Waterworld to allow the water to dissipate rather than build up and flood the building.

The model shows that although the blockage is the main flooding mechanism, the two DN750 culverts are still undersized. FLOW results indicate that the two DN750 outlets can only take approximately two thirds of the 1% AEP peak flow which causes the system to surcharge and flood at the overflow manhole, as shown by the long section in Figure 8-10. The inlet culvert and trifurcation culverts currently have sufficient capacity to take the flow. The overflow manhole is midway down the promenade steps and c. 2m below the car park and gully level and flow cascades onto the beach. This has been corroborated by CCC maintenance staff who have cleared away the blockages from the screens only for the backed up flood water to flow down the culvert and flood out from the overflow manhole. The DN750 outfall and overflow pipes are the restrictive pipes in the culvert.

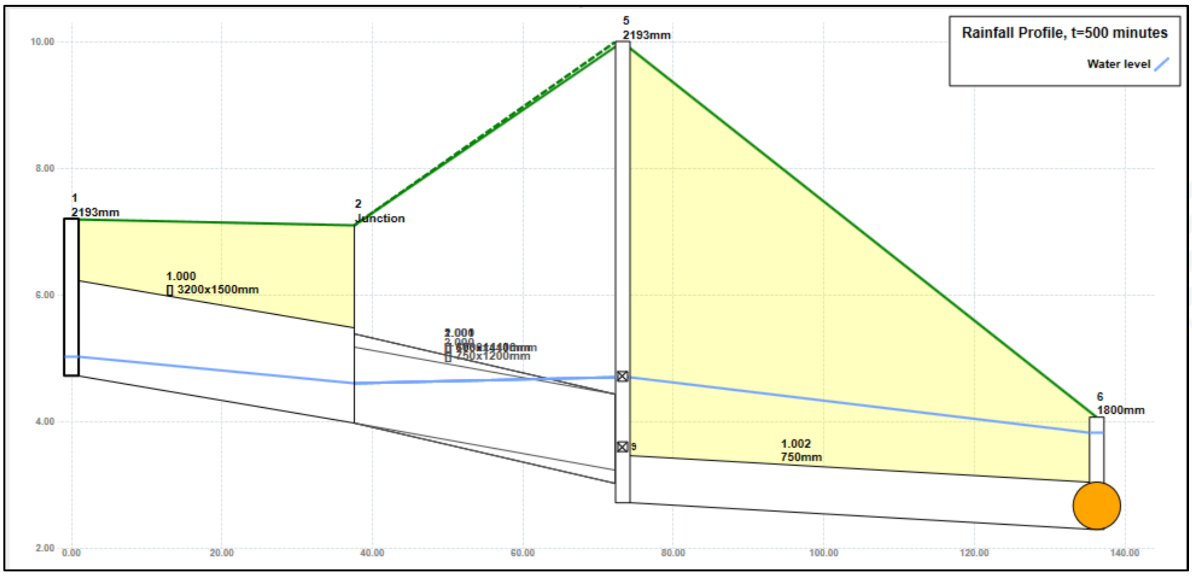


Figure 8-10: Long Section of max 1% AEP Water Level along Outfall Culvert

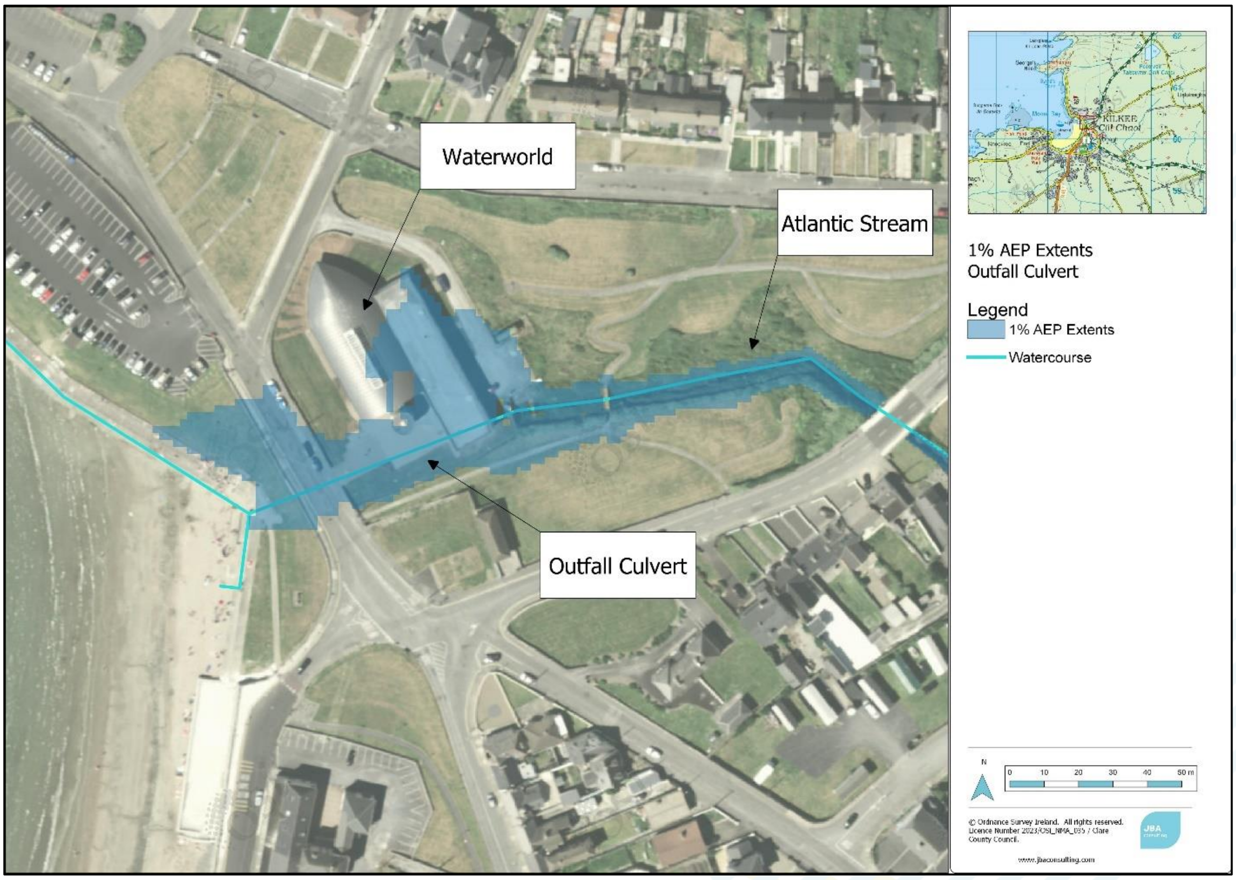


Figure 8-11: Atlantic Stream Outfall Culvert 1% AEP Flood Extents

FLOW would indicate that, if there are no blockages at the upstream security screen, surcharging of the overflow manhole would occur typically during the 50% AEP event.

8.2.2 Kilkee Bay Hotel Culvert

Significant flooding is modelled in the vicinity of the Kilkee Bay Hotel. Refer to Figure 8-12. This is a result of the hydraulic inefficiency of the 380m length, DN600 culvert which flows around the hotel and outfalls downstream of the caravan park. The high water levels downstream of the culvert also contribute to the restricting of flow through the culvert and ponding of flow upstream of the culvert. A CCTV survey of the culvert network in this area was undertaken by Clare Drains, this showed significant damage and silting in the culvert, the baseline model therefore includes increased Manning's and increased hydraulic loss at the inlet. Out of bank flow occurs at the inlet. The flood flows around the hotel building and out onto the road. There are no reported flooding incidents of the hotel.

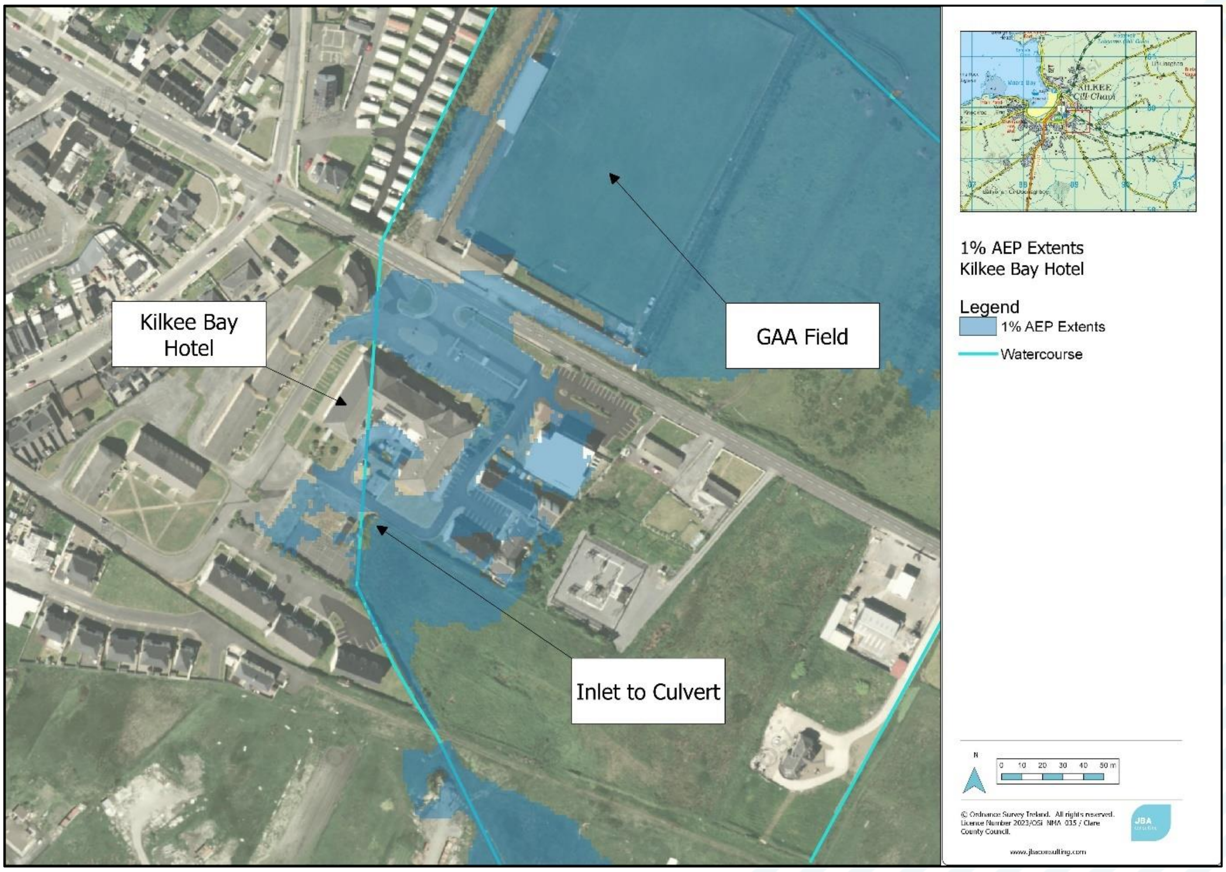


Figure 8-12: Atlantic Stream Kilkee Bay Hotel Culvert 1% AEP Flood Extents

8.2.3 GAA Field and Surrounding Area

Significant flooding is modelled at the GAA field and surrounding area. The main source is flooding is out of bank flow from the main Atlantic Stream channel, flooding from the tributary also contributes to the extents in this area. The flow is prevented from entering the adjacent caravan park due to the presence of a defence wall along the perimeter. Refer to Figure 8-13.

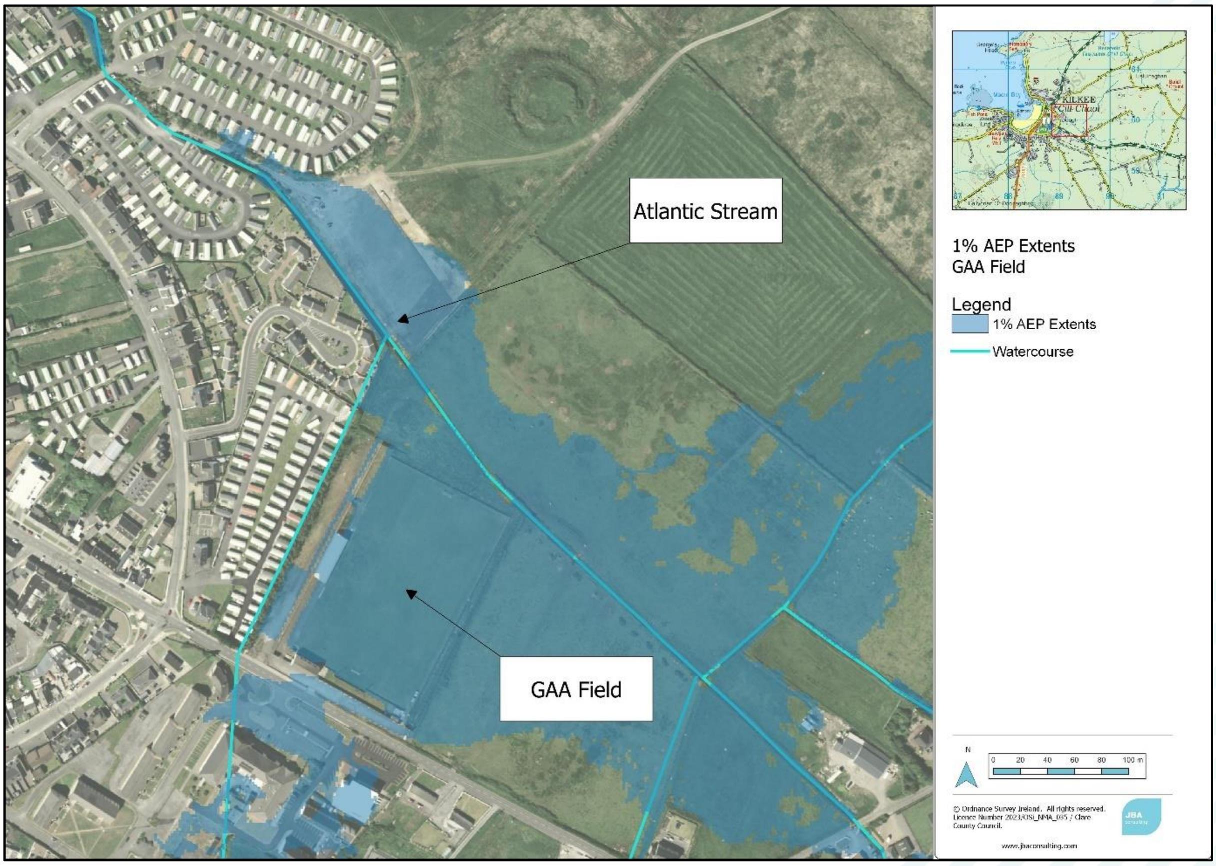


Figure 8-13: Atlantic Stream GAA Field 1% AEP Flood Extents

8.2.4 Meadow View Court

At Meadow View Court, the flood mechanism is insufficient culvert capacity from the 1200mmØ culvert. Floodwaters spill out at the upstream culvert inlet and flow in a northerly direction to the adjacent low-lying field. From this point, the model identified a number of properties that were within 300mm of the 1% AEP flood level. The cover level of an existing pump station was also identified as being lower than the modelled flood level. Note, that this model assumed that when floodwaters spill into this landbank, there is no means for it to return to the culvert from the field itself. The flood mechanism is illustrated in Figure 8-14.

On further review of this area, however, there was a number of informal connections identified that had not been included in the model output that is shown below. Therefore, there is an intention as part of the scheme to formalise these outfalls from the field into the culvert. This is detailed further in the Options Report.

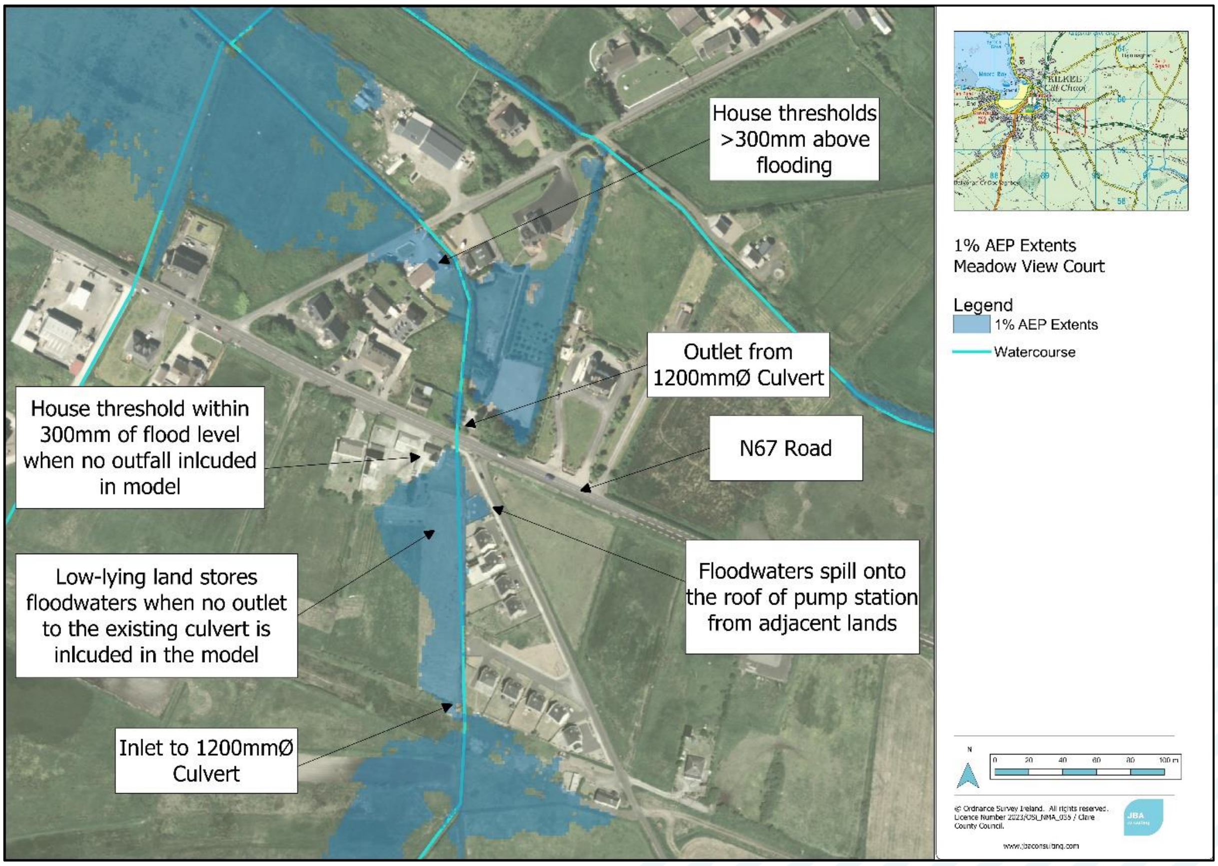


Figure 8-14: Atlantic Stream Meadow View Court Culvert 1% AEP Flood Extents

8.3 Climate Change

Impacts of climate change have been considered in the modelling process. The climate change factors applied for the Mid-Range Future Scenario (MRFS) and High-End Future Scenario (HEFS) are discussed in the corresponding Hydrology Report. Figure 8-15 and Figure 8-16 compare the fluvial 1% AEP flood extents for the current, MRFS and HEFS scenarios for the Victoria and Atlantic Streams, respectively. As expected, there is an increase in flooding with an increase in climate change severity. It is noted the modelled scenarios for the Victoria Model include the stop logs in place.

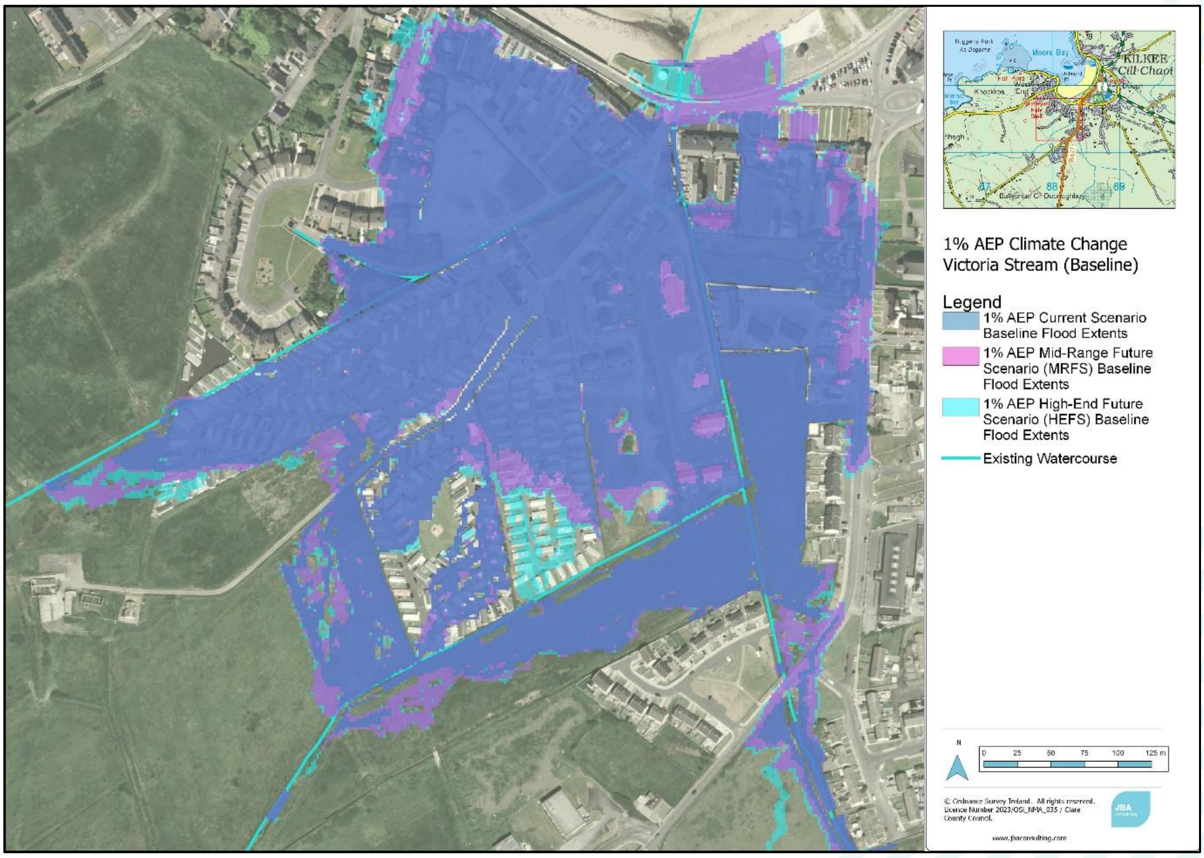


Figure 8-15: Fluvial 1% AEP / Tidal 50% AEP Climate Change Victoria Model

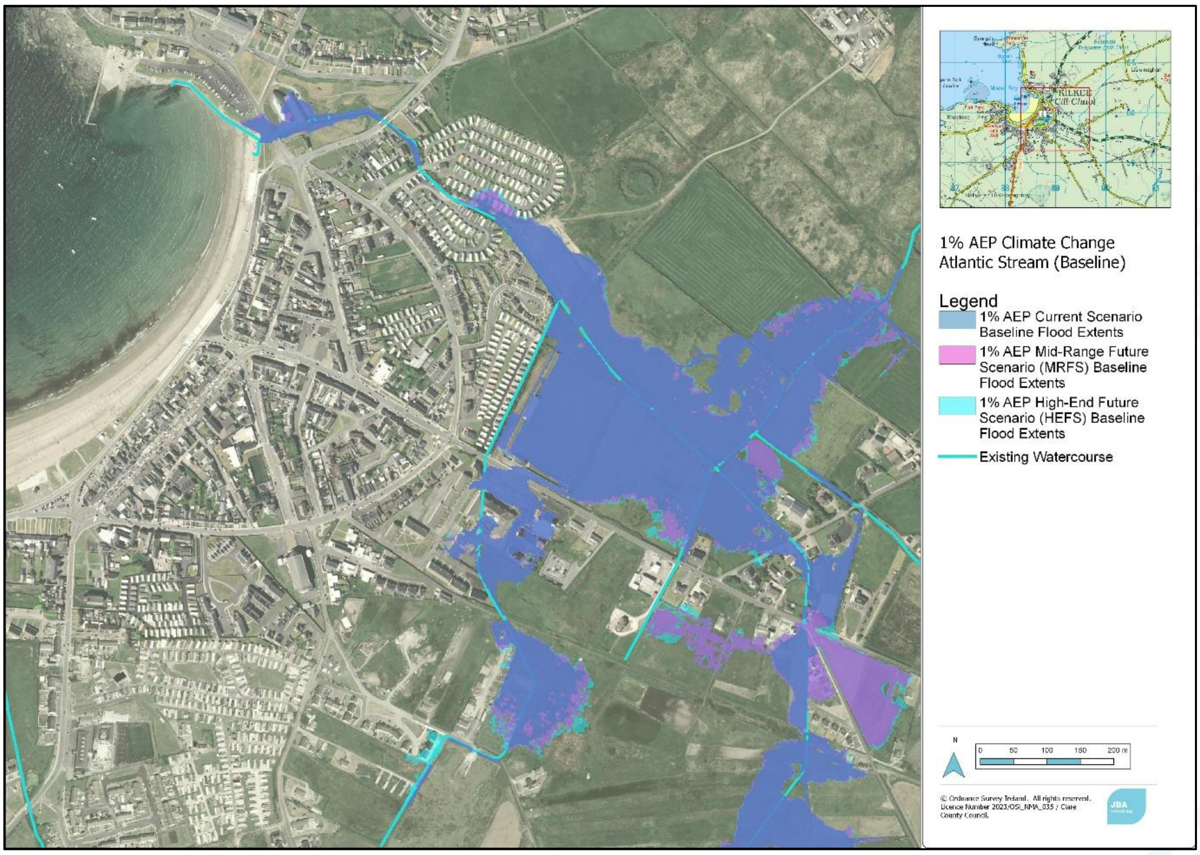


Figure 8-16: Fluvial 1% AEP Climate Change Atlantic Model

8.4 Storm water connection

There is additional risk as a result of the previously described storm water connection in Section 2.6. In the location shown in Figure 8-17 below the ground levels along the storm water pipe is less than the water level in the river at the point of connection. This may result in backing up of water in the pipe and increased flooding along the road.

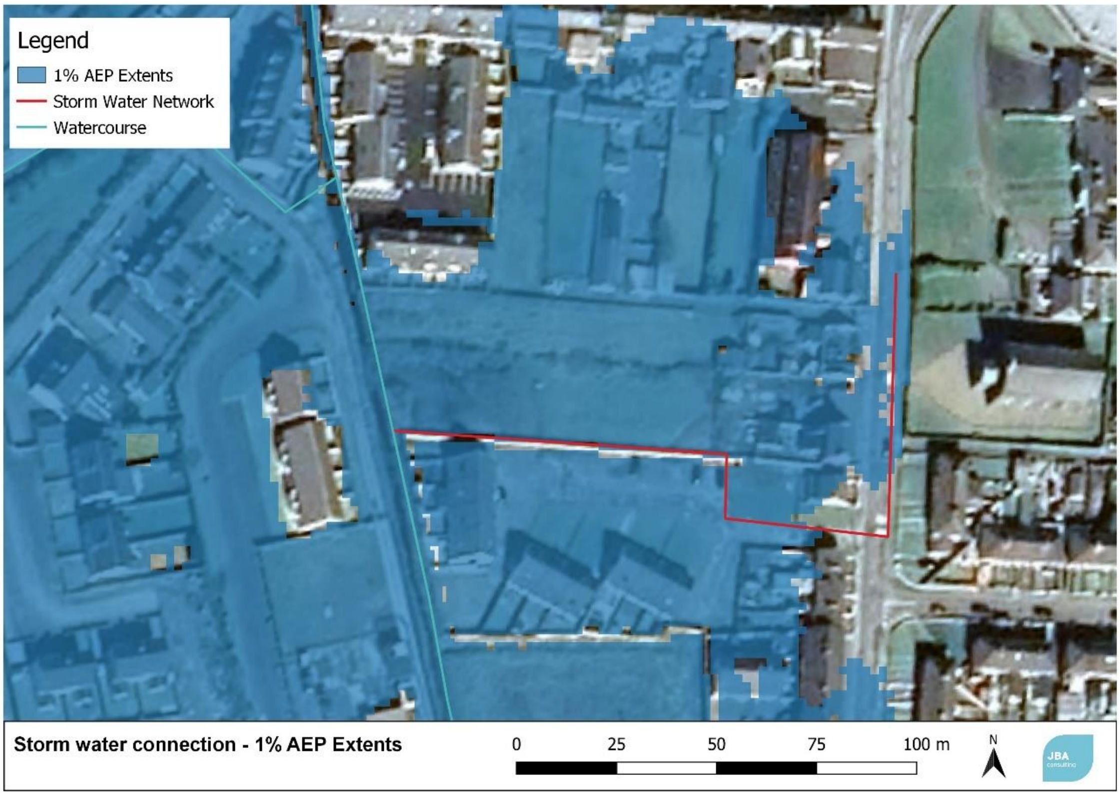


Figure 8-17: Stormwater connection and 1% AEP Flood Extents

8.5 Validation

A detailed calibration/validation cannot be undertaken as no storm has been recorded since the installation of the water level gauges in the Victoria and Atlantic Streams. However, a comparison can be completed with the estimated recorded extents from previous events. Figure 8-18 compares the 50% AEP and 10% AEP modelled flood extents. The historic flooding extents in the figure below have been developed from photographs and local knowledge of previous flood events. The extents match reasonably well with the approximate recorded historic flooding. The modelled extents are more extensive that what has been reported in the past particularly along the upstream of the Atlantic Stream.

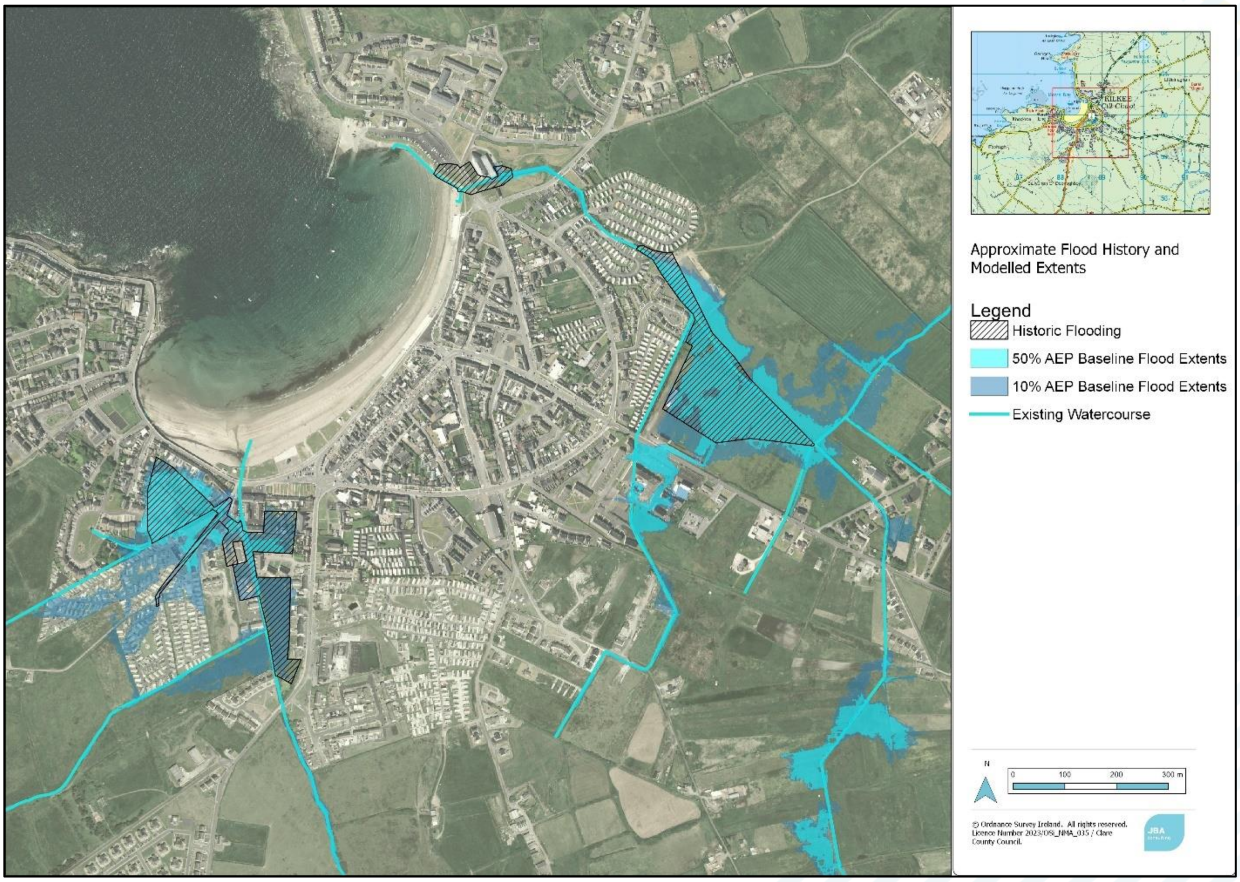


Figure 8-18: Approximate Flood History compared to 50% and 10% AEP extents

9 Summary

This hydraulic model report describes the methods and steps carried out to develop two linked 1D-2D hydraulic model for the Kilkee area. 9 watercourses have been modelled. A review of the available data and flood history for the area has been carried out and used to construct the new hydraulic model.

Flood Modeller Pro and TUFLOW software packages were used to build the 1D and 2D model components, respectively. All open channel watercourse and structures were modelled in 1D and details of each structure and its representation recorded. TUFLOW was used to model the floodplain and wider area.

The hydraulic programme FLOW has been used to model the culverted outfall from the Atlantic Stream.

Limitations of the modelling method have been highlighted, in particular the sections of the model which have been kept as 1D only.

A range of sensitivity tests were carried out. A number of hydraulic constraints and features have been identified.

Both hydraulic models are replicating observed flooding extents and depths that give confidence in using these models for assessment of the Flood Relief Scheme options.

A Appendix A – Hydraulic Results

A.1 Victoria Stream – Baseline Model

Reporting Locations - Victoria Stream

A.1.1 Water Level (mOD)

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27KILF00198	20.496	20.597	20.645	20.645	20.687	20.741	20.78	20.933
27KILF00158	17.088	17.145	17.171	17.171	17.199	17.236	17.259	17.365
02DNG00759	10.15	10.384	10.469	10.469	10.548	10.634	10.685	10.833
02DNG00352	7.055	7.197	7.249	7.249	7.298	7.35	7.386	7.523
02DNG00147	4.988	5.25	5.376	5.383	5.528	5.713	5.87	6.109
01DNG00332	4.54	4.787	4.875	4.928	4.96	5.033	5.087	5.298
27VIC00029	4.244	4.473	4.608	4.789	4.712	4.839	4.947	5.273
27VIC00019	4.061	4.303	4.488	4.767	4.646	4.812	4.935	5.272
01DNG00096	3.802	4.061	4.211	4.687	4.381	4.529	4.688	5.06
27VIC00010I	3.167	3.54	3.775	4.643	4.042	4.175	4.411	4.855
27VIC00005J	2.978	3.42	3.676	4.6	3.95	4.054	4.09	4.439
01FHG01100	27.316	27.471	27.571	27.571	27.635	27.671	27.69	27.771
01FHG00493	10.086	10.196	10.25	10.25	10.308	10.38	10.431	10.673
01FHG00184	6.457	6.608	6.69	6.69	6.732	6.774	6.801	6.872
01FHG00000	4.564	4.793	4.879	4.93	4.963	5.036	5.089	5.299
27KILG00120	36.838	36.968	37.027	37.027	37.092	37.167	37.223	37.457
27KILG00060	18.049	18.194	18.253	18.253	18.305	18.372	18.413	18.518
27KILG00001	10.15	10.384	10.469	10.469	10.548	10.634	10.685	10.833
27VTB100120	25.794	25.904	25.96	25.96	26.015	26.083	26.133	26.324
27VTB100065	11.191	11.279	11.32	11.32	11.361	11.412	11.451	11.59
27VTB100044I	6.98	7.19	7.281	7.281	7.358	7.443	7.517	7.626
27VTB100030J	4.41	4.555	4.628	4.8	4.722	4.856	4.962	5.278
27VTB100022	4.262	4.446	4.535	4.77	4.653	4.817	4.938	5.273
27VTB100016	4.223	4.439	4.531	4.769	4.652	4.817	4.937	5.273
27VTB100006I	4.189	4.434	4.528	4.769	4.65	4.816	4.937	5.334

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27VTB1A00006	4.264	4.446	4.535	4.77	4.653	4.817	4.938	5.28
27VTB1A00001	4.262	4.446	4.535	4.77	4.653	4.817	4.938	5.273

A.1.2 Flow (m^3/s)

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27KILF00198	0.285	0.421	0.495	0.495	0.578	0.689	0.773	1.169
27KILF00158	0.284	0.42	0.494	0.494	0.577	0.688	0.771	1.166
02DNG00759	1.553	2.294	2.7	2.7	3.148	3.75	4.192	6.338
02DNG00352	1.689	2.496	2.934	2.934	3.421	4.058	4.547	6.601
02DNG00147	1.773	2.611	3.07	3.07	3.589	4.239	4.756	5.207
01DNG00332	3.405	4.948	5.567	5.517	6.254	6.994	7.396	7.988
27VIC00029	3.419	4.964	5.503	5.426	5.907	6.237	6.372	6.713
27VIC00019	3.472	4.728	5.171	5.002	5.112	5.289	5.379	5.546
01DNG00096	3.954	5.249	6.325	7.602	7.314	8.501	9.417	12.05
27VIC00010I	4.061	5.369	6.431	7.931	7.432	8.62	9.544	12.312
27VIC00005J	4.194	5.636	6.463	8.011	7.518	8.62	9.544	13.35
01FHG01100	0.858	1.268	1.491	1.491	1.739	2.074	2.325	3.515
01FHG00493	1.182	1.745	2.048	2.048	2.393	2.851	3.204	4.847
01FHG00184	1.432	2.098	2.426	2.426	2.781	3.185	3.438	4.206
01FHG00000	1.588	2.3	2.587	2.586	2.937	3.35	3.614	4.217
27KILG00120	1.273	1.882	2.212	2.212	2.579	3.073	3.444	5.196
27KILG00060	1.271	1.88	2.209	2.209	2.575	3.067	3.436	5.184
27KILG00001	1.269	1.878	2.207	2.207	2.573	3.063	3.434	5.187
27VTB100120	0.78	1.153	1.356	1.356	1.582	1.887	2.115	3.198

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27VTB100065	0.779	1.151	1.354	1.354	1.58	1.884	2.111	3.178
27VTB100044I	0.778	1.15	1.329	1.328	1.486	1.635	1.668	1.769
27VTB100030J	0.778	1.15	1.329	1.329	1.486	1.626	1.667	1.767
27VTB100022	0.808	1.153	1.263	1.269	1.351	1.409	1.444	1.713
27VTB100016	0.813	1.013	1.074	1.079	1.107	1.142	1.164	1.202
27VTB100006I	0.671	0.691	0.626	0.524	0.506	0.505	0.504	0.503
27VTB1A00006	0.045	0.066	0.078	0.078	0.091	0.108	0.122	0.184
27VTB1A00001	0.041	0.064	0.076	0.076	0.088	0.102	0.111	0.323

A.1.3 Velocity (m^2/s)

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27KILF00198	0.739	0.823	0.853	0.853	0.897	0.944	0.972	1.089
27KILF00158	1.034	1.197	1.274	1.274	1.346	1.428	1.493	1.711
02DNG00759	0.953	0.962	0.962	0.962	0.962	0.975	1.012	1.215
02DNG00352	0.732	0.734	0.74	0.74	0.768	0.812	0.845	0.907
02DNG00147	0.586	0.585	0.578	0.567	0.567	0.566	0.565	0.563
01DNG00332	0.885	1.013	1.061	1.057	1.11	1.164	1.204	1.254
27VIC00029	0.84	0.882	0.919	0.919	0.95	0.976	0.984	1.012
27VIC00019	0.601	0.676	0.666	0.671	0.666	0.663	0.661	0.66
01DNG00096	1.103	1.249	1.356	1.451	1.444	1.541	1.614	1.803
27VIC00010I	1.427	1.55	1.636	1.671	1.704	1.777	1.828	1.959
27VIC00005J	3.914	4.42	4.873	4.83	5.214	5.481	5.799	4.633
01FHG01100	1.334	1.492	1.506	1.506	1.505	1.5	1.506	1.506
01FHG00493	0.959	1.083	1.135	1.135	1.18	1.228	1.261	1.35

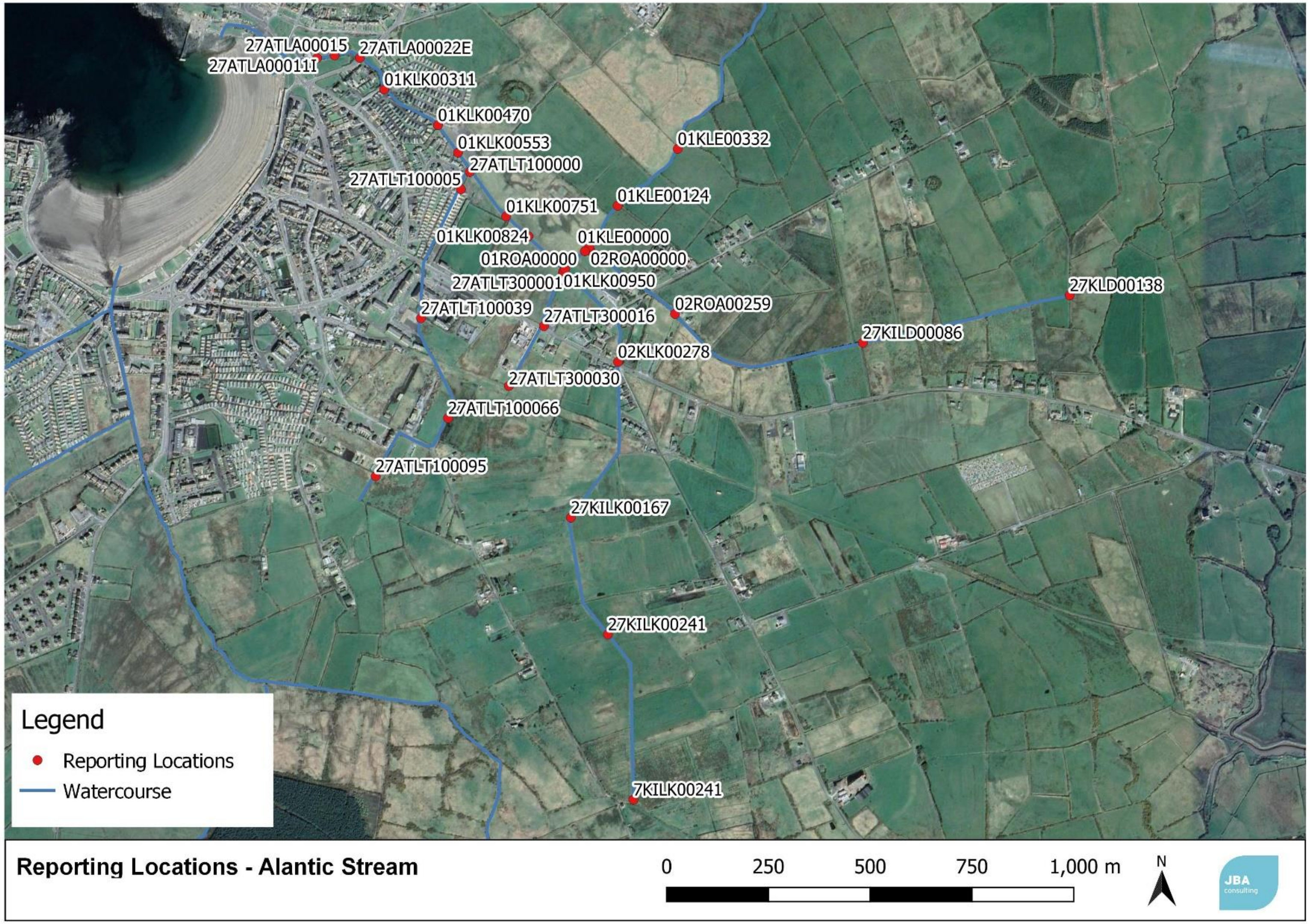
Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
01FHG00184	0.988	1.08	1.089	1.089	1.111	1.171	1.204	1.3
01FHG00000	0.966	0.982	0.985	0.985	0.987	1.001	1.007	1.022
27KILG00120	1.221	1.371	1.438	1.438	1.499	1.576	1.625	1.753
27KILG00060	0.714	0.744	0.745	0.745	0.745	0.748	0.751	0.793
27KILG00001	1.095	1.096	1.095	1.095	1.095	1.112	1.132	1.154
27VTB100120	0.955	1.074	1.119	1.119	1.169	1.223	1.259	1.401
27VTB100065	1.176	1.39	1.488	1.488	1.581	1.682	1.744	1.937
27VTB100044I	0.499	0.551	0.572	0.572	0.588	0.6	0.599	0.598
27VTB100030J	0.633	0.75	0.797	0.792	0.834	0.869	0.88	0.9
27VTB100022	0.377	0.426	0.46	0.462	0.48	0.499	0.51	0.529
27VTB100016	0.384	0.456	0.476	0.479	0.489	0.502	0.51	0.521
27VTB100006I	0.261	0.258	0.259	0.259	0.256	0.251	0.252	0.232
27VTB1A00006	0.411	0.425	0.43	0.43	0.438	0.445	0.444	0.475
27VTB1A00001	0.029	0.03	0.032	0.032	0.036	0.042	0.045	0.057

A.1.4 Froude Number

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27KILF00198	0.416	0.43	0.431	0.431	0.442	0.452	0.455	0.466
27KILF00158	0.719	0.757	0.775	0.775	0.789	0.802	0.816	0.842
02DNG00759	0.376	0.411	0.414	0.414	0.413	0.413	0.419	0.449
02DNG00352	0.408	0.418	0.414	0.414	0.411	0.412	0.411	0.418
02DNG00147	0.329	0.324	0.322	0.322	0.318	0.313	0.31	0.295
01DNG00332	0.288	0.309	0.318	0.317	0.328	0.338	0.345	0.367
27VIC00029	0.448	0.445	0.443	0.443	0.438	0.431	0.427	0.398

Reporting Location	50% AEP	20% AEP	10% AEP	10% Tidal with 0.5% Fluvial	5% AEP	2% AEP	1% AEP	0.1% AEP
27VIC00019	0.178	0.186	0.184	0.187	0.183	0.183	0.183	0.182
01DNG00096	0.297	0.314	0.326	0.335	0.335	0.343	0.349	0.365
27VIC00010I	0.639	0.77	0.869	0.999	0.957	1.056	1.13	1.339
27VIC00005J	3.119	3.305	3.579	3.548	3.782	3.925	4.122	2.886
01FHG01100	0.603	0.609	0.877	0.877	0.877	0.879	0.878	0.875
01FHG00493	0.502	0.514	0.519	0.519	0.524	0.53	0.535	0.595
01FHG00184	0.457	0.47	0.494	0.494	0.506	0.517	0.522	0.539
01FHG00000	0.501	0.509	0.51	0.51	0.512	0.52	0.522	0.518
27KILG00120	0.595	0.611	0.619	0.619	0.623	0.631	0.635	0.688
27KILG00060	0.38	0.403	0.408	0.408	0.407	0.407	0.408	0.496
27KILG00001	0.563	0.558	0.557	0.557	0.556	0.557	0.576	0.603
27VTB100120	0.503	0.514	0.517	0.517	0.523	0.526	0.529	0.544
27VTB100065	0.611	0.688	0.721	0.721	0.752	0.783	0.801	0.938
27VTB100044I	0.255	0.222	0.216	0.216	0.212	0.209	0.208	0.205
27VTB100030J	0.283	0.291	0.302	0.302	0.309	0.318	0.321	0.327
27VTB100022	0.182	0.182	0.189	0.19	0.196	0.203	0.207	0.213
27VTB100016	0.162	0.162	0.163	0.163	0.163	0.162	0.164	0.167
27VTB100006I	0.157	0.156	0.155	0.155	0.145	0.13	0.124	0.094
27VTB1A00006	0.413	0.42	0.423	0.423	0.426	0.427	0.426	0.447
27VTB1A00001	0.014	0.014	0.015	0.015	0.015	0.016	0.016	0.019

A.2 Atlantic Stream – Baseline Model



A.2.1 Water Level (mOD)

Reporting Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.1% AEP
02KLK00278	10.634	10.798	10.89	10.958	11.041	11.052	11.093
01KLK00950	9.136	9.211	9.284	9.38	9.497	9.568	9.865
01KLK00824	9.021	9.159	9.255	9.357	9.485	9.559	9.861
01KLK00751	8.987	9.145	9.239	9.342	9.465	9.541	9.848
01KLK00553	8.975	9.137	9.233	9.338	9.462	9.537	9.847
02ROA00259	10.528	10.588	10.631	10.67	10.726	10.766	10.947
02ROA00000	9.385	9.488	9.534	9.572	9.62	9.662	9.871
01ROA00085	9.385	9.488	9.534	9.572	9.62	9.662	9.871
01ROA00000	9.132	9.209	9.281	9.378	9.493	9.561	9.853
01KLE00332	11.348	11.531	11.598	11.667	11.747	11.801	11.96
01KLE00124	10.069	10.183	10.227	10.254	10.279	10.294	10.327
01KLE00000	9.385	9.488	9.534	9.572	9.62	9.662	9.871
27ATLA00022E	7.763	7.755	7.455	7.515	7.592	7.645	7.784
27ATLA00015	7.637	7.601	7.424	7.482	7.557	7.608	7.737
27ATLA00011I	7.225	7.273	7.416	7.487	7.578	7.628	7.76
27ATLT300030	10.486	10.978	11.293	11.589	11.862	11.876	11.904
27ATLT300016	9.626	9.683	9.725	9.795	9.888	9.96	10.169
27ATLT300001	9.136	9.211	9.283	9.38	9.498	9.568	9.866
27ATLT100095	11.498	11.56	11.603	11.641	11.692	11.735	11.91
27ATLT100066	10.551	10.696	10.786	10.877	11.007	11.096	11.322
27ATLT100039	10.284	10.326	10.345	10.36	10.376	10.384	10.415
27ATLT100005	8.978	9.138	9.234	9.339	9.465	9.539	9.869
27ATLT100000	8.977	9.138	9.233	9.338	9.462	9.538	9.848
27KILK00241	17.99	17.991	17.992	17.993	18.047	18.11	18.4
27KILK00198	14.807	14.862	14.895	14.928	14.97	14.992	15.053
27KILK00167	13.177	13.296	13.342	13.385	13.438	13.472	13.566
27KILD00138	13.813	13.898	13.945	13.991	14.048	14.08	14.194
27KILD00086	11.681	11.686	11.727	11.765	11.82	11.859	12.036

A.2.2 Flow (m^3/s)

Reporting Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.1% AEP
02KLK00278	1.117	1.567	1.845	2.119	2.428	2.471	2.538
01KLK00950	2.082	2.199	2.263	2.442	2.816	2.959	3.833
01KLK00824	1.809	1.849	1.868	1.867	1.858	1.848	1.841
01KLK00751	2.427	2.537	2.583	2.587	2.573	2.562	2.591
01KLK00553	1.803	1.791	1.367	1.323	1.274	1.254	1.218
02ROA00259	0.572	0.716	0.824	0.937	1.092	1.208	1.715
02ROA00000	0.577	0.726	0.913	1.258	1.712	2.043	2.64
01ROA00085	1.812	2.343	2.661	2.95	3.284	3.553	4.491
01ROA00000	1.469	1.671	1.772	1.844	1.901	1.929	2.022
01KLE00332	0.931	1.255	1.477	1.723	2.054	2.302	3.476
01KLE00124	1.312	1.621	1.77	1.875	1.969	2.015	2.107
01KLE00000	1.398	1.743	1.825	1.849	1.862	1.868	1.876
27ATLA00022E	4.641	4.852	4.957	5.504	6.219	6.688	8.305
27ATLA00015	11.787	12.363	5.67	6.225	6.886	7.304	8.764
27ATLA00011I	11.755	11.575	6.37	6.773	7.269	7.593	8.871
27ATLT300030	0.168	0.226	0.265	0.309	0.369	0.413	0.622
27ATLT300016	0.15	0.186	0.205	0.222	0.311	0.403	0.623
27ATLT300001	0.144	0.143	0.145	0.564	1.903	1.978	2.148
27ATLT100095	0.001	0.001	0.001	0.001	0.001	0.001	0.001
27ATLT100066	0.426	0.574	0.675	0.829	1.079	1.268	1.556
27ATLT100039	0.242	0.244	0.244	0.244	0.244	0.244	0.266
27ATLT100005	0.242	0.244	0.244	0.244	0.244	0.244	0.243
27ATLT100000	0.4	0.4	0.403	0.402	0.401	0.4	0.431
27KILK00241	0.001	0.001	0.001	0.001	0.001	0.001	0.001
27KILK00198	0.91	0.934	0.937	0.942	0.946	0.947	0.957
27KILK00167	1.034	1.337	1.522	1.712	1.887	1.96	2.199
27KILD00138	0.594	0.797	0.936	1.091	1.299	1.454	2.192
27KILD00086	0.593	0.796	0.931	1.083	1.288	1.44	2.176

A.2.3 Velocity (m²/s)

Reporting Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.1% AEP
02KLK00278	0.574	0.572	0.569	0.568	0.574	0.576	0.579
01KLK00950	0.658	0.652	0.654	0.652	0.652	0.651	0.643
01KLK00824	0.456	0.455	0.45	0.444	0.438	0.435	0.429
01KLK00751	0.617	0.635	0.647	0.647	0.641	0.636	0.635
01KLK00553	0.319	0.306	0.253	0.252	0.251	0.251	0.247
02ROA00259	0.555	0.581	0.594	0.608	0.62	0.626	0.629
02ROA00000	0.193	0.235	0.257	0.287	0.374	0.43	0.495
01ROA00085	0.514	0.59	0.634	0.673	0.716	0.749	0.848
01ROA00000	0.634	0.687	0.711	0.727	0.737	0.744	0.761
01KLE00332	0.6	0.677	0.72	0.787	0.871	0.928	1.147
01KLE00124	0.489	0.547	0.576	0.597	0.614	0.623	0.637
01KLE00000	0.565	0.6	0.618	0.663	0.713	0.748	0.849
27ATLA00022E	1.982	1.982	2.284	2.284	2.284	2.284	2.287
27ATLA00015	1.585	1.758	1.362	1.361	1.354	1.339	1.284
27ATLA00011I	1.059	1.05	0.851	0.851	0.851	0.851	0.83
27ATLT300030	0.243	0.289	0.318	0.347	0.382	0.403	0.507
27ATLT300016	0.42	0.483	0.519	0.548	0.614	0.661	0.808
27ATLT300001	0.241	0.243	0.241	0.24	0.568	0.574	0.516
27ATLT100095	0.003	0.003	0.003	0.003	0.003	0.003	0.003
27ATLT100066	0.22	0.249	0.268	0.302	0.362	0.419	0.497
27ATLT100039	0.439	0.439	0.438	0.441	0.441	0.441	0.438
27ATLT100005	0.081	0.081	0.081	0.08	0.08	0.08	0.081
27ATLT100000	0.144	0.144	0.145	0.145	0.144	0.144	0.141
27KILK00241	0.146	0.136	0.13	0.123	0.115	0.109	0.088
27KILK00198	0.621	0.631	0.638	0.648	0.663	0.676	0.731
27KILK00167	0.806	0.805	0.821	0.845	0.856	0.865	0.895
27KILD00138	0.85	0.926	0.965	1.005	1.042	1.069	1.093
27KILD00086	0.302	0.399	0.453	0.501	0.565	0.606	0.794

A.2.4 Froude Number

Reporting Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.1% AEP
02KLK00278	0.457	0.458	0.458	0.458	0.455	0.456	0.462
01KLK00950	0.338	0.312	0.3	0.314	0.335	0.341	0.346
01KLK00824	0.192	0.191	0.188	0.184	0.181	0.179	0.181
01KLK00751	0.235	0.248	0.252	0.254	0.254	0.253	0.257
01KLK00553	0.098	0.094	0.095	0.094	0.094	0.094	0.092
02ROA00259	0.313	0.314	0.314	0.314	0.314	0.314	0.314
02ROA00000	0.066	0.079	0.087	0.098	0.126	0.144	0.165
01ROA00085	0.17	0.207	0.219	0.229	0.241	0.25	0.282
01ROA00000	0.261	0.275	0.281	0.285	0.289	0.29	0.295
01KLE00332	0.413	0.44	0.453	0.484	0.523	0.548	0.634
01KLE00124	0.15	0.161	0.166	0.171	0.174	0.175	0.195
01KLE00000	0.288	0.293	0.297	0.3	0.303	0.308	0.359
27ATLA00022E	1.213	1.213	1.443	1.443	1.443	1.443	1.446
27ATLA00015	0.529	0.525	0.68	0.68	0.68	0.68	0.678
27ATLA00011I	1.669	1.652	1.316	1.316	1.316	1.316	1.383
27ATLT300030	0.202	0.233	0.252	0.271	0.293	0.304	0.366
27ATLT300016	0.234	0.264	0.282	0.295	0.317	0.334	0.359
27ATLT300001	0.145	0.146	0.144	0.144	0.183	0.185	0.162
27ATLT100095	0.002	0.002	0.002	0.002	0.002	0.002	0.002
27ATLT100066	0.082	0.087	0.088	0.093	0.108	0.123	0.145
27ATLT100039	0.447	0.449	0.449	0.449	0.449	0.449	0.472
27ATLT100005	0.04	0.04	0.04	0.04	0.04	0.04	0.039
27ATLT100000	0.053	0.052	0.053	0.053	0.053	0.052	0.051
27KILK00241	0.066	0.061	0.059	0.056	0.052	0.049	0.04
27KILK00198	0.356	0.359	0.362	0.365	0.37	0.374	0.394
27KILK00167	0.56	0.537	0.538	0.545	0.541	0.54	0.513
27KILD00138	0.446	0.46	0.483	0.509	0.554	0.599	0.931
27KILD00086	0.117	0.154	0.174	0.189	0.21	0.223	0.281

B Appendix B – Coastal Modelling and Wave Overtopping

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Kilkee Coastal Flood Modelling

Final Report

September 2023

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Contract

This report describes work commissioned by Office of Public Works (OPW), on behalf of Clare County Council (CCC). Rebecca Van Coppenolle and Ian Gaskell of JBA Consulting carried out this work.

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Purpose

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1 Introduction

1.1 Project background and aim

This coastal flood modelling report aims to provide technical details on the modelling undertaken to assess coastal flood risk at Kilkee to inform the Kilkee Flood Relief Scheme (FRS).

Kilkee was identified as an Area for Further Assessment (AFA) in the Shannon Catchment Flood Risk Assessment and Management (CFRAM) Study. It was concluded that an FRS would be viable and effective for the community. The purpose for the Kilkee FRS project is to design and submit for planning a scheme to alleviate the risk of flooding to the community of Kilkee to a determined Standard of Protection (SoP).

The Kilkee FRS involves both fluvial and coastal flood risks. This coastal report's primary use is to inform the development of the fluvial scheme, allowing it to understand the systems downstream boundary conditions and help inform the relationship between proposed fluvial defences and coastal overtopping volumes. It will also help inform the Kilkee coastal flood relief works, which will be proceeded as a separate entity.

To inform the Kilkee fluvial FRS a detailed assessment of coastal flood risk was required that considers the interaction of waves and tidal conditions, including surge and wave setup. This report details the data, methodology and results of the coastal flood modelling undertaken.

Detailed modelling of the Moore Bay area was undertaken as part of the Kilkee Coastal Wave and Water Level Modelling Study in 2021. This work mirrored that undertaken at other locations during Phase 3 of the Irish Coastal Wave and Water Level Modelling Study (ICWWS) 2018. The new modelling made use of the latest data sources, including recent hydrographic and topographic survey commissioned by Clare County Council of the Moore Bay area and the ICWWS 2018 extreme water level dataset. The modelling undertaken provided joint probability wave and water level conditions that were used as part of a wave overtopping analysis to inform the Kilkee FRS.

2 Model development

2.1 Modelling requirements

To assess coastal flood risk at Kilkee, a wave overtopping analysis was required to determine volumes of water passing over the top of the defence network. Flood inundation modelling was then used to map flood flow paths and resultant flood depths for a range of design events following the calibration to the 6th January 2014 storm event.

The remainder of this chapter describes the modelling software, methodology and key decisions made.

2.2 Datum and coordinate system

To be consistent with the fluvial modelling and available survey data, the coastal model data used, and results are presented in the Irish National Grid coordinate system to Malin Head datum (OSGM02). For reference, at Kilkee OSGM02 is 0.09m higher than OSGM15.

2.3 Wave overtopping software

The complexity of the physical processes leading to wave overtopping introduces a high degree of uncertainty into its quantification. As a result, the overtopping caused by individual waves is not typically calculated; instead, the average overtopping rate for a particular sea-state is estimated using empirical or physical models. An example of an empirical model is the European Overtopping Manual (EurOtop) Artificial Neural Network (ANN) first edition tool, which was used for this study. A second edition of the ANN tool is available but was not used based on experience of using the tool; results can be illogical due to the model trying to extrapolate the result when the structure and conditions are outside of the underlying training data.

The ANN allows for the assessment of complex multi-component defence structures. The tool requires the following input data to derive a mean wave overtopping discharge rate:

- Defence geometry
- Nearshore wave conditions at the toe of a defence structure
- Still water levels at the toe of a defence structure

2.4 Wave overtopping input data

2.4.1 Schematisation of coastal defence structures

A total of 13 separate defence sections were identified and schematised using the following topographic and bathymetric datasets:

- 2m resolution Digital Terrain Model (received on 14/01/2020)
- Topographic and bathymetric drone surveys and laser survey from McDonald Surveys Ltd (MCDS) 2020 5039-009[1].

The defence profile schematisation can be separated into three main sections. The upper section or crest, the middle section or berm and the lower section or toe. 18 parameters are used to describe the defence structure include crest height (Rc); armour height (Ac); armour width (Gc); berm elevation (hb); berm width (B); upper slope (αu); lower slope (αd); and roughness (γf). A typical beach profile and the parameters required for the schematisation of a NN profile are summarised in Figure 2-1.

The 13 defence sections were identified by assessing key changes in defence type, geometric shape, and crest and toe levels. The location of the 13 individual sections are shown on Figure 2-2 and the defence schematisations are displayed in Appendix A.1.

1 Bathy and Drone Survey Rev3

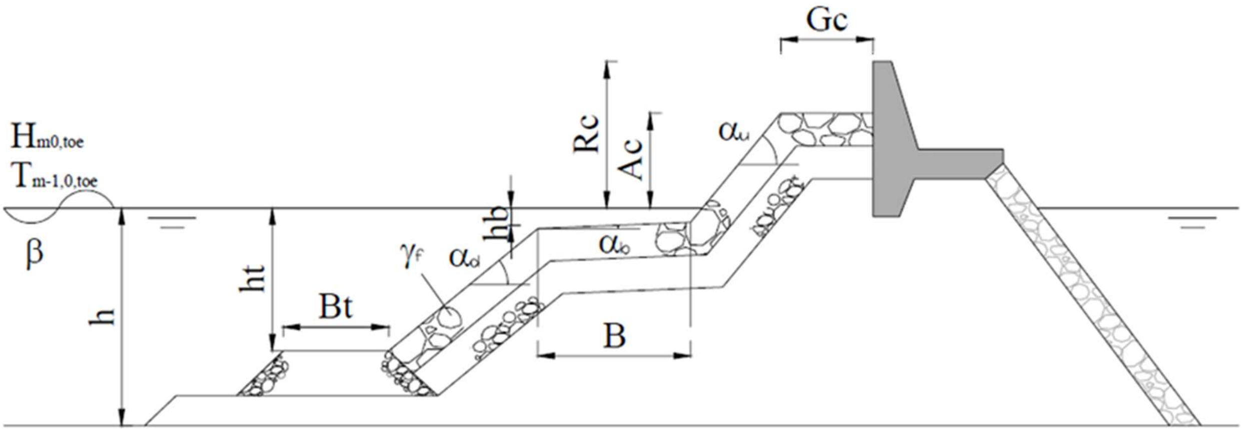

Figure 2-1: Schematisation of a typical defence section for analysis using the Neural Network overtopping tool

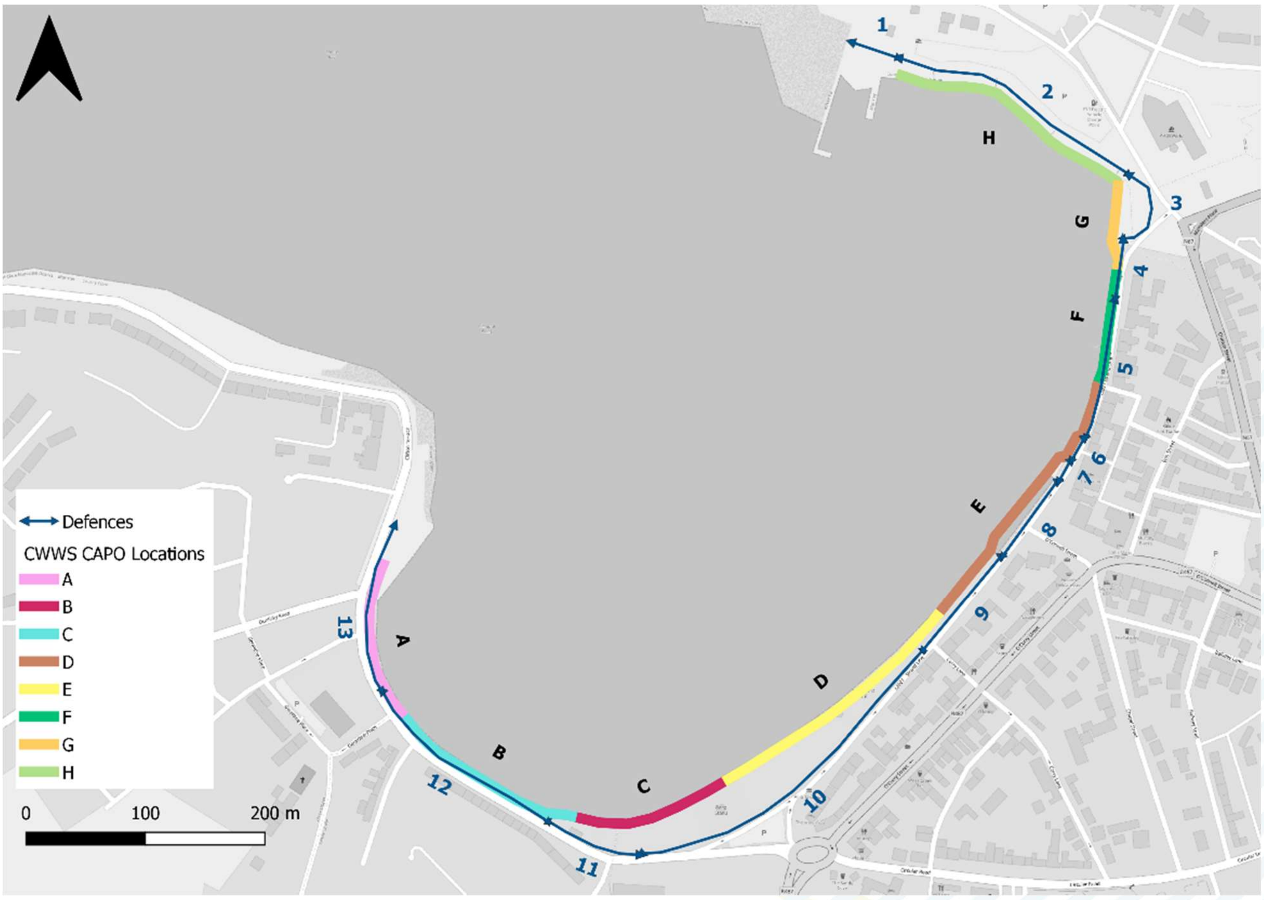

Figure 2-2: Location of the defence sections and CAPO along Kilkee's frontage

2.4.2 Nearshore wave and water level climate

The nearshore wave and water level climate was taken from the 2021 Kilkee Coastal Wave and Water Level Modelling Study[2]. The joint probability conditions included six combinations of wave height, period, direction and water level (OSGM02 used) that are equivalent to a specific Annual Exceedance Probability (AEP) event. The joint probability combinations were

[2] Kilkee Coastal Wave and Water Level Modelling Study August 2021 - IBE1781_CWWS_Kilkee_Rp01_D01

available for eight Coastal Areas Potentially Vulnerable to Wave Overtopping (CAPO) along Kilkee's frontage. An example of the 50% AEP joint probability conditions at CAPO A is shown on Figure 2-3. Each CAPO location is shown on Figure 2-2 and the CAPO nearshore conditions assigned to each of the 13 defence sections is detailed in Table 2-1.

The joint probability water levels account for wave setup, which was found to be significant at Kilkee, as discussed in Appendix A.2.

Present Day Scenario Water Level						
	Water Level (m)			Comb Wave Component		
AEP	MSL	OD Malin OSGM02	OD Malin OSGM15	Hm0 (m)	Tp (s)	MWD (°)
50%	3.13	3.00	2.91	1.05	16.05	24
	2.98	2.86	2.77	0.98	15.90	24
	2.95	2.82	2.73	0.93	14.41	25
	3.05	2.92	2.83	0.92	14.07	24
	3.06	2.93	2.84	0.87	12.77	24
	3.07	2.94	2.85	0.82	11.53	24

Figure 2-3: CAPO A 50% AEP event joint probability conditions

Table 2-1 Association of the CAPO and the defence sections

CAPO	Defence section
CAPO A	13
CAPO B	12
CAPO C	11
CAPO D	10
CAPO E	6, 7, 8, 9
CAPO F	4, 5
CAPO G	3
CAPO H	1, 2

For use in the wave overtopping modelling, design tidal water level time series curves were generated using the following datasets:

- Astronomical tide from the Admiralty TotalTide in Inishmore. The tide was taken for a peak corresponding to the Mean High-Water Spring (MHWS).
- Inishmore Design surge shape (refer to Appendix B for surge generation methodology).
- Extreme Sea Levels were taken from the joint probability dataset for the relevant CAPO and AEP.

An example of the 0.5% AEP design tidal water level time series curve is shown on Figure 2-4. The peak of the design surge was aligned between the trough and peak of the astronomical tide; this provides a wider and more conservative tidal curve volume than is aligned with the peak of the astronomical tide.

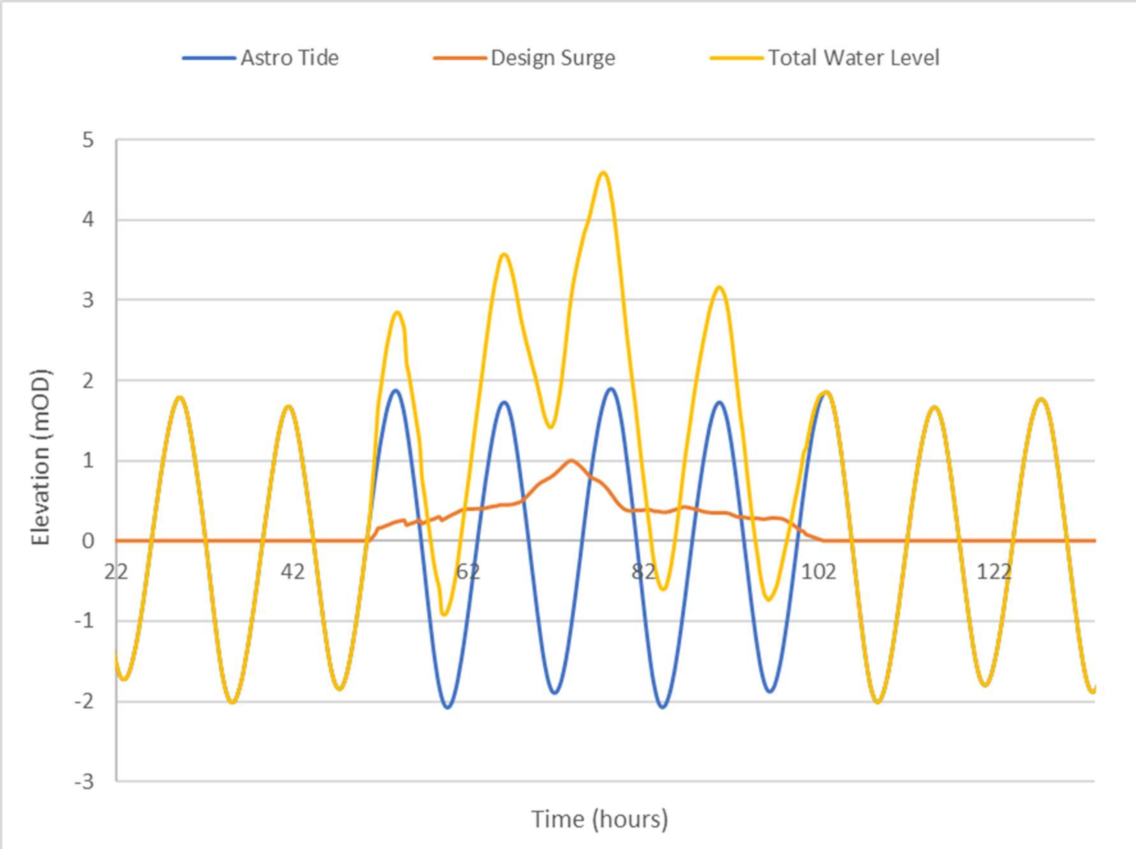

Figure 2-4: 0.5% AEP event design tidal water level time series curve

2.5 Wave overtopping calculations

Using the defence schematisations, nearshore joint probability wave and water level combinations and the tidal water level curves, mean wave overtopping discharges ($m^3/s/m$) were calculated for 13 defence sections along the Kilkee's frontage.

The first step involved calculating wave overtopping for all six joint probability combinations within each AEP event. The conditions that led to the worst-case overtopping rate for each AEP were then identified (refer to worst-case conditions tables in Appendix A.2) and used to generate a time-series of overtopping. To generate the time series for each AEP event, the overtopping rate was calculated at 15-minute intervals across the peak tidal cycle of the design water level time series curve (a 12-hour window). The maximum overtopping rate would therefore only occur at the water level peak for a short period of time.

Wave overtopping was calculated for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% Annual Exceedance Probability (AEP) events for present day, the Mid-Range Future Flood Risk (MFRS) and the High-End Future Scenario (HEFS).

2.6 Flood inundation modelling overview

A new Two-Dimensional (2D) 2m resolution TUFLOW hydraulic model was constructed to map the tidal overtopping flood risk. Note: this model was not linked to the 1D models of the Atlantic and Victoria Streams as the overtopping flood risk was considered a separate issue to the fluvial and not directly linked. The Victoria Stream was included a 2D channel within the model to allow for residual tidal ingress flood risk to be captured. The 2D model was simulated using TUFLOW version 2020-01-AB.

The January 2014 storm event was simulated in the coastal modelling suite to verify the performance of the models, and design simulations for the present day, MFRS and HEFS scenarios followed the verification.

2.6.1 Model domain and topography

The extent of the 2D model domain and coastal boundary conditions is presented in

Figure 2-5. The model was constructed using a 2m resolution fixed grid suitable to pick out the key overland coastal flood flow paths.

The base topography used for the model corresponds to the 2m resolution Digital Terrain Model (DTM) for the land area, and the 5039-009 combined topographic and bathymetric survey for the coastline and offshore areas (Figure 2-6).

Topographic features such as fluvial defences and modifications to the base LIDAR using topographic survey data was included from the 1D-2D fluvial models of the Atlantic and Victoria Streams. A full description of the topographic modifications included can be found in the Kilkee Flood Relief Scheme – Hydraulic Modelling Report.

Tidal defences along the coastal frontage were included based on the topographic survey data from MCDS Surveys Ltd 2020. The survey drawings 5039-009 Kilkee Topo_001 and 5039-009 Kilkee Topo_002 included a TOW level along the seawall that included top of wall levels, while the 5039-009 Kilkee Promenade Sea Wall Scan LAS also included wall crest levels. The wall crest levels were included in the TUFLOW model as distinct sections of constant crest level that align with those used in the wave overtopping modelling. A series of gaps in the tidal wall defence for pedestrian access were included in the model, based on the surveyed data. The coastal defence sections and location of the gaps are shown on Figure 2-7.

The Victoria Stream was included in the model to allow for tidal ingress and drainage of overtopped flood waters via low spots in the existing Victoria defences. The Victoria channel was included using a 2D gully line that acted to stamp a channel into the underlying model topography based on the river channel long section drawings 5039-009_27VIC_LS_001 and

5039-009-27VICTRIB1_REV1_LS_002 as well as the CFRAM long section 27KILF_KILFEARAGH_LS_1. To connect the channel to the Kilkee Bay an open culvert was included in 1D and connected to the 2D model grid based on drawing 5039-009_27VIC_XS_001. Two additional open culverts were included to allow for tidal ingress and drainage as shown on Figure 2-8. Culvert information was taken from 5039-009_27VICTRIB1_REV1_XS_001 (culvert 2 on Figure 2-8) and 27KILF_KILFEARAGH_XS_2 (culvert 3 on Figure 2-8).

The key culvert information used is detailed in Table 2-2.

Figure 2-5: Model domain and boundary location

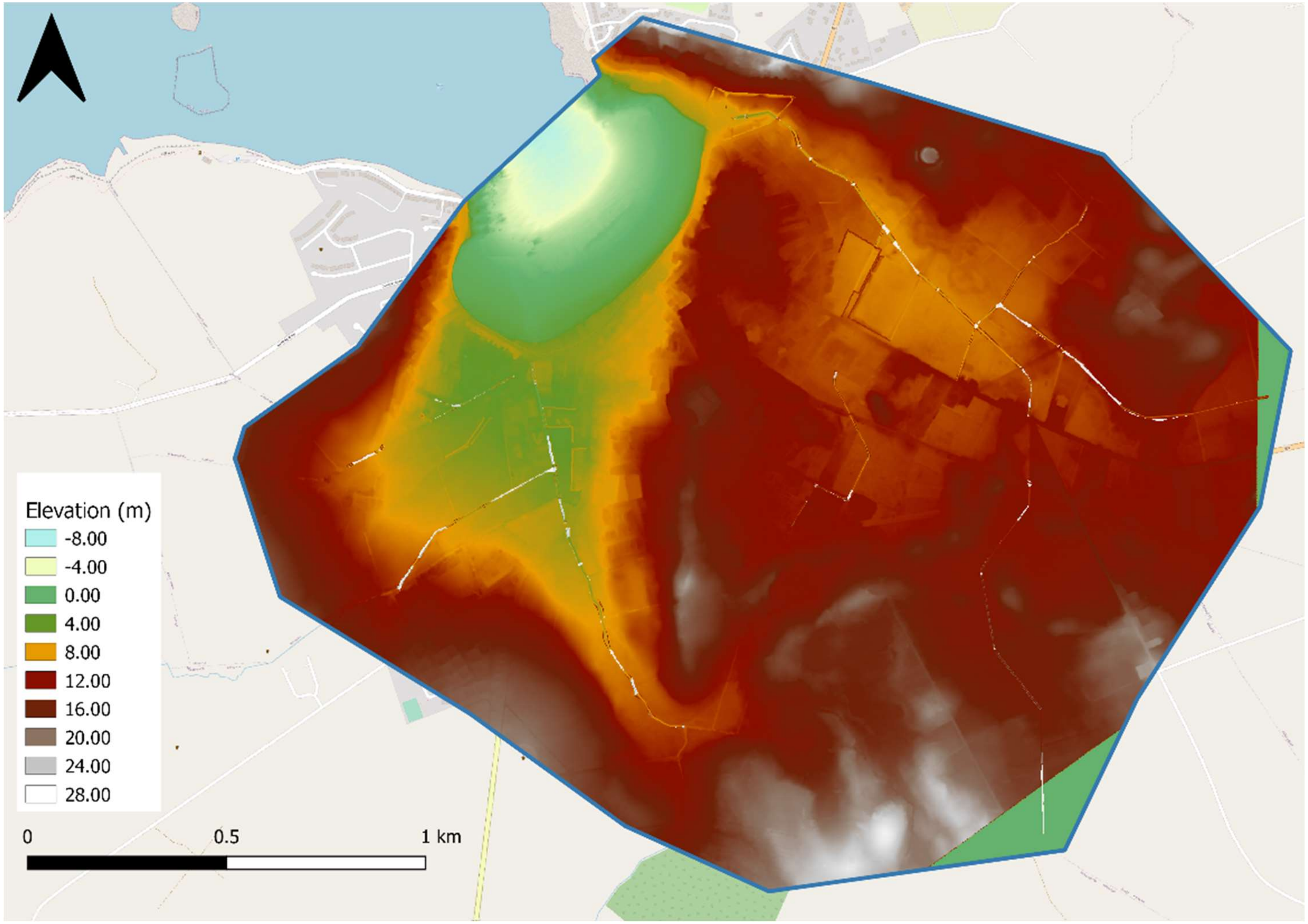

Figure 2-6: Model topography

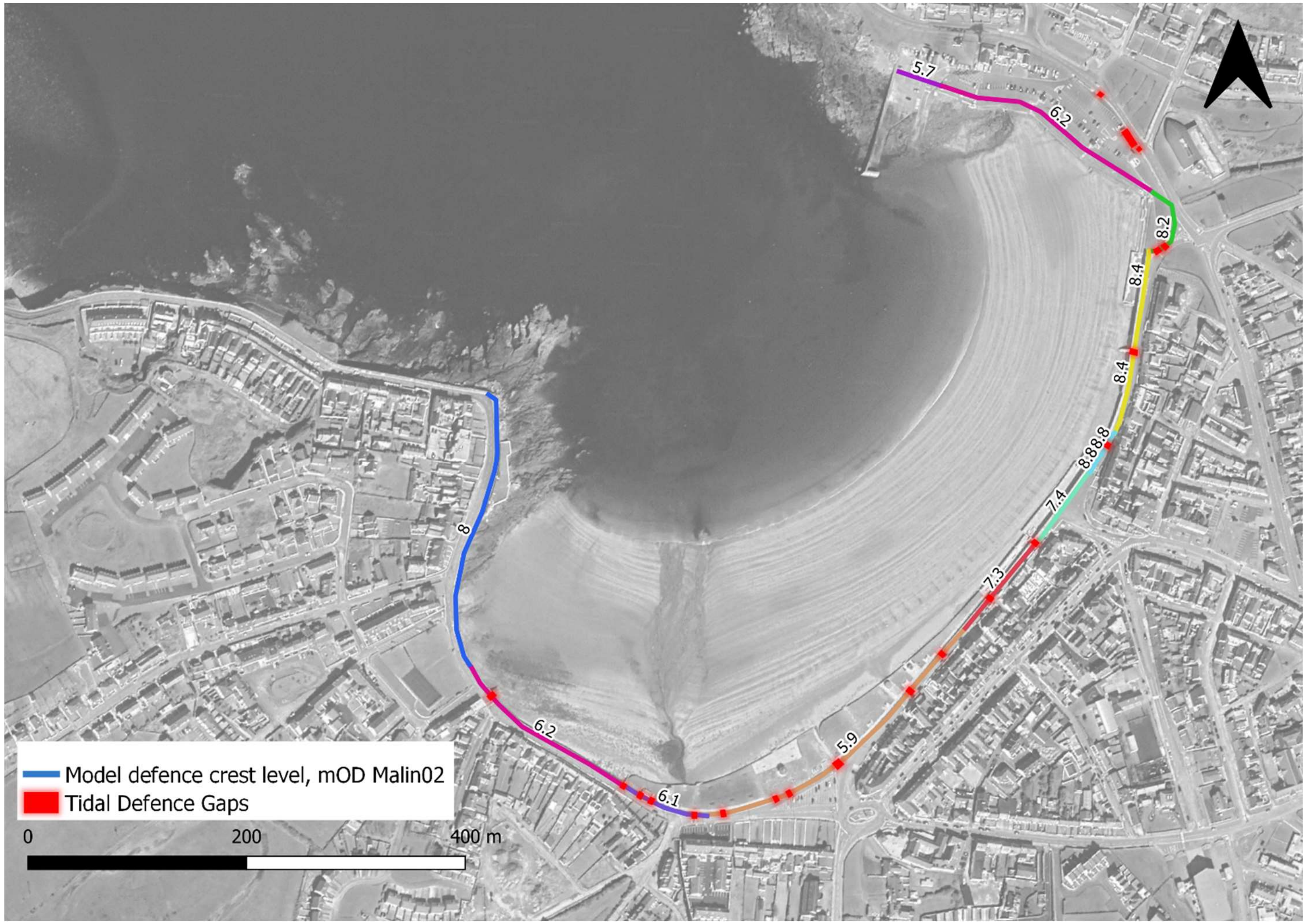

Figure 2-7: Coastal defence levels

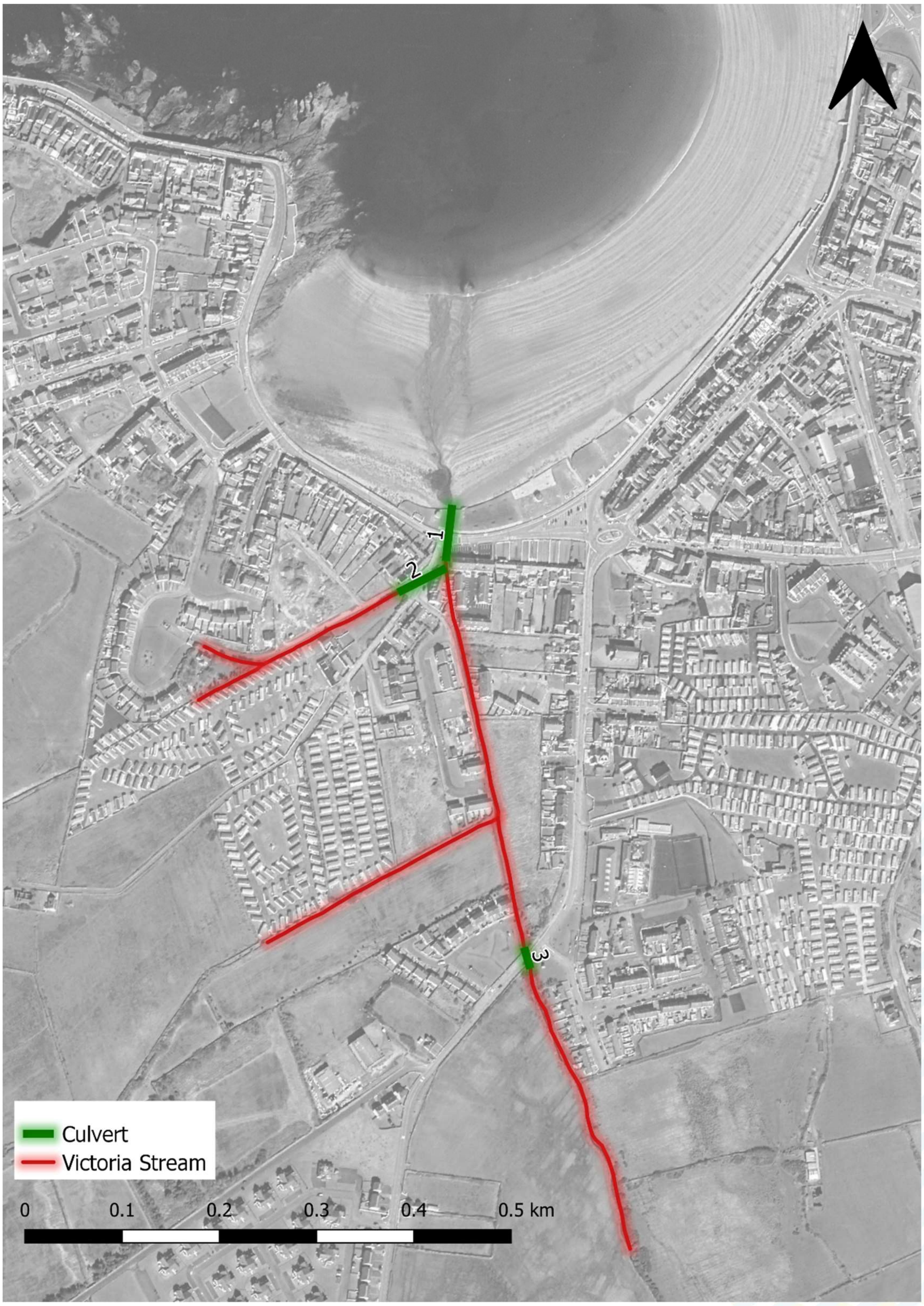

Figure 2-8: Victoria Stream and culvert locations

Table 2-2: Culvert dimensions

Culvert ID	Type	Length	Upstream invert	Downstream invert	Width	Height
1	Rectangular	51	2.40	2.23	4.66	1.64
2	Rectangular	50	2.90	2.53	0.668	0.417
3	Circular	20	4.30	4.13	1.20	1.20

2.6.2 Boundary conditions

Two tidal boundaries were included in the flood inundation model (see

Figure 2-5):

- A design water level time series applied offshore at the mouth of the bay using a level through time boundary: the largest extreme sea level for each AEP identified from the joint probability wave overtopping modelling was used in the flood inundation modelling. The tide curve generation process is detailed in chapter 2.3.2.

- Wave overtopping discharges applied using a flow through time boundary: mean overtopping discharges were taken from the wave overtopping model and injected on the landward side of the coastal defence. The overtopping rates varied throughout the peak water level tidal curve over a 12-hour period. The overtopping discharges were generated using the joint probability dataset and included wave setup. Note: wave overtopping outputs as m^3/s per meter of defence. Within the wave overtopping inflow boundary a multiplication factor of roughly the model cell size of 2m (this varies depending on how many model cells are activated by the boundary and the actual model defence length being modelled) was used to apply the correct overtopping volume into the model.

2.6.3 Topographic roughness

The surface roughness, including buildings and various land uses within the 2D model, has been applied using a 2D materials layer. This layer was adopted from the fluvial modelling and extended to cover the offshore area seaward of the Marine Parade. A full description of the roughness parameters applied can be found in the Kilkee Flood Relief Scheme – Hydraulic Modelling Report with a summary provided below.

The different Manning's n roughness values given to each land-use have been based on values from site visits, consultations of photographs, Chow 1959 and general values applied in hydrological modelling. Buildings and caravans have been modelled by applying a high roughness value to them (n=0.3) in order to ensure that water preferentially flows around buildings/caravans before moving through them to account for volume storage within the building footprint.

For model stability reasons the offshore area was defined as having a roughness coefficient of 0.045 and the foreshore area as 0.050. These values are typically higher than standard coefficients used in the offshore area but was deemed appropriate as the water level hydraulics were not important within the bay. The level simply rising and falling within the bay, rather than propagating up an estuary for example where the 2D roughness coefficient would be important.

A few areas of increased roughness were used to assist with model stability (2d_mat_Stability_R_002) as shown on Figure 2-9. These were primarily used offshore where the bumpy rocky foreshore was causing issues as tidal flows propagated into the bay. A few very small areas of increased roughness were used to slow the flow travelling back over the coastline defence seawards, where a drop down to the tidal water level led to some stability issues. The Victoria channel roughness was also increased to a value of 0.07 to stabilise rapid tidal flow up a very small channel in 2D.

It is expected that none of these stability fixes would impact on the maximum flood extents or depths and therefore deemed appropriate.

Figure 2-9: Location of increased topographic roughness for model stability

2.6.4 Model simulations

A summary of the flood inundation model simulations undertaken as part of this FRS are detailed in Table 2-3.

Table 2-3: Model simulations summary

Scenario	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
Present day	✓	✓	✓	✓	✓	✓	✓	✓

Scenario	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
(2020)								
Mid-Range Future Scenario			✓				✓	✓
High-End Future Scenario			✓				✓	✓

2.6.5 Calibration event modelling

The event of the 6th-7th of January 2014 was simulated in the coastal modelling suite. During the event, wave overtopping occurred along Kilkee's frontage, mainly affecting areas along the Strand Line to the North of the bay and the Marine Parade to the South of the bay. Kilkee suffered extensive flooding and overtopping. Figure 2-10 shows the overtopping that occurred during the event along the Strand Line between O'Connell Street and Ministers Place.

Figure 2-10: Overtopping along Strand Line between O'Connell Street and Ministers Place 6th-7th of January 2014

The wave and water level boundary conditions at the toe of the coastline defences for this calibration event were taken from the Kilkee Coastal Wave and Water Level Modelling Study. Wave conditions and water levels were provided for the 6th of January 2014 at 8am as presented in Figure 2-11. The tidal water level time series over the course of the event was also provided within Kilkee Bay as presented in Figure 2-12. Note that water level data were provided in OSGM15 and converted to OSGM02 (+0.09m).

The wave and water level climate for the event was simulated in the wave overtopping and 2D flood inundation models to test the capacity of the model to replicate the event of the 6th of January 2014 and to define the areas impacted by the storm based on anecdotal data. The modelled maximum overtopping rates for each defence section are displayed on Table 2-4. The modelled flood extent for the calibration event is shown on Figure 2-13 below.

Flooded areas correspond to the North of the bay along the Public Car Park, the Strand Line between Ministers Place to the North and the Strand restaurant to the South. The Band Strand to the South of the bay is impacted by the event, with flooding reaching Victoria Court and Well Road. The Marine Parade is also impacted, with flooding on the road.

The modelled flood extents for the calibration event replicate what happened to a suitable level of accuracy that we could have a degree of faith in the model’s ability to simulate design flood events. The modelled event data was compared against photographic and anecdotal evidence provided by Clare County Council, such as the picture of overtopping between O’Connell Street and Ministers Place, and an observation that the bandstand was destroyed by wave action. The modelled event outputs were provided to Clare County Council for consideration and approval.

No changes were made to the model as part of the calibration.

Location	Hm0 (m)	Tp (s)	MWD (°)	WL (m to MSL)	WL (m to OD Malin OSGM15)
CAPO A	0.97	18.28	25.70	3.32	3.10
CAPO B	1.16	19.49	7.78	3.32	3.11
CAPO C	1.41	19.28	335.81	3.31	3.10
CAPO D	1.55	19.06	326.69	3.30	3.08
CAPO E	1.67	18.56	310.25	3.30	3.08
CAPO F	1.61	18.06	282.29	3.31	3.09
CAPO G	1.47	18.14	269.76	3.31	3.10
CAPO H	1.39	17.58	262.70	3.30	3.09
Point 1	1.68	19.19	340.32	3.28	3.07
Point 2	1.73	18.87	321.48	3.28	3.06
Point 3	2.29	17.58	284.70	3.28	3.06

Figure 2-11: Table from the RPS report IBE1781 containing wave conditions and water levels to the study area on 6th January 2014 at 8am

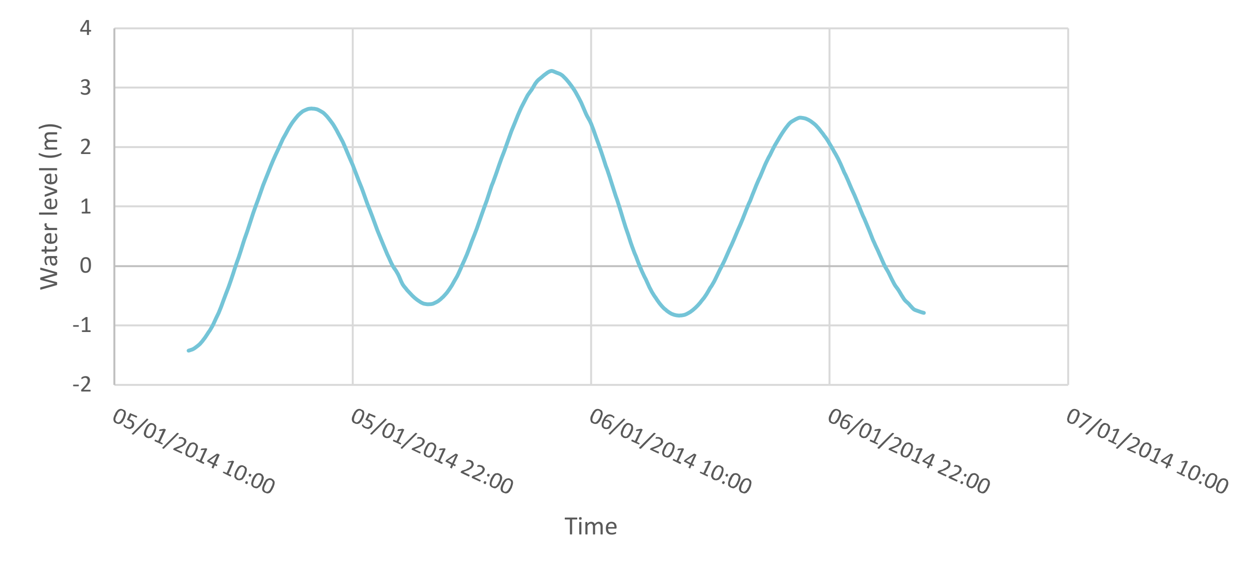

Figure 2-12: Tidal water level time series for the storm event of the 6th of January 2014

Table 2-4: Modelled overtopping rates at each defence section 6th of January 2014

Overtopping section	CAPO	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
1	H	1.39	17.58	262.7	3.18	0.00762
2	H	1.39	17.58	262.7	3.18	0.00210
3	G	1.47	18.14	269.76	3.19	0.00000
4	F	1.61	18.06	282.29	3.18	0.00008
5	F	1.61	18.06	282.29	3.18	0.00010
6	E	1.67	18.56	310.25	3.17	0.00000
7	E	1.67	18.56	310.25	3.17	0.00000
8	E	1.67	18.56	310.25	3.17	0.00000
9	E	1.67	18.56	310.25	3.17	0.00000
10	D	1.55	19.06	326.69	3.17	0.00000
11	C	1.41	19.28	335.81	3.19	0.00000
12	B	1.16	19.49	7.78	3.2	0.00045
13	A	0.97	18.28	25.7	3.19	0.00003

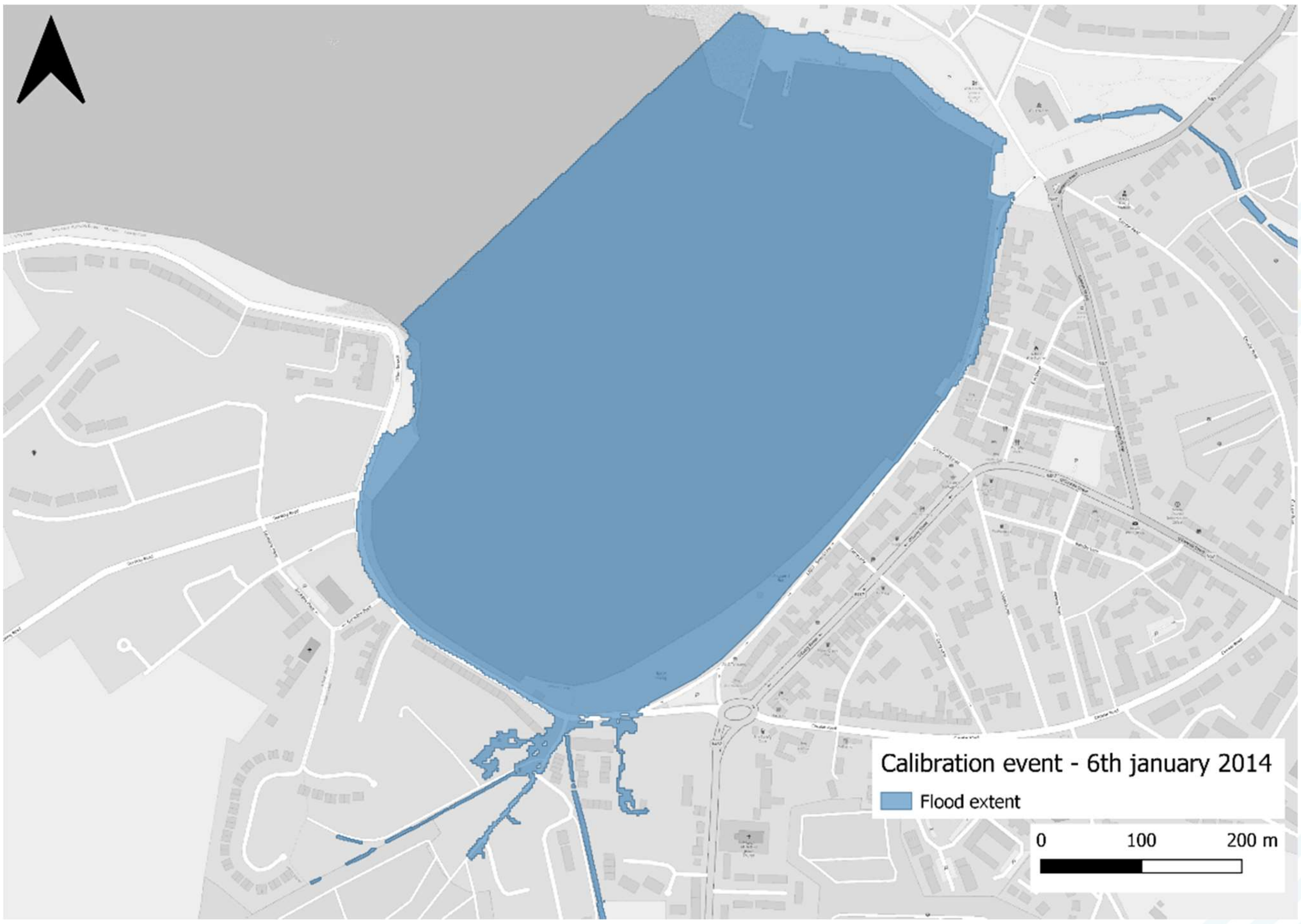

Figure 2-13: Flood extent for the calibration event of the 6th of January 2014

3 Model performance and limitations

3.1 Model performance

The model cell size is 2m resolution and the timestep used was 1 second.

The model shows no 2D negative depth waning messages for the present-day model simulations (Table 3-1). Under climate change simulations there are up to five negative depth warning messages. These warnings generally relate to flow passing back over the coastline defence crest in a seawards direction due to levels being higher on the landwards side. There is a drop down to the tidal water level on the seaward side leading to minor instability. These warning messages are not considered to impact on the modelled result.

Table 3-1: 2D negative depth warning messages

Scenario	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
Present day (2020)	0	0	0	0	0	0	0	0
Mid-Range Future Scenario	-	-	0	-	-	-	0	5
High-End Future Scenario	-	-	0	-	-	-	4	4

There are 80 checks prior to the model simulation for all model simulations. These relate to a 'CHECK 2370 – Ignoring coincident point found in Create TIN layer'. Checks were undertaken and this check relates to the ztins for heavy vegetation, compound and field where points are closer together than the 2m model cell size. It made sense to keep in the more detailed ztin points and therefore no changes were made to remove these check messages, but no model result impact is expected.

There are 9 warnings prior to the model simulation for all model simulations. These relate to a 'WARNING 2073 – Null shape object ignored. Only Regions, Lines, Polylines & Multiple Polylines used'. Checks were undertaken to try and identify what shape objects were being ignored but were not identified. The warning messages we're also located outside of the model domain and were therefore considered to have no impact on the modelled results.

The tolerance limit for Mass Balance Error (MBE) is +/- 1.0%. it is notable that upon model start-up there is a significant inflow into the model, a result of a 2D initial water level not being implemented. However, this does not impact on model results. The MBE for all model simulations is well within the tolerable limit of +/-1%. By way of example, the 0.5% AEP present day simulation shows a maximum MBE of -0.022.

3.2 Limitations and assumptions of coastal modelling method

The approaches taken in this study incorporate standard practice methods currently used around the Irish coast to inform coastal flood risk on the scale of the study area. However, the results are only as accurate as the input data that are used. Whilst all due care and diligence was taken to use appropriate data and methods, the results should be viewed with a margin of caution given the inherent uncertainty in the estimation of wave overtopping.

Assumptions and limitations include:

- Extrapolation of limited data records to extreme values used to generate boundary conditions.

- The overtopping discharges assume that the wind and wave conditions remain constant throughout the duration of the tidal event. Overtopping rates vary as a consequence of the water level variations only although depth limitation of wave conditions is accounted for.
- Offshore winds are accounted for in the offshore and surf zone wave transformation as these are included in the boundaries of the wave models. In the nearshore the local winds may also impact on wave overtopping discharge rates and the extent over which the overtopping impacts behind a defence when there is a strong onshore wind blowing spray over the defences. These local wind affects are not accounted for in the wave overtopping modelling.
- The ANN first edition tool has limitations on the defence profile schematisation. The software is developed based on a EurOtop-dataset of more than 13,000 tests on wave overtopping over all kind of structures for a range of wave and water level conditions. The tool has some restrictions of defence profile schematisation, for example, slopes upwards and downwards from the berm that are shallower than a 1 in 8 slope are not considered, while the armour crest level must be above the extreme water level. Berm and armour crest width also have limits based on the wave height. The overtopping schematisations have been developed based on surveyed data and adjusted to best fit the tool within its restrictions based on modelling experience. In some cases, the schematised profile can vary from the underlying survey data significantly, often where we limit the slope from the berm to a 1 in 8, while in reality it is much shallower. We consider the schematisations in these cases to provide a conservative overtopping value, rather than an underestimate, although it is acknowledged that the schematisation impacts the overtopping rate and volume.
- The coastal flood inundation model was undertaken using a 2D only version of the model. This was deemed appropriate as the overtopping flood risk was considered a separate issue to the fluvial and not directly linked. Tidal propagation up the Atlantic Stream was not considered based on the fluvial modelling work which considered the Atlantic Stream would not provide a residual tidal risk. Tidal propagation up Victoria Stream was considered as a residual risk and the channel was included as a 2D 'gully line' lowering the model topography to represent channel bed level based on the 1D model cross sectional data.
- The interaction between tidal and fluvial flows was also not considered as part of the coastal flood modelling.

4 Results

Present-day coastal flood extents for eight different AEPs are shown on Figure 4-1 and Figure 4-2. Overtopping occurs in the south of Kilkee Bay on Marine Parade from the 20% AEP event. Flood water flows towards the junction of Wells Road and eventually reaches the entrance of the caravan park. In the east of the bay the flood water is limited to the northern end of Strand Line and the Strand restaurant. Due to the nature of the topography in Kilkee, where Marine parade and its associated coastal defences are raised up from the lower lying topography on the landward side, wave overtopping flood waters tend to run south and inundate the southwestern area of Kilkee. The flood extents in this area of Kilkee gradually increases as the overtopping volume increases for each AEP as described below.

During the 10% AEP event, overtopping has increased along Strand Line from Kilkee Water World to The Esplanade. Flood inundation is more extensive to the areas surrounding Wells Road, reaching Victoria Park, inundating the brownfield site and back gardens of the properties on Marine Parade. Flooding increases along Wells Road, Geraldine Place, and into the caravan park during the 5% AEP and overtopping is present along much of the Strand Line. During the 2% AEP there is increased flooding on Wells Road, reaching further into the caravan park and inundating Victoria Park. During the 1% AEP overtopping is present along the length of the Strand Line and Marine Parade.

The 0.5% and 0.1% AEPs show more extensive flooding in the southwestern area with the 0.1% AEP having the largest flood extent which extends south as far as Marion Estate.

In the north of Kilkee Bay wave conditions are generally smaller due to sheltering effects of the bay and breakwater. Flood risk in this area is similar for all AEPs due to the nature of the topography limiting flood flows and does not extend further than Kilkee public car park and Kilkee Water World.

The impact of climate change on the 0.5% AEP event is shown on Figure 4-3 and Figure 4-4. Flood extents increase with the medium and high-end climate change scenarios as would be expected, although flood extents are not drastically different due to the topography rising more sharply from the outer extents of the 0.5% AEP present day event. A comparison of flood depths at key locations for the 0.5% AEP for present day and climate change conditions are shown in Table 4-1 below. While flood extents are not that different flood depths are shown to increase.

Table 4-1: Flood depths for the 0.5% AEP for each epoch

ID	Location	0.5% AEP flood depth (m)			X	Y
		Present day	MRFS	HEFS		
1	Wells Road	0.94	1.44	1.50	88247.4	159709.0
2	Church Road	0.61	1.13	1.17	88402.8	159614.2
3	Caravan Park	0.93	1.44	1.50	88113.2	159524.9
4	Victoria Park	0.82	1.33	1.39	88264.2	159472.7
5	Marine Parade	0.89	1.00	1.04	88165.0	159781.5
6	Kilkee Water World	0.09	0.11	0.13	88710.0	160300.2

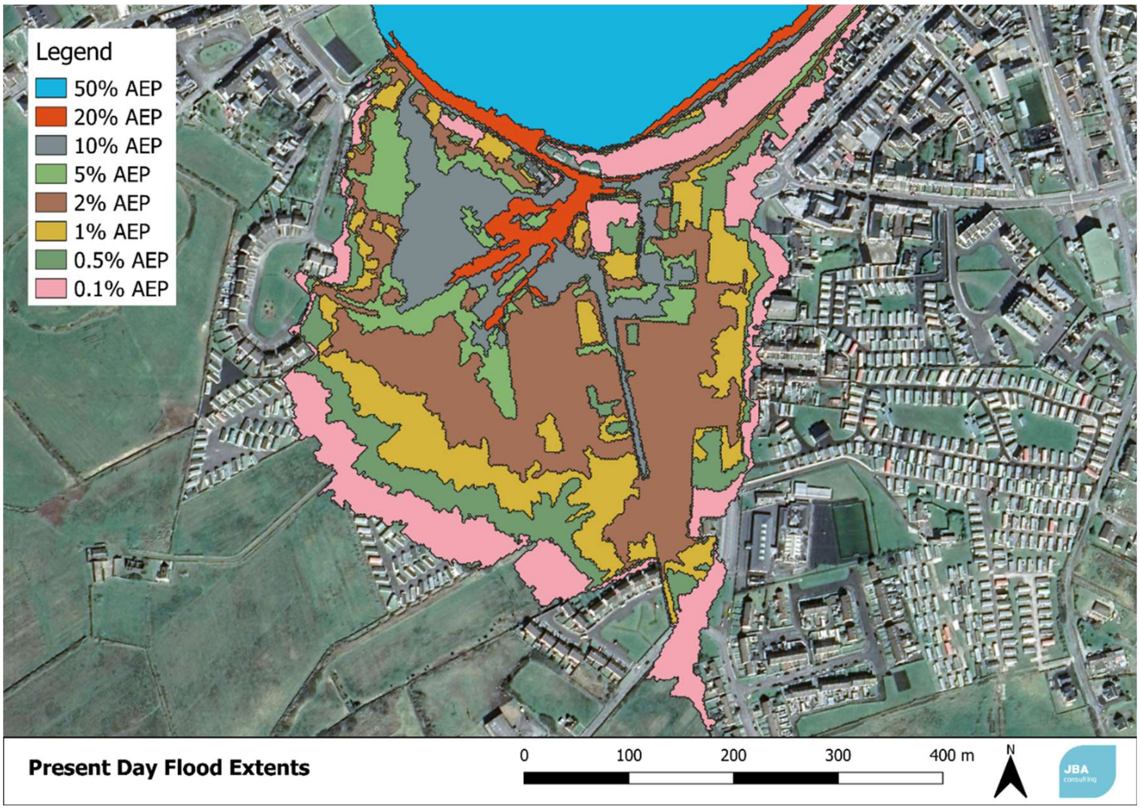

Figure 4-1: Present day flood extents for all AEP events – southern extent

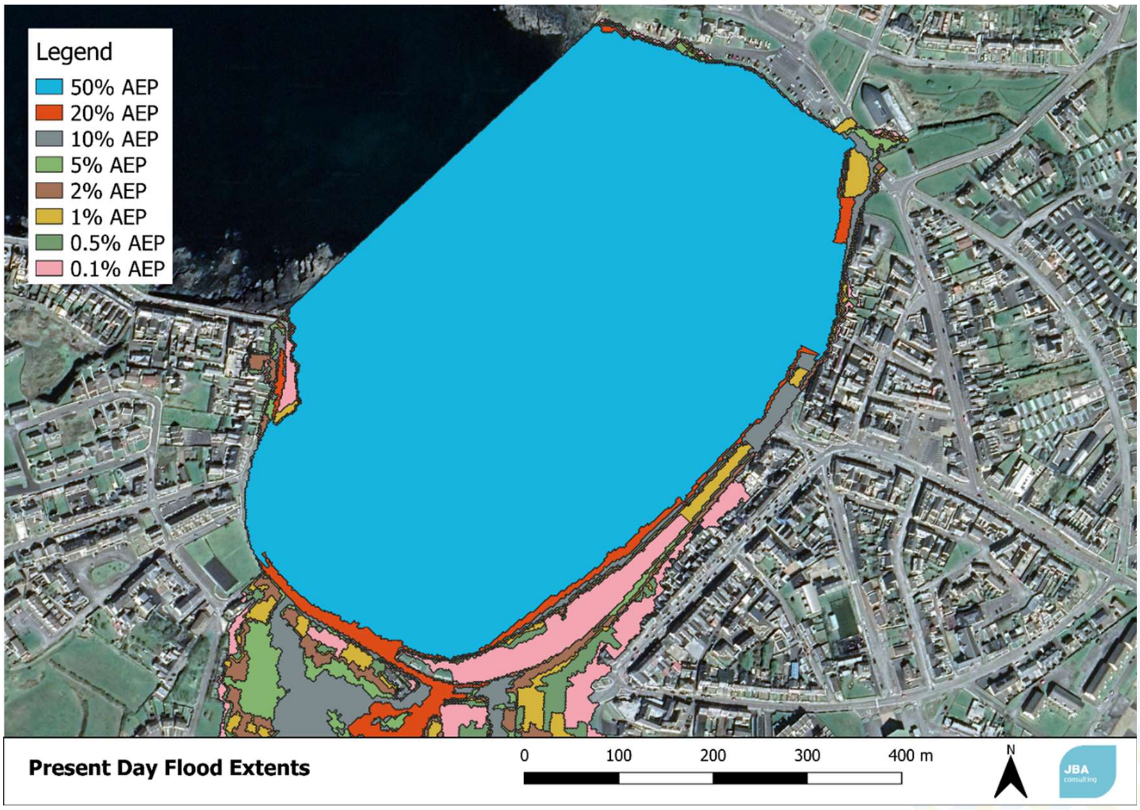

Figure 4-2: Present day flood extents for all AEP events – northern extent

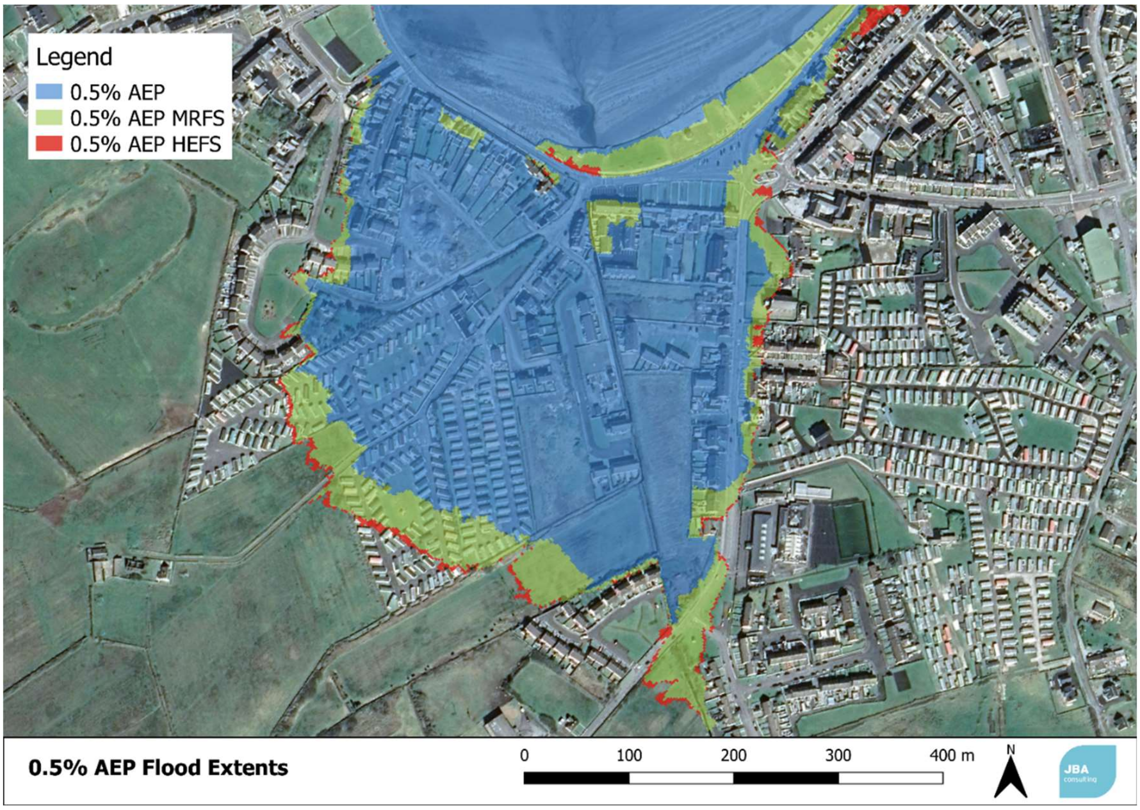

Figure 4-3: 0.5% AEP flood extents under climate change – southern extent

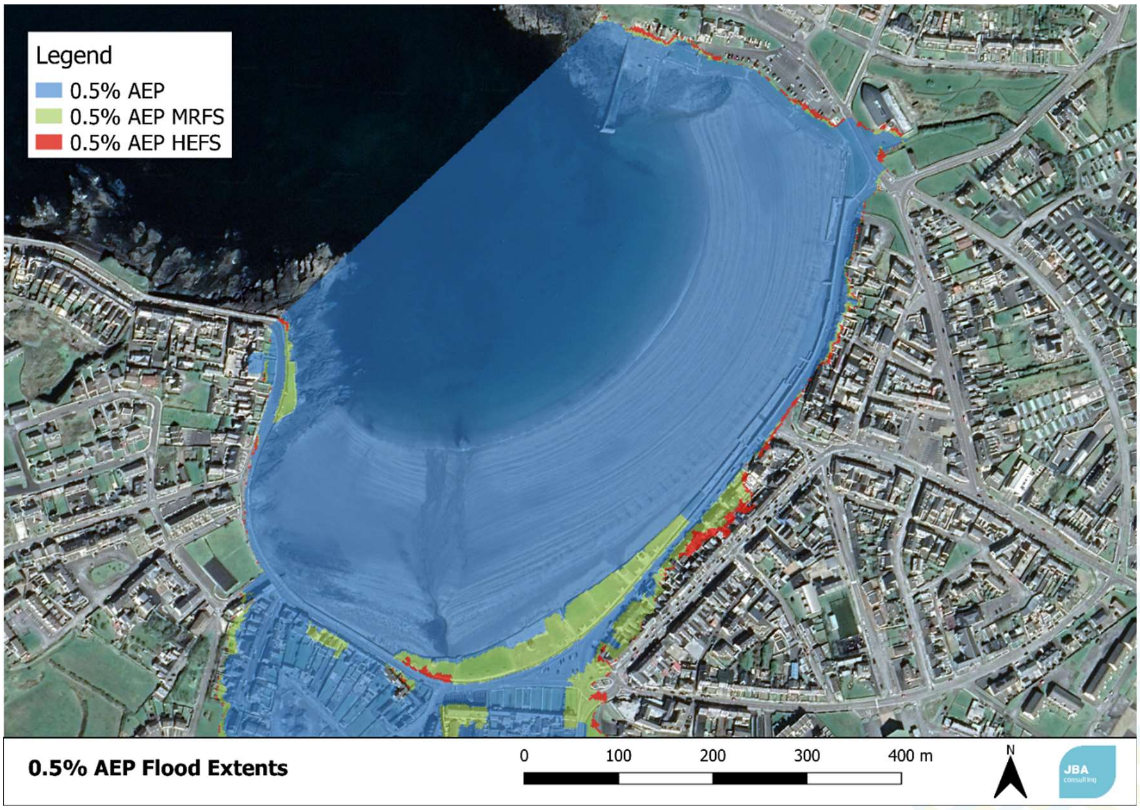

Figure 4-4: 0.5% AEP flood extents under climate change – northern extent

5 Sensitivity test – wave setup

As detailed in the 2021 Kilkee Coastal Wave and Water Level Modelling Study wave setup was shown to be significant at Kilkee. What is less well defined is the duration that the wave setup impacts on the water levels.

The 0.5% AEP water level of 4.59mOD Malin02 as detailed in the joint probability dataset includes a significant increase for wave setup, being 1.25m higher than the 0.5% extreme level as detailed in the SW49 ICWWS 2018 data point. In reality the impact of wave setup will vary depending on the water level and wave conditions occurring throughout a storm event. As a sensitivity test the 0.5% AEP wave overtopping event was simulated using a 0.5% AEP tidal curve generated using the ICWWS level of 3.25mOD Malin. This tidal curve was then uplifted over a 15-minute time period at the peak of the tide curve to match the wave setup level of 4.50mOD Malin (as shown on

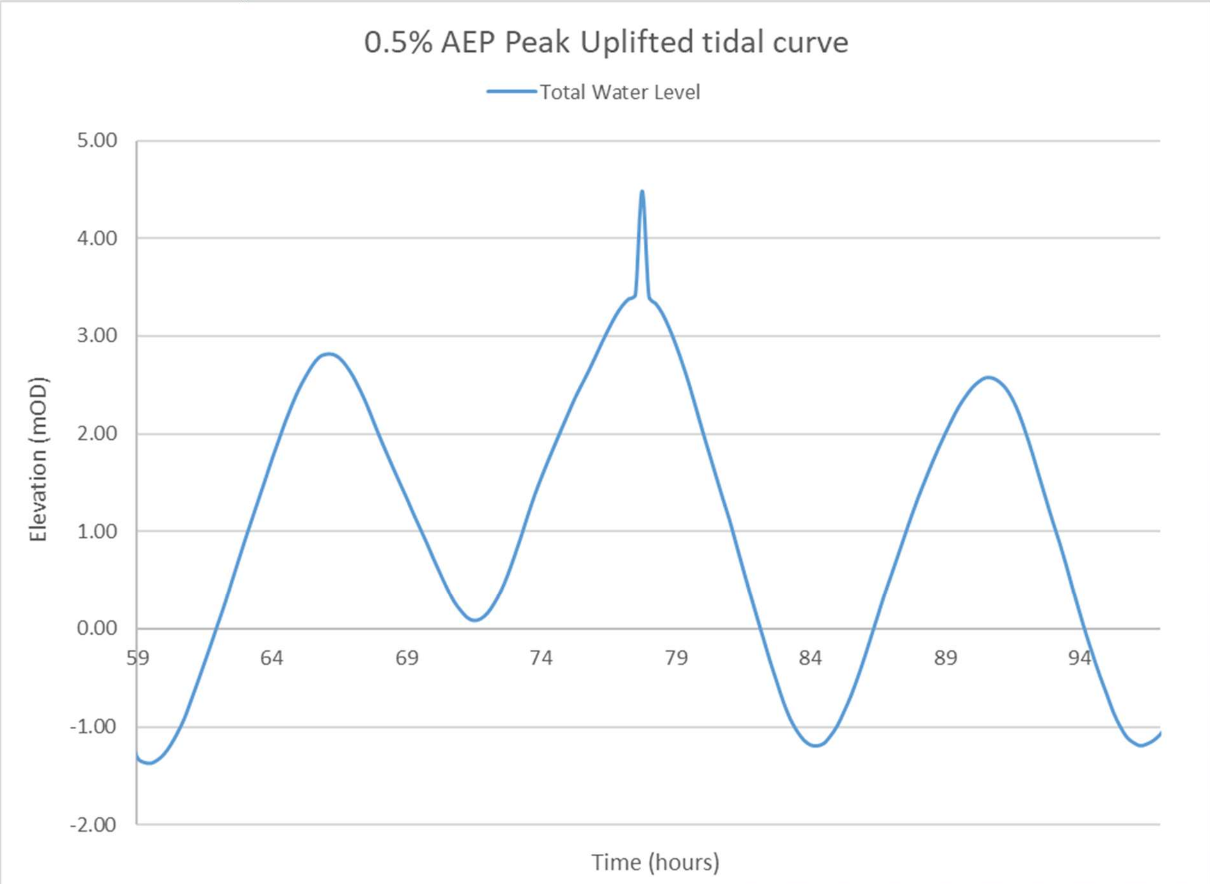

Figure 5-1). The flood model was then simulated, and the result shows a significant reduction in wave overtopping volume (red extent on Figure 5-2) when compared to the blue extent which was generated using a tidal curve generated using the joint probability level of 4.59m (as shown on Figure 5-3).

The sensitivity test showed that the impact of wave setup is significant at Kilkee. Both overtopping methods generated the same peak overtopping rate, but the overtopping volume generated is significantly less if the wave setup acts for a shorter time period.

Due to the lack of understanding of the impact of wave setup, the standard tidal curve generation approach and more conservative flood risk outputs were adopted as the final coastal flood risk outputs for this study.

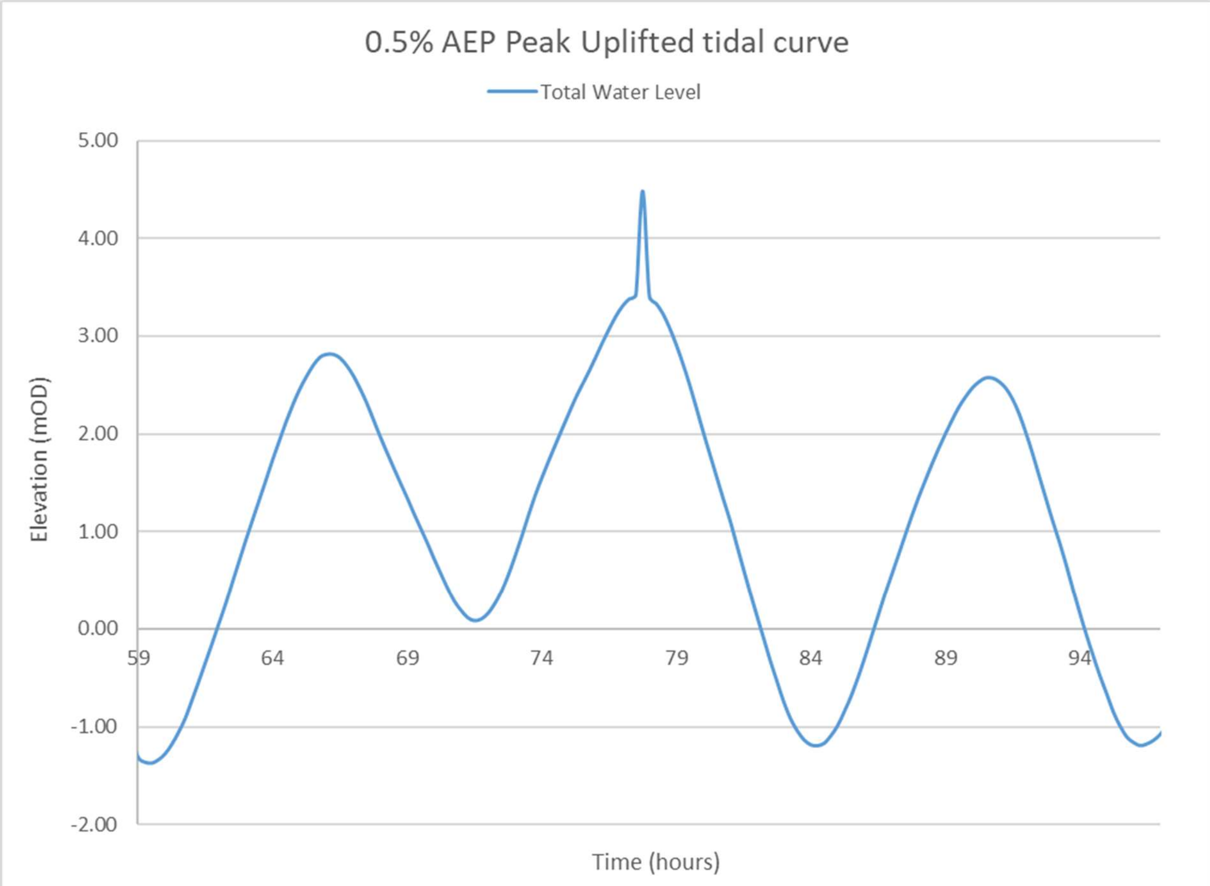

Figure 5-1: 0.5% peak uplifted tidal curve

Figure 5-2: 0.5% AEP comparison of wave setup

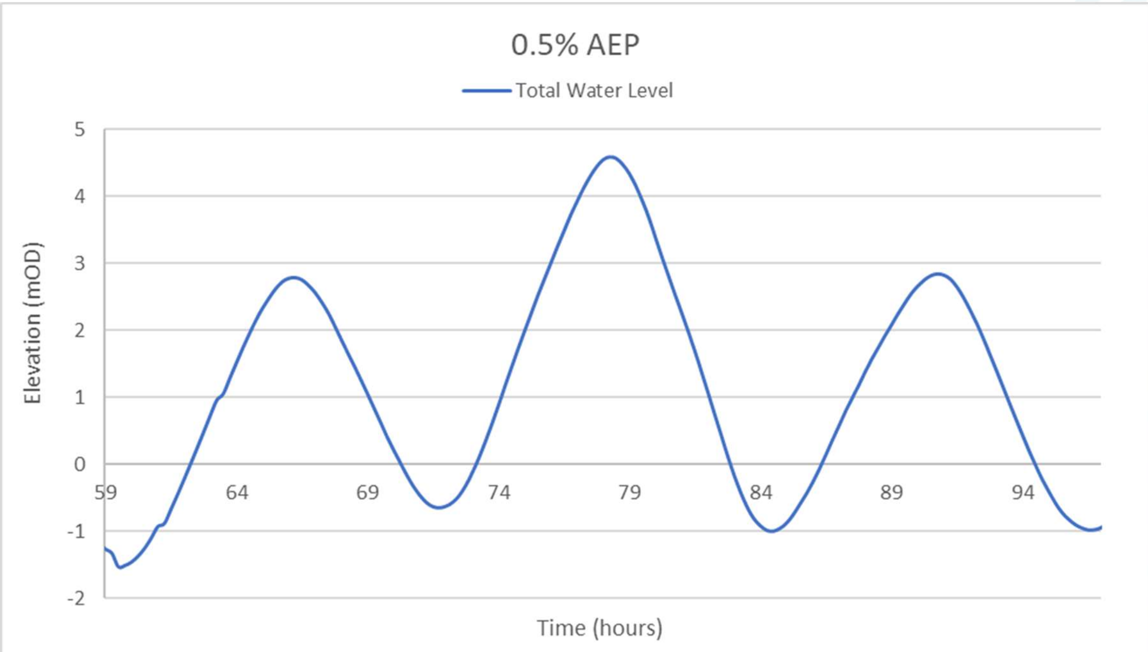

Figure 5-3: 0.5% tidal curve

A Appendix A – Wave overtopping

A.1 Defence's schematisations

Note: the black dashed line is the topography and bathy combined survey, and blue dashed line is extracted from 2m resolution LIDAR. Defence geometry has been schematised such that it fits within the range of the ANN tool, in some cases the defence profile cannot be represented accurately.

Defence 1

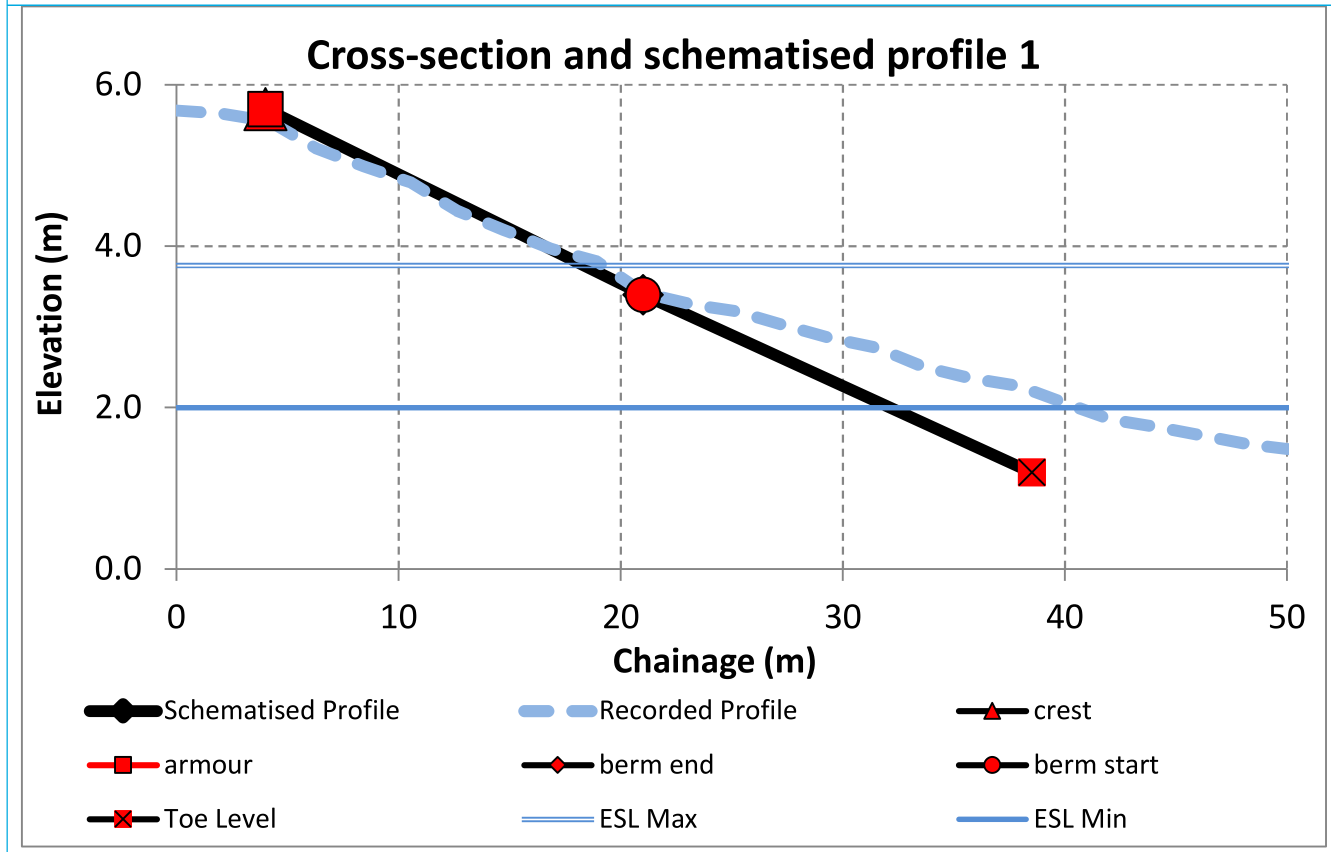

Figure A1: Bathymetry profiles and Neural Network schematisation for defence 1

Figure A2: Defence image

Table A1: Neural Network parameters for defence 1

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	1.20	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	1.20	Crest level (mAOD)	5.70
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	198.95
Slope downward of Berm (cotangent), cot ad	7.95	Wave return wall (yes=1, No=0)	0.00
Berm level (mAOD)	3.40	Break in Roughness 1	4.00
Width of Berm (m), B	0.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	7.39	Roughness 2	1.00
Armour crest level (mAOD)	5.70	Roughness 3	1.00

Defence 2

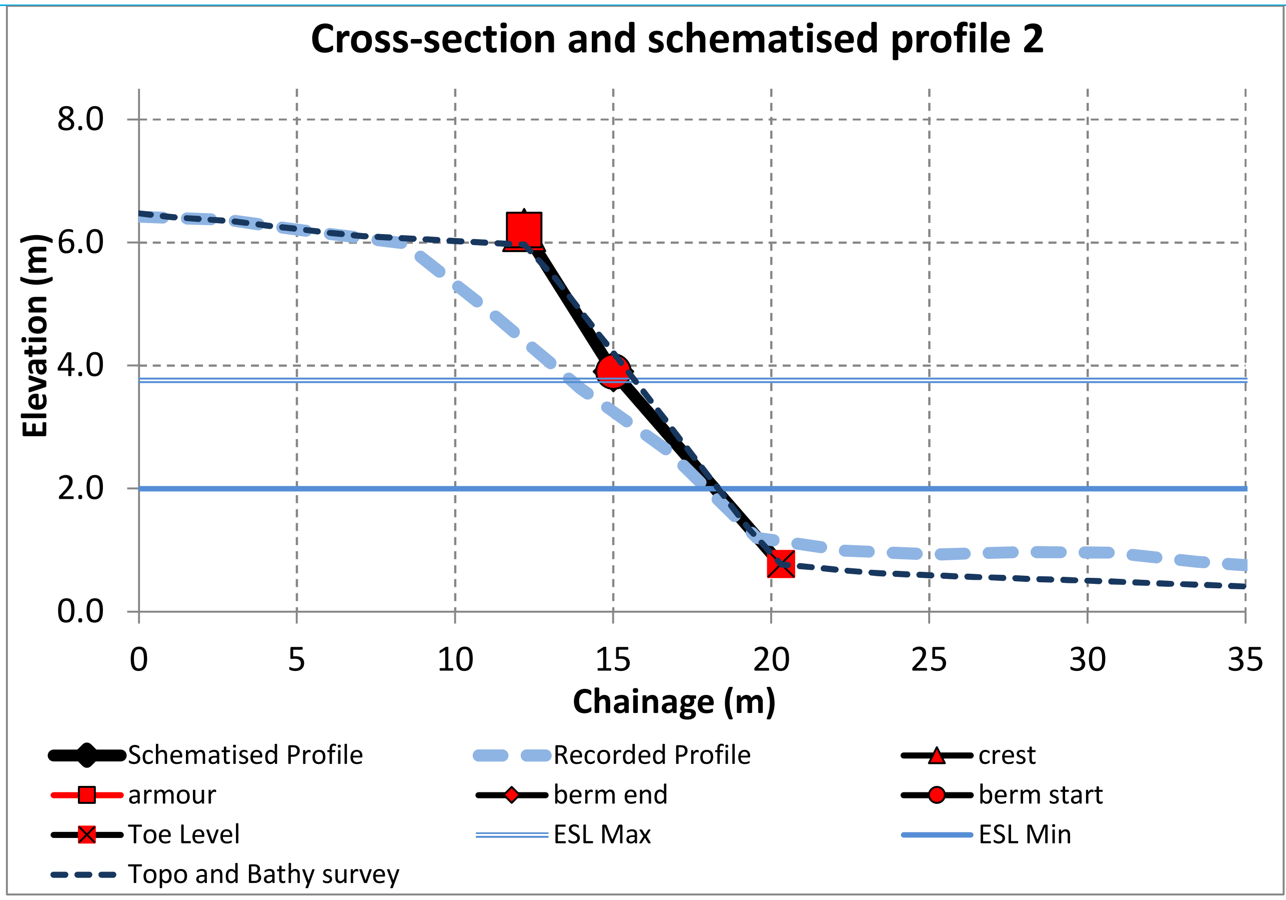

Figure A3: Bathymetry profiles and Neural Network schematisation for defence 2

Figure A4: Defence image

Table A2: Neural Network parameters for defence 2

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	0.77	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	0.77	Crest level (mAOD)	6.20
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	206.71
Slope downward of Berm (cotangent), cot ad	1.50	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	-9.00	Break in Roughness 1	6.20
Width of Berm (m), B	0.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	0.65
Slope upward of Berm (cotangent), cot au	1.50	Roughness 2	0.65
Armour crest level (mAOD)	6.20	Roughness 3	0.65

Defence 3

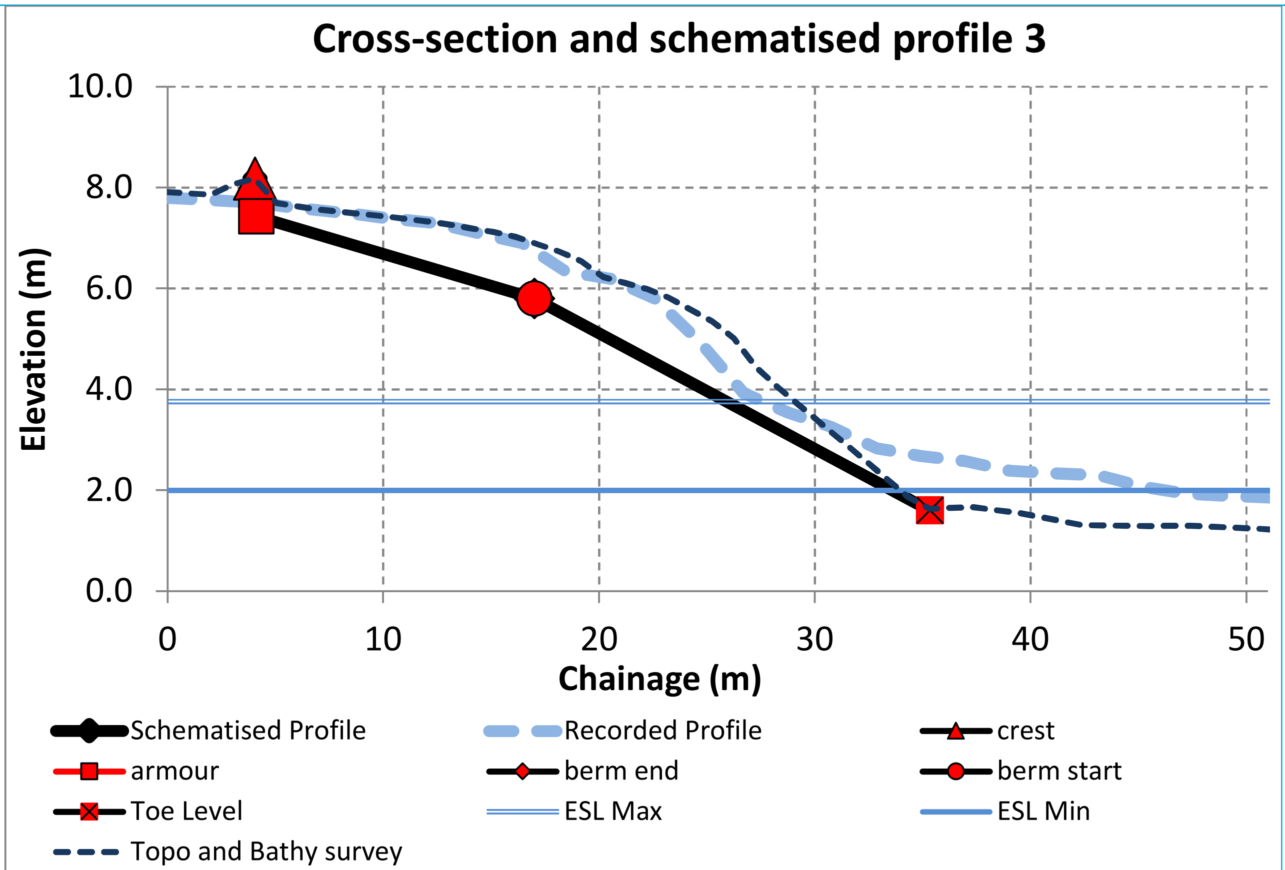

Figure A5: Bathymetry profiles and Neural Network schematisation for defence 3

Figure A6: Defence image

Table A3: Neural Network parameters for defence 3

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	1.60	Width of Armour crest (m), Gc	0.06
Toe level (mAOD)	1.60	Crest level (mAOD)	8.18
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	276.39
Slope downward of Berm (cotangent), cot ad	4.36	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	5.80	Break in Roughness 1	5.00
Width of Berm (m), B	0.00	Break in Roughness 2	3.80
Slope of Berm (tangent), tan aB	0.00	Roughness 1	2.00
Slope upward of Berm (cotangent), cot au	7.91	Roughness 2	2.00
Armour crest level (mAOD)	7.43	Roughness 3	1.00

Defence 4

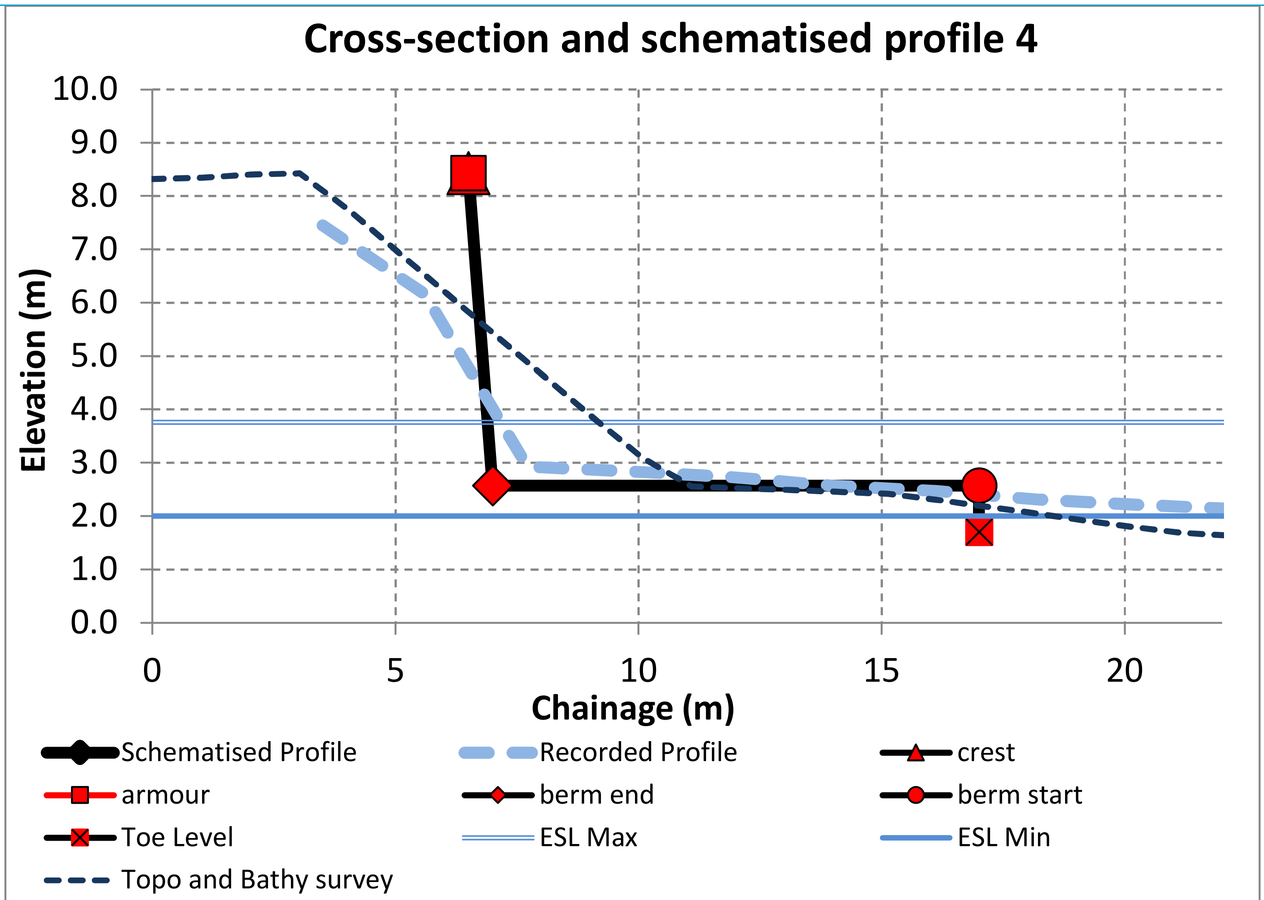

Figure A7: Bathymetry profiles and Neural Network schematisation for defence 4

Figure A8: Defence image

Table A4: Neural Network parameters for defence 4

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	1.70	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	1.70	Crest level (mAOD)	8.43
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	278.90
Slope downward of Berm (cotangent), cot ad	0.00	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	2.57	Break in Roughness 1	4.00
Width of Berm (m), B	10.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.09	Roughness 2	1.00
Armour crest level (mAOD)	8.43	Roughness 3	1.00

Defence 5

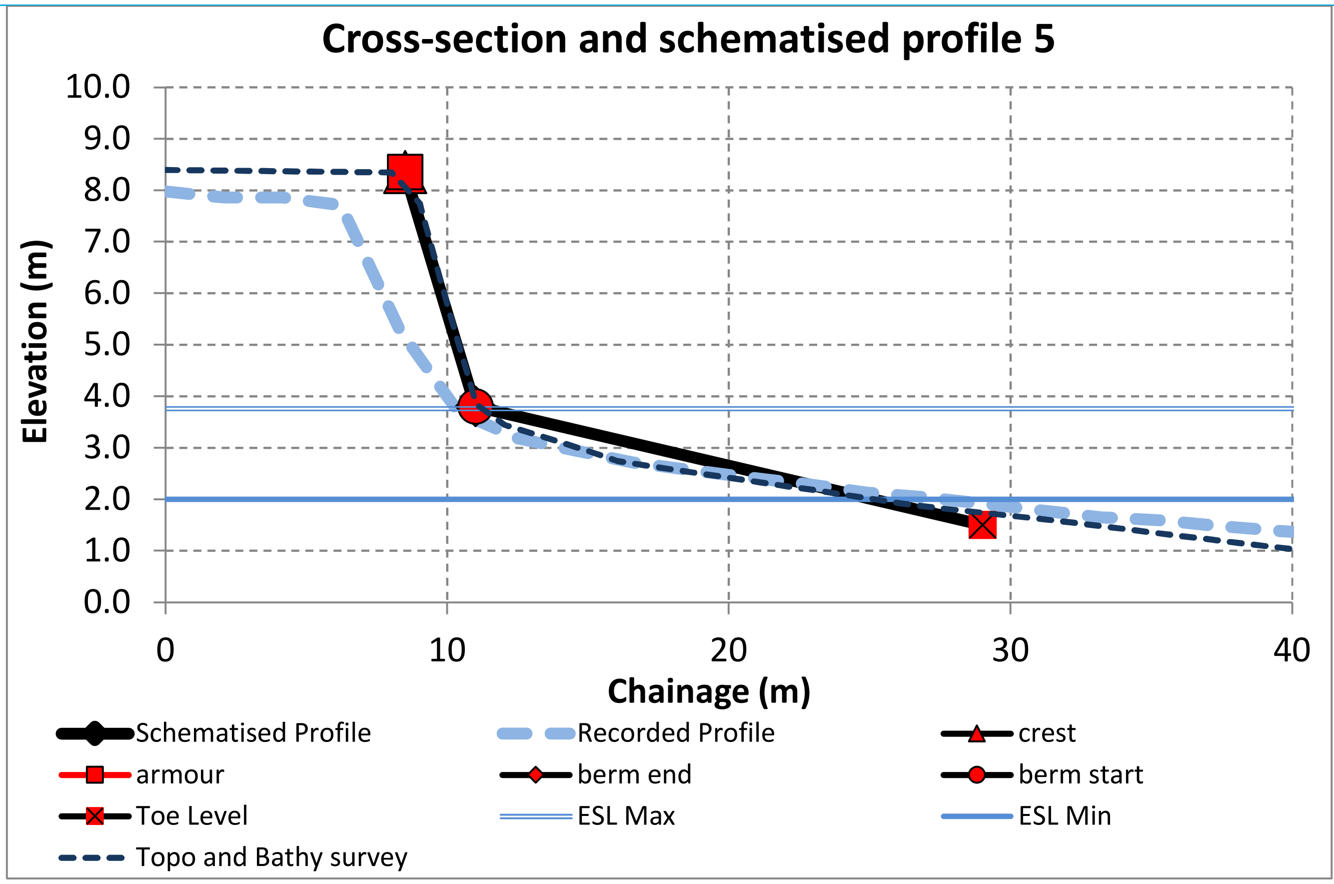

Figure A9: Bathymetry profiles and Neural Network schematisation for defence 5

Figure A10: Defence image

Table A5: Neural Network parameters for defence 5

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	1.50	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	1.50	Crest level (mAOD)	8.35
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	282.01
Slope downward of Berm (cotangent), cot ad	7.83	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	3.80	Break in Roughness 1	5.00
Width of Berm (m), B	0.00	Break in Roughness 2	3.80
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.55	Roughness 2	1.00
Armour crest level (mAOD)	8.35	Roughness 3	0.80

Defence 6

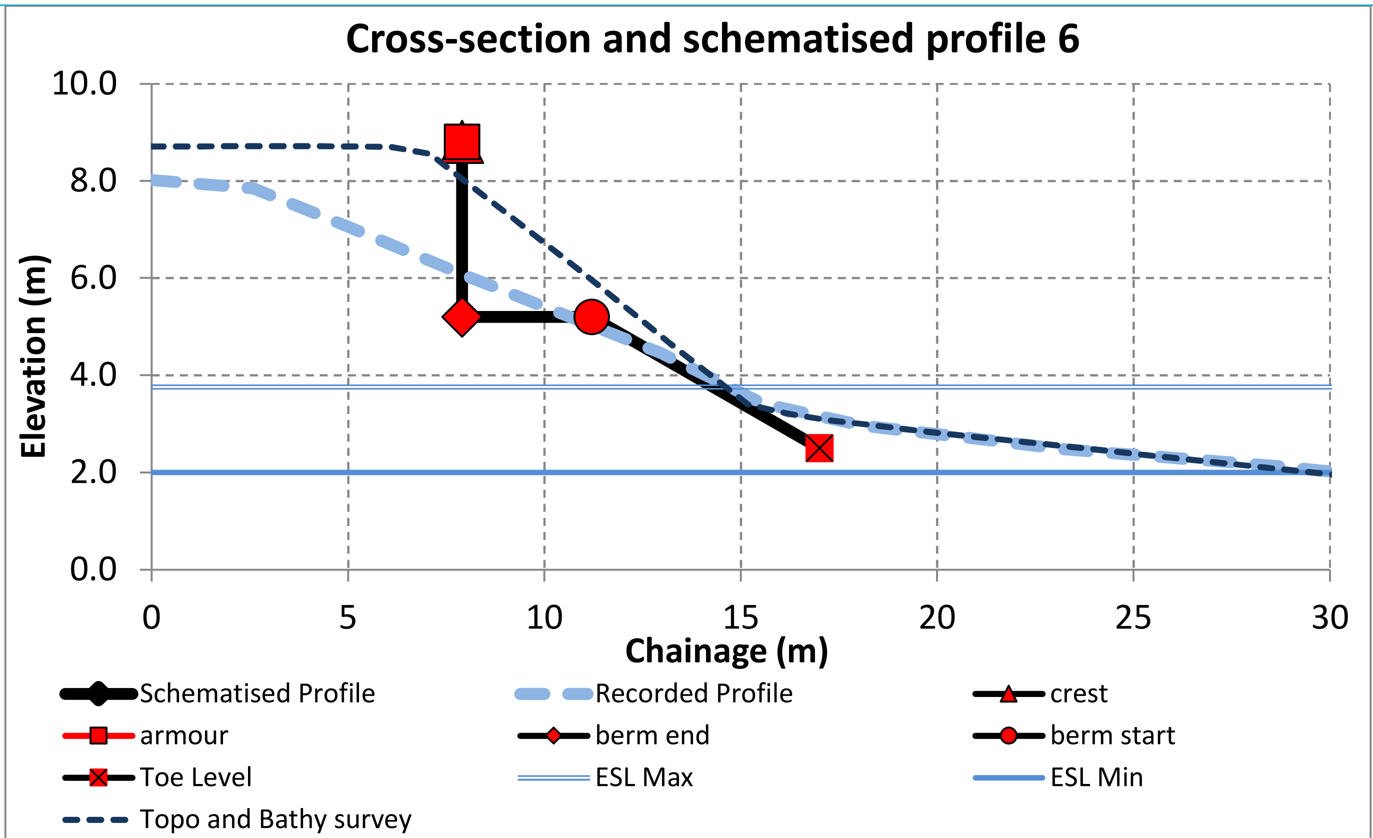

Figure A11: Bathymetry profiles and Neural Network schematisation for defence 6

Figure A12: Defence image

Table A6: Neural Network parameters for defence 6

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	2.50	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	2.50	Crest level (mAOD)	8.80
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	299.17
Slope downward of Berm (cotangent), cot ad	2.15	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	5.20	Break in Roughness 1	4.00
Width of Berm (m), B	3.30	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	8.80	Roughness 3	1.00

Defence 7

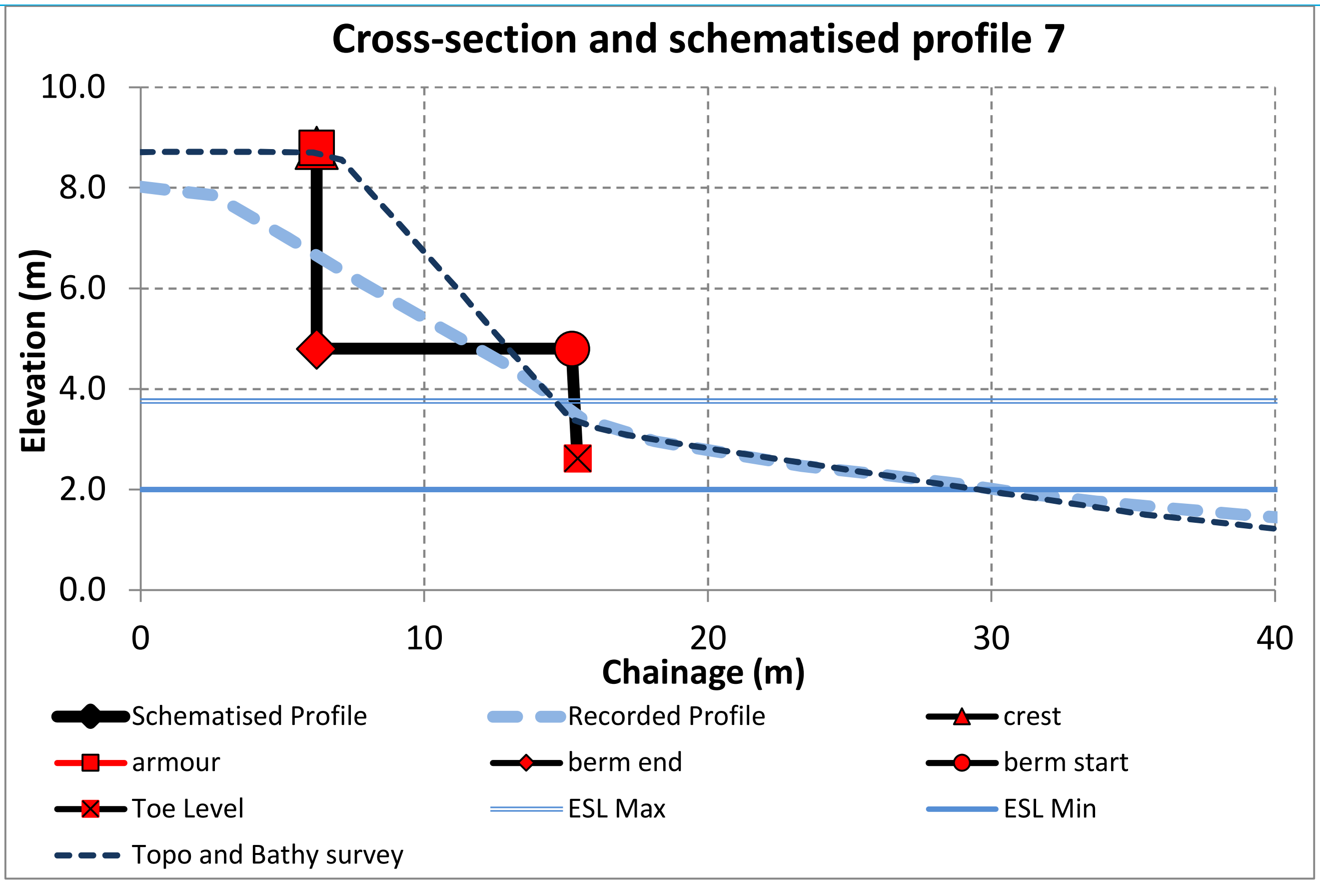

Figure A13: Bathymetry profiles and Neural Network schematisation for defence 7

Figure A14: Defence image

Table A7: Neural Network parameters for defence 7

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	2.62	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	2.62	Crest level (mAOD)	8.80
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	299.29
Slope downward of Berm (cotangent), cot ad	0.09	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	4.80	Break in Roughness 1	4.00
Width of Berm (m), B	9.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	8.80	Roughness 3	1.00

Defence 8

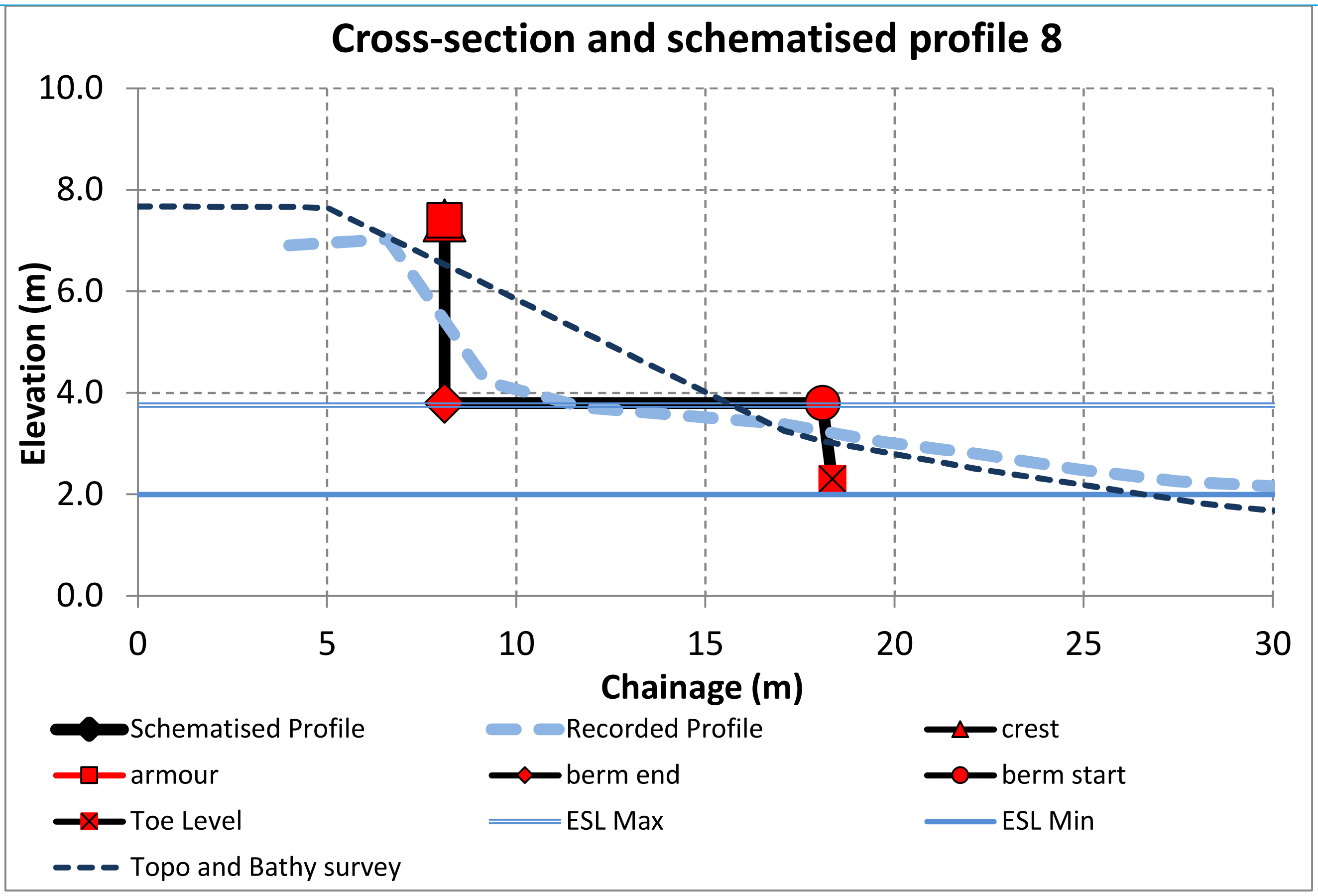

Figure A15: Bathymetry profiles and Neural Network schematisation for defence 8

Figure A16: Defence image

Table A8: Neural Network parameters for defence 8

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	2.30	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	2.30	Crest level (mAOD)	7.40
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	305.92
Slope downward of Berm (cotangent), cot ad	0.17	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	3.80	Break in Roughness 1	4.00
Width of Berm (m), B	9.99	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	7.40	Roughness 3	1.00

Defence 9

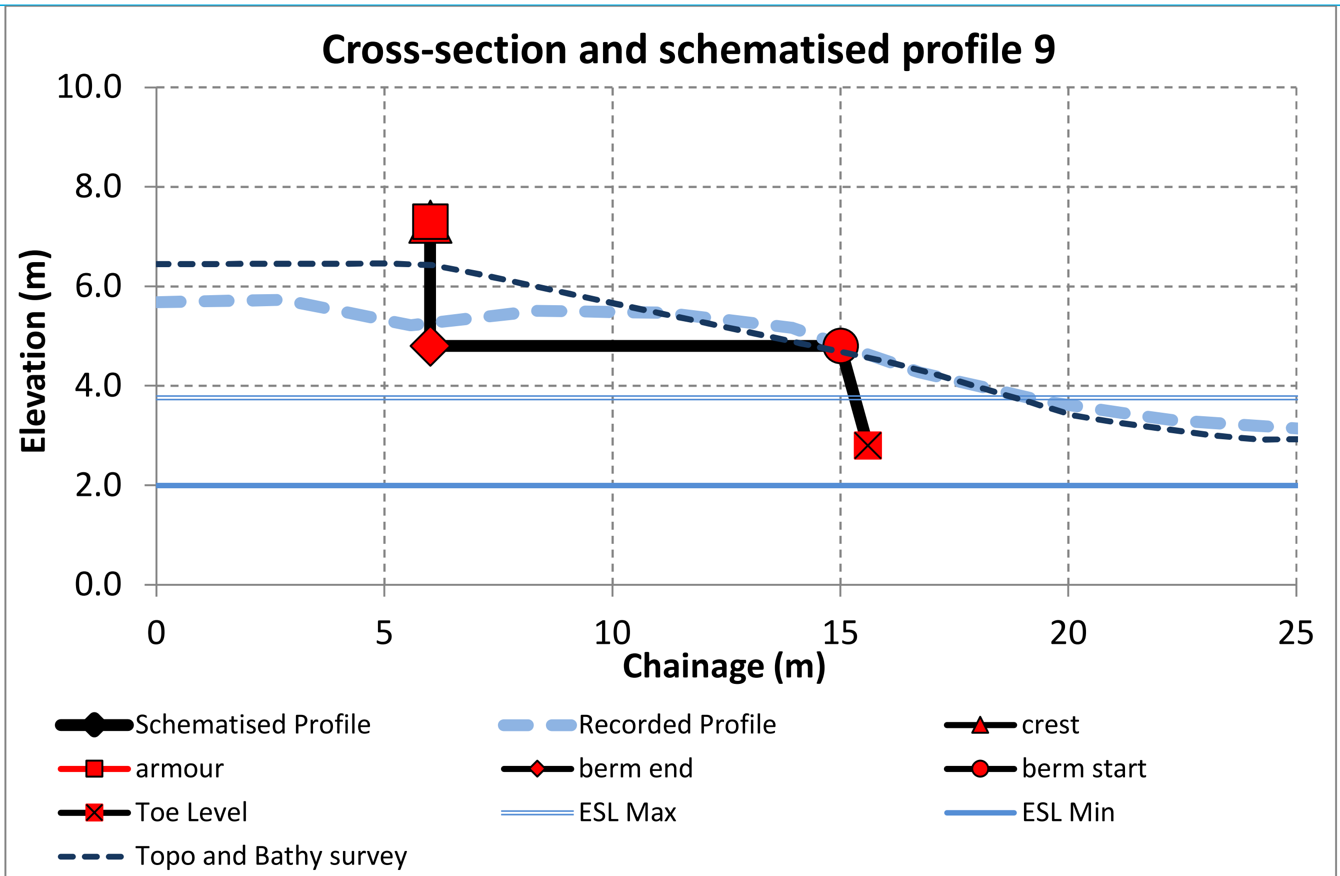

Figure A17: Bathymetry profiles and Neural Network schematisation for defence 9

Figure A18: Defence image

Table A9: Neural Network parameters for defence 9

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	2.80	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	2.80	Crest level (mAOD)	7.30
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	309.92
Slope downward of Berm (cotangent), cot ad	0.30	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	4.80	Break in Roughness 1	7.30
Width of Berm (m), B	9.00	Break in Roughness 2	2.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	7.30	Roughness 3	1.00

Defence 10

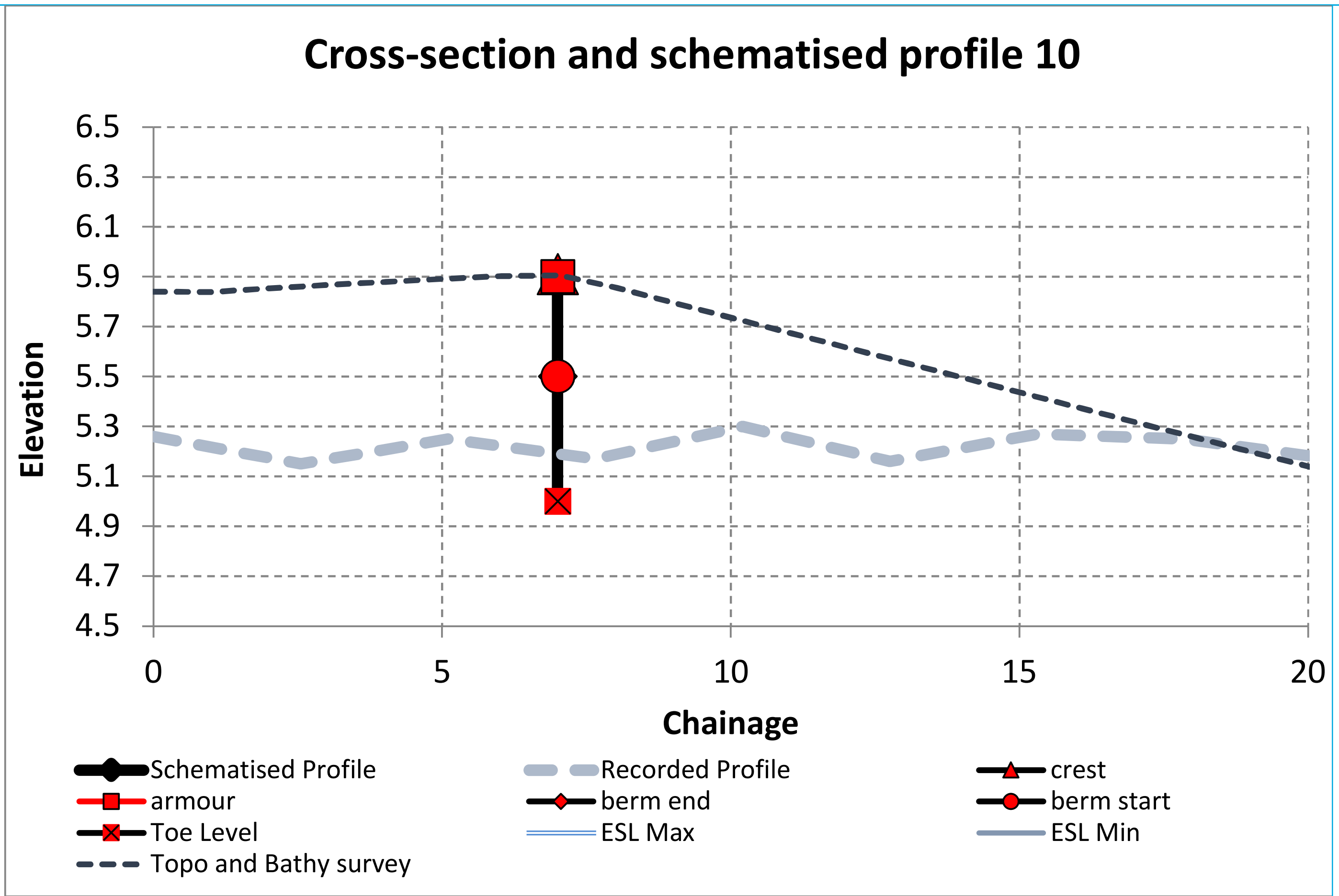

Figure A19: Bathymetry profiles and Neural Network schematisation for defence 10

Figure A20: Defence image

Table A10: Neural Network parameters for defence 10

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	5.00	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	5.00	Crest level (mAOD)	5.91
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	323.80
Slope downward of Berm (cotangent), cot ad	0.00	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	-9.00	Break in Roughness 1	5.90
Width of Berm (m), B	0.01	Break in Roughness 2	4.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	5.70	Roughness 3	1.00

Defence 11

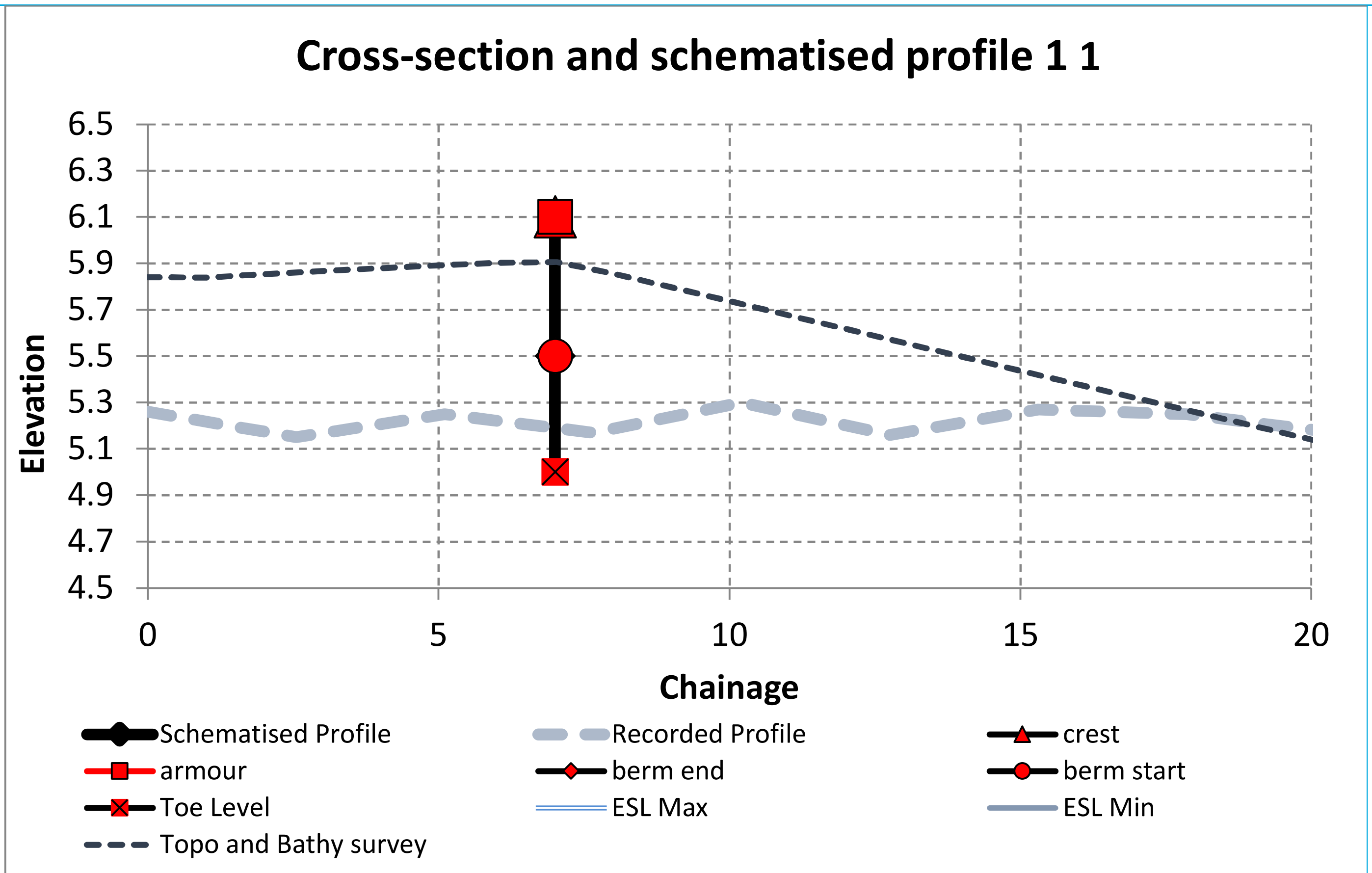

Figure A21: Bathymetry profiles and Neural Network schematisation for defence 11

Figure A22: Defence image

Table A11: Neural Network parameters for defence 11

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	5.00	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	5.00	Crest level (mAOD)	6.10
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	19.67
Slope downward of Berm (cotangent), cot ad	0.00	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	-9.00	Break in Roughness 1	3.00
Width of Berm (m), B	0.01	Break in Roughness 2	2.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	6.10	Roughness 3	1.00

Defence 12

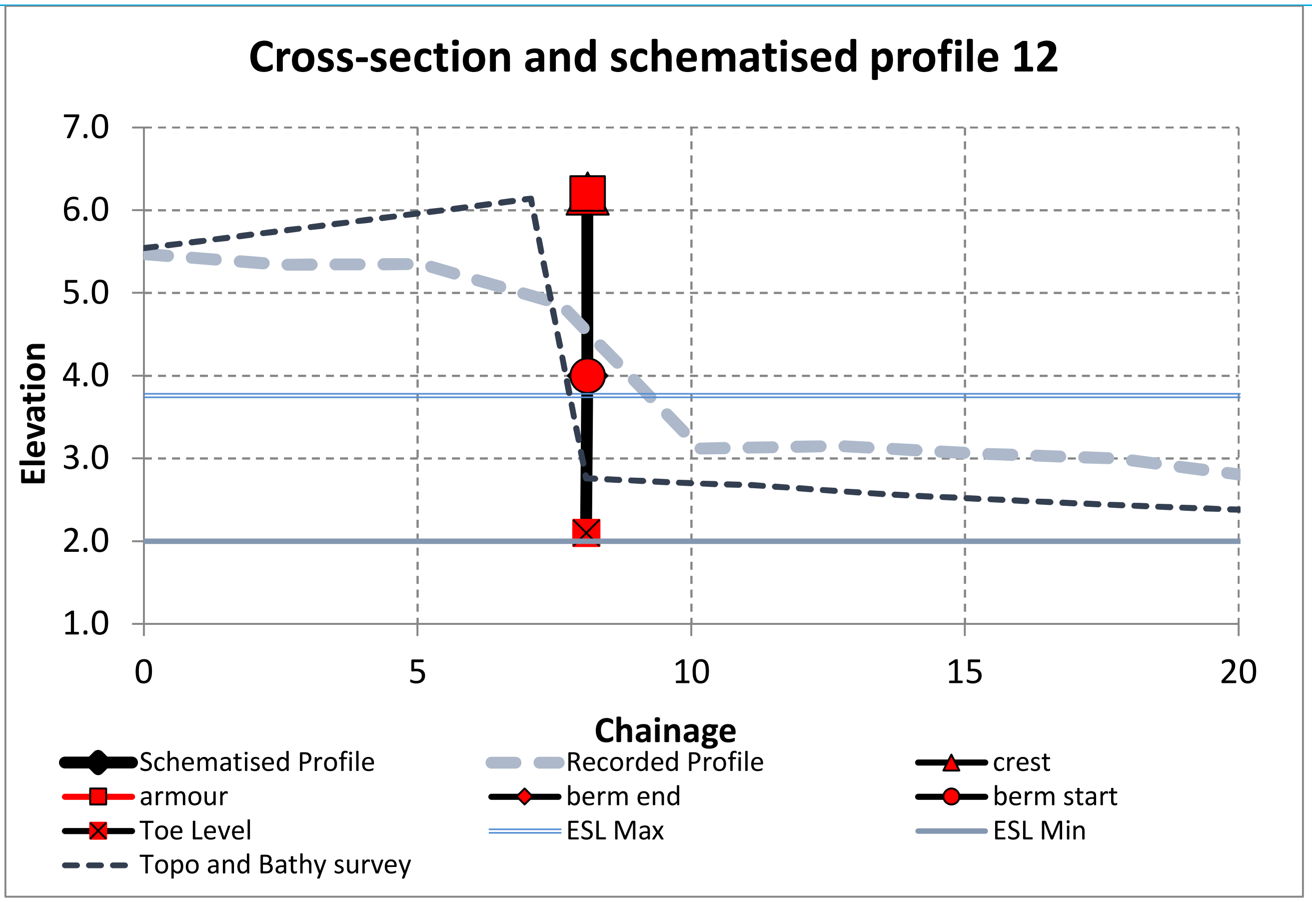

Figure A23: Bathymetry profiles and Neural Network schematisation for defence 12

Figure A24: Defence image

Table A12: Neural Network parameters for defence 12

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	2.10	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	2.10	Crest level (mAOD)	6.20
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	37.95
Slope downward of Berm (cotangent), cot ad	0.01	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	-9.00	Break in Roughness 1	4.00
Width of Berm (m), B	0.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.01	Roughness 2	1.00
Armour crest level (mAOD)	6.20	Roughness 3	1.00

Defence 13

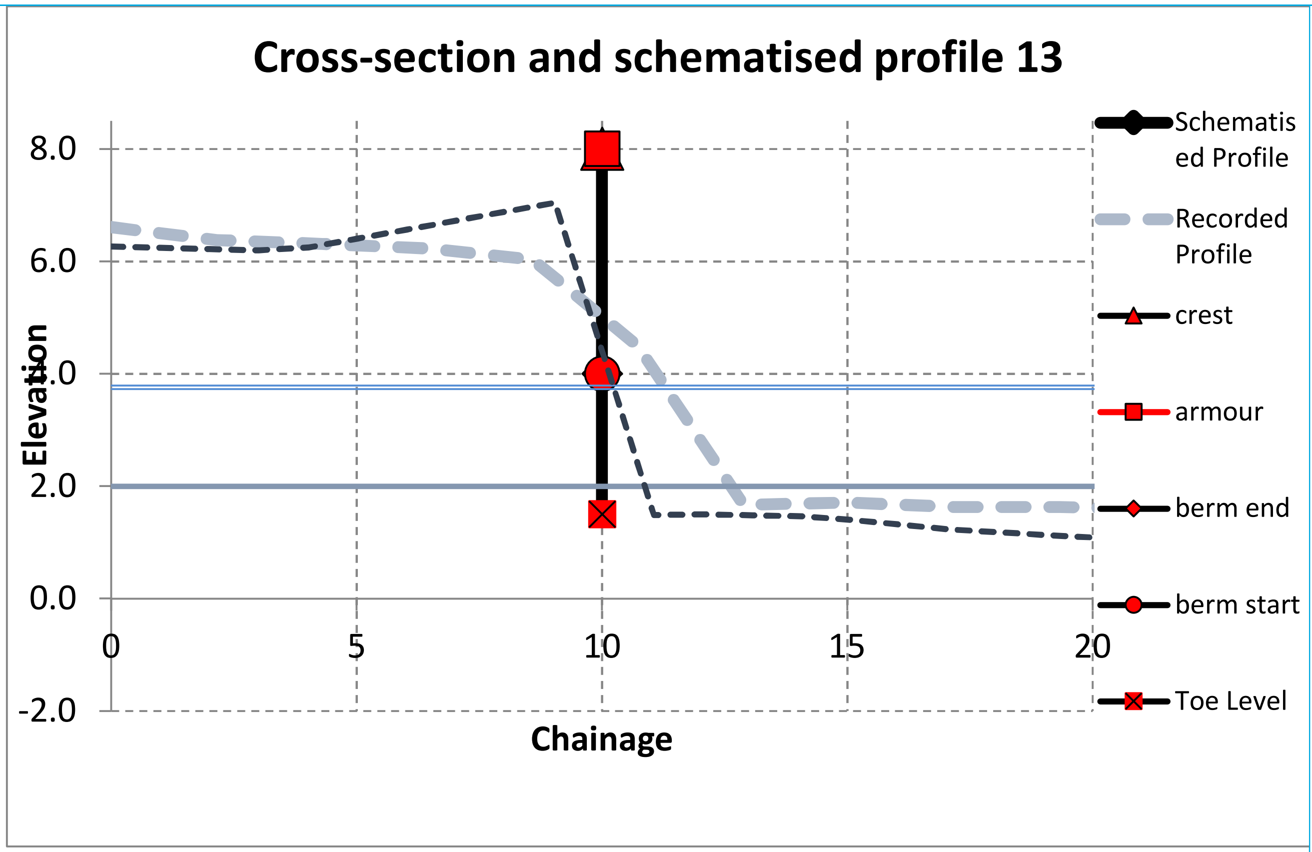

Figure A25: Bathymetry profiles and Neural Network schematisation for defence 13

Figure A26: Defence image

Table A13: Neural Network parameters for defence 13

Defence's parameters (Neural Network inputs)	Value	Defence's parameters (Neural Network inputs)	Value
Front of toe level (mAOD)	1.50	Width of Armour crest (m), Gc	0.00
Toe level (mAOD)	1.50	Crest level (mAOD)	8.00
Width of toe (m), Bt	0.00	Normal angle of defence (degrees from North)	95.09
Slope downward of Berm (cotangent), cot ad	0.00	Wave return wall (yes=1, No=0)	0
Berm level (mAOD)	-9.00	Break in Roughness 1	4.00
Width of Berm (m), B	0.00	Break in Roughness 2	3.00
Slope of Berm (tangent), tan aB	0.00	Roughness 1	1.00
Slope upward of Berm (cotangent), cot au	0.00	Roughness 2	1.00
Armour crest level (mAOD)	8.00	Roughness 3	1.00

A.2 Worst case conditions tables and wave setup discussion

This section provides for each defence the shoreline wave climate and water level conditions as used for the overtopping discharges used in the flood inundation modelling and taken from the Kilkee Coastal Wave & Water Level Modelling Study (KilkeeCWWS).

For each AEP event, the KilkeeCWWS provides six combinations of wave and water levels conditions. The condition number in the worst-case tables below refers to the joint probability combination that led to the worst-case overtopping. For example, in Table A 14 the 50% AEP worst case overtopping rate related to the first joint probability combination from the CAPO H present day data. The worst-case results tended to be the first or second combination (which generally related to the highest water levels and wave conditions being paired together). As each overtopping defence section is considered in isolation, the worst-case combination can vary as described. The 0.5% overtopping rate can be generated by different water level and wave conditions at neighbouring defence sections, which would not be expected to occur at the same time, or during the same event. This approach is a conservative approach, designed to determine a specific AEP overtopping rate for each defence section and provide a worst-case flood risk along the coastal frontage for a given AEP event. In reality a coastal event may not lead to all defences experiencing the modelled impacts at the same time. To inform defence standards and design it is necessary to consider the worst-case at each defence and adopt a sensibly conservative approach to flood risk and resultant mapping.

It is important to note that the water levels in the joint probability tables, and therefore used in the wave overtopping modelling, includes an uplift for wave setup.

The specific morphology of Kilkee bay produces a wave set-up within the bay, i.e., the mean water levels increase when getting closer to the coastline defences. The Kilkee Wave and Water Level Modelling Study simulated storm events using both a Coupled Model and an Uncoupled Model, to determine the impact the wave climate had on the water level i.e. the wave set-up. It was found that under certain conditions a range of wave set-up values were possible for the various AEP events. The wave setup was determined based on the 280° directional sector as this provided the worst-case wave conditions. The resultant wave set-up has been included in the final wave model joint probability simulations for all AEPs (present and future). Due to the significant effects of wave set-up in the Bay, when the input joint probability wave heights are at their largest, the set-up is at its largest, which results in the largest waves and water levels being paired together.

Due to the significant wave setup, as the flood inundation water level boundary the highest extreme water level across all CAPO locations for the AEP event being modelled was used. Note; this was considerably higher than the equivalent ICWWS 2018 extreme sea levels due to wave setup. For example, the 0.5% AEP wave setup value used was 4.50mOD Malin while the 0.5% AEP ICWWS level is 3.25mOD Malin.

A.2.1 Defence 1 – CAPO H

Table A 1 Wave and water level conditions and associated overtopping rate for defence 1

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (mOD)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.06	16.05	265	3.00	0.0009
20	1	1.10	17.70	265	3.27	0.0022
10	1	1.16	17.91	266	3.55	0.0059
5	2	1.15	17.78	266	3.87	0.0120
2	2	1.20	17.92	267	4.22	0.0300
1	2	1.26	18.80	267	4.40	0.0497
0.5	2	1.31	20.01	268	4.56	0.0741
0.1	3	1.36	20.01	268	4.99	0.1581
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.08	16.05	266	3.50	0.0034
20	1	1.13	17.70	266	3.77	0.0087
10	2	1.19	17.91	267	4.05	0.0206
5	2	1.20	17.78	267	4.37	0.0399
2	1	1.26	17.92	268	4.72	0.0882
1	1	1.32	18.79	268	4.90	0.1313
0.5	1	1.38	20.01	269	5.06	0.1662
0.1	3	1.48	22.33	269	5.36	0.1738
HIGH-END FUTURE SCENARIO						
50	1	1.13	16.05	267	4.00	0.0146
20	1	1.19	17.70	267	4.27	0.0321
10	2	1.19	16.06	268	4.65	0.0670
5	2	1.28	17.78	268	4.87	0.1158
2	1	1.39	19.88	268	5.06	0.1673
1	1	1.43	20.04	269	5.23	0.1699
0.5	1	1.47	20.16	269	5.39	0.1719
0.1	3	1.56	20.01	270	5.99	0.4252

A.2.2 Defence 2 – CAPO H

Table A 2 Wave and water level conditions and associated overtopping rate for defence 2

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.06	16.05	265	3.00	0.0001
20	1	1.10	17.70	265	3.27	0.0003
10	1	1.16	17.91	266	3.55	0.0010
5	2	1.15	17.78	266	3.87	0.0023
2	2	1.20	17.92	267	4.22	0.0084
1	2	1.26	18.80	267	4.40	0.0182
0.5	2	1.31	20.01	268	4.56	0.0323
0.1	2	1.40	21.03	268	4.95	0.0814
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.08	16.05	266	3.50	0.0005
20	1	1.13	17.70	266	3.77	0.0014
10	1	1.19	17.91	267	4.05	0.0050
5	2	1.20	17.78	267	4.37	0.0122
2	2	1.26	17.92	268	4.72	0.0362
1	2	1.32	18.79	268	4.90	0.0622
0.5	2	1.38	20.01	269	5.06	0.0907
0.1	1	1.48	21.02	269	5.45	0.1618
HIGH-END FUTURE SCENARIO						
50	1	1.13	16.05	267	4.00	0.0027
20	1	1.19	17.70	267	4.27	0.0090
10	1	1.26	17.91	268	4.55	0.0256
5	2	1.28	17.78	268	4.87	0.0507
2	2	1.35	17.92	269	5.22	0.1010
1	2	1.42	18.78	269	5.40	0.1391
0.5	2	1.48	20.01	269	5.56	0.1596
0.1	1	1.60	22.33	270	5.86	0.1726

A.2.3 Defence 3 – CAPO G

Table A 3 Wave and water level conditions and associated overtopping rate for defence 3

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.20	16.05	272	3.00	0.0000
20	1	1.29	17.70	272	3.27	0.0000
10	1	1.38	17.91	273	3.55	0.0000
5	1	1.42	18.07	273	3.73	0.0000
2	1	1.51	19.88	273	4.06	0.0000
1	2	1.51	18.80	274	4.40	0.0044
0.5	2	1.57	20.01	274	4.56	0.0070
0.1	2	1.65	21.04	274	4.95	0.0163
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.29	16.05	273	3.50	0.0000
20	1	1.36	17.70	273	3.77	0.0000
10	1	1.44	17.91	273	4.05	0.0000
5	2	1.44	17.78	274	4.37	0.0030
2	2	1.56	19.88	274	4.56	0.0069
1	2	1.57	18.79	274	4.90	0.0113
0.5	2	1.63	20.01	275	5.06	0.0174
0.1	2	1.73	21.02	275	5.45	0.0393
HIGH-END FUTURE SCENARIO						
50	1	1.36	16.05	273	4.00	0.0000
20	1	1.43	17.70	274	4.27	0.0000
10	1	1.51	17.91	274	4.55	0.0054
5	2	1.51	17.78	274	4.87	0.0086
2	2	1.58	17.92	275	5.22	0.0182
1	2	1.65	18.79	275	5.40	0.0297
0.5	2	1.72	20.01	275	5.56	0.0441
0.1	2	1.82	21.02	276	5.95	0.0908

A.2.4 Defence 4 – CAPO F

Table A 4 Wave and water level conditions and associated overtopping rate for defence 4

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.39	16.04	283	3.00	0.0000
20	2	1.48	16.04	283	3.29	0.0001
10	2	1.57	16.06	284	3.65	0.0007
5	2	1.69	17.77	284	3.87	0.0015
2	2	1.78	17.92	284	4.22	0.0032
1	2	1.87	18.79	285	4.40	0.0053
0.5	2	1.95	20.00	285	4.56	0.0082
0.1	2	2.07	21.03	285	4.95	0.0194
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.54	16.04	284	3.50	0.0004
20	1	1.65	17.69	284	3.77	0.0011
10	1	1.76	17.90	284	4.05	0.0024
5	2	1.78	17.78	285	4.37	0.0040
2	2	1.87	17.92	285	4.72	0.0088
1	2	1.96	18.78	285	4.90	0.0145
0.5	2	2.04	20.00	285	5.06	0.0218
0.1	2	2.17	21.01	286	5.45	0.0477
HIGH-END FUTURE SCENARIO						
50	1	1.67	16.04	284	4.00	0.0017
20	1	1.77	17.69	284	4.27	0.0033
10	1	1.88	17.90	285	4.55	0.0067
5	2	1.89	17.78	285	4.87	0.0117
2	2	1.98	17.92	285	5.22	0.0248
1	2	2.08	18.77	286	5.40	0.0382
0.5	2	2.16	20.01	286	5.56	0.0546
0.1	2	2.29	21.01	286	5.95	0.1073

A.2.5 Defence 5 – CAPO F

Table A 5 Wave and water level conditions and associated overtopping rate for defence 5

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.39	16.04	283	3.00	0.0000
20	2	1.48	16.04	283	3.29	0.0002
10	2	1.57	16.06	284	3.65	0.0009
5	2	1.69	17.77	284	3.87	0.0021
2	2	1.78	17.92	284	4.22	0.0052
1	2	1.87	18.79	285	4.40	0.0090
0.5	2	1.95	20.00	285	4.56	0.0143
0.1	2	2.07	21.03	285	4.95	0.0334
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.54	16.04	284	3.50	0.0006
20	1	1.65	17.69	284	3.77	0.0015
10	1	1.76	17.90	284	4.05	0.0036
5	2	1.78	17.78	285	4.37	0.0065
2	2	1.87	17.92	285	4.72	0.0148
1	2	1.96	18.78	285	4.90	0.0244
0.5	2	2.04	20.00	285	5.06	0.0364
0.1	2	2.17	21.01	286	5.45	0.0767
HIGH-END FUTURE SCENARIO						
50	1	1.67	16.04	284	4.00	0.0024
20	1	1.77	17.69	284	4.27	0.0052
10	1	1.88	17.90	285	4.55	0.0115
5	2	1.89	17.78	285	4.87	0.0194
2	2	1.98	17.92	285	5.22	0.0397
1	2	2.08	18.77	286	5.40	0.0610
0.5	2	2.16	20.01	286	5.56	0.0860
0.1	2	2.29	21.01	286	5.95	0.1611

A.2.6 Defence 6 – CAPO E

Table A 6 Wave and water level conditions and associated overtopping rate for defence 1

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.69	16.03	310	3.00	0.0000
20	1	1.86	17.68	311	3.27	0.0000
10	1	2.01	17.89	311	3.55	0.0000
5	1	2.11	18.06	311	3.73	0.0000
2	3	2.12	16.06	311	4.26	0.0005
1	3	2.27	17.78	311	4.43	0.0012
0.5	3	2.39	17.99	311	4.59	0.0024
0.1	3	2.62	20.00	311	4.99	0.0117
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.89	16.03	311	3.50	0.0000
20	2	2.04	17.68	311	3.77	0.0000
10	2	2.09	16.05	311	4.15	0.0003
5	3	2.11	15.97	311	4.38	0.0009
2	3	2.24	16.06	311	4.76	0.0048
1	3	2.41	17.78	311	4.93	0.0095
0.5	3	2.53	17.99	311	5.09	0.0172
0.1	3	2.77	20.00	311	5.49	0.0619
HIGH-END FUTURE SCENARIO						
50	1	2.07	16.04	311	4.00	0.0000
20	2	2.16	16.03	311	4.29	0.0006
10	2	2.26	16.05	311	4.65	0.0030
5	3	2.27	15.97	311	4.88	0.0078
2	3	2.40	16.06	311	5.26	0.0290
1	3	2.57	17.78	311	5.43	0.0529
0.5	3	2.70	17.99	311	5.59	0.0823
0.1	3	2.95	20.00	311	5.99	0.2040

A.2.7 Defence 7 – CAPO E

Table A 7 Wave and water level conditions and associated overtopping rate for defence 7

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.69	16.03	310	3.00	0.0000
20	1	1.86	17.68	311	3.27	0.0000
10	1	2.01	17.89	311	3.55	0.0000
5	1	2.11	18.06	311	3.73	0.0000
2	3	2.12	16.06	311	4.26	0.0002
1	3	2.27	17.78	311	4.43	0.0004
0.5	3	2.39	17.99	311	4.59	0.0008
0.1	3	2.62	20.00	311	4.99	0.0039
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.89	16.03	311	3.50	0.0000
20	2	2.04	17.68	311	3.77	0.0000
10	2	2.09	16.05	311	4.15	0.0001
5	3	2.11	15.97	311	4.38	0.0003
2	3	2.24	16.06	311	4.76	0.0016
1	3	2.41	17.78	311	4.93	0.0032
0.5	3	2.53	17.99	311	5.09	0.0059
0.1	3	2.77	20.00	311	5.49	0.0226
HIGH-END FUTURE SCENARIO						
50	1	2.07	16.04	311	4.00	0.0000
20	2	2.16	16.03	311	4.29	0.0002
10	2	2.26	16.05	311	4.65	0.0010
5	3	2.27	15.97	311	4.88	0.0026
2	3	2.40	16.06	311	5.26	0.0105
1	3	2.57	17.78	311	5.43	0.0191
0.5	3	2.70	17.99	311	5.59	0.0307
0.1	3	2.95	20.00	311	5.99	0.0795

A.2.8 Defence 8 – CAPO E

Table A 8 Wave and water level conditions and associated overtopping rate for defence 8

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.69	16.03	310	3.00	0.0000
20	1	1.86	17.68	311	3.27	0.0000
10	2	1.94	16.05	311	3.65	0.0001
5	3	1.98	15.97	311	3.88	0.0004
2	3	2.12	16.06	311	4.26	0.0029
1	3	2.27	17.78	311	4.43	0.0066
0.5	3	2.39	17.99	311	4.59	0.0126
0.1	3	2.62	20.00	311	4.99	0.0464
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.89	16.03	311	3.50	0.0000
20	2	1.98	16.03	311	3.79	0.0003
10	2	2.09	16.05	311	4.15	0.0017
5	3	2.11	15.97	311	4.38	0.0052
2	3	2.24	16.06	311	4.76	0.0230
1	3	2.41	17.78	311	4.93	0.0397
0.5	3	2.53	17.99	311	5.09	0.0611
0.1	3	2.77	20.00	311	5.49	0.1413
HIGH-END FUTURE SCENARIO						
50	1	2.07	16.04	311	4.00	0.0008
20	2	2.16	16.03	311	4.29	0.0035
10	2	2.26	16.05	311	4.65	0.0157
5	3	2.27	15.97	311	4.88	0.0340
2	3	2.40	16.06	311	5.26	0.0887
1	3	2.57	17.78	311	5.43	0.1281
0.5	3	2.70	17.99	311	5.59	0.1684
0.1	3	2.95	20.00	311	5.99	0.2871

A.2.9 Defence 9 – CAPO E

Table A 9 Wave and water level conditions and associated overtopping rate for defence 9

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.69	16.03	310	3.00	0.0000
20	1	1.86	17.68	311	3.27	0.0000
10	1	2.01	17.89	311	3.55	0.0000
5	1	2.11	18.06	311	3.73	0.0000
2	3	2.12	16.06	311	4.26	0.0005
1	3	2.27	17.78	311	4.43	0.0014
0.5	3	2.39	17.99	311	4.59	0.0032
0.1	3	2.62	20.00	311	4.99	0.0186
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.89	16.03	311	3.50	0.0000
20	2	2.04	17.68	311	3.77	0.0000
10	2	2.09	16.05	311	4.15	0.0003
5	3	2.11	15.97	311	4.38	0.0010
2	3	2.24	16.06	311	4.76	0.0072
1	3	2.41	17.78	311	4.93	0.0151
0.5	3	2.53	17.99	311	5.09	0.0270
0.1	3	2.77	20.00	311	5.49	0.0810
HIGH-END FUTURE SCENARIO						
50	1	2.07	16.04	311	4.00	0.0001
20	2	2.16	16.03	311	4.29	0.0006
10	2	2.26	16.05	311	4.65	0.0043
5	3	2.27	15.97	311	4.88	0.0123
2	3	2.40	16.06	311	5.26	0.0452
1	3	2.57	17.78	311	5.43	0.0713
0.5	3	2.70	17.99	311	5.59	0.1015
0.1	3	2.95	20.00	311	5.99	0.2012

A.2.10 Defence 10 – CAPO D

Table A 10 Wave and water level conditions and associated overtopping rate for defence 10

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.70	16.04	331	3.00	0.0000
20	1	1.89	17.69	331	3.27	0.0000
10	1	2.07	17.90	330	3.55	0.0000
5	1	2.18	18.06	330	3.73	0.0000
2	1	2.39	19.87	330	4.06	0.0000
1	1	2.48	20.03	330	4.23	0.0000
0.5	1	2.57	20.15	330	4.39	0.0000
0.1	1	2.83	22.33	330	4.86	0.0000
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.93	16.04	330	3.50	0.0000
20	1	2.11	17.69	330	3.77	0.0000
10	1	2.28	17.90	330	4.05	0.0000
5	1	2.39	18.06	330	4.23	0.0000
2	3	2.59	19.87	330	4.56	0.0000
1	3	2.68	20.03	330	4.73	0.0000
0.5	3	2.77	20.15	330	4.89	0.0000
0.1	3	2.93	20.00	330	5.49	0.0055
HIGH-END FUTURE SCENARIO						
50	1	2.15	16.04	330	4.00	0.0000
20	1	2.33	17.69	330	4.27	0.0000
10	1	2.49	17.90	330	4.55	0.0000
5	1	2.60	18.06	330	4.73	0.0000
2	3	2.52	16.06	329	5.26	0.0001
1	3	2.71	17.78	329	5.43	0.0025
0.5	3	2.85	17.99	329	5.59	0.0161
0.1	3	3.12	20.00	329	5.99	0.0991

A.2.11 Defence 11 – CAPO C

Table A 11 Wave and water level conditions and associated overtopping rate for defence 11

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.56	16.04	340	3.00	0.0000
20	1	1.74	17.69	340	3.27	0.0000
10	1	1.90	17.90	340	3.55	0.0000
5	1	2.01	18.06	340	3.73	0.0000
2	1	2.20	19.88	340	4.06	0.0000
1	1	2.29	20.04	340	4.23	0.0000
0.5	1	2.37	20.16	340	4.39	0.0000
0.1	1	2.62	22.33	340	4.86	0.0000
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.78	16.04	340	3.50	0.0000
20	1	1.95	17.69	340	3.77	0.0000
10	1	2.10	17.90	340	4.05	0.0000
5	1	2.21	18.06	340	4.23	0.0000
2	3	2.39	19.88	340	4.56	0.0000
1	3	2.48	20.04	340	4.73	0.0000
0.5	3	2.56	20.16	340	4.89	0.0000
0.1	3	2.71	20.01	339	5.49	0.0025
HIGH-END FUTURE SCENARIO						
50	1	1.98	16.04	340	4.00	0.0000
20	1	2.15	17.69	340	4.27	0.0000
10	1	2.30	17.90	340	4.55	0.0000
5	1	2.40	18.07	340	4.73	0.0000
2	3	2.32	16.06	339	5.26	0.0000
1	3	2.50	17.78	339	5.43	0.0010
0.5	3	2.63	17.99	339	5.59	0.0095
0.1	3	2.88	20.01	339	5.99	0.0551

A.2.12 Defence 12 – CAPO B

Table A 12 Wave and water level conditions and associated overtopping rate for defence 12

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.28	16.04	13	3.00	0.0001
20	2	1.39	16.04	11	3.29	0.0008
10	2	1.51	16.06	10	3.65	0.0055
5	3	1.55	15.98	9	3.88	0.0158
2	3	1.66	16.07	8	4.26	0.0585
1	3	1.80	17.79	8	4.43	0.0950
0.5	2	1.98	20.01	8	4.56	0.1324
0.1	2	2.15	21.04	7	4.95	0.2456
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.46	16.05	11	3.50	0.0026
20	2	1.54	16.04	10	3.79	0.0107
10	1	1.63	16.06	8	4.15	0.0435
5	2	1.78	17.78	8	4.37	0.0838
2	1	1.90	17.92	7	4.72	0.1488
1	1	2.01	18.80	7	4.90	0.2021
0.5	1	2.11	20.01	6	5.06	0.2551
0.1	1	2.29	22.33	6	5.36	0.2963
HIGH-END FUTURE SCENARIO						
50	1	1.62	16.05	9	4.00	0.0259
20	2	1.69	16.04	8	4.29	0.0646
10	1	1.88	17.91	7	4.55	0.1235
5	2	1.92	17.78	6	4.87	0.1739
2	1	2.11	19.88	6	5.06	0.2547
1	1	2.18	20.04	6	5.23	0.2794
0.5	1	2.25	20.16	6	5.39	0.2864
0.1	1	2.45	22.33	5	5.86	0.3112

A.2.13 Defence 13 – CAPO A

Table A 13 Wave and water level conditions and associated overtopping rate for defence 13

AEP event (%)	Condition number	Hm0 (m)	Tp (s)	Mean wave direction (°)	Water level (m)	Overtopping rate ($m^3/m/s$)
PRESENT-DAY						
50	1	1.05	16.05	24	3.00	0.0000
20	1	1.18	17.70	24	3.27	0.0001
10	1	1.29	17.91	23	3.55	0.0004
5	2	1.35	17.78	22	3.87	0.0008
2	2	1.45	17.92	22	4.22	0.0021
1	2	1.54	18.81	22	4.40	0.0040
0.5	2	1.61	20.01	21	4.56	0.0068
0.1	2	1.74	21.05	21	4.95	0.0174
MEDIUM-RANGE FUTURE SCENARIO						
50	1	1.20	16.05	23	3.50	0.0002
20	1	1.31	17.70	23	3.77	0.0006
10	1	1.41	17.91	22	4.05	0.0014
5	2	1.45	17.78	21	4.37	0.0026
2	2	1.55	17.92	21	4.72	0.0065
1	2	1.64	18.80	20	4.90	0.0116
0.5	2	1.72	20.01	20	5.06	0.0183
0.1	2	1.85	21.04	20	5.45	0.0410
HIGH-END FUTURE SCENARIO						
50	1	1.33	16.05	22	4.00	0.0008
20	1	1.44	17.70	22	4.27	0.0021
10	1	1.53	17.91	21	4.55	0.0048
5	2	1.57	17.78	20	4.87	0.0085
2	2	1.67	17.92	20	5.22	0.0187
1	2	1.76	18.80	20	5.40	0.0304
0.5	2	1.84	20.01	19	5.56	0.0443
0.1	2	1.98	21.04	19	5.95	0.0850

B Appendix B – Design surge shape

A design surge shape was calculated based on the analysis of multiple past surge events for use in the generation of tidal water level time series curves at Kilkee.

The tide observations and predictions were taken from the Inishmore station that is located North of Kilkee and is the closest station with both predicted and observed tides.

Six past events were identified from the Inishmore tide gauge and used for the creation of the design surge. They correspond to the following surge events:

- 8th of February 2019
- 30th of October 2019
- 3rd of October 2020
- 3rd of December 2020
- 25th of December 2020
- 7th of February 2021.

The creation of the design surge shape was based on the following steps:

- For each event, surge residuals were identified by subtracting the predicted tide level (taken from Admiralty TotalTide at Inishmore) from the observed tide levels as recorded at the Inishmore tide gauge.
- The residuals were smoothed over 2 hours using a moving average method.
- The positive residuals were normalised with the peak of the surge set to a value of 1.
- The peaks of the normalised residuals were aligned for the six events.
- The normalised residuals were averaged over the six events to give the design surge (Figure B 1).

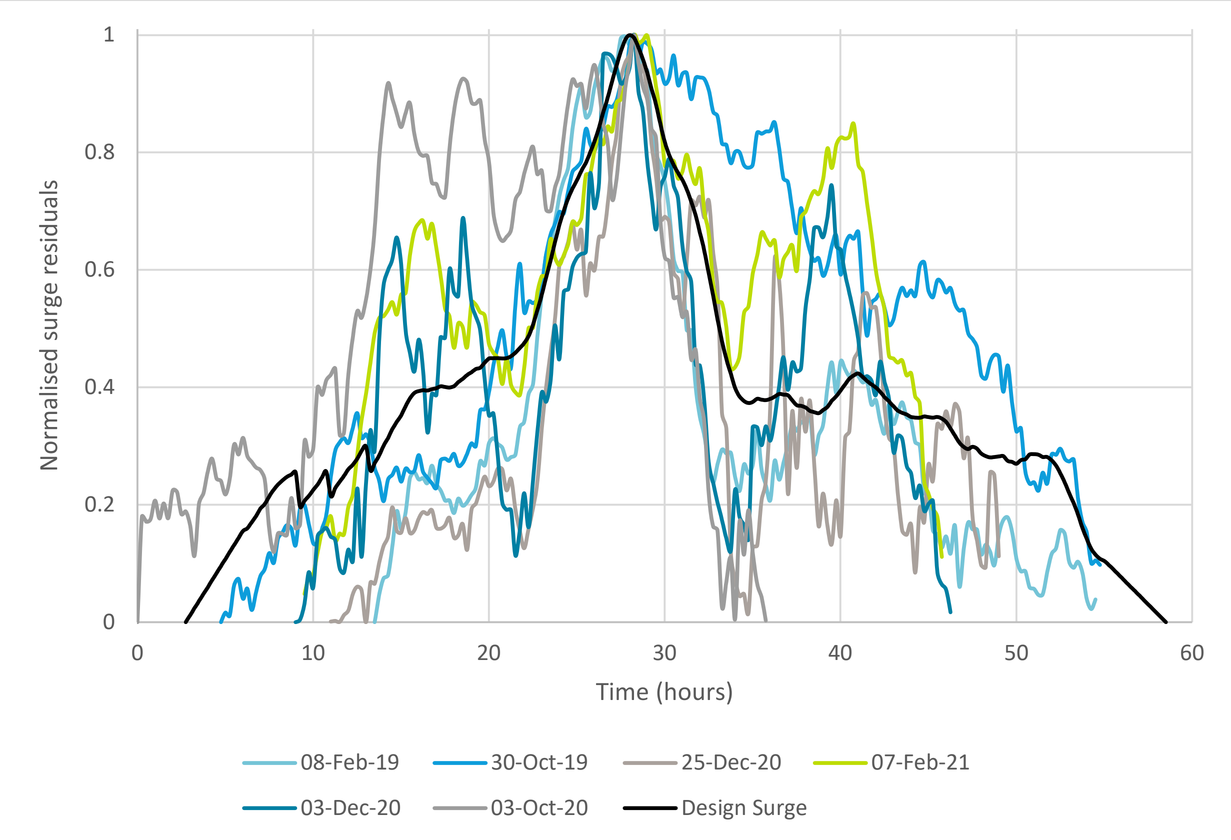

Figure B 1 Normalised surge residuals for the six past events and averaged design surge (black) at the Inishmore tide gauge

There is a station in Doonbeg, closer to Kilkee than Inishmore that has tide observations over the last couple of years, but no predictions. As a comparison, the same method was applied with observed tidal data recorded at the Doonbeg gauge and predicted tidal data at Inishmore from Admiralty TotalTide. The results of both analyses produced similar design surge shapes. For consistency between the gauge stations, the design surge shape created based on the observed and predicted tidal data at Inishmore was used in the generation of design tidal water level time series curves.

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