Lochgilphead Flood Study

Phase 2: Baseline Modelling

06 December 2019

Quality information

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1. Introduction

AECOM have been commissioned by Argyll and Bute Council to undertake a Flood Study for the town of Lochgilphead. This report outlines the work undertaken for Phase 2 (see list below).

The project will be undertaken using a phased approach, and includes the following main tasks:

- Phase 1 – Data review and gap analysis
- Phase 2 – Baseline existing flood conditions
- Phase 3 – Long list to short list selection
- Phase 4 – Option development and modelling

Phase 2 of the study, which is detailed in this report, contains the following elements:

- Background of the project
- Historic flooding
- Canal flooding assessment
- Coastal modelling assessment
- Fluvial and coastal baseline flooding assessment
- Recommended next steps

2. Project Background

The study area is outlined in Figure 2-1 below and encompasses the town of Lochgilphead and the A816 to the north of the town. The purpose of this study is to identify the areas at risk from tidal and fluvial flooding (which also includes potential inflow from the Crinan Canal) during the current day and climate change scenarios.

The main fluvial risk in the study area is from the Badden Burn which runs alongside the A816, diverting east upstream of the Meadows area, before returning to run through the town. A stretch of the Cuilarstich Burn, which joins the Badden Burn in Lochgilphead, has also been assessed. This watercourse sits in a well-defined channel upstream of Bishopton Road and the assessment of flooding was cropped to this location. The Peddie Burn, that is partially culverted through the western side of Lochgilphead, is outwith the scope of this study. The Crinan Canal runs along the western edge of the study area. An overflow weir is located between Cairnbaan and Lochgilphead which discharges into the Badden Burn.

The southern section of Lochgilphead, at the Front Green and along Poltalloch Street, faces an intertidal area and is likely to be susceptible to coastal flooding.

Further background data on the study and Lochgilphead can be found in the Phase 1 report.

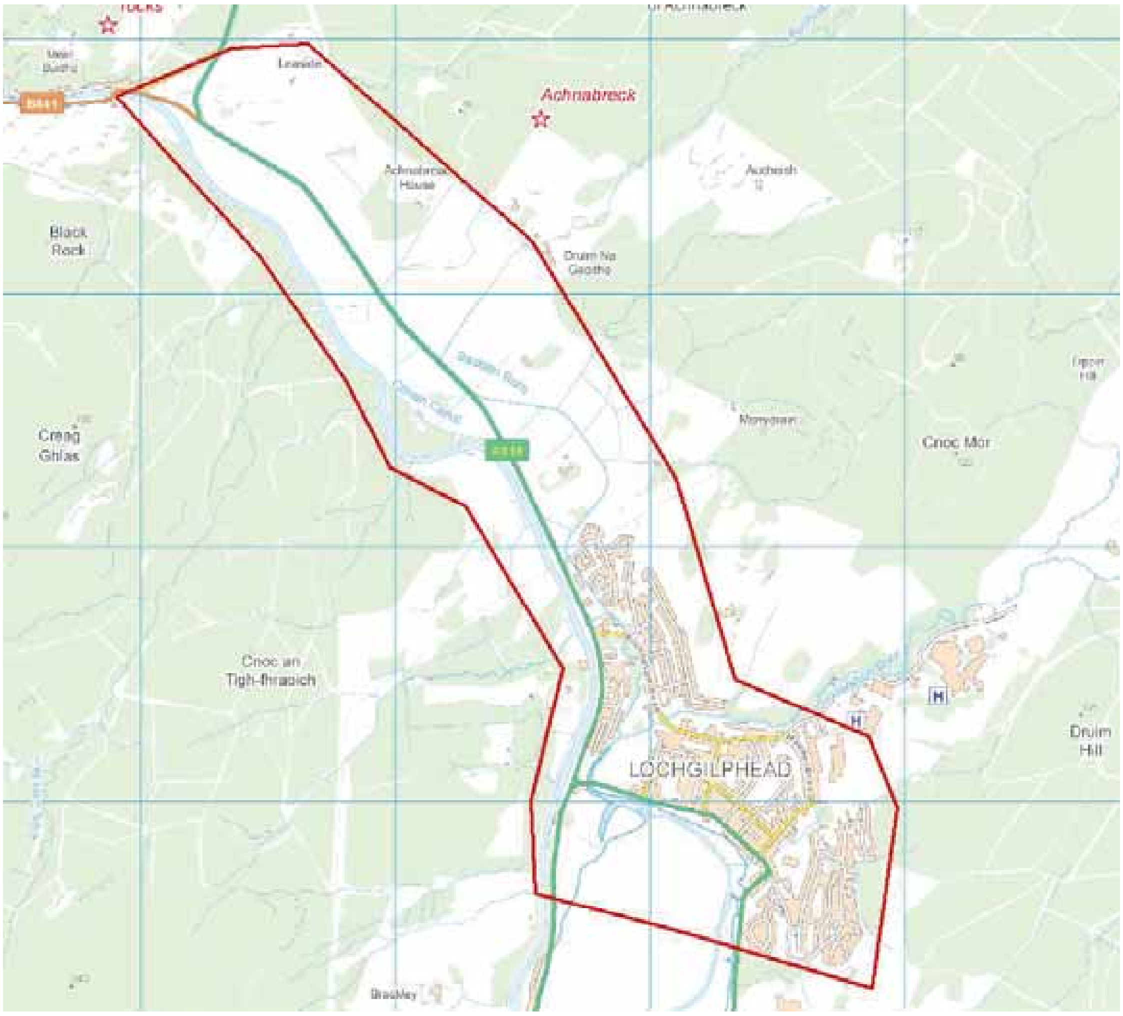

Figure 2-1: Study area

2.1 Site Visit

A site walkover was undertaken in July 2018 to establish the general topography and constraints on the Badden Burn from Cairnbaan to Lochgilphead, the Cuilarstich Burn and along the frontage of the town. Sections of the Crinan Canal were also assessed, with particular attention paid to the overflow weirs. During the walkover, a review of possible flood flow routes and assessment of the viability of potential options was undertaken.

The Badden Burn in the reach that runs alongside the A816 in canalised and uniform. At the time of the site visit, the banks and channel were overgrown with long grass and reeds. At this point, the channel is relatively narrow, approximately 5m, with a stony bed. Several small access tracks cross the watercourse upstream of Lochgilphead which may act as flow constrictions. However, no blockages were observed.

In Lochgilphead, the Badden Burn joins with the Cuilarstich Burn, which flows from the north east, to form a channel that is well defined and approximately 13m wide. Again, the majority of the channel bed is made up of stony material with mature and relatively dense vegetation and trees on the banks. Several large roads as well as smaller footbridges cross the watercourses through the town. No blockages were noted at these structures. The watercourse flows into an intertidal area of Loch Gilp, where it continues in an easterly direction parallel to Poltalloch Street before heading south out to sea when tides are low. This intertidal area is a combination of sand, gravel and larger rocks, with permanent, vegetated ground adjacent to where the watercourse outfalls.

The Crinan Canal runs parallel to the A816. This reach is described in Scottish Canal's Water Control Manual (WCM) as the Eastern reach and extends over 5.5km and is bounded by locks upstream and downstream. No additional locks are located along this eastern stretch. The upstream locks separate the eastern reach and the summit pound and the downstream locks separate the eastern reach from Loch Gilp. The overflow weir that discharges into the Badden Burn is located approximately 1.2km downstream of Cairnbaan. The weir consists of 3 arches, 1.7m in width, with a contained concrete spillway immediately downstream. The spillway then changes into a more naturalised channel further downstream. At the time of the site visit, the spillway was relatively densely vegetated with long grass and reeds. Each of the openings has 2 weir boards of 300mm in height, which can be removed or installed to control levels in the canal. All other weirs on the eastern reach of the canal discharge into Loch Gilp.

Photographs can be found in Appendix A.

2.2 Historic Flooding and SEPA flood maps

Coastal flooding is predicted by the SEPA online Flood Risk Management Maps[1] (FRM maps), encroaching on roads including Poltalloch Street and the A816 and land around the caravan park. The SEPA maps also predict fluvial flooding from the Badden Burn around the Meadows area, extensive sections of the A816 and eastern sections of Lochgilphead.

The SEPA floodmaps are backed up with the historic flood reports set out in The Flood Risk Management Strategy, where flooding has been noted at properties, community facilities, utilities, agricultural land, designated areas and transport networks. Fluvial flooding has caused particular issues on the A816 between Lochgilphead and Cairnbaan. The Crinan Canal is also seen to act as source of flooding where it discharges over a waste weir into the Badden Burn catchment increasing overall flow. Anecdotal evidence also describes flooding on the A83 and on the Front Green caused by high tides, surface water and fluvial events. It should be noted that surface water may be the cause of some of the localised flooding within Lochgilphead.

SEPA and ABC have provided the following records:

- 28th November 2018 – Low pressure system causing high tides and strong winds which resulted in flooding on the Front Green;
- 15th November 2015 – A816 – Flooding between Lochgilphead and Cairnbaan as a result of overtopping on the ditches and Badden Burn causing road closures;
- July 2009 – Flooding of the A816 around the Meadows due to overflowing road ditches and the Badden Burn as a result of heavy rain. The flooding lead to a road closure;
- July 2009 – Flooding of the A83 at the Lorne Street junction. Flooding of the carriageway/footway caused by high tide combined with heavy rainfall;
- 19th January 2004 – A816 – Flooding on road between Cairnbaan and Lochgilphead caused by overflowing ditches likely caused by heavy rain (no indication of blockage);
- Unknown dates – occasional localised flooding in the caravan site of unknown source;
- Yearly – anecdotal accounts of flooding on the Front Green up to Polltalloch Street. This was observed due to high tides but was noted that flood waters infrequently spill onto the road.

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[1] http://map.sepa.org.uk/floodmap/map.htm

3. Fluvial Hydrological Assessment

3.1 Methodology overview

The Flood Estimation Handbook (FEH) gives guidance on rainfall and river flood frequency estimation in the UK and also provides methods for assessing the rarity of notable rainfalls or floods. A number of methods of flood estimation are presented, including the FEH statistical method and the FEH rainfall-runoff method. Subsequent publications have presented the ReFH and ReFH 2 rainfall-runoff method, updating the FEH rainfall-runoff method.

The statistical method relies on deriving a representative growth curve for the subject site from a pooled group of hydrologically similar catchments for which there is gauged information. This means that the accuracy of the method and resulting flow estimate depends on there being a sufficient number of similar catchments contained in the gauging station database. The method assumes that the flood statistics within the periods of record in the pooling group are representative of the flooding regime in the future, i.e. that the data is stationary. However, the method is based on actual observed flood data, and is therefore considered to be more robust than the more conceptual rainfall-runoff methods.

The statistical method consists of two parts; estimation of the median annual flood (QMED), i.e. the flood event with an annual exceedance probability of 50% (1 in 2 year return period), and the derivation of a pooled or single-site growth curve. The growth curve is then multiplied by the QMED estimate to provide a flood frequency curve for the subject site for a range of AEP events.

The best estimate of QMED is determined using flood data at the site if such local data exists. Alternatively, if no such data exists, QMED can be estimated from FEH catchment descriptors and improved by data transfer from a suitably hydrologically similar donor gauge.

WINFAP-FEH is a software tool that supports the statistical flood frequency estimation methods as presented in Volume 3 of the FEH. It provides single-site and pooled group methods of frequency analysis based on annual maxima data from a database of gauged catchments. WINFAP-FEH files (Version 7) gauging station data was used to generate a hydrologically similar pooling group of sites using WINFAP-FEH v4 software. The database contains the annual maximum (AMAX) series data for each station in the database giving AMAX series up to and including the 2015 water year for the majority of UK gauges. Each station within the pooling group was checked and found to have AMAX series up to and including the 2016 water year. Some stations now have more recent data. Similarity is judged using a distance measure derived from the difference in floodplain extent (FPEXT), rainfall (SAAR) and catchment area (AREA) between the subject site and the gauging station sites. The total data record from the resulting group should amount to around 500 years of data as recommended in *Science Report SC050050*. The pooling group is then used to predict a growth curve, which is combined with the index flood QMED to provide flow estimates for flood events of varying severity.

The FEH rainfall-runoff method has been updated with the ReFH2 rainfall-runoff method. This method utilised catchment specific descriptors to assess catchment functioning and runoff. ReFH2 is now accepted by SEPA as it incorporates a larger Scottish dataset, includes more small-catchment data, utilises the most up to date 2013 rainfall DDF (Depth, Duration, Frequency) model data and incorporates an improved method for assessing urban losses.

Given the size of the full catchment to the downstream extent at Poltalloch Street is 22.9km^2, both the statistical and ReFH2 methods were suitable and both were undertaken for a comparison before finalising the choice of method. Flow estimates for the whole catchment were determined at the downstream extent of the study area for flow reconciliation purposes. A range of return periods were required. These included the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP events with and without climate change.

Each subcatchment that contributed to the main watercourses was added as a separate ReFH2 unit, where critical storm duration testing was undertaken. All catchments were then scaled, with the aim of matching the total catchment peak flows at the downstream extent of the model. Further details of this process can be found in the following sections.

Climate change was also assessed to determine likely flood conditions in the future.

Throughout this report, flooding events will be described in terms of their Annual Exceedance Probability (AEP). Table 3-1 sets out how these AEP events correspond to flood return periods.

Table 3-1: AEP and return period equivalent

Annual Exceedance Probability (AEP) event	Return Period
50%	2
20%	5
10%	10
5%	20
3.33%	30
2%	50
1%	100
0.5%	200
0.1%	1000

3.2 Total catchment hydrology

In order to determine the peak flows at the downstream extent of the model, where the watercourse discharges into Loch Gilp, a hydrological assessment of the total catchment area of the Badden Burn and Cuilarstich Burn was undertaken.

The numerous sub-catchments within these larger catchments were to be applied to the model separately at specific locations and the peak flows identified for the downstream extent of the total catchment would be used to reconcile these subcatchment flows.

3.2.1 FEH Catchment Descriptors

The Badden Burn and Cuilarstich Burn catchment descriptors and GIS catchment boundary to the tidal limit was obtained from the FEH web service. The line of the watershed was verified against OS mapping and amended where appropriate based on survey, LiDAR data, the Scottish Canal's Water Control Manual and OS mapping. Based on a review of Scottish Canal's Water Control Manual (WCM) and through consultation with Scottish Canals, it was found that several watercourses feed into the canal rather than flowing underneath and into the Badden Burn as shown in the default FEH Web Service delineation. These uncontrolled canal feeder catchments were not picked up correctly in the default FEH Web Service delineation and were removed in the updated manual delineation so as not to overestimate flow reaching the Badden Burn catchment. Further details of the canal catchments can be found in Section 4.2. The areas removed from the fluvial catchments were mainly those that were found to be intercepted by the Crinan Canal.

Figure 3-1 displays the FEH default and the manually amended catchment area.

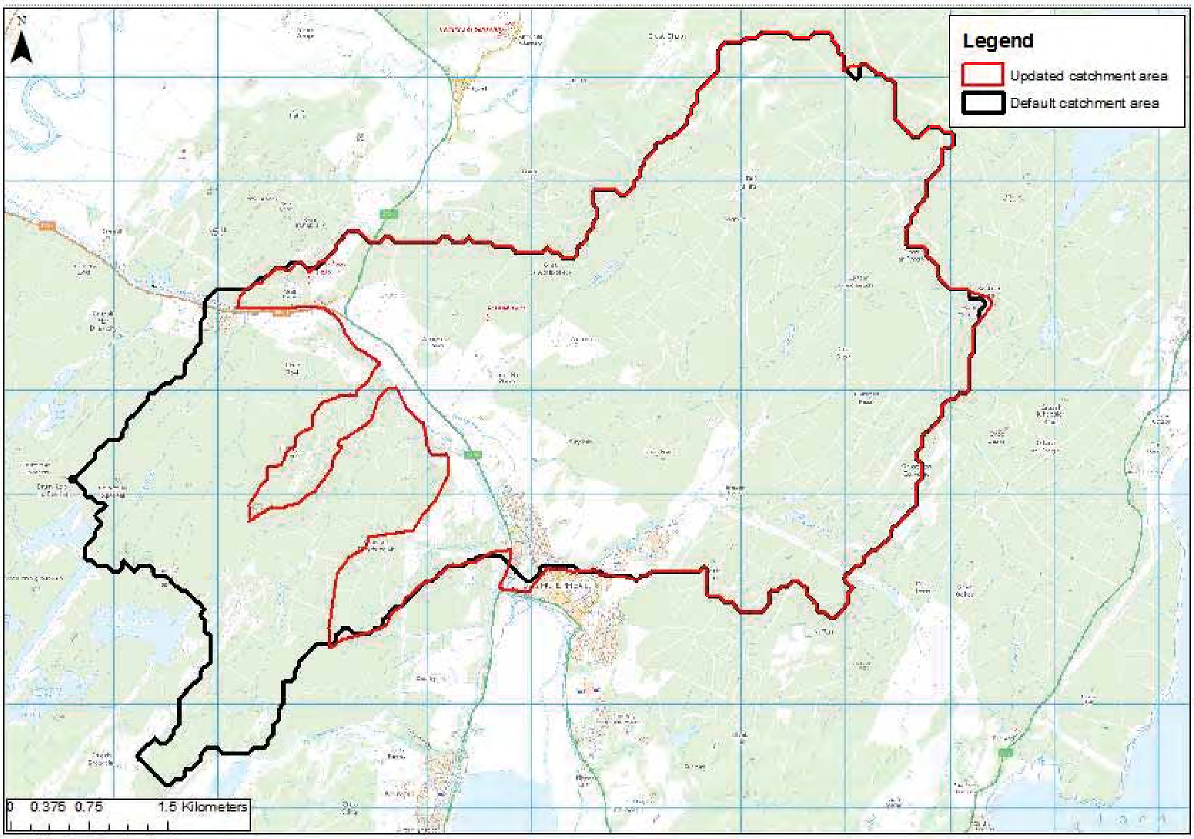

Figure 3-1: FEH default catchment and updated catchment to downstream extent

The catchment area value was updated from the downloaded descriptors to reflect the amended catchment delineation. DPLBAR is inherently linked to area. However, when adjusting DPLBAR to correspond which the new area using the linking equation in FEH Vol 4, it was found to increase from the default, which is unlikely to be correct due to the decrease in catchment area. For that reason, DPLBAR was left as default. The URBEXT value was also updated to current day.

Catchment descriptors used in the total catchment hydrological assessment to the downstream extent of the model are shown in Table 3-2.

Table 3-2: FEH catchment descriptors

Catchment parameters	**Total catchment descriptors**
NGR	*185900 688250*
AREA	*22.91* updated from default of 30.645*
ALTBAR	*122*
ASPBAR	*252*
ASPVAR	*0.09*
BFIHOST	*0.41*
DPLBAR	*4.73*
DPSBAR	*116.8*
FARL	*0.994*
FPEXT	*0.0653*
FPDBAR	*0.929*
FPLOC	*0.742*
LDP	*8.37*
PROPWET	*0.75*
RMED-1H	*10.5*
RMED-1D	*44.7*
RMED-2D	*60*
SAAR	*1807*
SAAR4170	*1638*
SPRHOST	*40.34*
URBEXT1990	*0.0052*
URBEXT2000	*0.0058* updated to current day*

3.2.2 Statistical analysis

Statistical analysis was undertaken on the total catchment to establish peak flow estimates. Given the size of the catchment, which is relatively small, there are inherent uncertainties using this method as there are only a small number of similar small catchments in the data set. The catchments within the final pooling group should be assessed to establish their similarity to the subject catchment.

Catchment descriptors and delineations were obtained from the FEH Web Service and updated as set out in Section 3.2.1.

3.2.2.1 QMED estimation

QMED is defined as the median annual flood, i.e. the flood event with an AEP of 50%.

With no gauged data in the catchment, the FEH recommended method to derive QMED is by data transfer from a hydrologically similar donor catchment. This donor transfer was carried out within the WIN-FAP v4 software, using both the multiple station method and the single station method. The single site donor urbanised adjustment method has been selected as the preferred method because the single donor catchment was the most similar to the total catchment and contained good gauge suitability comments. It was therefore a strong candidate. The donor catchment used in this study for establishing QMED is the Little Eachaig @ Dalinlongart. Details of the donor selection can be found in Appendix B. Transferred QMED values are shown in Table 3-3.

Table 3-3: QMED donor transfer

Water Course	QMEDcd	QMEDadj (6 sites)	QMEDadj (1 site)	**QMEDadj (1 site) (urbanised)**
Badden Burn and Cuilarstich Burn total catchment	24.70	23.89	24.56	**24.72**

3.2.2.2 Pooled growth curve

The resulting default pooling group was reviewed in WINFAP and a number of adjustments were made. This mainly comprised of removing sites with low FARL values and short records. Details of the default and reviewed pooling groups are included in Appendix B.

The best fitting statistical distribution was the GL (Generalised Logistic), and the heterogeneity measure H_2 was 0.23, acceptably homogeneous.

Table 3-4 shows the growth curve and flood frequency curve for the total catchment using the statistical method.

Table 3-4: Growth curve and flood frequency curve for the total catchment

AEP (%) event	Pooling group growth curve for total catchment (GL)	Peak flows for total catchment (m^3/s)
50%	1.00	24.72
20%	1.32	32.58
10%	1.55	38.21
5%	1.79	44.20
2%	2.15	53.06
1%	2.46	60.75
0.5%	2.81	69.43
0.2%	3.35	82.67
0.1%	3.82	94.31

3.2.3 ReFH2

The Revitalised Flood Hydrograph Method 2 (ReFH2) was undertaken as a comparison to the flow estimates for the total catchment generated using the Statistical method. ReFH2 is now accepted by SEPA and is suitable to use on catchments of this size.

Catchment descriptors and delineations were obtained from the FEH Web Service and updated as set out in Section 3.2.1. ReFH2 software, using the winter storm profile given the rural nature of the catchment, was used. The standard ReFH2 derived hydrograph shapes were used as there was no gauged data on the watercourse to be used to generate an alternative.

Peak flows from the analysis are shown in Table 3-5.

Table 3-5: ReFH2 Peak flows

AEP (%) event	Peak flows for total catchment (m^3/s)
50%	19.90
20%	26.69
10%	31.54
5%	36.47
2%	43.44
1%	49.45
0.5%	56.28
0.2%	67.92
0.1%	80.15

3.2.4 Hydrological method selection

The Statistical method gave higher peak flow estimates than the ReFH2 method. The size of the catchment to the downstream extent is relatively small, which can make it more problematic to find similar sized gauged catchments for use in the statistical method, and therefore may increase uncertainty in the results. However, assessment of the pooling group found that the group catchments were also relatively small, with the largest catchment being 38km^2, and that other key descriptors were also similar. The Statistical method is also based on actual observed data rather than conceptual rainfall-runoff methods. For these reasons, the Statistical analysis peak flows were deemed to be the most appropriate for the total catchment in this study. Table 3-6 displays the comparison of the Statistical analysis and ReFH2 peak flow estimates. The statistical analysis peaks were to be used to reconcile the subcatchment flows at the downstream extent of the model.

Table 3-6: Statistical and ReFH2 comparison

AEP (%) event	**Statistical peak flows for total catchment (m^3/s)**	ReFH2 peak flows for total catchment (m^3/s)
50%	**24.72**	19.90
20%	**32.58**	26.69
10%	**38.21**	31.54
5%	**44.20**	36.47
2%	**53.06**	43.44
1%	**60.75**	49.45
0.5%	**69.43**	56.28
0.5% +CC	**95.12**	77.10
0.2%	**82.67**	67.92
0.1%	**94.31**	80.15

3.3 Climate change

The United Kingdom Climate Projections 2018 (UKCP18) dataset was published in December 2018 and outlines updated probabilistic projections of climate change impact for the 2020's, 2050's and 2080's based on various emissions scenarios and probability percentiles. United Kingdom Climate Projections (UKCP09[2]) is a previous version of the projections and since little guidance (at the time of this analysis) had been provided on how these new 2018 uplifts in rainfall relate to increases in flow, using the UKCP09 flow uplifts as a basis is still considered to be appropriate.

Outlined in their Flood Modelling Guidance for Responsible Authorities, SEPA commissioned CEH to undertake a study assessing Scottish catchments vulnerability to climate change. Within this study the UKCP09 projections were run through models to provide flow uplift for hydraulic basins. This provides a more accurate representation of the increase to fluvial flows.

Based on the medium emission scenario 2080s, 50th percentile, the study area is reported to have a 37% change in flood peak as shown in table 10-2 of the SEPA Flood Modelling guidance. For the purpose of this flood study, a 37% uplift has been adopted for climate change scenario to come in line with SEPA's Technical Flood Risk Guidance. The medium emission scenario 2080, 90th percentile change in flood peak will also be applied to the modelling to assess sensitivity. This is reported to be 60% for the study area.

SEPA published guidance on climate change uplifts for use in Flood Risk Assessments (FRA) at the end of April 2019. Within this guidance, it is recommended that for small catchments of less than 30km^2, that peak rainfall intensity should be uplifted rather than peak river flow. For Argyll and Bute, this uplift in rainfall intensity is 55%. This guidance is focussed towards FRAs and it is still recommended that a range of scenarios be tested for Flood Studies. It should be noted that a sensitivity test on a flow uplift of 60% will be undertaken as part of this study as outlined above.

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[2] UK Climate Projections, *United Kingdom Climate Projections (UKCP09) Rainfall Maps*, June 2009

3.4 Fluvial sub-catchment hydrology

To replicate the main flow entry points along the length of the Badden Burn, subcatchments inflows were applied to several points in the model rather than applying the full catchment flow at the upstream extent. Applying the full catchment flow at the upstream extent of the model was considered to be overly conservative and would result in artificially high flood depths along the A816.

An assessment of pluvial flooding was not undertaken as part of this study. However, rain falling on the watercourse catchments was picked up within the point inflow hydrology.

The hydrological assessment in previous sections was undertaken for the total catchment of the Badden Burn and Cuilarstich Burn to the downstream boundary of the model. These peak flows were calculated so that the subcatchment flows could be reconciled in the model, to match this downstream peak estimate. Calculating the total catchment inflow, and reconciling the subcatchments inflows to match, increases confidence in hydrological estimates. The total catchment is much larger than each subcatchment which reduces the uncertainties associated with small catchment hydrology.

The following sections outline the reconciliation process.

3.4.1 Delineation and representation

Sub-catchments of the Badden Burn were delineated using LiDAR. As part of this assessment, the watercourses to the west of the canal were also assessed to establish which areas fed into the canal and which went underneath and fed into the Badden Burn. Full details of the canal assessment can be found in Section 4.

A total of 5 subcatchments of the Badden Burn were identified. Some of these sub catchments were watercourses such as the Auchoish Burn and some of the sub catchments were runoff areas with no associated watercourse. The run off areas were identified as separate sub catchments to allow flow to be added to specific sections in the model so that flow was not overrepresented in the upper portions. In the case of the runoff catchment named ‘Downstream’, this inflow was applied as a lateral inflow at 3 separate 1D model nodes based on % of that contributing catchment. The subcatchment inflow locations and extents are shown in Figure 3-2.

Throughout this report, the unnamed watercourse running off Creag Ghlas hill, underneath the canal and into the Badden Burn is referred to as the Creag Ghlas Burn. This is not the same as the Craig Glass Burn that is denoted on the OS mapping to the south and which runs into the canal as an uncontrolled feeder.

Catchment descriptors were downloaded for the tributary watercourses from the FEH Web Service and area and DPLBAR were updated where appropriate based on catchment delineation using the LiDAR. These downloaded descriptors were also used as the descriptors for the runoff areas either by modifying the area and DPLBAR of the nearest and most similar watercourse catchment or by undertaking an intermediate catchment assessment. Catchment descriptors of the subcatchments are shown in Table 3-7.

Each subcatchment was represented using an ReFH2 unit which provided both the peak flow for each subcatchment as well as the hydrograph shape.

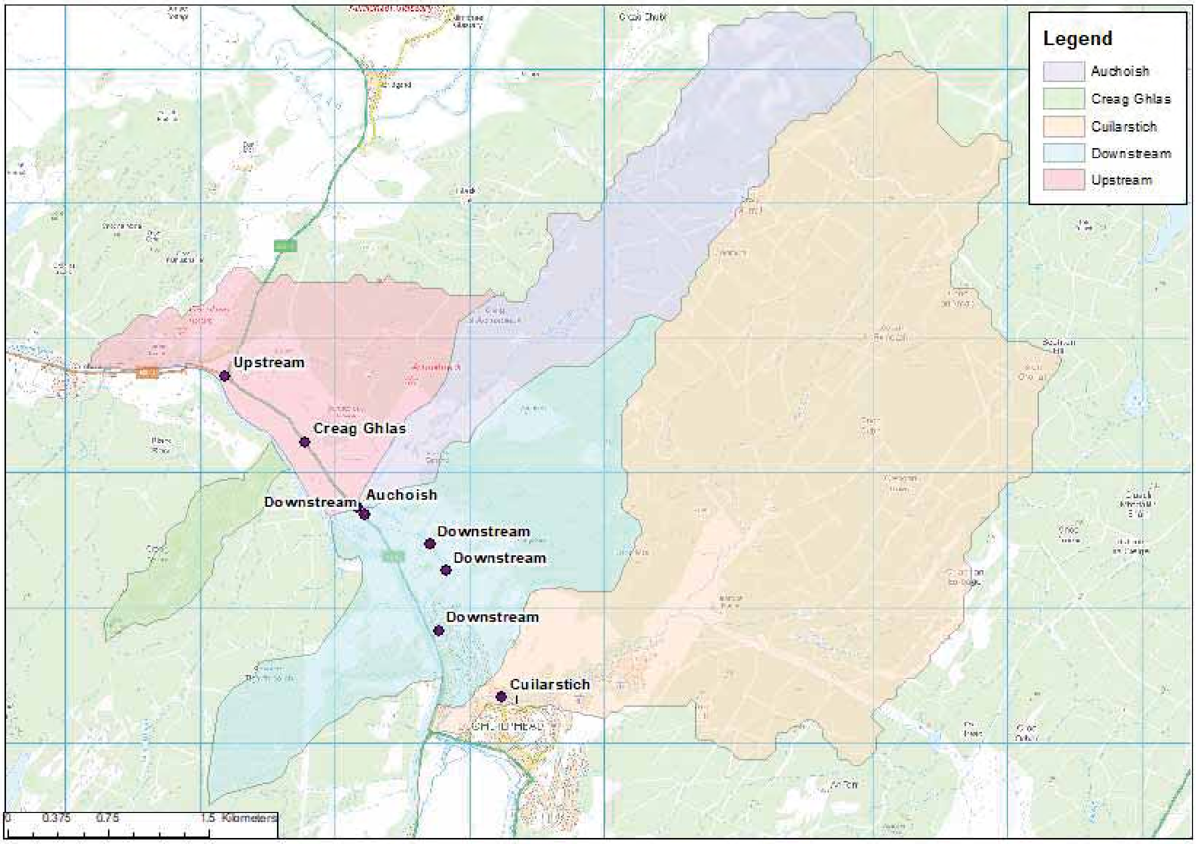

Figure 3-2: Subcatchments and inflow locations into model

Table 3-7: Sub catchment descriptors

Catchment parameters	Upstream	Creag Ghlas	Auchoish Burn	Downstream	Cuilarstich
NGR	*185101 689809*	*184600 690150*	*185250 689750*	*185873 688311*	*185950 688250*
AREA	*2.561*	*0.603*	*3.516*	*4.342*	*11.89*
ALTBAR	*37*	*139*	*133*	*113*	*132*
ASPBAR	*232*	*48*	*232*	*232*	*230*
ASPVAR	*0.51*	*0.62*	*0.44*	*0.44*	*0.29*
BFIHOST	*0.448*	*0.382*	*0.403*	*0.421*	*0.414*
DPLBAR	*1.67*	*1.2*	*3.5*	*2.2*	*4.7*
DPSBAR	*77.2*	*172.8*	*122.7*	*95.3*	*111.1*
FARL	*1*	*1*	*1*	*1*	*0.985*
FPEXT	*0.2368*	*0.0043*	*1.0404*	*0.0404*	*0.0422*
FPDBAR	*5.132*	*0.051*	*0.513*	*0.513*	*0.511*
FPLOC	*0.76*	*-999999*	*0.47*	*0.47*	*0.848*
LDP	*2.52*	*2.07*	*6.26*	*6.26*	*8.32*
PROPWET	*0.75*	*0.75*	*0.75*	*0.75*	*0.75*
RMED-1H	*10.6*	*10.3*	*10.7*	*10.7*	*10.5*
RMED-1D	*44.7*	*44.3*	*44.4*	*44.4*	*44.4*
RMED-2D	*59.9*	*59.3*	*59.4*	*59.4*	*59.7*
SAAR	*1688*	*1782*	*1828*	*1768*	*1827*
SAAR4170	*1577*	*1582*	*1636*	*1636*	*1659*
SPRHOST	*28.65*	*47.58*	*42.01*	*34.78*	*39.87*
URBEXT1990	*0*	*0*	*0*	*0.0258*	*0.0051*
URBEXT2000	*0*	*0*	*0*	*0.0098*	*0.0091*

3.4.2 Critical storm duration testing

Critical storm duration on a catchment refers to the design storm which provides the highest flood discharges/flood levels for a source of flooding. It is important to assess the critical storm duration so that maximum flood depths at key receptors can be established.

The individual sub catchments had a range of default critical storm durations due to the difference in areas and catchment descriptors. Given the relatively small size of the overall catchment, and therefore likelihood of only one storm event affecting the total catchment, one critical duration for the entire catchment had to be established.

This was achieved by running the linked 1D/2D model, with the same critical duration applied to all sub catchment inflows for the 0.5% AEP event. The spill from the canal overflow weir and any spill over the embankment was also included in this critical duration testing. Full details of this assessment can be found in Section 4. A range of durations were run in order to identify the critical event at key receptors such as within town and on the A816.

It was found that the 7h15 duration produced the greatest flood depths within town, the Meadows housing area and around the caravan park.

The critical duration was found to be slightly longer for the areas upstream of the Meadows on the A816, equating to a 7h45 duration event. However, flood levels on the A816 were not seen to be overly sensitive to change in durations, with average variations between the 7h15 and 7h45 of 3-8mm. The flooding in this area around the A816 was also not seen to affect any properties. The small variation in depth between critical durations on the A816 would also not materially alter the findings of the damage assessment. For these reasons it was deemed appropriate to adopt the 7h15 duration event as the critical duration for all flood receptors.

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3.4.3 ReFH2 reconciliation

3.4.3.1 Application of uplifts to match downstream total catchment peak

Once the critical duration had been established, the ReFH2 sub catchment inflows were iteratively scaled within the 1D/2D model, by uplifting the whole hydrograph, until the flow at the downstream extent of the model matched the statistical analysis peak flow estimates outlined in section 3.2.4. The canal inflows were not included in the analysis.

However, after several iterations, it became clear that scaling the subcatchment peaks to match the Statistical Analysis peaks required extremely large uplifts that were unlikely to be realistic. For example, the 1 in 200yr event required all ReFH2 inflows to be upscaled by a factor of 1.65 to allow the flow to match the downstream Statistical peaks.

A range of sensitivity runs were undertaken to establish whether the Statistical Analysis peaks were overly conservative and whether the high uplifts to achieve reconciliation were appropriate or unrealistic.

3.4.3.2 Volume sensitivity check

When comparing the flood extents of the simulations with the default uplift factor of 1 and the reconciled uplift factor of 1.65 for the 0.5% AEP event, it could be seen that the flood extent upstream of Lochgilphead was significantly larger in the uplifted simulation due to a constriction in the channel. This mechanism was expected to some extent, however, it appeared that the flow exiting the channel upstream of Lochgilphead accounted for the vast majority of all additional flow resulting from the uplifted inflows. The large increase in floodplain storage upstream of Lochgilphead explained the requirement for the significant uplifts to flows to match the statistically derived peaks at the downstream extent.

A volume calculation of the gross rainfall compared to the total inflow was undertaken for the critical duration of 7h15 on the Creag Ghlass Burn to assess whether the uplift of 1.65 was realistic or technically impossible. The total inflow from a hydrograph is made up of the net rainfall plus the baseflow which accounts for normal flow conditions without rapid runoff from rainfall. The gross rainfall accounts for no losses in the catchment and in using this value, the assessment is conservative. Table 3-8 shows the total volume of the gross rainfall hyetograph and the total volumes of the default and uplifted inflows. The baseflow has been subtracted from the inflow volumes as this flow is not caused by the rainfall event and therefore not linked to the gross rainfall. In the Creag Ghlass catchment, the model inflow volume of the default hydrograph was lower than the gross rainfall volume, which is expected. However, the volume of the uplifted hydrograph inflow was higher than the gross rainfall which is not possible. This is a known limitation of the ReFH2 unit where generated flow can exceed total rainfall.

This assessment has shown that an uplift in hydrographs of 65% is not realistic.

Table 3-8: Volume calculations

	Total volume (m^3)
Gross Rainfall	41,800
Inflow with default uplift factor (minus baseflow)	23,200
Inflow with 1.65 uplift factor (minus baseflow)	50,500

A longer critical duration of 14h15 was also run to assess whether the uplift factors could be reduced if the flood mechanism was altered and floodplain storage was used earlier in the event. This was found to reduce peak flows at the downstream extent compared to the shorter duration event. Higher uplift factors would therefore be required to match the downstream peak.

As a final check, the 50% AEP event was run to assess the uplifts required on an event that was largely within bank, and therefore had little floodplain interaction. It was found that an uplift of 1.23 was required to achieve a match with the downstream statistical peak which is more in line with expected model uplifts, and also technically feasible when comparing with the gross rainfall volumes.

3.4.3.3 Assessment of floodplain extent

The pooling group had previously been scrutinised as outlined in Section 3.2.2.2. However, given the complexities relating to flow reconciliation, the pooling group was further assessed to determine whether it was producing artificially high peak flow estimates as a result of stations in the group having dissimilar floodplain or soil characteristics. Both of these descriptors have the potential to significantly alter the growth curve.

FPEXT is defined as the fraction of the catchment that is estimated to be inundated by a 1% AEP event and can therefore be important in establishing how flood waters are stored or conveyed during an event. Table 3-9 displays the FPEXT values for the pooling group. It was found that all but 2 catchments had a lower FPEXT (floodplain extent) descriptor than the Badden Burn catchment which could mean that the growth curve may be underestimating floodplain extent and overestimating peak flows. It was not however felt that the FPEXT values of the pooling group varied significantly enough from the Badden Burn catchment to warrant an update of the pooling group.

Table 3-9: FPEXT values for the Statistical pooling group

Station	**FPEXT**
21017 (Ettrick Water @ Brockhoperig)	0.012
25012 (Harwood Beck @ Harwood)	0.021
206006 (Annalong @ Recorder)	0.024
48004 (Warleggan @ Trengoffe)	0.035
27032 (Hebden Beck @ Hebden)	0.040
46005 (East Dart @ Bellever)	0.042
72007 (Brock @ Upstream of a6)	0.054
73009 (Sprint @ Sprint Mill)	0.061
49003 (de Lank @ de Lank)	0.064
Badden Burn catchment	**0.065**
73015 (Keer @ High Keer Weir)	0.075
72014 (Conder @ Galgate)	0.082

3.4.3.4 Assessment of subcatchment inflows and permeability

The 0.5% AEP inflow for each subcatchment was divided by its area to establish a flow per km^2 for comparison. This was done to establish if any of the subcatchments were responding differently to the others and warranted additional evaluation.

Table 3-10 displays the peak flow per km for each subcatchment, which are seen to show significant variation.

Table 3-10: Subcatchment flow per km^2

	200yr flow (m^3/s) per km^2
Upstream catchment	2.33
Cuilarstich Burn	2.47
Downstream catchment	2.69
Auchoish	2.80
Full catchment	**3.30**
Creag Ghlas Burn	3.44

The permeability of a catchment can significantly alter the total runoff generated. Given the variation in the subcatchment flow per km, SPRHOST was assessed for all subcatchments to establish whether the variation in flow per km^2 was explained by variation in soil permeability and whether there were any discrepancies within the downloaded descriptors that could be affecting the generated inflows.

Table 3-11 displays the SPRHOST values for each of the sub catchments. The soils in the subcatchments were checked using the Soil Survey of Scotland map and the Institute of Hydrology soil classifications. The upstream catchment SPRHOST was seen to be particularly low when compared to the overall catchment. The predominant soil in the upstream catchment was noncalcareous gleys and humic gleys which generates a low SPRHOST value of 25.3. This confirms the lower SPRHOST value generated in the FEH Web Service download. The values for all subcatchments downloaded from the FEH Web Service were deemed reasonable.

Table 3-11: Subcatchment SPR and area.

Catchment	SPRHOST (low to high)	Area
Upstream catchment	28.65	2.56
Downstream catchment	34.78	4.34
Cuilarstich Burn	39.87	11.89
Full catchment	**40.34**	**22.91**
Achnabreck	42.01	3.52
Creag Ghas Burn	47.58	0.60

Table 3-12 displays the BFIHOST values for the pooling group catchments. BFIHOST is the baseflow index derived using the HOST classification and describes the portion of the streamflow that is sustained between precipitation events. The pooling group is seen to be within a reasonable range, with the Badden Burn catchment being in the middle of the group. The pooling group is therefore considered to reasonably represent the Badden low flows.

Table 3-12: BFIHOST values for the Statistical pooling group

Station	**BFIHOST**
27032 (Hebden Beck @ Hebden)	0.252
25012 (Harwood Beck @ Harwood)	0.261
72007 (Brock @ Upstream of a6)	0.319
206006 (Annalong @ Recorder)	0.336
46005 (East Dart @ Bellever)	0.363
49003 (de Lank @ de Lank)	0.379
206006 (Annalong @ Recorder)	0.336
21017 (Ettrick Water @ Brockhoperig)	0.421
Badden Burn catchment	**0.41**
72014 (Conder @ Galgate)	0.443
73009 (Sprint @ Sprint Mill)	0.452
73015 (Keer @ High Keer Weir)	0.486
48004 (Warleggan @ Trengoffe)	0.499

3.4.3.5 Subcatchment final uplifts

The flood mechanism was found to be heavily influenced by the floodplain upstream of Lochgilphead and sensitivity testing demonstrated that the standard approach of uplifting subcatchment flows to reconcile to the downstream peak flow was not appropriate in this study, with a required uplift for the 0.5% AEP event being 1.65.

Given the Statistical Analysis was deemed the most appropriate method of determining peak flow, subcatchment flows, which were ReFh2 units, were uplifted by the ratio of the downstream Statistical Analysis to the ReFh2 peak flow estimates.

The ReFH2 units were uplifted by between 1.18 and 1.24. Table 3-13 displays the uplift for each AEP event.

These uplifts were deemed reasonable as they corresponded well to the uplift value of 1.23 for the 50% AEP event, where there was little floodplain interaction.

Table 3-13: Sub catchment ReFH2 inflow uplifts

AEP event %	% uplift of ReFH2 sub catchment inflows	Resultant peak flows at downstream extent of model (m^3/s)
50%	1.24	24.79
20%	1.22	30.65
10%	1.21	34.48
5%	1.21	38.45
2%	1.22	44.09
1%	1.23	48.38
0.5%	1.23	54.00
0.1%	1.18	70.18

As an additional check, and to reduce any uncertainty in these lower uplifts, the 68% and 95% confidence intervals were assessed for the downstream boundary flow estimates using the Environment Agency's technical guidance 'Using local data to reduce uncertainty in flood frequency estimation'. Values from table 1 in the technical guidance were applied to the statistically derived peak flows, as this was the preferred method identified in 3.2.4, and based on using 1 donor. Figure 3-3 displays the 68% and 95% confidence band flows for the downstream statistical peaks along with peak flows derived using the lower uplifts. The flows resulting from the lower uplifts are within the 68% confidence band which improves certainty in the uplift values.

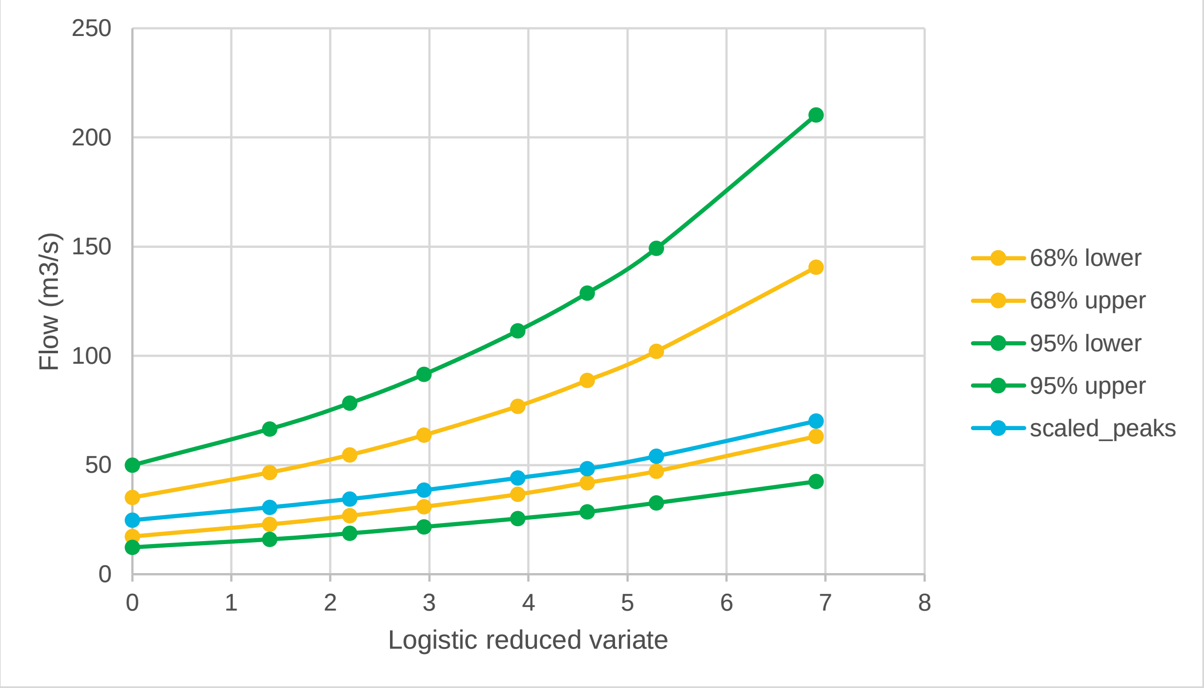

Figure 3-3: Confidence bands for the downstream peak flows

Flood history has been assessed and the flood maps from the calculated flows have been compared. Recorded accounts of road closures on the A816 are noted every 3-4 years although anecdotal accounts suggest that shallower flooding occurs more frequently. This is in line with the flood maps derived from hydrology calculated in this study from the 50% AEP onwards and further improves confidence in the hydrology.

Flooding at the caravan park site is not seen to occur until the 2% AEP event, however flooding has been noted more frequently. Due to the very low-lying nature of the land, it is reasonable to assume that this flooding here may be pluvial.

4. Canal Modelling

The Crinan Canal discharges into the Badden Burn catchment via waste weir 3 in between Cairnbaan and Lochgilphead. There is also the potential for the canal to overtop its embankment should levels build sufficiently. Both sources will contribute to flood risk in and around Lochgilphead and require to be added into the 1D/2D model.

The following sections outline the schematisation of the canal, detailing upstream catchments, hydrological inflows and structure dimensions.

4.1 General layout

The Crinan Canal catchment is a complex system of uncontrolled and controlled feeders, many of which are influenced by the numerous reservoirs in the upstream sections of the catchment. The canal is split into a summit pound, an eastern reach and a western reach, which are separated by a series of locks that remain closed to maintain depths required for navigation. Figure 4-1 shows the extents of each of the reaches.

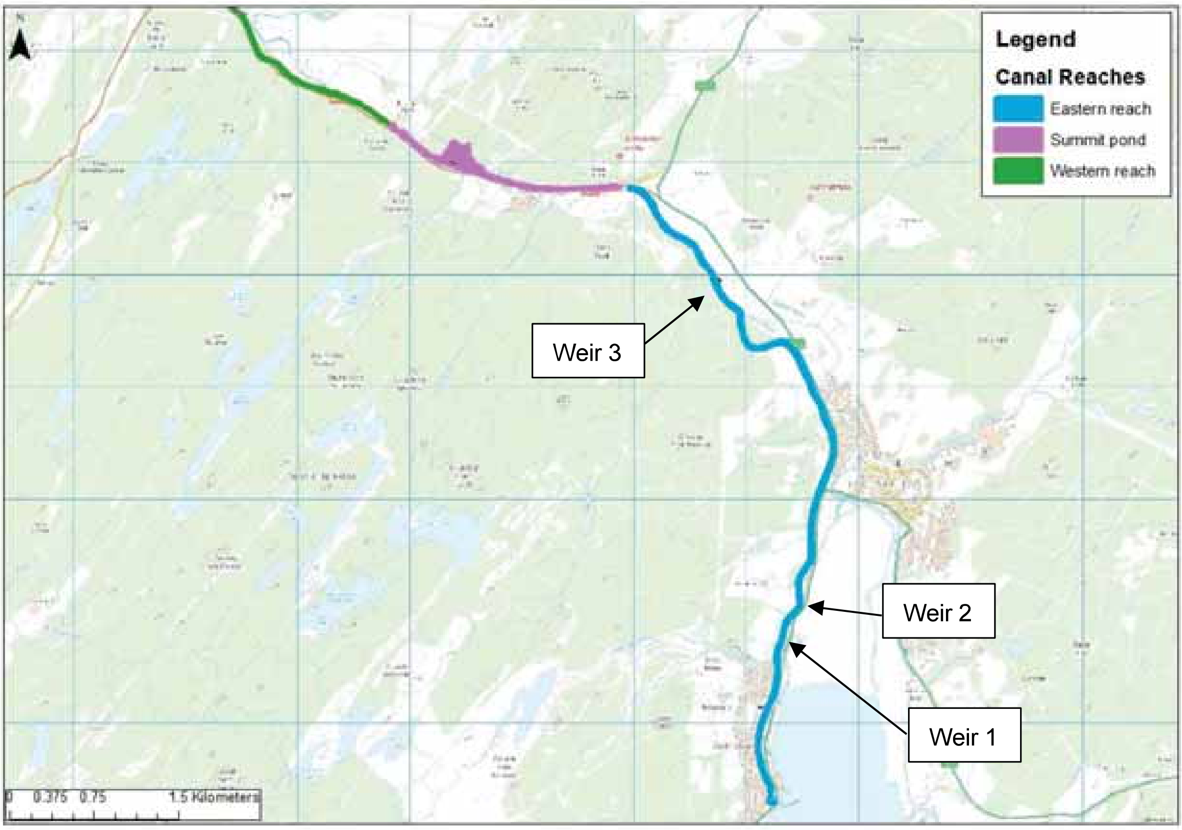

Figure 4-1: Crinan Canal reaches

Spill from the eastern reach has the potential to affect Lochgilphead and the A816. A total of 3 waste weirs are located along this reach, however only 1 discharges into the Badden Burn, with the other 2 discharging into Loch Gilp.

Spill over the canal embankment also has the potential to cause flooding in and around Lochgilphead.

Any spill from the western reach will not affect flooding in the study area and has not been considered further.

4.2 Contributing catchments delineation

Due to the many artificial modifications in the catchment, such as the reservoirs and watercourses being diverted under the canal, the FEH Web Service catchment is not representative. For this reason, the catchments that discharged into the canal were manual delineated using Lidar, information from Scottish Canal's Water Control Manual (WCM,) which outlines the canal schematisation and their operating procedures, and through conversations with Scottish Canals.

4.2.1 Summit pound

The summit pound has 2 controlled feeders: the Carindubh Burn (feeder 2) and the Dunardy Burn (feeder 1), as well as other smaller burns and runoff areas. The upstream portions of these feeder catchments consists of 10+ interconnected reservoirs, many of which are not actively controlled.

During normal conditions, all reservoirs spill into Daill Loch, where the outfall forms the Dunardy Burn (feeder 1). Water can exit Gleann Loch into the Cairndubh Burn (feeder 2) but only when water levels are too low to allow flow into Loch na Faoilinn and on towards Daill Loch. This would not reasonably be expected to occur during winter. As stated in the WCM, when a storm is forecast, the outfall sluice to Daill Loch is closed so that no flow enters the Dunardy Burn. As Daill Loch fills due to the closure of the sluice leading to the summit pound, spill from the upstream reservoir, Loch an Add, then flows west along the Barnagad Burn out west to sea Loch Sween.

Through assessment of the WCM operation guides and conversations with Scottish Canals, it was concluded that during a storm event, rainfall from catchments upstream of Daill Loch would be unable to enter into the summit pound. For this reason, the contributing catchments into the summit pound consisted of the Cairndubh and Dunardy Burn catchments downstream of the reservoirs.

Figure 4-2 shows the contributing catchments into the summit pound and the eastern reach. A schematisation covering the upstream Lochs can be found in Appendix C.

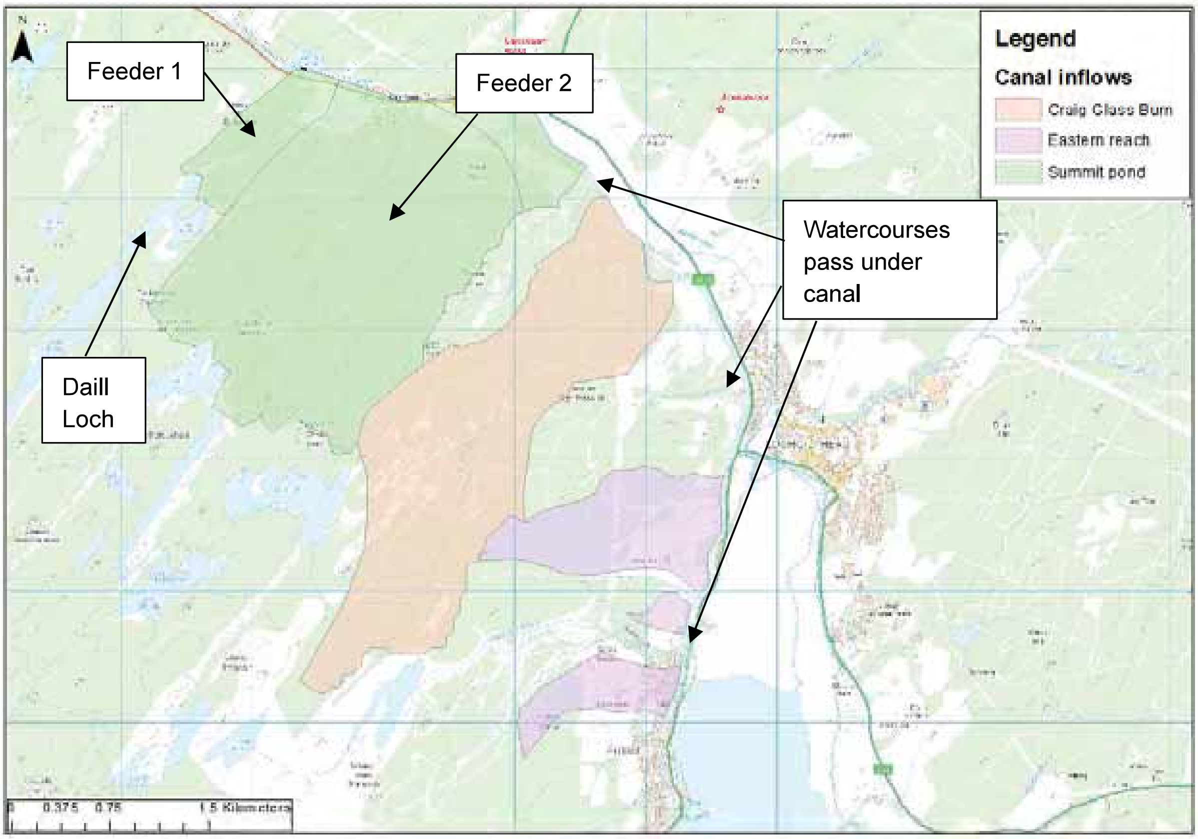

Figure 4-2: Contributing catchments to the canal eastern reach and summit pound

4.2.2 Eastern reach

Based on the details set out in the WCM and interrogation of the LiDAR, the Creag Ghlas Burn (coming off the Creag Ghlas hill) and an unnamed watercourse near to Oakfield at Lochgilphead run underneath the canal and have been included as inflows into the Badden Burn and not into the Crinan Canal. The Kilduskland Burn is also seen to run underneath the canal and discharge into Loch Gilp and has also been discounted as a contributing catchment to the canal. No details of the structures under the canal were available and it is possible that some of these catchment’s flows could enter the canal and not the Badden Burn. To be conservative, it was assumed that the structures were large enough to pass the flows into the Badden Burn catchment.

The Craig Glass Burn along with several other smaller unnamed watercourses and runoff areas do flow into the eastern reach of the canal as uncontrolled feeders and have been included in the contributing catchment hydrological assessment of the canal. It should be noted that the Craig Glass Burn low flow is diverted into the summit pound during summer, but this diversion does not occur in the winter, which is the season being considered in this study. For this reason, all flow from the Craig Glass Burn is applied to the eastern reach.

Figure 4-2 displays the contributing catchments.

4.3 Contributing catchment inflow representation

Given the simplified schematisation of the canal (details of the model build can be found in Section 4.4), inflow location was not critical. The catchments identified above were split into 3 groups and represented by 3 ReFH2 units. Full details of the catchment descriptors can be found in Appendix C.

The inflows were grouped as follows:

- Summit pound inflow
- Craig Glass Burn inflow
- Eastern reach additional inflow (runoff areas and small watercourses)

During the critical duration testing exercise outlined in Section 3.4.2 a range of durations were run through the 1D canal model to produce varying outputs from the canal to be used as inflows for the 1D 2D hydraulic model. This allowed for an overall critical duration for the entire Badden Burn catchment to be established.

Table 4-1 displays the peak inflows for each canal subcatchment for the 7h15 critical storm event.

Table 4-1: Canal inflows for each subcatchment

	Summit pound	Craig Glass Burn	Eastern reach
AREA	**5.5**	**3.73**	**1.57**
50%	7.94	5.41	2.46
20%	10.33	7.04	3.20
10%	11.99	8.17	3.72
5%	13.70	9.34	4.25
2%	16.12	10.99	5.00
1%	18.17	12.38	5.64
0.5%	20.53	13.99	6.38
0.1%	28.75	19.60	8.93

4.4 Canal model build

A simplified 1D model of the Crinan Canal Eastern reach and Summit pound, which includes Loch a Bharain, was developed in Flood Modeller in order to establish if overflow from the canal contributed to flow in the Badden Burn. A schematisation of the model can be found in Appendix C.

The Crinan Canal longitudinal section is shown in Figure 4-3 which displays lock and weir numbers. The eastern reach is on the left hand side of the diagram.

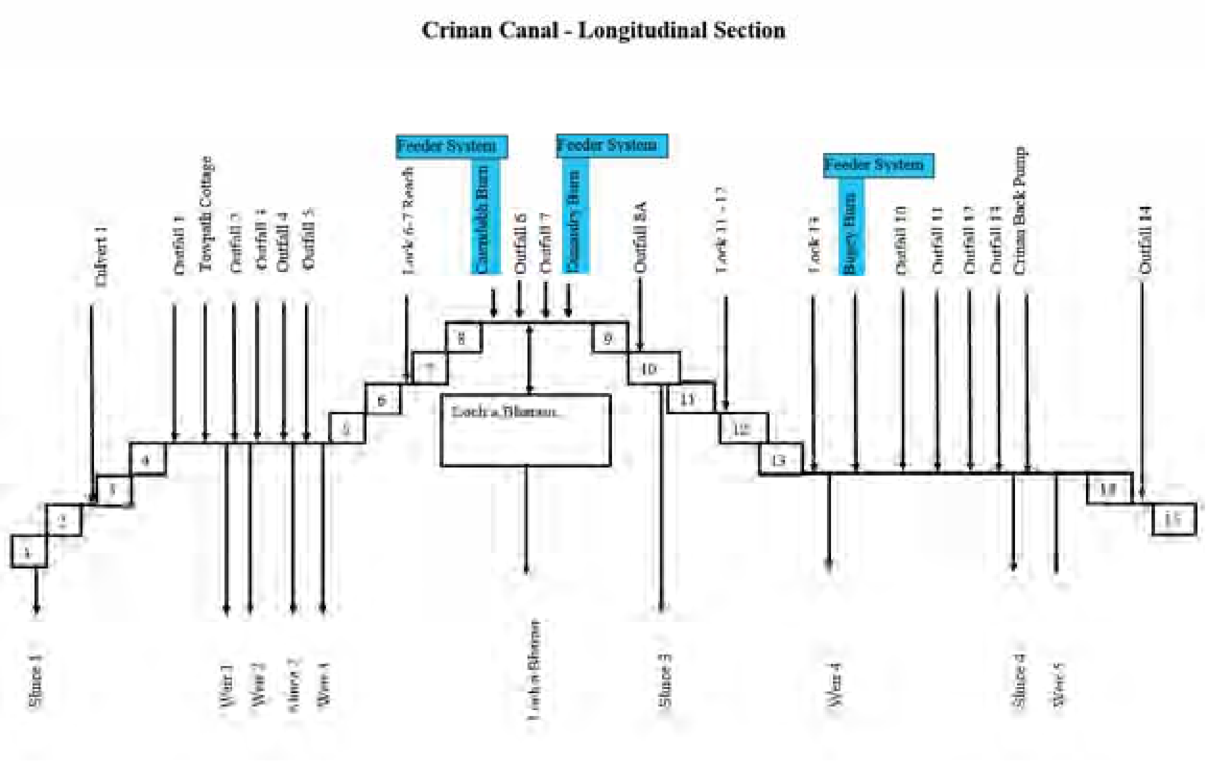

Figure 4-3: Crinan canal longitudinal section

The summit pound and eastern reach were represented by separate reservoir units based on surface area and starting at ‘normal’ canal water level based on site photographs and topographic surveyed levels. Loch a Bharain and the Summit pound reach are connected and maintain the same water level. To allow spill out of the summit and eastern reaches, the lock gates were represented using weir units set to the width and elevation of the upstream lock in the cascade. The lock dimensions in the model are outlined in Table 4-2.

Table 4-2: Canal structure dimensions

	Location description	Location coordinates	Width (m)	Elevation (mAOD)
Lock 4 (top of gate)	Downstream extent of Eastern reach at Ardrishaig	185105, 685806	7.60	10.46
Weir at Loch a Bharain	Overflow weir at Loch a Bharain discharging into Western reach	182361, 691168	9.60	20.80
Lock 8 (top of gate)	Lock separating the summit pound and eastern reach	183296, 690767	6.49	21.031
Lock 9 (top of gate)	Lock separating the summit pound and western reach	182326, 690999	8.3	21.011
Overflow weir 1	Weir discharging into Loch Gilp	185317, 686712	7.56	9.98
Overflow weir 2	Weir discharging into Loch Gilp	185478, 687066	3.00	9.99
Overflow weir 3	Weir discharging into Badden Burn	184739, 689943	5.10	9.89
Eastern reach embankment				Range between 10.34 – 12.05

Three overflow weirs are located on the Crinan Canal Eastern reach and the dimensions included in the model are outlined in Table 4-2. These weirs were represented as 'general weirs' and used default parameters. Only Weir 3 is seen to discharge into the Badden Burn, with weirs 1 and 2 discharging directly into Loch Gilp. 'Dummy sections' based on lidar and normal depth boundaries were applied downstream of the weirs to allow flow from the reservoir unit, representing, the reach in the model. This approach is likely to be slightly conservative as it assumes free flow over the weirs. Sweetening inflows of 0.1m^3/s were also applied downstream of the weirs to allow the model to run at the early stages of the simulation when no water was spilling. These sweetening inflows did not affect flows over the weir or canal embankment that were extracted to be used in the 1D/2D model.

The canal embankment bounding the eastern side of the canal has been surveyed by Scottish Canals and was included as 2 separate spill units leading from the reservoir unit representing the eastern reach. The embankment was split into 2 sections to separate potential flow over the embankment that would spill onto the land and flow that would spill into Loch Gilp.

The full range of return periods, with and without climate change, were run through the canal model for the critical duration of 7h15 (which had been established as the overall critical duration of the model) to establish inflows into the Badden Burn catchment from the overflow weir and spill over the embankment.

The model was run unsteady, representing a full hydrograph, at a 1 second timestep. No alterations were made to default run parameters.

4.4.1 Assumptions

A number of assumptions were made when modelling the Crinan Canal due to uncertainty relating to operation during flood conditions. These assumptions are outlined below:

- Weir boards are in place at weir 1 and weir 3 on the eastern reach of the canal to maintain levels. Weir 2 does not have boards in place. Based on the WCM, these boards should all be removed early in an event once a trigger level is reached on the eastern reach. It has been assumed that these boards are removed, and the weir crest elevations are the spill levels rather than the top of the board elevations. This assumption is conservative as the lower spill will allow increased flow from the canal;

- There was no information on normal water levels in the Crinan Canal so levels were set based on an average of levels from site photographs and water levels from historic surveys;

- Weir boards are also in place at Loch a Bharain. However, there was little information about the operational procedures at this weir. For this reason, the weir crest was taken as top of boards as this would result in higher levels in the summit pound. This approach was deemed appropriately conservative;

- The WCM states that once an initial trigger level in the eastern reach is met, the feeders into the summit pound are stopped. Given that it is also the case that the western lock sluices are opened whilst the eastern reach sluices remain closed, it is likely that minimal flow will spill from the summit pound into the eastern reach. This assumption will be tested within the modelling;

- The WCM states that once a second trigger level is met at each of the 3 overflow weirs, that the sluices in the locks leading out to Loch Gilp are opened to alleviate levels in the eastern reach. However, it was deemed appropriate not to include this functioning in the canal model for several reasons:

    - Opening the sluices relies on manual operation. It is unclear how long after the trigger level is met that this would occur therefore how much higher water levels in the eastern reach would rise;

    - The impact of opening the sluices could not be readily assessed and was also not proportional to this study. Each sluice was approximately 1 x 0.6m but invert levels were not known. The inflow into the canal is relatively high and the downstream face of the sluice would be submerged; limiting flow out to sea;

    - The trigger level is well below the top of the canal embankment and it is likely that the influence of opening the sluice gates would draw down water in the eastern reach before it reached the embankment crest level. It was therefore considered conservative to not include the sluice operation in the model as this would represent the worst case situation;

        - As an example, the water level in the eastern reach during a 0.5% AEP event reaches 10.44mAOD. The trigger level at which the sluices out to sea would be opened is 10.3m.

- Breach of the canal embankment and upstream reservoirs have not been modelled;

- The modelling is based on a conservative assumption that the sluice gates at Ardrishaig are not operated during the event. This would be contrary to the operational regime of the canal and therefore unlikely and demonstrates a worst case scenario with regard to Lochgilphead.

Whilst it is not proportional to assess the canal interaction in more detail within this study, it is recommended that Scottish Canals be consulted and any relevant information be passed on for future use that may aid operations and flood alleviation in the future.

4.5 Canal outflow results

As stated above, flow into the summit pound is stopped once an initial trigger level is met in the eastern reach, with flow being diverted to the west from the reservoirs and out to sea. The western sluices are also opened to draw down the summit pound whilst the eastern sluices remain closed. No flow is observed to spill from the summit pound into the eastern reach before the trigger level is met for all modelled return periods. Therefore, no flow is modelled as entering the eastern reach from the summit pound during a storm event.

Table 4-3 displays the peak flows over weir 3 and over the canal embankment with and without climate change. Spill over the canal embankment is only observed between Cairnbaan and 350m upstream of the Meadows. No spill is observed downstream of the Meadows. As discussed previously, it is likely that the flow over the embankment is conservative as it assumes the operational procedure outlined in the WCM of opening the sluice gates to Loch Gilp is not undertaken. A sensitivity run was undertaken in the 1D/2D hydraulic model to understand how inclusion of this spill affects flooding in Lochgilphead.

Excluding all spill over the canal embankment for the 0.5% AEP fluvial event, was seen to reduce flooding upstream of the Meadows area around the A816 by up to 100mm when compared with the simulation that does include the canal spill. A reduction in flood extent is also noted at the ABC plant yard. Within town, the caravan park is shown to have no change in flood depth, with minor decreases in the open land between the caravan park and Bishopton Road. Given that few key receptors are affected by the removal of the canal spill, it was deemed appropriately conservative to maintain the spill over the canal embankment due to the uncertainties relating to the sluice operations.

These spills were input into the model as 2D inflow hydrographs as outlined in Section 7.1.2.

Table 4-3: Peak spills from the canal

AEP % event	Spill over weir 3 – current day (m^3/s)	Spill over weir 3 – climate change (m^3/s)	Spill over embankment to Meadows area– current day (m^3/s)	Spill over embankment to Meadows area – climate change (m^3/s)
50%	2.15	2.94	0.00	0.58
20%	2.81	3.31	0.13	4.03
10%	3.10	3.44	1.52	6.51
5%	3.27	3.54	3.42	8.96
2%	3.43	3.64	6.20	12.13
1%	3.51	3.87	8.31	13.42
0.5%	3.6	4.31	10.62	14.48
0.1%	4.41	6.04	14.69	18.44

It can be seen from Table 4-3 that significant flow is modelled to spill over the canal embankment. However, through the sensitivity testing of removing the spill, it was seen that the fluvial catchment, and its key receptors, was not sensitive to changes in the flow over the embankment. For the purposes of this study, it is appropriately conservative to maintain all spill from the canal.

5. Joint probability

A joint probability assessment was undertaken to establish how the sea and watercourses interacted. The likelihood of coastal and fluvial interactions can be assessed in a number of ways, through statistical approaches or sensitivity testing within the model. This analysis will determine whether flooding from both sources occurring together is likely or not.

Using the DEFRA/Environment Agency guidance (Technical report FD2308/TR2) it is seen that there are no sites that are particularly close to Lochgilphead, with the closest sites seen to show a modest correlation between fluvial and coastal events. There is also no gauged data from the surrounding areas that could be used to establish joint probability likelihood. Therefore, the most appropriate way to determine joint probability is through sensitivity testing.

The naming of events in this joint probability testing is as follows:

$$Q_x \ T_x$$

Q refers to the fluvial event, T refers to the tidal event. A range of fluvial and tidal events will be run to assess the dependence of the two sources.

The sensitivity testing involved running a high fluvial event with a range of coastal levels to identify if there were any key areas where flooding got worse during a combined event (i.e. $Q_{05\%}$ $T_{10\%}$) than from a single source event ($Q_{0.5\%}$ T_{MHWS}). The same combinations were run for a Tidal event, with a high tide of 0.5% AEP with a range of fluvial flows. This analysis was also undertaken for climate change scenarios. For the single source simulations, a tide level of the Mean High Water Spring (MHWS) level was applied to the high fluvial flows, and a flow equating to QMED / 2 was applied to the high tidal levels.

MHWS level was taken from the nearest point to Lochgilphead in the 2018 CFB extreme sea level dataset. The 2018 CFB estuary dataset contains a point closer to Lochgilphead within the intertidal area but does not contain MHWS levels. Therefore, the ratio of uplift between the closest point to Lochgilphead and the closest point to the extreme sea level point was applied to the MWHS level. This raised the MHWS level from 1.9 to 2.05mAOD. The MHWS level with climate change was uplifted by the same uplift as in the 50% AEP event and equates to 2.68mAOD.

This sensitivity testing would be used to establish whether the fluvial and tidal events were independent of each other or whether there were areas of interest where flood depths were greatest during a combined event.

The sensitivity simulations undertaken are as follows:

High fluvial event	High tidal event
• $Q_{0.5\%}$ x $T_{0.5\%}$	• $Q_{0.5\%}$ x $T_{0.5\%}$
• $Q_{0.5\%}$ x $T_{10\%}$	• $Q_{10\%}$ x $T_{0.5\%}$
• $Q_{0.5\%}$ x T_{MHWS}	• Q_{Low} x $T_{0.5\%}$

5.1 Present day joint probability simulations

Assessing the results of the fluvial simulations, it was found that changing the tidal event did not affect flood depths upstream of Bishopton Road on the Badden Burn. Tidal levels also did not affect flood depths upstream of the Bowling Green on the Cuilarstich Burn. These areas were therefore considered as fluvially dominated. Downstream of Bishopton Road and the Bowling Green, flood depths were seen to vary depending on the tidal event, which signified these areas are tidally dominated.

On comparing the tidal results, changing the fluvial event was found to alter flood depths in some areas downstream of Bishopton Road and the Bowling green. Depths in the caravan park increased by approximately 7mm and the open areas adjacent to Highbank Park increased by up to 40mm when comparing the $Q_{10\%}$ $T_{0.5\%}$ and Q_{Low} $T_{0.5\%}$ events. However, given there were no flood receptors in this area, this increase was not considered significant and it was concluded that it was not fluvially sensitive in these areas.

For the present day scenario, there were no areas of interest that showed material increases flooding in a combined event.

Greatest flood depths upstream of Bishopton Road and the Bowling Green were experienced during a fluvial event and areas downstream of Bishopton Road and the Bowling Green experienced the greatest flood depths during a tidal event.

5.2 Climate change joint probability simulations

Assessing the results of the fluvial simulation, it was again found that changing the tidal event did not affect flood depths upstream of Bishopton road or the Bowling Green. These areas were therefore considered as fluvially dominated. Downstream of Bishopton Road and the Bowling Green, flood depths were seen to vary depending on the tidal event, which signified these areas are tidally dominated.

On comparing the tidal results, changing the fluvial event was found to alter flood depths in some areas downstream of Bishopton Road and the Bowling green. Depths in the caravan park increased by approximately 3mm and the open areas adjacent to Highbank Park increased by up to 10mm when comparing the $Q_{10\%}$ $T_{0.5\%}$ and Q_{Low} $T_{0.5\%}$ events.

Only one area of interest in a combined event compared to a fluvial only or tidal only simulation showed increased flooding. This affected a number of properties at Highbank Park. Comparison the $Q_{10\%}$ $T_{0.5\%}$ and Q_{Low} $T_{0.5\%}$ flood depths at properties showed that the $Q_{10\%}$ $T_{0.5\%}$ produced depths up to 50mm higher than the Q_{Low} $T_{0.5\%}$ depths. Given the very localised nature of this situation, with approximately 3 properties affected, it was considered appropriate to undertake sensitivity testing in the damage assessment to understand how this difference would affect the overall damages. The damages would be assessed in phase 3 of this study.

5.3 Joint probability conclusions

It was seen that in both the present day and climate change scenarios that the areas upstream of Bishopton Road and the Bowling Green were fluvially dominated and the areas downstream of those points were tidally dominated. Through the sensitivity testing, it was shown that the fluvial and tidal scenarios are independent of each other.

Therefore, a range of fluvial simulations, with and without climate change, and a range of tidal simulations, with and without climate change, will be run to fully assess flooding in the study area.

6. Coastal Modelling

6.1 Introduction

The main objective of the coastal modelling exercise is to establish the nearshore extreme sea level and wave characteristics along the frontage at Lochgilphead. In order to achieve this, AECOM has undertaken a numerical modelling study to investigate the existing and future (up to the year 2100) wave climate.

Specifically, this exercise includes the following:

- Review of existing information available to support the study including previous investigations;
- Long-term (38 years) wave transformation modelling to determine local marginal wave extremes;
- Joint probability analysis of waves and water levels local to Lochgilphead for present day and future (2100) epoch; and
- Overtopping assessment at four locations along the frontage of Lochgilphead shown later in this section;

6.2 Data review

The first step in the coastal modelling study was to establish any relevant information over the project area and wider environment. As part of the data review AECOM undertook a site visit to establish first-hand the conditions and constraints affecting conditions at the site. Photographs can be found in Appendix A.

AECOM identified a number of key bathymetric and topographic datasets that were available for use in the study which are outlined in the following sections.

6.2.1 Bathymetry and Topographic Data

A topographic survey was undertaken as part of this study and this data was used in combination with existing data. The key datasets that have been used include:

- C-MAP data - The bathymetry data has been obtained through the C-MAP digital archive which AECOM has access to under licence. The C-MAP dataset is derived from digitized contours and sounding data taken from published Admiralty charts. Datasets are provided in XYZ format. Figure 6-1 shows the coverage and density of data points included the C-MAP dataset;
- Bathymetric survey of the intertidal area (AECOM, 2018); and
- Topographic long sections of representative cross sections perpendicular and parallel to the frontage as well as spot heights surveyed in 2018 as part of this study. Data shown in Appendix D.

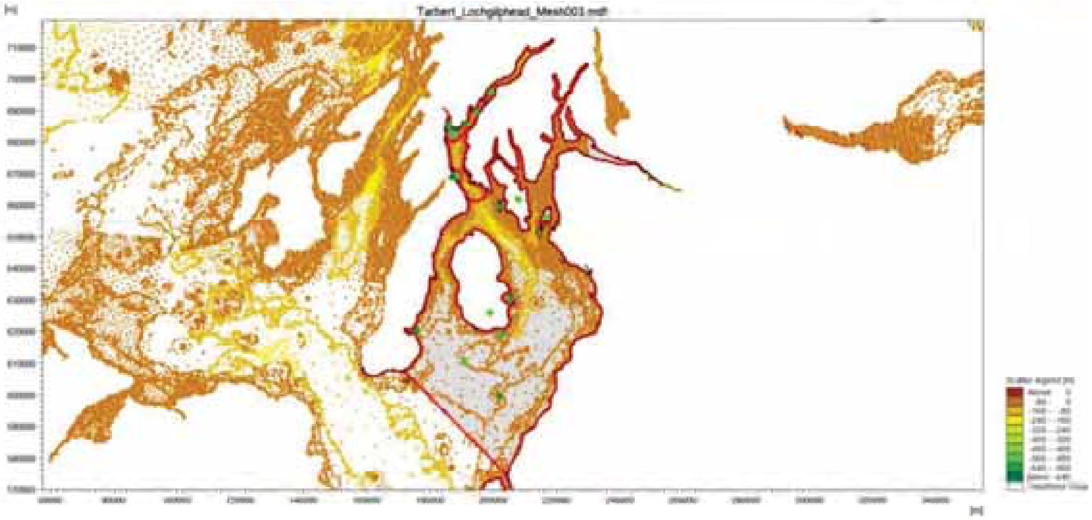

Figure 6-1: Tarbert and Lochgilphead Offshore Bathymetry Data –Vertical Datum m ODN

6.2.2 Water Levels

Extreme water levels include tides, sea level rise and surge. The extreme values for present day (2018) at Lochgilphead are available from the Coastal Flooding Boundary (CFB) dataset, Scottish Environment Protection Agency (SEPA). The surge shape has been taken from the closest site of Millport. The CFB dataset has been developed to inform work around the coast ranging from coastal flood modelling, scheme design, strategic planning and flood risk assessments. Figure 6.2 shows closest available data points of still water level. The point labelled '1876-8-Main-A' has been selected for Lochgilphead.

The present day sea levels were calculated by adjusting the SEPA CFB dataset, with a baseline year of 2008, by adding an allowance for sea level rise. In order to consider climate change for the future time epoch in 2100, the present day extreme water levels were factored with UKCP09 95th percentile high emission scenario (including surge) sea level rise projections. Sea level rise was added to present day and no multiplier was applied. The results show that the corresponding increase in sea level is approximately 630mm in 2100 at Lochgilphead. Table 6-1 provides extreme water levels for present day (2018) and long term epoch in 2100.

Revised climate change from recently issued guidance on UK Climate Projections (UKCP18), has been made publicly available which comes after the modelling was completed for the Lochgilphead Flood Study. UKCP18 provides the most up-to-date consideration of how the climate may change up to 2100 and beyond. The UKCP18 guidance has been reviewed to consider any implications to the modelling completed for the Lochgilphead study.

To make a comparison between the work undertaken to date and UKCP18 output, the Sea Level Rise (SLR) was downloaded from the UKCP18 website (https://ukclimateprojections-ui.metoffice.gov.uk/). Sea level rise relative to 2008 were derived based on RCP8.5 scenario (unmitigated emissions) 50th percentile and 95th percentile which estimates global average temperature rise of 4.3 deg. by 2100 compared to pre-industrial period. RCP8.5 is most similar to UKCP09 high emissions scenario. Sea level rise from RCP8.5 scenario is 0.52m (50th percentile) and 0.86m (95th percentile) respectively. Sea level rise used in this study was 0.63m based on UKCP09 95th percentile high emission scenario, which is located between 0.52m (50th percentile) and 0.86m (95th percentile) from RCP8.5 scenario. Considering the small difference and climate change uncertainty, the recalculations for wave overtopping and updating the inundation modelling is not considered to be necessary.

Figure 6-2: Available data points still water levels

Water level profiles are required to provide an estimate of flood duration along the study frontage. The water level profiles at Lochgilphead were derived using the CFB database. The surge shape at the closest site Millport has been adopted. Tide data at Lochgilphead was obtained from Admiralty Tide Tables (ATT) for the underlying astronomical tide curve Figure 6-3 and Figure 6-4 show the water level profiles for 2018 and 2100 respectively.

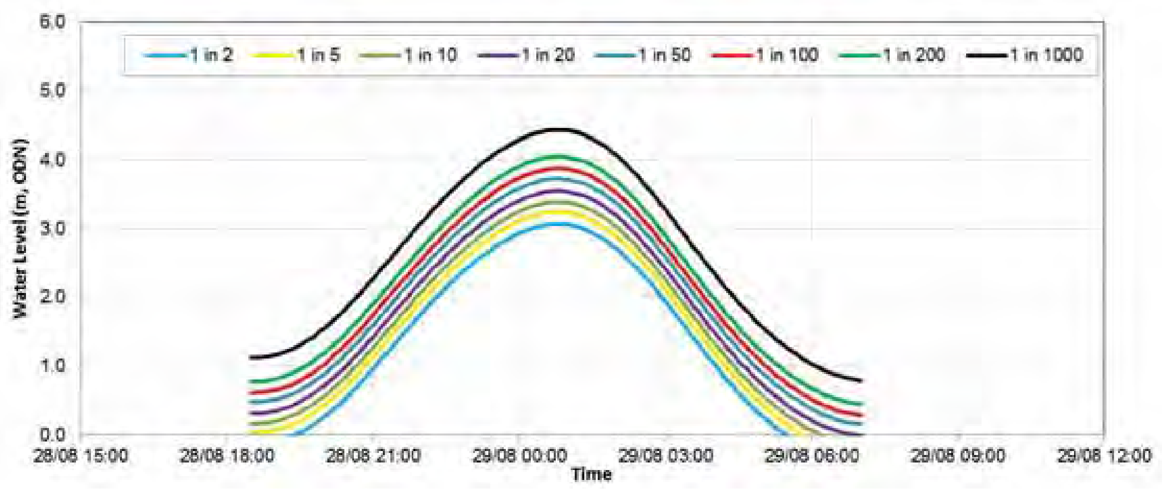

Figure 6-3: Water level profiles (2018)

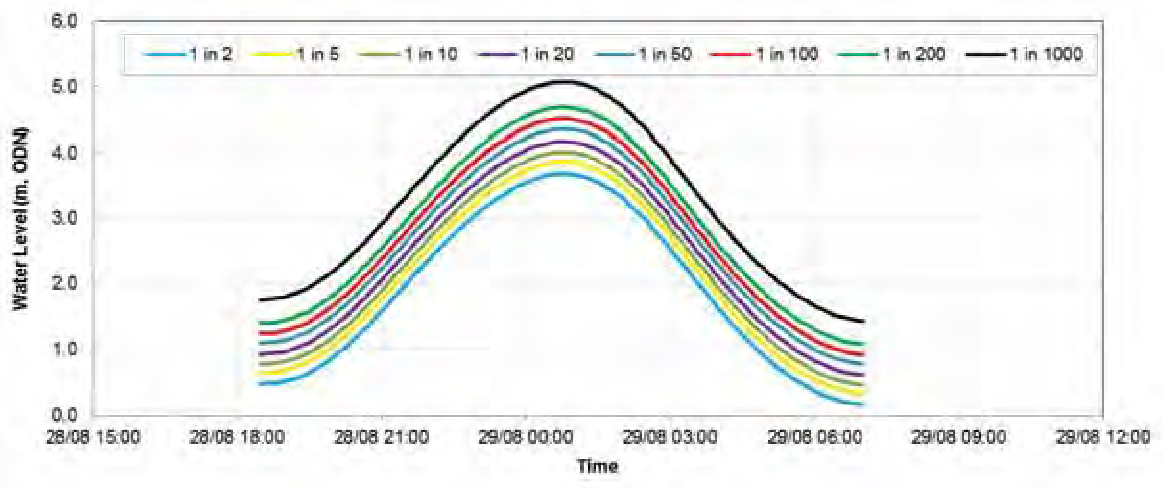

Figure 6-4: Water level profiles (2100)

Table 6-1: Coastal Boundary extreme water levels for chainage 1876-Main-A

% Annual Exceedance Probability (Return Period)	Present Day (2018) Level m AOD	Future (2100) High Emission Level m AOD
50 (2)	3.07	3.68
20 (5)	3.25	3.87
10 (10)	3.39	4.01
5 (20)	3.55	4.18
2 (50)	3.73	4.36
1 (100)	3.88	4.52
0.5 (200)	4.05	4.69
0.1 (1000)	4.43	5.08

6.2.3 Offshore Wave Conditions

Offshore wave conditions have been obtained under licence from the Met Office WaveWatch III European Waters model. The model has a grid resolution of approximately 8 km which is adequate for representing wave fields in the deep waters of the Atlantic and Continental Shelf. The model provides a 38 year dataset for the period January 1980 to December 2017 with wind and wave parameters provided at 3 and 1-hourly intervals, depending on the period. The wave component of the hindcast is generated using the WaveWatch III third-generation spectral wave model (Tolma, 2009)[3]. The model uses a WAM Cycle-4 source term scheme as described by Bidlot (2012)[4] and includes parameterizations for shallow water effects on the wave field.

Information has also been provided to AECOM from the Met Office wave model at three positions (Table 6-1 and Figure 6-3) for use on the project, specifically as input conditions for the regional wave model. Also shown are the times of the predicted largest peak Hs and Tp values.

Table 6-2: Met Office data extract

Position	Latitude	Longitude	Largest Significant Wave Height (Hs)	Maximum Peak Period (Tp)
2161	55.1970	-5.3880	5.69m (8.48 sec, 5th December 2013)	19.23 sec (4.50m, 10th December 2014)
2309	55.6057	-4.9930	3.93m (8.20 sec, 15th January 2015)	18.18 sec (0.01m, 31st August 2011)
2345	55.7597	-5.2860	3.22m (5.78 sec, 5th December 2013)	8.55 sec (0.002m, 6th January 1999)

6.2.4 Wind Data

A 38-year record of wind data at 3-hourly intervals was also obtained from the UK Met Office European Wave model at the positions shown in Figure 6-5 covering the period 1980 to 2017. Figure 6-5 shows the predicted wind rose at the closest available position to Lochgilphead based on a directional resolution of 20°. Winds are seen to be predominately from the South to North Westerly directions. Average wind speeds are around 6 m/s with a maximum wind speed of 30 m/s predicted.

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[3] Tolman, 2009. User Manual and System Documentation of WaveWatch III Version 3.14. Environmental Modelling Centre Marine Modelling and Analysis Branch

[4] Bidlot, J.R. (2012). Present Status of Wave Forecasting at ECMWF. ECMWF Workshop on Ocean Waves

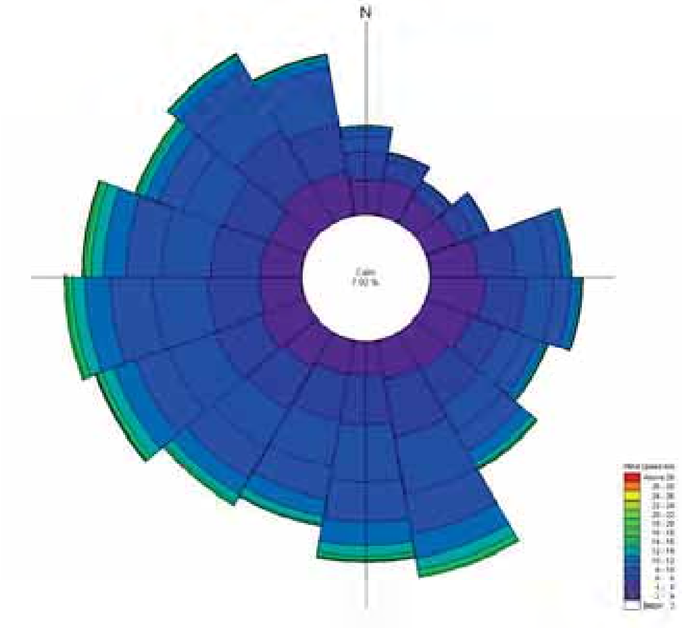


Figure 6-5: Wind rose plot at 2345

Wind time-series data were also obtained from the Allt Dearg Windfarm, located to the South of Lochgilphead, and compared against the Met Office data (Position 2345) for the same available period (2013-2017). Generally the pattern of wind direction was consistent but with larger variations in speeds seen. This difference was attributed to the positioning of the Allt Dearg Windfarm gauge at some 420-450m AOD.

Allt Dearg windfarm is located well above sea level, which reduces usefulness of the data where wind at 10m above sea level is required. The longer time-series Met Office data was therefore deemed most suitable for use in the subsequent modelling exercise and was generally consistent with the measured data noting that differences between the data are likely to be attributed to the significant difference in elevation. Due to the complex nature of local topographical features, a simple correction for the elevation of the windfarm anemometer data would not appropriate.

6.2.5 Climate Change

As agreed in the proposed methodology (AECOM, July 2018), climate change has been considered based on the current UKCP09 / Defra guidance on changes to relative sea levels, wind and wave climate in the future. The UKCP18 data was published after the climate change modelling had been undertaken and has yet to be adopted by SEPA. All data was downloaded from the Defra website (http://ukclimateprojections-ui.metoffice.gov.uk/ui/) for Lochgilphead. Relative sea levels are shown to rise from a baseline year of 2018 based on the ‘High Emissions Change Factor 95th percentile’ (Table 6-1).

The greater extents experienced in the climate change scenarios can be attributed to the significant rise in sea levels, which is predicted to rise by between 0.6 – 0.65m over the next 80 years (Section 6.2.3). This rise will mean that sea levels currently associated with rare events, will become a lot more frequent, resulting in an increase in the frequency of disruptive flooding as 2100 approaches. Table 6-3 sets out what a current day AEP event will correspond to in 2100.

Table 6-3: comparison AEP events – current day vs 2100 predication.

Current day (2018) AEP	Equivalent 2100 AEP event
50%	-
20%	-
10%	-
5%	-
2%	50%
1%	20%
0.5%	10%
0.1%	2%- 1%

6.3 Joint Probability Analysis

6.3.1 Correlation coefficient

Joint probability refers to the chance of two or more conditions occurring at the same time. In this instance, with coastal flood risk management in mind, the coincidence of extreme wave condition and extreme water level is of interest. A Joint Probability Analysis (JPA) of wave and water level was therefore undertaken to provide the possible combination of wave and water level. The simplified JPA approach, as described in the guidance (Use of Joint Probability Methods in Flood Management: A Guide to Best Practice – R&D Technical Report FD2308/TR2, 2005), has been used for the standard set of return periods. The guidance provides two graphs (Figure 6-6 and Figure 6-7) showing the strength of correlation between waves and sea level in UK waters. For all wave directions combined the corrections at Port Patrick and Millport are 'strong correlated'. No information of correlation coefficient is specifically given at Lochgilphead. 'strong correlated' is what has been adopted for this site.

Excel spreadsheets provided with the guidance were used to calculate the tables of joint exceedence extremes and curves, using available information on marginal (single variable) extremes and an estimate of the dependence between the two variables. Inputs supplied by the user include:

- Marginal extremes of wave height and water level;
- Dependence parameters (well);
- Number of records per year;
- Joint exceedence return periods required.

Figure 6-6: Correlation coefficient (Wave Height & Sea Level), All wave direction combined

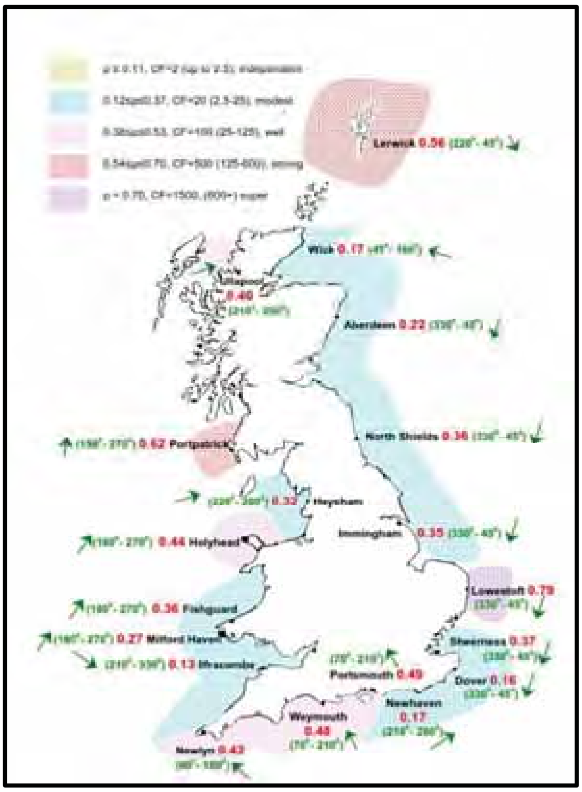


Figure 6-7: Correlation coefficient (p, wave height & sea level) Wave direction where dependence is highest.

6.4 Spectral Wave Modelling

6.4.1 Overview

AECOM has used the MIKE21 Flexible Mesh Spectral Wave (SW) model in the present modelling study. The software was developed by the Danish Hydraulics Institute (DHI).The Spectral Wave model is a state-of-the-art wave transformation model based on triangular mesh elements which are able to provide enhanced resolution covering important features such as local variations in bathymetry. The wave model simulates growth, decay and transformation of wind-generated waves and swells in offshore and coastal areas. The model is capable of reproducing the combined effects of shoaling, refraction, diffraction, reflection, wave breaking and directional spreading. For the present Lochgilphead modelling study, the main processes are shoaling, refraction, wave breaking and directional spreading.

Detailed wave transformation modelling and assessment work have been carried out to estimate the wave overtopping rates along Lochgilphead frontage. The study derived the wave climate and extreme sea levels for the required range of return periods of 2, 5, 10, 20, 50, 100, 200 and 1000 years, with and without climate change. Extreme wave heights have been estimated based on the Weibull probability distribution involving the selection of individual storm events from the peaks over threshold method. Joint probability analysis was then undertaken to establish the possible combinations between wave heights and water levels.

A regional model of the wider Loch Fyne area was modelled as well as a local model, at a finer resolution, of the Lochgilphead intertidal area.

The calculations of wave overtopping were undertaken using EurOtop (2018) ‘Manual on wave overtopping of Sea Defences and Related Structures’ to determine the mean overtopping discharge (l/s/m) for a range of structure types. The manual incorporates new techniques to predict wave overtopping at seawalls, flood embankments, breakwaters and other shoreline structures. Wave overtopping rates along the Lochgilphead defence structures were provided to establish the extent of flooding and identify the level of risk from coastal flooding for a range of return periods.

6.4.2 Model setup

The regional wave model mesh (Figure 6-8Table 6-8) has a resolution of between 1000 and 100 m, with the highest mesh resolution applied in the areas of interest adjacent to Lochgilphead. The Spectral Wave model was used to simulate the full data period available from the Met Office (38 years) Wave Watch III European Wave Model.

The model has been configured based on the recommended default parameters within the model setup. A directionally decoupled approach was adopted in which the full directional spectrum of wave directions were considered. This approach allows for waves coming from any direction to be considered within the model.

6.4.3 Regional modelling results

The model domain covers the entire Firth of Clyde. Figure 6-8 shows the model flexible mesh generated using the MIKE-Zero Mesh Generator. The model is based on two types of bathymetric data sources, C-MAP and LiDAR survey data. For the majority of the model domain water depth information was taken from C-Map digital chart data. This has been supplemented with local LiDAR survey data made available by the client. These datasets were interpolated onto the model mesh. The bathymetry is referenced to the British National Grid horizontal projection and ODN vertical datum.

Hindcast offshore wave data (wave height, period and direction) and wind conditions (speed and direction) have been taken from the Met Office WaveWatch III (WW3) model. The time-series data is available at hourly intervals for the years 2001 to 2017 and 3-hourly between 1980 and 2001. Figure 6-9 and Figure 6-10 show wave and wind rose at the model boundary. The directional resolution in each wave and wind rose plot is 15º.

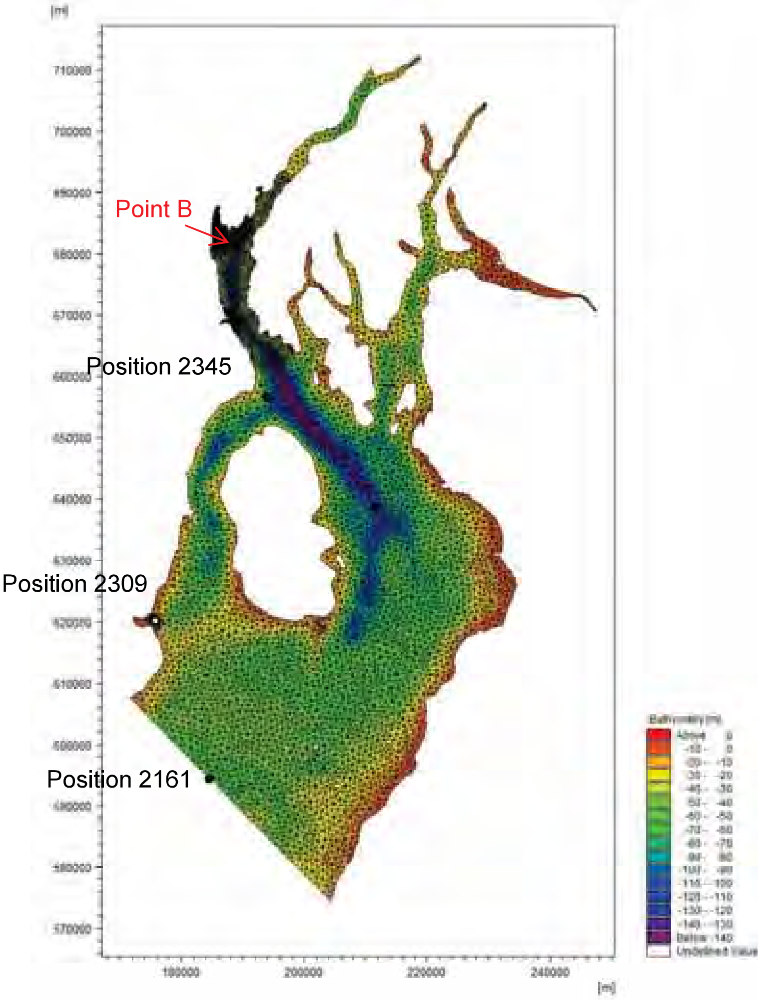


Figure 6-8: Model domain and data points

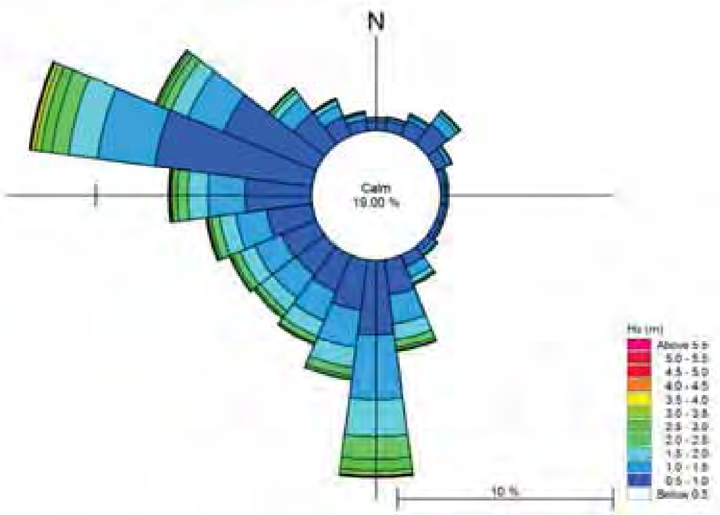


Figure 6-9: Wave rose (entrance to Loch Gilp)

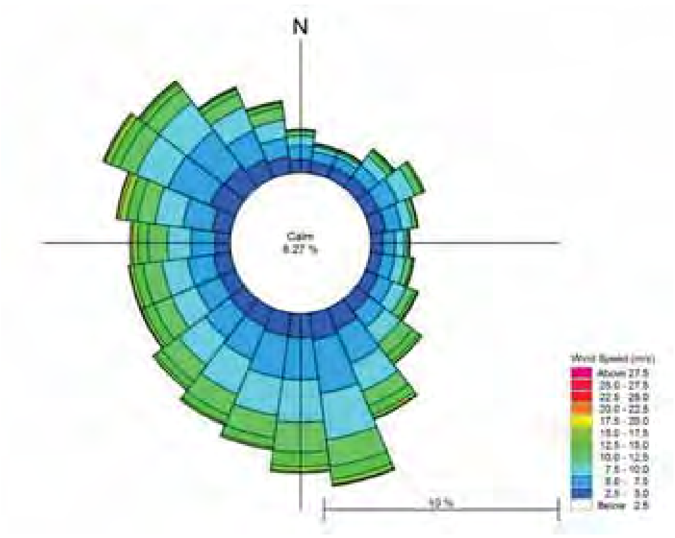


Figure 6-10: Wind rose (entrance to Loch Gilp)

In the absence of a long time-series of water levels (tide and surge), the regional wave transformation modelling was initially undertaken with a constant water level of 2.7m ODN equivalent to a 1 in 1 year return period. Wave data at Point B (Figure 6-8) near the entrance of Lochgilphead were extracted for the local wave transformation. At this location, the water depth is -23m and it is not expected that depth limited wave breaking would occur.

Wave statistics along the study frontage are simulated using a near field SW model, as described later.

6.4.4 Joint probability analysis

The transformed wave conditions derived from the modelling exercise were obtained for a point adjacent to the entrance of the channel leading to Lochgilphead (Table 6-4) and applied in the joint probability analysis for Lochgilphead.

Table 6-4: Results extraction location

Extraction Site	Easting (km)	Northing (km)
Lochgilphead Entrance	180500	685000

The results of joint probability analysis in Sector 165° are given in Table 6-5 to Table 6-8. Table 6-5 and Table 6-7 provide the marginal distributions of significant wave height and still water level for a combination of extreme wave height and water level for the present day (2018) and time epoch in 2100. Table 6-6 and Table 6-8Table 6-9 present the joint exceedance return periods for the present and climate change scenarios respectively. They are the likely 84 combinations between wave and water level, which have been used as inputs for wave overtopping calculations. Figure 6-11 and Figure 6-12Table 6-12 show the plots of joint probability distribution for present day scenario and climate change scenario in 2100.

Table 6-5; Marginal wave height and water level extremes in Sector 165° (21018)

RP (years)	Significant Wave Height [m]	Still Water Level [m ODN]
2	2.43	3.07
5	2.73	3.25
10	2.96	3.39
20	3.20	3.55
50	3.53	3.73
100	3.77	3.88
200	4.03	4.05
1000	4.64	4.43

Table 6-6: Distribution of wave height and water level in Sector 165° (2018)

Hs (m)	Joint exceedence return period (years)							
	2	**5**	**10**	**20**	**50**	**100**	**200**	**1000**
	Still water Level from SEPA [m ODN]							
0.79	3.07	3.25	3.39	3.55	3.73	3.88	4.05	4.43
1.13	3.07	3.25	3.39	3.55	3.73	3.88	4.05	4.43
1.46	3.05	3.25	3.39	3.55	3.73	3.88	4.05	4.43
1.61	2.89	3.19	3.39	3.55	3.73	3.88	4.05	4.43
1.93	2.69	3.00	3.22	3.44	3.73	3.88	4.05	4.43
2.21	2.54	2.84	3.08	3.29	3.59	3.81	4.05	4.43
2.43	2.39	2.69	2.92	3.15	3.44	3.66	3.89	4.43
2.73		2.50	2.72	2.95	3.25	3.47	3.69	4.26
2.96			2.57	2.80	3.11	3.32	3.56	4.09
3.20				2.65	2.96	3.18	3.40	3.93
3.53					2.76	2.99	3.21	3.72
3.77						2.84	3.07	3.59
4.03							2.91	3.43
4.64								3.10

Table 6-7: Marginal wave height and water level extremes in Sector 165° (2100)

RP (years)	Significant Wave Height [m]	Still Water Level [m ODN]
2	2.68	3.68
5	3.01	3.87
10	3.26	4.01
20	3.52	4.18
50	3.88	4.36
100	4.15	4.51
200	4.43	4.69
1000	5.10	5.08

Table 6-8: Distribution of Wave Height and Water Level for the Worst Case 165°N direction (2100)

Hs (m)	Joint exceedence return period (years)							
	2	5	10	20	50	100	200	1000
	Still water Level from SEPA [m ODN]							
0.87	3.68	3.87	4.01	4.18	4.36	4.51	4.69	5.08
1.25	3.68	3.87	4.01	4.18	4.36	4.51	4.69	5.08
1.61	3.66	3.87	4.01	4.18	4.36	4.51	4.69	5.08
1.77	3.50	3.80	4.01	4.18	4.36	4.51	4.69	5.08
2.13	3.30	3.61	3.83	4.06	4.36	4.51	4.69	5.08
2.44	3.14	3.45	3.69	3.91	4.22	4.44	4.69	5.08
2.68	2.99	3.30	3.53	3.77	4.06	4.29	4.52	5.08
3.01		3.10	3.33	3.57	3.86	4.09	4.32	4.90
3.26			3.18	3.41	3.72	3.94	4.18	4.73
3.52				3.26	3.57	3.80	4.02	4.56
3.88					3.36	3.60	3.83	4.35
4.15						3.44	3.69	4.21
4.43							3.52	4.05
5.10								3.72

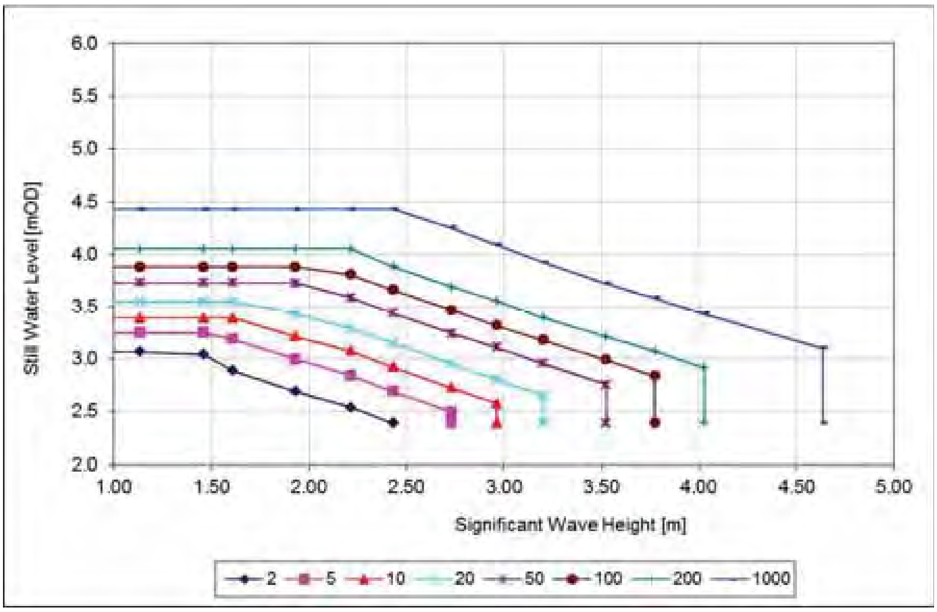


Figure 6-11: Joint probability distribution of wave height and still water level for the 165°N direction (2018)

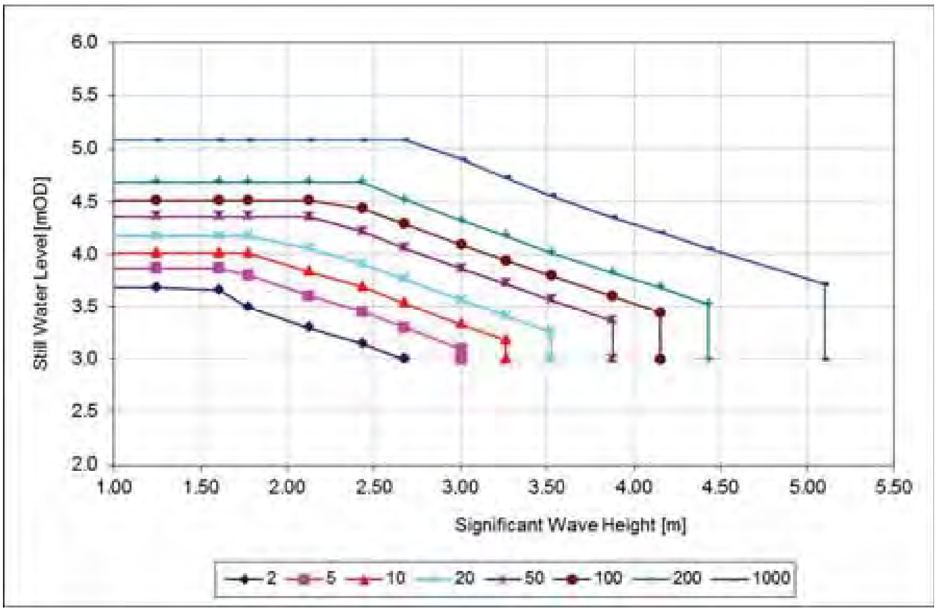


Figure 6-12: Joint probability distribution of wave height and still water level for the 165°N direction (2100)

6.4.5 Local wave model

In some circumstances wave heights at the toe of the defence structure will be depth limited and use of a fixed water level may lead to an underestimate of wave heights at the toe. Wave statistics were analysed and derived at Point B (-23.0mAOD contour line) where no depth-limited wave breaking occur. Joint probability analysis was used to generate all combinations of water level and wave height at Point B.

A local model (Figure 6-13) was set up to transform offshore waves to the toe of coastal structures for the range of water level and wave conditions. The mesh resolution across the local model domain is variable with high resolution around Lochgilphead. In the offshore area this resolution is around 50 m increasing to 10 m for the areas of most interest (Figure 6-14).

Figure 6-15 shows the wave rose for a location on the local model boundary. The directional resolution of the rose plot is 15º. The wave rose indicates that prevailing waves come from the sector between south-east and south. Wave conditions have been investigated for three sectors with directional resolution of 15°, i.e. 150°, 165° and 180°. The sensitivity study indicated that waves from Sector 165° will generate the largest wave overtopping rates at Lochgilphead. This shows that the regional and local model correlate.

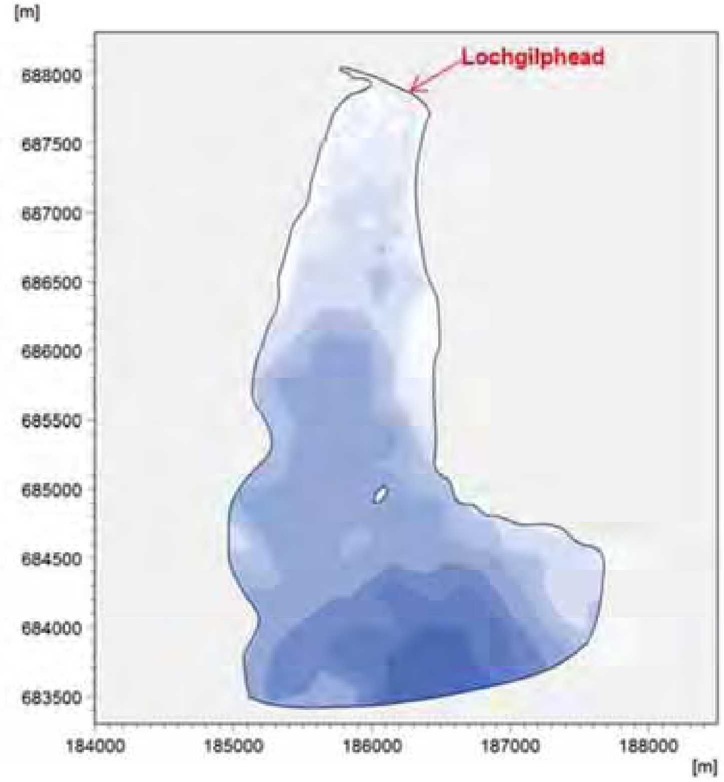


Figure 6-13: Local Wave Model Domain

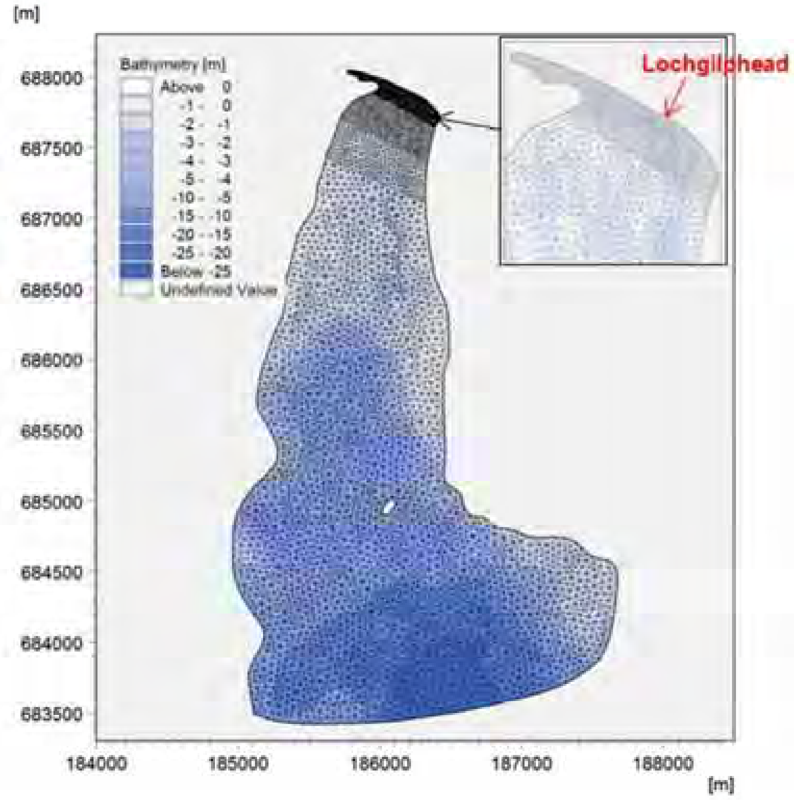


Figure 6-14: Local Wave Model Mesh

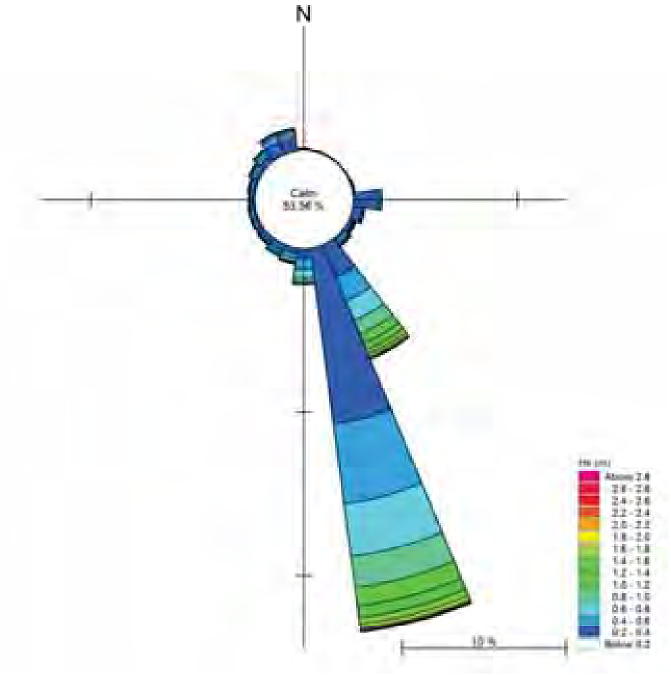


Figure 6-15: Wave rose at the local model boundary (165°N direction)

For winds and waves, there is no specific guidance available to estimate their increase under a high emission scenario. The allowances provided in Environment Agency Guidance ‘Flood risk assessments: climate change allowances’ (https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances) were adopted to estimate the increase, which is accepted by SEPA. This requires a 10% increase in wind speed and wave height. The present and future predicted extreme water levels and wave heights at the local model boundary are presented in Table 6-9.

Table 6-9: Wave conditions in Sector 165° (present day and 2100)

RP (years)	Significant Wave Height [m] (present day)	Significant Wave Height [m] (2100)
2	2.43	2.68
5	2.73	3.01
10	2.96	3.26
20	3.20	3.52
50	3.53	3.88
100	3.77	4.15
200	4.03	4.43
1000	4.64	5.10

The results from the regional modelling have been used to provide boundary conditions for the local wave model of Lochgilphead. Figure 6-16 shows an example 1 in 100 year RP event – Present day condition from the near field model. Wave heights are predicted to rapidly reduce as they enter Lochgilphead due to the limited water depths over the shallow intertidal areas.

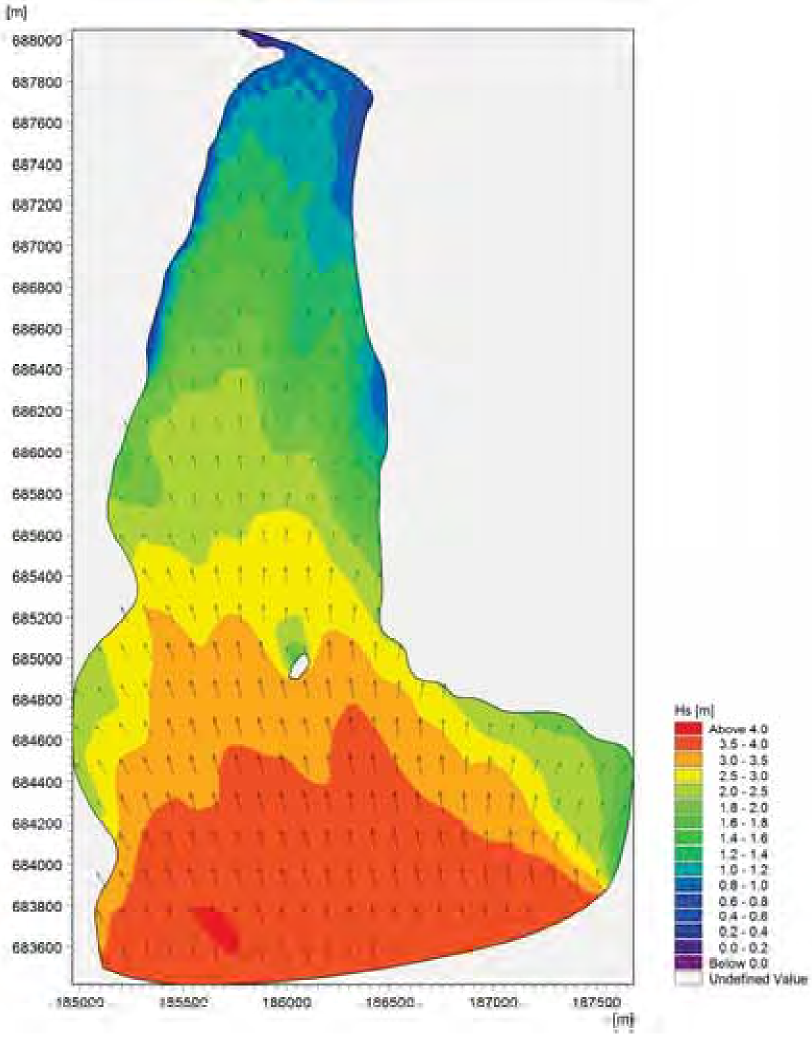


Figure 6-16: Example of Significant Wave Heights and Directions predicted for a 1 in 100 Return Period Event

6.5 Wave overtopping assessment

Wave overtopping calculations for the coastal defence structures were undertaken to identify the level of risk from coastal flooding for a range of return periods. The calculations were carried out using the empirical formulae provided in EurOtop (2018) 'Manual on wave overtopping of Sea Defences and Related Structures' to determine the overtopping discharge (l/s/m) along the Lochgilphead frontage.

At present, the EurOtop guidance (2018) is regarded as best practice within the industry. The required inputs to the calculation vary according to structure type. For the vertical defence structure at Lochgilpead, the inputs typically consist of:

- Significant wave height (m);
- Mean wave period (s);
- Wave direction;
- Structure freeboard (m);
- Water depth at the structure toe (m).

For each of the 4 identified cross-sections shown in Figure 6-17, the typical geometry of the defence structure (crest height, bed level, slope etc.) was established using the detailed topographical survey information (AECOM, 2018). The estimated wave overtopping rates based on EurOtop (2018) are presented in Table 6-10 and Table 6-11.

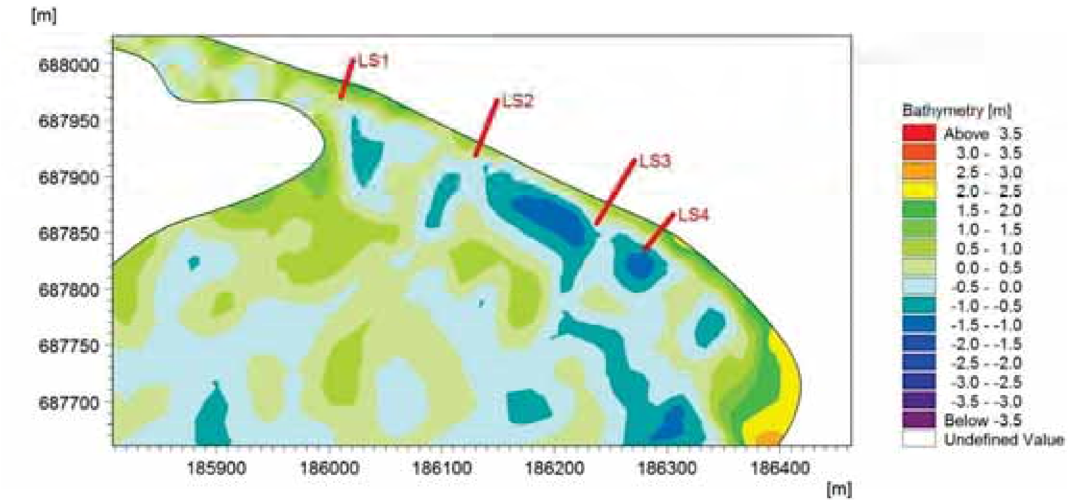


Figure 6-17: Overtopping Cross-section locations

Table 6-10: Present Day (2018) Maximum Wave Conditions at Cross-Sections LS1, LS2, LS3 and LS4

2018

RP (yrs)	LS1			LS2			LS3			LS4		
	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)
2	0.41	3.4	2.7	0.61	3.8	2.7	0.54	3.5	2.7	0.58	3.8	2.7
5	0.5	3.8	2.7	0.69	4.2	2.7	0.67	4.0	2.7	0.65	4.0	2.7
10	0.54	4.0	2.7	0.74	4.4	2.7	0.72	4.2	2.7	0.70	4.0	2.7
20	0.55	4.4	2.7	0.76	4.7	2.7	0.78	4.4	2.7	0.73	4.0	2.7
50	0.61	4.6	2.8	0.82	4.9	2.8	0.84	4.8	2.8	0.73	4.0	2.8
100	0.62	4.7	2.8	0.87	5.0	2.8	0.87	5.1	2.8	0.74	4.0	2.8
200	0.66	4.7	2.9	0.92	5.1	2.9	0.91	5.2	2.9	0.75	4.0	2.9
1000	0.78	4.7	3.1	1.05	5.2	3.1	1.04	5.2	3.1	0.84	4.1	3.1

* Hs is wave height, Tm is mean wave period and WL is wave length

Bed levels at cross-section LS4 are at approx. +1m ODN, whereas at LS2 and LS3 there are around +0.5m ODN, the greater water depths result in slightly higher wave conditions fronting sections LS2 and LS3.

Table 6-11: Future (2100) Maximum Wave Conditions at Cross-Sections LS1, LS2, LS3 and LS4

2100

RP (yrs)	LS1			LS2			LS3			LS4		
	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)	Hs (m)	Tm (s)	WL (m)
2	0.59	4.1	3.0	0.82	4.3	3.0	0.80	4.3	3.0	0.58	3.5	3.5
5	0.65	4.3	3.1	0.89	4.6	3.1	0.87	4.6	3.1	0.64	3.6	3.8
10	0.69	4.6	3.2	0.94	4.8	3.2	0.93	4.8	3.2	0.65	3.6	4.0
20	0.73	4.7	3.3	1.00	4.9	3.3	0.98	4.9	3.3	0.66	3.6	4.2
50	0.79	4.8	3.4	1.07	5.1	3.4	1.05	5.1	3.4	0.77	3.8	4.4
100	0.84	5.0	3.4	1.12	5.2	3.4	1.10	5.3	3.4	0.78	3.8	4.5
200	0.89	5.0	3.5	1.18	5.3	3.5	1.16	5.3	3.5	0.88	4.0	4.6
1000	1.03	5.0	3.7	1.24	5.4	3.7	1.31	5.4	3.7	1.10	4.8	4.7

* Hs is wave height, Tm is mean wave period and WL is wave length

Table 6-12: Present Day (2018) Maximum Overtopping Rates (l/s/m)

RP	LS1 (XS_001)	LS2 (XS_002)	LS3 (XS_003)	LS4 (XS_004)
2	64.5	119.3	99.2	0.6
5	88.8	148.4	142.6	1.2
10	100.7	157.2	160.1	1.9
20	113.2	176.9	180.5	3.5
50	137.8	207.2	201.8	8.0
100	143.8	232.3	221.4	17.5
200	161.1	257.0	244.7	43.7
1000	207.6	319.9	304.3	196.3

The overtopping rates at section 4 are lower than the other sections due to smaller wave heights and wave periods as the wave break earlier due to the topography.

Table 6-13: Future (2100) Maximum Overtopping Rates (l/s/m) (2100)*

RP	LS1 (XS_001)	LS2 (XS_002)	LS3 (XS_003)	LS4 (XS_004)
2	119.9	180.5	189.3	4.0
5	144.3	218.5	203.1	9.3
10	166.3	250.2	233.0	23.3
20	187.5	280.7	261.6	48.3
50	218.7	324.7	302.7	136.0
100	244.5	359.7	335.7	231.2
200*	270.9	395.5	369.2	470.0
1000*	338.7	484.5	453.4	586.3

*Maximum overtopping rates for the 200 and 1000 RP events for LS1, LS2 and LS3 are limited to lower water levels conditions which do not exceed the crest elevation of these cross-sections. At section LS4, the higher crest elevtaion allows for the maximum water level and wave conditions.

The frequnecy of overtopping, seen from the 1 in 2yr/ 50% AEP event, ties in with anecdotal accounts from both SEPA and resiendents.

6.6 Conclusions

A regional wave model was run to establish the offshore wave heights at Lochgilphead under present day conditions. The regional modelling results show that the wave climate at the entrance to Lochgilphead (around Ardrishaig) is moderate (< 3 m) although a maximum significant wave height of 3.5 m was predicted over the available 38 year hindcast period. A joint probability analysis of wave heights and water levels was undertaken for present day condition and a future (2100) epoch. The results from this extremes analysis were then used as boundary conditions for a local high resolution model.

The findings from the local model for the present day scenario show that wave conditions within Lochgilphead are small, with a 1% AEP event producing wave heights in the region of 0.8 m. The small waves can be attributed to the extensive shallow bathymetry within Lochgilphead.

Wave overtopping was undertaken to consider the impact on the nearshore environment around Lochgilphead where current crest elevations (around +2.8m ODN for sections LS1 to LS3) are similar to current day highest astronomical tidal levels, therefore, these small wave events will have a notable impact on local flood risk. Crest levels at position LS4 are significantly higher (+4.5m ODN) resulting in much smaller wave overtopping volumes.

The final analysis from the coastal modelling involves the calculation of the overtopping rates based on the findings from the local wave modelling. These results have shown that overtopping rates of 232.2 l/s/m (2018) and 359.7 l/s/m (2100) can be expected for an extreme 1 in 100yr event at Lochgilpead respectively. The findings from the overtopping analysis have been used as boundary conditions in the inundation mapping (Section 7), showing extent and depth of water intrusion into Lochgilphead.

Tidal levels to be applied to the 1D/2D model are shown in Table 6-14.

Table 6-14: Tidal levels to be used in 1D/2D model

% Annual Exceedance Probability (Return Period)	Present Day (2018) Level m AOD	Future (2100) High Emission Level m AOD
50 (2)	3.07	3.68
20 (5)	3.25	3.87
10 (10)	3.39	4.01
5 (20)	3.55	4.18
2 (50)	3.73	4.36
1 (100)	3.88	4.52
0.5 (200)	4.05	4.69
0.1 (1000)	4.43	5.08

7. Fluvial and coastal inundation hydraulic modelling

7.1 Model Schematisation

A single hydraulic model has been constructed of the Badden Burn and Cuilarstich Burn consisting of a one dimensional element representing the river channels built in Flood Modeller and a two dimensional element representing the floodplain constructed in Tuflow, both being industry standard hydrodynamic modelling software.

7.1.1 One dimensional channel model

A one dimensional model of the Badden and Cuilarstich Burns was constructed using surveyed river cross sections and inline structures. Cross sections, structures and a long section can be found in Appendix E. The survey was undertaken as part of this study and was used to represent the channel geometry and to define the top of bank at each surveyed section. The model consists of 50 surveyed river sections, 9 bridges and 2 culverts with approximately 100m spacing throughout the reach as the channel was relatively uniform. All structures in the modelled reach were included in the model. Where the downstream section of the bridge was not surveyed, a copy of the upstream face was used. During the model runs, all structures were assumed to be clear of obstruction.

The upstream extent of the model on the Badden Burn is located at the junction of the B841 and A816 near Cairnbaan, to allow for flooding along the A816 to be fully assessed. On the Cuilarstich Burn, the upstream extent of the model is located 200m upstream of the bridge at Bishopton Road. The model was not extended further on this watercourse as the banks rise steeply upstream of this point. The 1D model extends to the tidal limit of the watercourses, downstream of Poltalloch Street. Section labelling can be seen in Figure 7-1 and Figure 7-2 will be referenced later in the report to describe flood risk at certain locations.

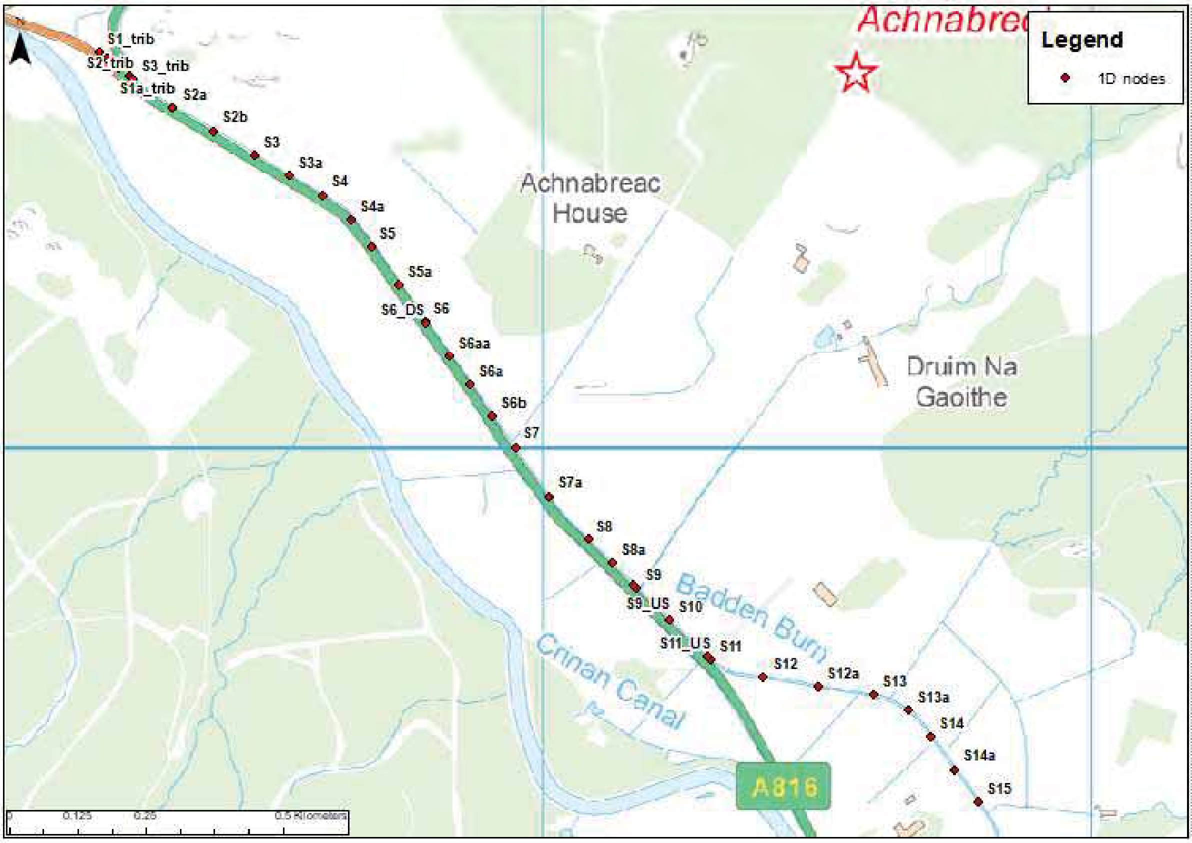


Figure 7-1: 1D node locations

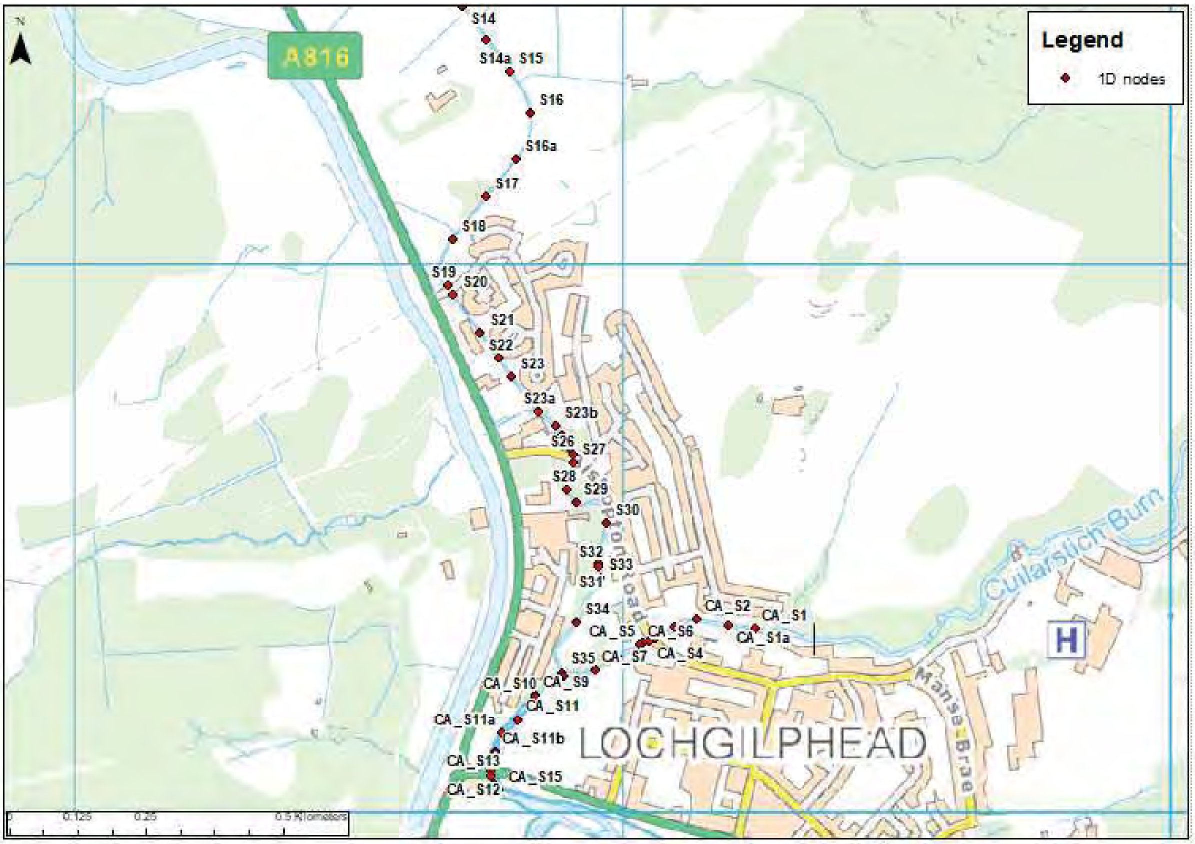


Figure 7-2: 1D node locations

The inflow hydrographs, as calculated in Section 3, were applied at the locations shown in Figure 3-2. The downstream boundary was represented as a Head Time boundary, representing the tidal cycle established in the coastal assessment detailed in Section 6.

Channel and bank Manning's 'n' roughness values were selected based on photographs and the site visit. The channel is seen to vary from relatively natural to artificially straightened and dredged. Due to the weedy, stony nature of the channel bed, roughness was set between 0.035 - 0.045. Only minor sections of floodplain and banks were represented in the 1D model. Long grass and reed banks were set at 0.045 and denser vegetated banks set at 0.05.

7.1.2 Two dimensional flood plain and coastal model

The 1D channel model was linked to a 2D domain (ground surface model) to model the overland flood mechanisms. The 2D hydraulic model contained the following elements:

- Ground surface using 1m LiDAR Digital Terrain Model (DTM);
- 1D/2D links to allow free flow between the river channel and floodplain based on surveyed top of bank elevations and LiDAR;
- Roughness layer depicting different surfaces based on OS Mastermap data representing buildings (n = 0.5), roads (n= 0.025), wooded areas (n=0.08), reeds (n=0.045), scrub (n=0.05), sand (n=0.025) and grassland (n= 0.035);
- Downstream boundary – a tidal boundary was applied 1km out from the shoreline of Lochgilphead. This tidal boundary adopted the same tidal cycle as set in the 1D downstream boundary;
- Buildings are represented by a roughness of 0.5;
- Additional topographic survey was used to refine the LiDAR along the Front Green;
- A 2D inflow point to represent the overflow from the Crinan Canal at weir 3. Full details of this inflow and model can be found in Section 4.5;

- A 2D polygon inflow for the spill over the canal embankment. Full details of this inflow and model can be found in Section 4.5;
- The intertidal area was set at 1mAOD to improve stability. This does not affect flood levels in Lochgilphead as this level is well below the level of the current defences and bank lines;
- Further LiDAR refinements around bridge desks and at gaps in the LiDAR to best represent the topography.

Overtopping analysis from the coastal modelling exercise outlined in Section 4 was not included in the 1D/2D model. This was due to the topography of the coastal frontage. As Tuflow does not have the ability to model wave momentum, any overtopping input in the model would simply fall out to sea rather than replicating any wave run up. Still water level is also seen to inundate the areas and would cause far greater flooding than overtopping. The impact of wave overtopping will be assessed during phase 3 and phase 4 of this study.

7.1.3 Ground truthing

Ground truthing of the 1m LiDAR used to represent the 2D floodplain was undertaken using surveyed top of bank levels as well as levels along the Front Green and throughout town. This exercise was undertaken to ensure that the LiDAR provided a reasonable representation of the ground surface from which the floodmaps would be generated.

In channel survey levels and levels around bridges were removed from the assessment as these were likely to be inaccurately represented in the LiDAR due to the resolution of the data and the inability for channel beds to be picked up.

A total of 675 points were compared and the differences between the LiDAR and the surveyed levels were found to vary between -1.14 and 0.63m. The largest differences were noted throughout the reach with no obvious pattern or specific areas of concern. Table 7-1 shows that of the 675 points compared,90% displayed a difference of less than 300mm between the LiDAR and the survey, with 54% of the compared points having a difference of less than 100mm. Again, there was no locational pattern to where these smaller differences were positioned.

Based on this assessment, the LiDAR is considered to be a reasonable representation of the ground surface and suitable for the modelling at this stage of the Flood Study. If a scheme is found to result from this study, additional topographic survey may be required to improve representation in localised areas.

Table 7-1: Results of ground truthing (LiDAR vs topographic survey)

	Number of points out of the total points tested	Percentage
LiDAR more than 300mm higher than the survey	50 / 675	7%
LiDAR between 100 – 300mm higher than the survey	180 / 675	27%
LiDAR within 100mm of the survey	361 / 675	54%
LiDAR between 100 – 300mm lower than the survey	62 / 675	9%
LiDAR more than 300mm lower than the survey	22 / 675	3%

7.1.4 Model runs parameters

The 2D domain was set at a 4m grid size. This was deemed appropriate as the majority of the study area was rural or open space and there were few complex alley ways or walled off details that required a finer resolution.

The 1D/2D model was run unsteady, i.e. time varying flow, for the required return periods set out in Section 3. This allowed for the flood progression to be fully assessed in both the 1D channel and the 2D floodplain. A 1 second 1D timestep and 2 second 2D timestep were used for all simulations.

Model parameters were kept as default with the exception of the theta and alpha parameters within the IEF run file. These parameters were changed from defaults of 0.7 to 0.9 theta and 0.55 alpha. These values are advised for numerical dampening in tidal situations.

7.2 Sensitivity analysis

No calibration data was available for Lochgilphead. Therefore, sensitivity checks were carried out on the hydraulic model parameters where they are considered to be inherently uncertain to explore the effect on model results.

The aim is to understand the range of model results that could be obtained with variation of these parameters. The intention is not to evaluate an accuracy range or otherwise quantify uncertainty; but to give an indication of the influence certain parameters have and identify if there are significant or disproportionate influences.

In line with SEPA guidance, the model parameters tested were:

- Flow,
- Manning’s Roughness,
- Structure Blockages.

7.2.1 Flow

7.2.1.1 SEPA recommended uplift of 20% - sensitivity test of model parameters

SEPA recommend in their modelling guidance technical note, that model sensitivity to flow be tested with a 20% increase for the 10% AEP and 0.5% AEP events.

Tabulated results with changes to channel water elevations at various locations are displayed in Table 1 Appendix F.

Increasing the flow by 20% increased the 10% AEP event channel water levels by up to 160mm and the 0.5% AEP event channel water levels by up to 220mm. These maximum increases are relatively consistent throughout the modelled reach.

During the 10% AEP event, flood extent is seen to increase in some areas upstream of the Meadows area around the A816, with floodplain depths increased by up to 120mm from the baseline event. Extents are not materially altered downstream of Bishopton Road, with small areas experiencing increases to depth of approximately 100mm.

Floodplain depths are seen to increase from the baseline in the 0.5% AEP event, with depth increases of up to 200m in the area upstream of the Meadows area. Flood extents are not however seen to be significantly altered. Downstream of Bishopton Road, flood extents are seen to increase, with increased flow exiting the Cuilarstich Burn. Depths in these new areas of flooding reach up to 50mm. In areas that previously flooded, such as the caravan park, depth increases from baseline are in the order of 110mm.

Channel water levels are seen to increase as a result of an uplift in flow during both the 10% AEP and 0.5% AEP events. However, these uplifts do not result in a significant increase in flood risk to key receptors as they primarily affect rural areas. The flows used in the baseline model are deemed to be appropriate as they are based on best practice methodologies. It is recommended that uncertainty in design flows are addressed by adoption of an appropriate freeboard allowance.

7.2.1.2 60% uplift – sensitivity of the climate change uplift

Further to SEPA's recommended flow uplift of 20%, an increase in flow of 60% to the 10% AEP and 0.5% AEP event was also tested as this corresponded with Medium emission scenario, 90th percentile climate change uplift. This sensitivity run was compared with the baseline climate change run that was undertaken using the medium emission 50th percentile of 37% to assess how sensitive flooding in Lochgilphead is to selection of climate change scenario.

Tabulated results with changes to channel water elevations at various locations are displayed in Table 2 Appendix F.

The maximum difference in channel water level between the 37% and 60% uplifts were 600mm and 700mm for the 10% AEP and 0.5% AEP respectively. These uplifts are considered to be significant.

During the 10% AEP event, flood extent was not seen to increase significantly in the area upstream of the Meadows, around the A816. Depths in this area are seen to increase by up to 40mm. Downstream of Bishopton Road, flood extents were seen to increase at the caravan park and surrounding properties, with depths increases of 110 mm.

Floodplain extents were not seen to increase significantly in the area upstream of the Meadows, with depth increases in the order of 50mm during the 0.5% AEP event. Similarly, downstream of Bishopton Road, flood extents did not increase markedly, with increases in depth of up to 60mm in and around the caravan park.

Channel water levels are seen to increase as a result of an uplift in flow that represents a climate change scenario that is unlikely to be exceeded. Whilst increases to channel water levels are seen to be significant, this does not translate into increases in floodplain depth and extent. The increase in depths do not materially alter flood risk to key receptors with the exception of an increase in depth on the A816 and several properties around the caravan park.

The climate change uplifts of 37% to flow are deemed to be appropriate as they are based on best practice methodologies at the time of assessment. It is recommended that uncertainty in climate change uplifts are addressed by adoption of an appropriate freeboard allowance.

7.2.1.3 Manning's roughness

In line with SEPA's modelling guidance Manning's 'n' roughness was increased by 40% in both the 1D channel and 2D floodplain for the 10% and 0.5% AEP events. Roughness was increased in both the fluvial and the tidal simulations. A total of 4 runs were assessed.

Tabulated results with changes to channel water elevations at various locations are displayed in Table 3 Appendix F.

Increasing the roughness by 40% in the fluvial simulations increased the 10% AEP and 0.5% AEP event channel water levels by up to 180mm and 230mm and respectively. This increase was relatively uniform along the reach.

During the fluvial simulations, the 10% AEP event flood extent was seen to increase upstream of the Meadows area. Depths increased in this location by up to 100mm. Downstream of Bishopton Road, flood extents and depths are also increased, with a small area of additional flooding at the caravan park. Depths are increased by up to 150mm in parkland near the confluence of the two watercourses. During the 0.5% AEP event, flood extents were seen to increase marginally upstream of the Meadows area, with an increase in depths of up to 100mm. Downstream of Bishopton Road, extents are seen to increase more significantly as a result of additional out of bank spill from the Cuilarstich Burn. Increases in depths of up to 200mm are experienced in the area in and around the caravan park.

During the tidal simulations, floodplain depths and extents were not seen to markedly change, with increases in depth of up to 3mm during both the 10% and 0.5% AEP events.

Reasonable increases in depths in the channel and on the floodplain indicate that the model is moderately sensitive to the choice of roughness for the fluvial simulations. The model is not seen to be sensitive to changes in roughness for the tidal simulations. Whilst the increase in flood depth is not insignificant, the roughness values used in the baseline model are deemed to be appropriate as they are based on channel type and geometry. It is recommended that uncertainty in channel and floodplain roughness is addressed by adoption of an appropriate freeboard allowance.

7.2.1.4 Blockages

Blockage scenarios were tested for the 0.5% AEP event to assess the impacts on flooding should a structure become partially blocked during a flood event. A total of 5 structures were identified around Lochgilphead as having the potential to cause increased flooding if they became partially blocked. Details of the structures can be found in Appendix E. Not all structures were assessed in the blockage sensitivity testing. Those that weren't blocked included small footbridges and structures where significant flooding already occurred and a change in flow mechanism would not materially alter flood risk. Where two structures were positioned in close proximity, the structure with the narrowest opening was taken forward for the blockage assessment. There were no anecdotal accounts of any significant blockages at any of the structures, which were also seen to be relatively large and in reasonable condition.

Structures were modelled as partially blocked to 50% of the flow area by reducing the cross sectional area accordingly. This blockage scenario is considered conservative.

Each blockage scenario was run separately, which assumes that a significant blockage would not occur on 2 structures at once. Blockage locations are shown in Figure 7-3. Tabulated results with changes to channel water elevations at various locations are displayed in Table 4 Appendix F.

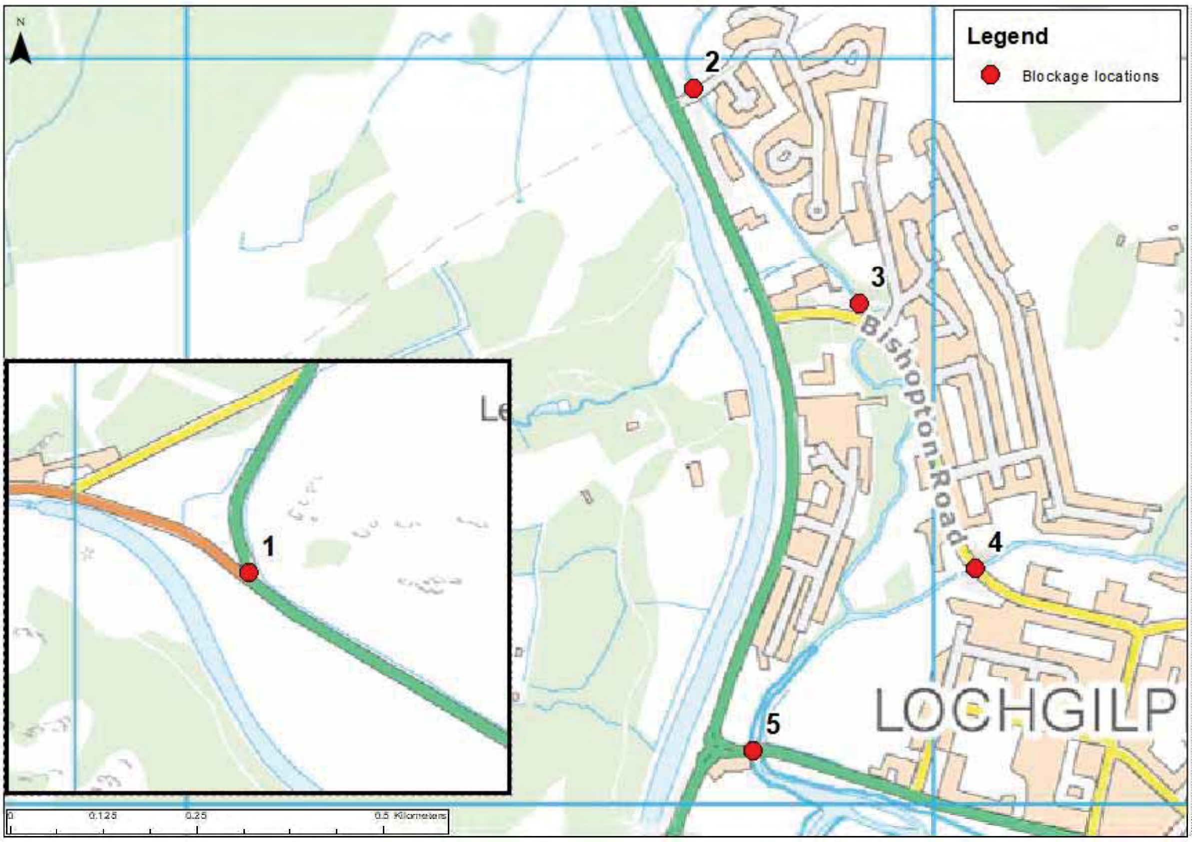


Figure 7-3: Blockage locations

Decreasing the cross sectional area at the 5 structures increased channel water levels by 48mm, 185mm, 540mm, 1000mm and 145mm for blockage scenarios 1, 2, 3, 4 and 5 respectively.

During blockage scenario 1, no real difference in floodplain depth was observed, with floodplain depths increasing by up to 10mm in the area local to the culvert around the junction at Cairnbaan. The area around this culvert currently floods and blockage of the structure does not materially alter the flood mechanism.

During blockage scenario 2, minor increases to the flood extent are observed upstream of the Meadows, with increases in depth of up to 120mm. Due to the topography, the increase in flood depths are not seen to cause flooding to areas that were not previously flooded in the baseline scenario. An increase in water being stored upstream of the structure as a result of the blockage means that floodplain depths are slightly reduced immediately downstream of the culvert.

During blockage scenario 3, increased flood depths and extents are observed upstream of the structure around Bishopton Road and the ABC plant yard. These increases to floodplain depths are experienced up to 1km upstream of the structure of up to 450mm. Floodplain depths are slightly reduced downstream of the structure due to the change in flow mechanism.

During blockage scenario 4, there is a significant change in flood mechanism. Out of bank flow as a result of the blockage occurs upstream of the bridge at Bishopton Road, affecting areas on both the left and right banks up to depths of 900mm where there had previously been no flooding. Floodplain depths in the caravan park are also seen to increase by up to 30mm with flow travelling overland in a south westerly direction from upstream of the bridge.

During blockage scenario 5, floodplain extents are seen to slightly increase, with localised increases in depth of up to 130mm in the caravan park. Other areas are not affected by this blockage.

This sensitivity analysis is not an analysis on the likelihood of blockage, but an assessment of the severity of flooding impacts should a blockage occur at a particular structure. It was found that structures 3 and 4 were sensitive to a blockage of 50% which may be relatively conservative given no blockage history has been recorded. Identifying the structures where blockage may result in increased flooding is useful for identifying structures that would benefit from either extra maintenance or additions such as trash screens.

7.2.1.5 1D/2D connection

The link between the 1D and 2D models has the potential to influence flood depths and extents as the rate of flow across the boundary can vary depending on the discharge coefficient used.

The flow over the 1D/2D link was tested by adjusting the ‘a’ attribute in the boundary line, which can be used to adjust the energy loses over the boundary when the link line is and ‘HX’ type, as it is in Lochgilphead. To test the sensitivity of the 1D/2D link, the results of the default value of 0 was compared to the results using a value of 5, which is stated as an acceptable raise in the TUFLOW manual.

Tabulated results with changes to channel water elevations at various locations are displayed in Table 3 Appendix F.

Altering the discharge coefficient across all link lines from 0 to 5 resulted in changes to water levels between 10 - 150mm for both the 10% and 0.5% AEP events. There was not seen to be a pattern to whether the channel water levels decreased or increased as a result of the energy loss, however the greatest reduction in levels were experienced around Cairnbaan.

Floodplain levels vary between 6-12mm around town and 3-10mm in the Meadows area for the 0.5% AEP event. For the 10% AEP event, the variation was much smaller, in the order of 1-5mm. Variation in flood extents was not significant.

Varying the parameter that influences the discharge coefficient across the 1D/2D link was not seen to signifncantly affect flood depth or extent on either a frequent and rarer event. It was concluded that the model was not sensitive to a change in this parameter.

8. Results

8.1 Baseline – Fluvial simulations

During the 0.5% AEP fluvial event, which includes any flow from the canal, flooding first occurs from the overflow weir 3 on the canal and from overtopping of the culvert near Cairnbaan on the Badden Burn (location 1, Figure 8-1). Out of bank spill then occurs upstream of the Meadows area into the rural land on either side of the watercourse (location 2). Flooding from these spill locations combines with spill from the canal and causes much of the land and the A816 between Lochgilphead and Cairnbaan to become inundated up to depths of 1.2m. Whilst flooding continues to spread upstream of Lochgilphead, out of bank flow is first noted at the confluence of the Badden Burn and Cuilarstich Burn, building to depths in the order of 900mm (location 3). Further spill commences slightly later in the event on the left bank at the caravan park (location 4). Flood waters build in the caravan site, with depths ranging between 80- 700mm before flowing over the A83 and out to sea. Additional small pockets of flooding are noted in the open land between the confluence of the watercourses and Bishopton Road, with depths in the order of 400mm. Later in the event, an additional area of flooding is observed around the junction of Bishopton Road and the A816 around the ABC plant yard (location 5).

The joint probability analysis showed that during a fluvial event, it was likely there would be a low tide. No flooding is observed during a fluvial event as a result of a MHWS sea level as the tide levels do not reach the top of the coastal defences along the Front Green.

The 0.5% AEP event plus climate change follows the same flood mechanism as the 0.5% AEP present day event, albeit with an increase in flood depths. Flooding in the area upstream of the Meadows around the A816 is seen to reach depths of up to 1.6m, with depths ranging from 150mm – 950mm in the caravan park. Additional areas of flooding on both banks originates from the Cuilarstich Burn at the Bishopton Road (location 6) crossing which occurs slightly later in the event than spill into the caravan park. Out of bank spill on the left bank travels in a southerly direction across Lorne Street and Stag Park, with average depths of 150mm, before combining with flood waters in the caravan park. On the right bank, floodwaters are seen to affect properties on Bishopton Road and the SSE site, with depths ranging from 400-900mm.

The joint probability analysis showed that during a fluvial event, it was likely there would be a low tide. No flooding is observed during a fluvial event as a result of a MHWS sea level as the tide levels do not reach the top of the coastal defences along the Front Green.

The 0.5% AEP event plus climate change flood extent is shown in Figure 8-1. A full range of flood maps can be seen in Appendix G.

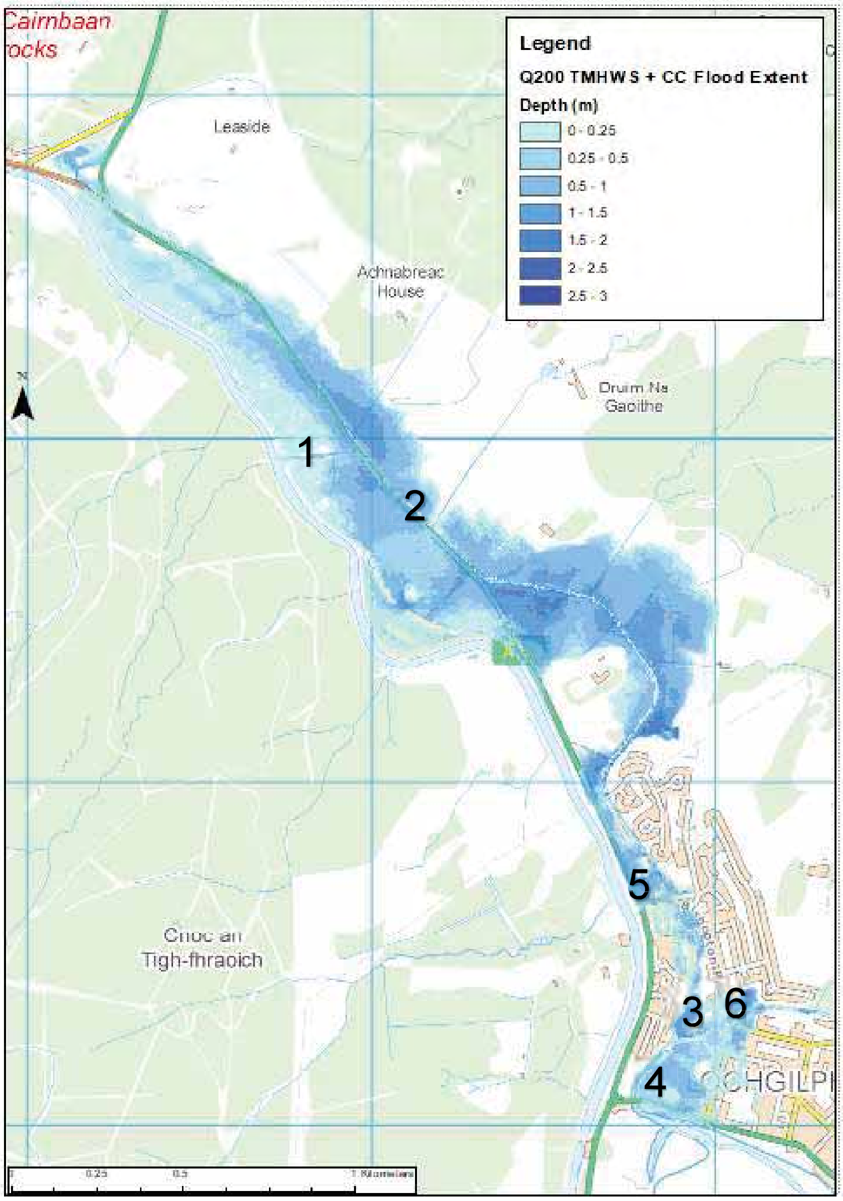


Figure 8-1: 0.5% AEP plus climate change fluvial flood event

During higher frequency events, flooding first occurs on the floodplain surrounding the A816 and at the confluence of the Badden Burn and Cuilarstich Burn from the 50% AEP event. More notable flooding is experienced upstream of the Meadows area from the 10% AEP event. By the 2% AEP event, a large portion of the floodplain upstream of the Meadows is inundated, and spill into the caravan park from the Badden Burn commences. By the 1% AEP event, a large percentage of the caravan park is inundated.

8.2 Baseline – Tidal simulations

During the 0.5% AEP tidal event, with a low fluvial flow, flooding first occurs around the confluence of the Badden and Cuilarstich Burn due to tidal levels backing up in the channel. Lower lying sections of the Front Green become inundated 3hours before the peak tide, with the entire area becoming flooded 2 hours before peak tide. Maximum depths on the Front Green reach 1.4m. Around this time, tidal waters spill over the A83 and over the left bank of the watercourse into the caravan park, which builds to depths ranging from 0.5 – 1.3m. Tidal floodwaters begin to encroach onto properties along the A83 approximately 1.5hours before the peak of the event, before spreading up Argyll Street. Depths in these urban areas range from 190 – 500mm.

The joint probability analysis showed that during a tidal event, it was likely there would be a low flow in the watercourses and from the Crinan Canal. Some minor flooding is noted on the A816 from the canal overflow weir and from overtopping of the culvert at Cairnbaan during a tidal event. Average depths are in the region of 100mm.

The 0.5% AEP event plus climate change follows the same flood mechanism as the 0.5% AEP present day event, albeit with an increase in flood depths. Flooding in the lower lying areas of the Front green become inundated 3.5 hours before the peak tide, with the entire area becoming inundated 2.5 hours before peak tide. Maximum depths on the Front Green reach 2m. Spill enters the caravan park in the same manner as the present day event with depth ranging from 1.1 – 1.8m. A larger portion of the urban area Lochgilphead is affected in the climate change scenario, with floodwaters reaching Lorne Street and Union Street. Properties are seen to be affected by tidal flooding 2.5hours before the peak of the event. Maximum depths in this urban section of Lochgilphead range from 0.2 – 1.1m

The joint probability analysis showed that during a tidal event, it was likely there would be a low flow in the watercourses and from the Crinan Canal. Some minor flooding is noted on the A816 from the canal overflow weir and from overtopping of the culvert at Cairnbaan during a tidal event. Average depths on the floodplain to the west of the A816 are 200mm, with minor flooding up to 40mm on the A816. It should be noted that road and field drains have not been included in this model and the flood depths on the road are likely to be conservative.

The 0.5% AEP event plus climate change flood extent is shown in Figure 8-2. A full range of flood maps can be seen in Appendix G.

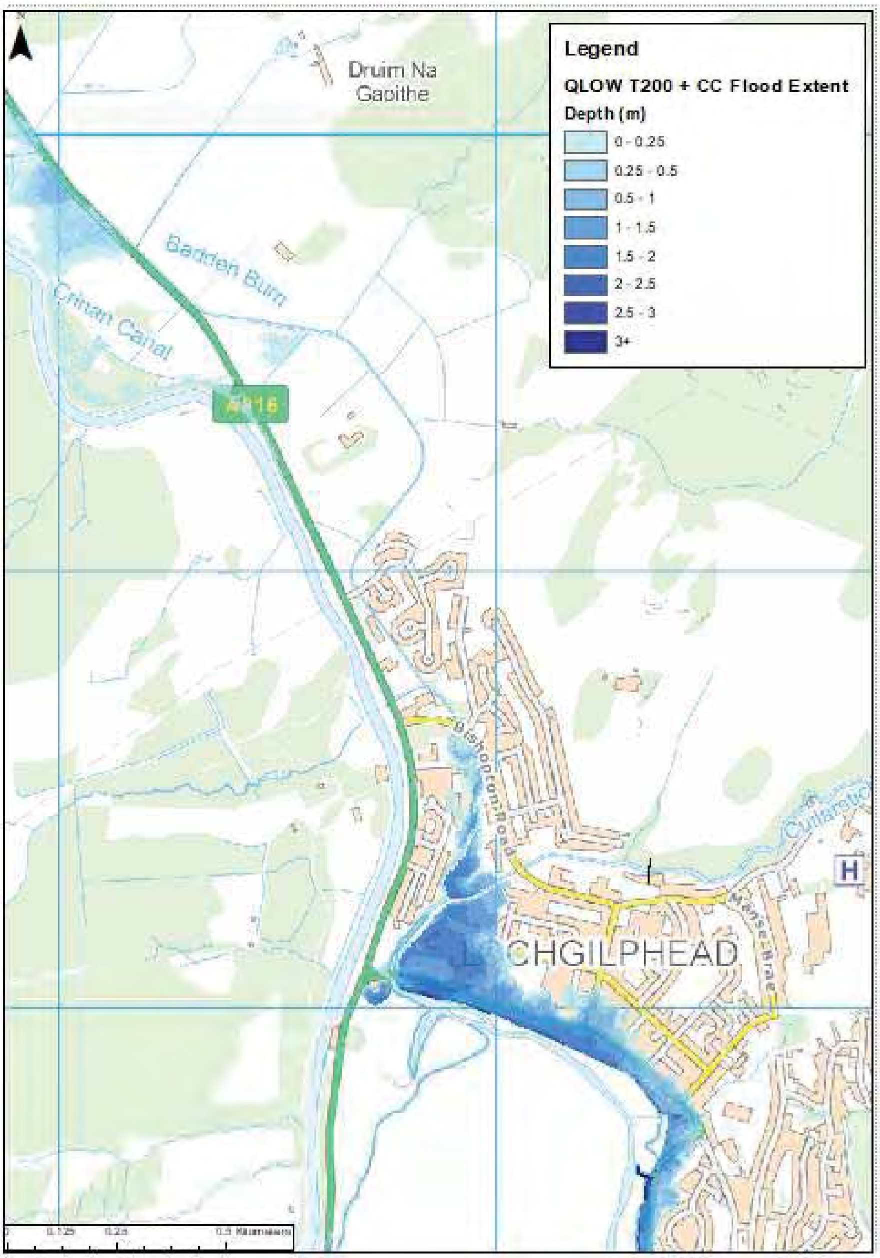


Figure 8-2: 0.5% AEP plus climate change tidal flood extent

During higher frequency events, flooding occurs on the Front Green from the 50% AEP event. By the 20% AEP event, the whole Front Green is inundated, and tidal levels have exceeded the levels on Poltalloch Street and overflowed into the caravan park, causing inundation across approximately 60% of the site. Properties along Poltalloch Street are seen to be affected from the 10% AEP onwards, with flooding becoming more widespread through Lochgilphead by the 2% AEP event.

9. Conclusions

9.1 Model inflows

9.1.1 Subcatchment hydrology

Subcatchments were delineated using LiDAR, OS mapping and information from Scottish Canal's Water Control Manual (WCM) as the natural catchment of the Badden Burn has been modified through introduction of reservoirs and the canal.

Peak flows were calculated for the total catchments of the Badden Burn and Cuilarstich Burn at the downstream extent of the model. Given that the catchment was relatively small, both the Statistical and ReFH2 methods were undertaken for comparison. These peak flows were then used to undertake the flow reconciliation exercise on the subcatchments for the full range of return periods.

9.1.2 Canal modelling

A 1D representation of the Crinan Canal summit pound and eastern reach was constructed to determine any inflows from the canal into the Badden Burn catchment. This model contained elements such as lock gates, waste weirs and inflows from the contributing catchments.

It was found that spill from waste weir 3 and spill over the canal embankment discharged into the Badden Burn from low AEP events. Spill over the canal embankment is only observed between Cairnbaan and upstream of the Meadows as the embankment is higher between the Meadows and Ardrishaig. As discussed within this report, the spill over the canal embankment is likely to be overly conservative but represented a worst case scenario whereby part of the flood control operations were not undertaken.

The spills from the canal were input into the 1D/ 2D hydraulic model as 2D inflows.

Whilst it is not proportional to assess the canal interaction in more detail within this study, it is recommended that Scottish Canals be consulted and any relevant information be passed on for future use that may aid operations and flood reductions in the future.

9.1.3 Coastal modelling

The main objective of the coastal modelling exercise is to establish the nearshore extreme sea level and wave characteristics along the frontage at Lochgilphead during existing and future climates. In order to achieve this, a spectral wave model was constructed using surveyed bathymetry, C-MAP data and topographic data.

Water level profiles are required to provide an estimate of flood duration along the study frontage. The water level profiles at Lochgilphead were derived using the CFB database. The surge shape at the closest site Millport has been adopted. Tide data at Lochgilphead was obtained from Admiralty Tide Tables (ATT) for the underlying astronomical tide curve for present day and future scenarios.

A regional wave model of the wider Loch Fyne areas was run to establish the offshore wave heights at Lochgilphead under present day conditions. The regional modelling results show that the wave climate at the entrance to Lochgilphead is moderate (< 3 m) although a maximum significant wave height of 3.5 m was predicted over the available 38 year hindcast period. The results from this extremes analysis were then used as boundary conditions for a local high resolution model of the intertidal area.

The findings from the local model for the present day scenario show that wave conditions within Lochgilphead are small, with a 1% AEP event producing wave heights in the region of 0.8 m. The small waves can be attributed to the extensive shallow bathymetry within the intertidal area.

Wave overtopping for 4 sections along the frontage was also undertaken to consider the impact of waves on the Front Green and properties on Poltalloch Street. This analysis calculated the overtopping rates based on the findings of the local wave model. Due to the topography of the frontage, wave overtopping inflows were not added into the 1D/2D model but will be used in subsequent stages of this study.

The tidal levels and tide shape established in the coastal modelling assessment have been used as boundary conditions in the 1D/2D hydraulic model of Lochgilphead.

9.2 1D/2D hydraulic modelling

A single hydraulic model was constructed of the Badden Burn and Cuilarstich Burn consisting of a one dimensional element representing the river channels and a two dimensional element representing the floodplain.

The hydraulic model consisted of surveyed river and structure sections, LiDAR representing the floodplain, and various inflows and boundaries as discussed above.

A range of sensitivity tests were undertaken to assess model parameters as there was no calibration data available.

9.3 Baseline flood risk

9.3.1 Fluvial

During the 0.5% AEP fluvial event, which includes any flow from the canal, flooding occurs in the area upstream of the Meadows around the A816 and at the caravan park. Additional small pockets of flooding are noted in the open land between the confluence of the watercourses and Bishopton Road. Depths in the rural land upstream of the Meadows reach 1.2m, with depths ranging from 80 – 700mm on the caravan park. Later in the event, an additional area of flooding is observed around the junction of Bishopton Road and the A816 around the ABC plant yard.

The joint probability analysis showed that during a fluvial event, it was likely there would be a low tide. No flooding is observed during a fluvial event as a result of a MHWS sea level as the tide levels do not reach the top of the coastal defences along the Front Green.

The 0.5% AEP event plus climate change follows the same flood mechanism as the 0.5% AEP present day event, albeit with an increase in flood depths. Flooding is observed upstream of the Meadows, at the caravan park and in smaller pockets of open land between the confluence of the watercourses and Bishopton Road. Later in the event, an additional area of flooding is observed around the junction of Bishopton Road and the A816 around the ABC plant yard. Depths in the rural land upstream of the meadows reach 1.6m, with depths ranging from 150 – 950mm on the caravan park.

Additional areas of flooding during the climate change event occurs on both banks of the Cuilarstich Burn at the Bishopton Road crossing. Out of bank spill on the left bank travels in a southerly direction across Lorne Street and Stag Park, with average depths of 150mm, before combining with flood waters in the caravan park. On the right bank, floodwaters are seen to affect properties on Bishopton Road and the SSE site, with depths ranging from 400-900mm.

The joint probability analysis showed that during a fluvial event, it was likely there would be a low tide. No flooding is observed during a fluvial event as a result of a MHWS sea level as the tide levels do not reach the top of the coastal defences along the Front Green.

9.3.2 Tidal

During the 0.5% AEP tidal event, with a low fluvial flow, flooding occurs along the Front Green, Poltalloch Street, Argyll Street and areas around the caravan park. This flooding is observed to occur between 3 and 1.5hours before the peak tide. At the peak of the event, maximum depths on the Front Green reach 1.4m with lower depths of between 0.5 – 1.3m and 190 – 500mm in the caravan park and urban areas of the town respectively.

The joint probability analysis showed that during a tidal event, it was likely there would be a low flow in the watercourses and from the Crinan Canal. Some minor flooding is noted on the A816 from the canal overflow weir and from overtopping of the culvert at Cairnbaan during a tidal event. Average depths are in the region of 100mm.

The 0.5% AEP event plus climate change follows the same flood mechanism as the 0.5% AEP present day event, albeit with an increase in flood depths and earlier onset of flooding. At the peak of the event, maximum depths on the Front Green reach 2m with lower depths of between 1.1 – 1.8m and 0.2 – 1.1m in the caravan park and urban areas of the town respectively.

The joint probability analysis showed that during a tidal event, it was likely there would be a low flow in the watercourses and from the Crinan Canal. Some minor flooding is noted on the A816 from the canal overflow weir and from overtopping of the culvert at Cairnbaan during a tidal event. Average depths are in the region of 200mm.

10. Next Steps

The next phase of work is to undertaken the long list to short list selection. Using the baseline conditions set out in this study, a long list of feasible options will be developed. This option development will aim to:

- address the flooding issues;
- consider the wider benefits;
- Identify the cost, environmental and legal implications.

As part of this stage of work, the baseline economic assessment will be undertaken looking at the baseline flood damages. The stakeholders and the public consultations will also be undertaken.

Appendix A – Photographs

Photograph 1: Weir 3 on the Crinan Canal

Photograph 2: Bridge across the Badden Burn upstream of the Meadows

Photograph 3: upstream Badden Burn

Photograph 4: Auchoish Burn confluence

Photograph 5: lower reaches of the combined watercourses within Lochgilphead

Photograph 6: Downstream of Poltalloch Road Bridge

Photograph 7: tidal limit of Badden Burn

Photograph 8: tidal defences along Front Green

Appendix B – Hydrological analysis

Donors assessed in QMED calculations taken from WINFAP selection

Station name	*Station no.*	*Area*	*SAAR*	*BFIHOST*	*SPRHOST*	*FARL*
Total catchment (Badden Burn and Cuilarstich Burn) – subject catchment		*22.91*	*1807*	*0.41*	*40.34*	*0.994*
DONORS:						
Glazert Water @ Milton of Campsie	84020	*51.93*	*1560*	*0.414*	*45.24*	*0.991*
Ruchill Water @ Cultybraggan	*16003*	*101.07*	*1888*	*0.433*	*44.13*	*1*
Little Eachaig @ Dalinlongart	***86001***	***31.83***	***2341***	***0.393***	***47.17***	***1***

Acceptable / Not acceptable

Default pooling group						
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
49003 (de Lank @ de Lank)	0.218	51	14.324	0.225	0.206	0.417
46005 (East Dart @ Bellever)	0.388	53	39.242	0.157	0.057	1.458
25012 (Harwood Beck @ Harwood)	0.567	48	32.945	0.191	0.234	0.834
48004 (Warleggan @ Trengoffe)	0.571	48	9.983	0.258	0.257	1.483
48009 (st Neot @ Craigshill Wood)	0.592	12	8.469	0.245	0.373	1.225
73009 (Sprint @ Sprint Mill)	0.618	48	42.091	0.18	0.199	0.607
27032 (Hebden Beck @ Hebden)	0.669	51	4.052	0.204	0.247	0.763
72007 (Brock @ Upstream of a6)	0.716	39	28.011	0.202	0.214	0.51
76811 (Dacre Beck @ Dacre Bridge)	0.716	17	34.73	0.205	0.241	0.924
48001 (Fowey @ Trekeivesteps)	0.81	48	17.465	0.22	0.276	0.222
206006 (Annalong @ Recorder)	0.812	48	15.33	0.189	0.052	1.982
72014 (Conder @ Galgate)	0.902	49	16.283	0.22	0.111	1.575
Total		512				
Weighted means				0.207	0.197	

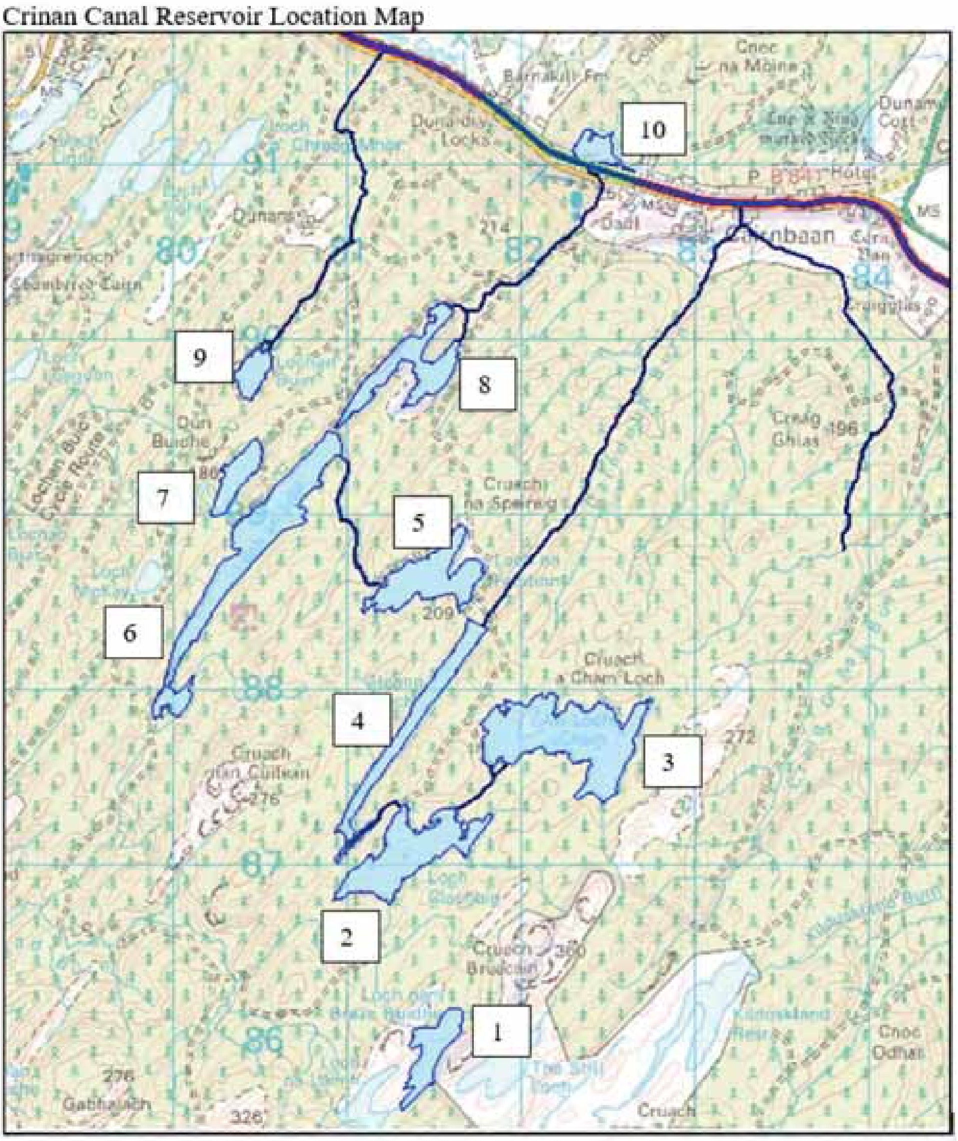
Removed stations from default pooling group. All stations added as a result of removal were checked for all parameters to ensure suitability

Station no.	***Reason***
48009	*Too few years on record*
48001	*Removed due to low FARL*
76811	*Removed as excess years in the pooling group*

Amended pooling group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
49003 (de Lank @ de Lank)	0.218	51	14.324	0.225	0.206	0.501
46005 (East Dart @ Bellever)	0.388	53	39.242	0.157	0.057	1.395
25012 (Harwood Beck @ Harwood)	0.567	48	32.945	0.191	0.234	0.74
48004 (Warleggan @ Trengoffe)	0.571	48	9.983	0.258	0.257	1.736
73009 (Sprint @ Sprint Mill)	0.618	**48**	**42.091**	**0.18**	**0.199**	**0.394**
27032 (Hebden Beck @ Hebden)	0.669	51	4.052	0.204	0.247	0.965
72007 (Brock @ Upstream of a6)	0.716	39	28.011	0.202	0.214	0.523
206006 (Annalong @ Recorder)	0.812	48	15.33	0.189	0.052	2.238
72014 (Conder @ Galgate)	0.902	49	16.283	0.22	0.111	1.441
73015 (Keer @ High Keer Weir)	0.943	26	12.285	0.177	0.178	0.338
21017 (Ettrick Water @ Brockhoperig)	0.945	41	60.364	0.203	0.276	0.729
Total		502				
Weighted means		963		0.201	0.184	

Appendix C – Canal information



1.	Loch Nam Breac Buidhe
2.	Loch Clachaig
3.	Cam Loch
4.	Gleann Loch
5.	Loch Na Faoilinn
6.	Loch an Add
7.	Loch Na Bric
8.	Daill Loch
9.	Lochan Duin
10.	Loch a Bharain

Figure 1: Reservoir Location map

Table 1: Crinan Canal feeder catchment descriptors

	Summit pound catchment descriptors	Creag Ghlas catchment descriptors	Eastern Reach catchment descriptors
AREA	5.503	3.73	1.567
ALTBAR	200	200	200
ASPBAR	53	53	53
ASPVAR	0.47	0.47	0.47
BFIHOST	0.341	0.341	0.341
DPLBAR	2.85	2.85	1.279
DPSBAR	134.6	134.6	134.6
FARL	1	1	1
FPEXT	0.0178	0.0178	0.0178
FPDBAR	0.223	0.223	0.223
FPLOC	0.765	0.765	0.765
LDP	5.3	5.3	5.3
PROPWET	0.75	0.75	0.75
RMED-1H	10.6	10.6	10.6
RMED-1D	45.4	45.4	45.4
RMED-2D	60.9	60.9	60.9
SAAR	1909	1909	1909
SAAR4170	1750	1750	1750
SPRHOST	50.61	50.61	50.61
URBCONC1990	-999999	-999999	-999999
URBEXT1990	0	0	0
URBLOC1990	-999999	-999999	-999999
URBCONC2000	-999999	-999999	-999999
URBEXT2000	0	0	0
URBLOC2000	-999999	-999999	-999999
C	-0.018	-0.018	-0.018
D1	0.45223	0.45223	0.45223
D2	0.34771	0.34771	0.34771
D3	0.45902	0.45902	0.45902
E	0.25613	0.25613	0.25613
F	2.48941	2.48941	2.48941

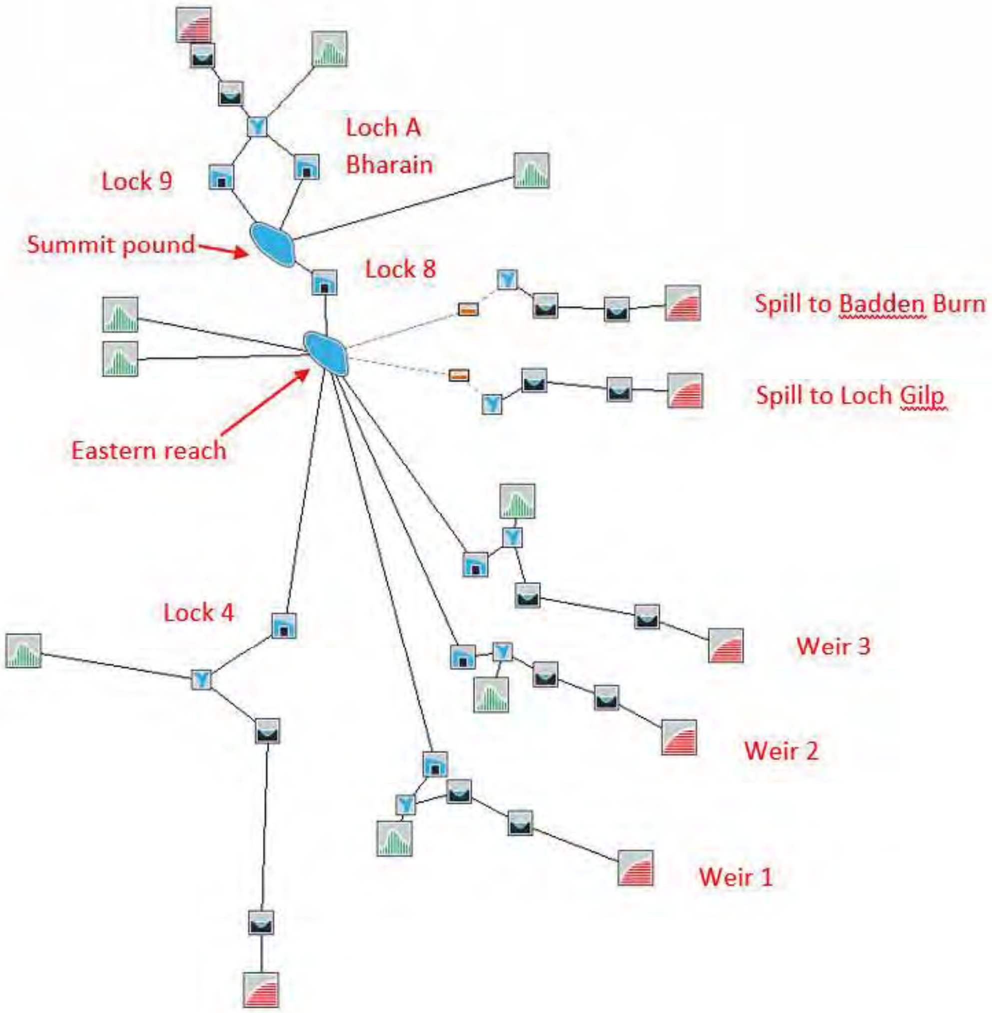


Figure 2: Simplified canal model schematisation