Dumfries and Galloway Council

Newton Stewart Flood Study

Revised Final Report

22 April 2015

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Executive Summary

Dumfries and Galloway Council commissioned Kaya Consulting Ltd. in May 2013 to undertake a study to assess flooding risk from the River Cree and Penkiln Burn within the urban areas of Minnigaff and Newton Stewart.

The River Cree overtopped its banks and flooded areas of Newton Stewart in November 2012. This was the largest flooding event in recent memory. A smaller flood event occurred in December 2013. These flood events followed a Strategic Flood Risk Assessment undertaken by Dumfries and Galloway Council in 2007 for the whole council area which identified Newton Stewart and Minnigaff as a priority area in terms of the number of properties potentially at risk of flooding from a 1 in 200 year flood event in the River Cree.

This study presents the results of a detailed flood study of Newton Stewart and Minnigaff that considers flooding risk from the River Cree and Penkiln Burn. The assessment undertakes a detailed hydrological assessment for the River Cree and Penkiln Burn, develops a linked 1D/2D flood model of the river through the two settlements, produces flood inundation maps for a range of return period flood events, assesses a range of possible flood alleviation measures and presents an initial cost-benefit analysis for the preferred flood alleviation options.

Design flows for the River Cree for use in the study were developed and agreed with SEPA. The 200 year flow for River Cree in Newton Stewart was predicted as 485 m^3/s. This compares with a peak flow of 387 m^3/s for the November 2012 event, which had a return period of around 50 to 85 years. The December 2013 event had a peak flow of the order of 290 m^3/s, with a corresponding return period of the order of 5 years.

An integrated mathematical model of the River Cree and Penkiln Burn and their floodplains through Newton Stewart and Minnigaff was developed using the ISIS 1D/2D software package. The model was calibrated against recorded flood level and flood extent information from the November 2012 event. The model predicted flood extent matches well to the recorded flood extent during the 2012 event.

The calibrated model was used to simulate inundation during floods with a range of return periods (2, 5, 10, 25, 50, 100, 200, 200 plus climate change, and 1000 year return periods). Flood maps were prepared for each event, showing the areas which would be affected by flooding during each event.

The model predictions showed flooding of properties along Arthur Street, along Millcroft Road, along Victoria Street and Riverside Road, and in the Holmpark area on the opposite bank. Flooding of large areas of undeveloped land on both banks of the river was predicted towards the downstream end of Newton Stewart, including the Scottish Water Sewage Pumping Station. The metal footbridge close to the south end of Riverside Road was predicted to surcharge for flows in excess of the 2 year return period, and surcharging of the bridge was observed during the 2012 and 2013 events.

The model predicted that 142 properties would be affected during a 200 year flood, almost equally split between residential and non-residential properties. For a 2 year flood event, 3 residential properties were predicted to flood. The number of properties predicted to flood is significantly smaller than indicated in previous studies, due to the improved methods and datasets used in the current assessment.

A number of flood mitigation options were considered, including; flood storage upstream of Newton Stewart; direct defences where flood risk areas could be protected by flood walls and embankments; removal of a gravel berm (island) just downstream of Cree Bridge; increasing the flow passing capacity of the A75 Road Bridge; and raising the deck of the metal footbridge near Riverside Road.

Modelling work indicated that the removal of the gravel berm, increasing the flow capacity of the A75 bridge and raising the deck level of the metal footbridge had limited local effects on peak water levels only. Either individually or collectively these flood alleviation options would not reduce flooding risk to the urban areas of Newton Stewart or Minnigaff significantly. However, they could be considered as part of a wider scheme, but it should be noted that if removed the gravel berm will likely form again in due course.

A potential location for a large upstream flood storage area was identified in the River Cree Valley. The storage area was shown to be able theoretically to reduce peak flows passing downstream by up to 70 m^3/s, resulting in a 200 year flow being reduced to 75 year flow. However, the storage area option was not predicted to be able to reduce 200 year flows sufficiently to prevent widespread flooding in the urban areas of Newton Stewart and Minnigaff. In addition, it is not considered economically viable (too high a cost for benefit provided) and it would have other environmental and social effects that were not considered in this report. This option, if considered, would need to be combined with direct defences in the urban areas to provide protection against a 200 year flood.

Direct defences within the urban areas would provide flood protection. It was calculated that a total of 1.65 km of flood walls and 0.25 km of flood embankments would be required to protect all the flood risk areas in Newton Stewart and Minnigaff from a 200 year flood. Wall heights would generally be up to 2 m high (above existing ground level), except at Reid Place where the required height including freeboard would be approximately 2.3 m. Options were also considered for defence schemes that provided lower levels of protection. In comparison, a scheme which would provide a 10 year level of protection would require defence heights of up to 1 m on average.

An initial cost-benefit analysis was undertaken, based on the model results and conceptual level flood alleviation options. Hence, the cost-benefit analysis should be considered as initial only, with a high degree of uncertainty. A bias factor of 60% was added to cost estimates for the flood defence schemes as per standard practise for initial cost-benefit analyses. This increase in cost estimates aims to covers unknown factors that could be encountered during the later detailed design processes, e.g., difficult ground conditions.

Flood damage costs for a range of return periods were estimated using the standard Multi-coloured Manual 2010. The assessment indicated that flood damage costs from a 200 year flood would be of the order of £3.7M. For a 10 year flood, the corresponding flood damage costs would be of the order of £0.26M.

Costs for construction of flood walls and embankments able to contain a 200 year flood event were estimated to be £16.1M which is equivalent to £25.8M, with 60% bias. A scheme with walls able to provide protection up to a 1 in 50 year event was estimated to cost £18M, with 60% bias. A scheme with walls able to provide protection up to a 1 in 10 year event was estimated to cost £5.6M, with 60% bias. In comparison the costs for an upstream flood storage pond were estimated to be between £7.3M and £20.3M (both with 60% bias), depending on the sophistication of the flow control mechanism.

The costs and benefits of each scheme were assessed and a benefit-cost ratio calculated for each scheme. Benefit-cost ratios need to be greater than 1 before they would normally be considered as being economically feasible and able to attract grant aid from the Scottish Government. The benefit-cost assessment concluded that;

- The benefit-cost ratios for schemes with flood storage areas and with flood defence walls protecting properties up to the 1 in 200 year or 1 in 50 year level would be less than 1. If other intangibles such as social and environmental benefits were included the resultant benefit-cost ratio would be higher, but unlikely to be significantly higher than 1.
- The benefit-cost ratio for a scheme with flood walls to provide a 1 in 10 year level of protection was calculated to be greater than 1, even without the addition of intangible benefits.

The conclusion was that a scheme providing 1 in 10 year protection could be economically feasible and has the potential to attract Scottish Government grant aid.

It should be noted that how the Scottish Government will allocate grants for such schemes in the future is not known at present. If flood mitigation schemes are compared nationally, those with lower benefit-cost ratios may not attract grant until such time other schemes with higher benefit-cost ratios are complete.

Based on the outline cost-benefit analysis undertaken, a scheme consisting of direct defences and providing an approximately 1 in 10 year level of protection would appear technically and economically feasible and worth further consideration. This could be enhanced with property level protection.

1 Introduction

1.1 Background

Kaya Consulting Ltd was commissioned by Dumfries and Galloway Council to undertake a detailed flood study for the towns of Newton Stewart and Minnigaff, focussing on flooding risk of the urban areas from the River Cree and lower part of Penkiln Burn.

The council commissioned a Strategic Flood Risk Assessment (SFRA) for Dumfries and Galloway in 2007. This study ranked Newton Stewart as the number 1[1] priority in terms of number of properties potentially at risk of flooding based on a weighted priority list. The assessment was based on 5 categories; economics, social, environmental, planning and frequency of flood risk for all towns within the council area. In total 353 properties were identified to lie within the 1 in 200 year flood outline at that time. In 2011, as part of the Flood Risk Management (Scotland) Act 2009, SEPA has completed a National Flood Risk Assessment and identified Newton Stewart as a Potentially Vulnerable Area (PVA) with 306 residential properties and 46 non-residential properties identified at flooding risk.

In November 2012 there was a large flood event on River Cree that caused flooding within Newton Stewart. This event heightened awareness of flood risk within the town.

In response to the results of the SFRA and National Flood Risk Assessment and due to recent flooding within the town, Dumfries and Galloway Council commissioned this detailed flood risk assessment for Newton Stewart.

1.2 Aims and Objectives

The main aim of the study is to identify the risk of flooding from the River Cree and Penkiln Burn within Newton Stewart and to develop outline flood mitigation measures, including outline costing and cost-benefit analyses. The findings of the study will be used by Dumfries and Galloway Council to make a decision on what further actions can be taken to mitigate flood risk in the town.

The Terms of Reference for the study identified 10 key Tasks. The Tasks are summarised in Table 1, which identifies where in this report each of the Tasks are addressed.

[1] Dumfries and Galloway Council, Strategic Flood Risk Appraisal, Final Report, Table 11-1

Table 1: Key Tasks

No.	Task	Where Addressed in Report
1	A review of historical flooding incidents in Newton Stewart, including a door to door survey of properties identified as at-risk to flooding.	Chapter 2
2	Fluvial hydrological study of the River Cree, Penkiln Burn and tributaries;	Chapter 4
3	Mathematical modelling and flood inundation mapping (for scenarios with and without hydraulic structures).	Chapter 5
4	Calibration and verification of models using SEPA data and results of review of historical flooding;	Chapter 5; Section 5.7
5	Inundation Mapping, focusing on urban areas of Newton Stewart and Minnigaff for 1:2 , 1:5, 1:10, 1:25, 1:50, 1:100, 1:200, 1:200 + Climate Change and 1:1,000 year return periods;	Chapter 5; Section 5.9 and Appendix C
6	Assessment of amount of storage above normal water level available upstream of the town;	Chapter 6; Section 6.1
7	Assessment of flood risk management benefits of removing gravel berm to the south of the weir downstream of Bridge of Cree.	Chapter 6; Section 6.3
8	Review of Scottish Water GIS data for wastewater network and indication of effect of 200 year flood levels on the network. Reference to be made to Drainage Area Plan for Newton Stewart;	Chapter 5
9	Economic analysis to develop preliminary Cost Benefit Analysis (CBA) for identified options for flood protection schemes;	Chapter 7
10	Assessment of suitability and implementation costs of flood warning scheme for Newton Stewart;	Chapter 8
11	Structural assessment of existing retaining walls along banks of Cree and assessment of their suitability as flood walls, including recommendations for remedial works. The walls should be considered as not offering any benefit in CBA calculations;	Chapter 9
12	Outline design of three feasible options for flood protection schemes to achieve; • A 0.5% AEP (including allowance for climate change) level of protection; • A 2.0% AEP level of protection; • A level of protection for the greatest benefit/cost ratio of feasible options for an event return period between 1:1 and 1:200 + climate change;	Chapter 10

1.3 Extent of Study Area and Description

Newton Stewart is located approximately 55 km west of Dumfries, in Wigtonshire, Dumfries and Galloway. The town has a population of approximately 3,500 with the main local industries being agriculture, forestry and tourism. The town is situated on the western bank of the River Cree which runs south through the town, before reaching the Solway Firth.

Minnigaff is a small village which is located on the opposite (eastern) bank of the Cree to Newton Stewart. The village is much smaller than Newton Stewart comprised of mainly residential properties. The Penkiln Burn flows along the north-western boundary of Minnigaff before joining the River Cree towards the northern end of Newton Stewart. Minnigaff is linked to Newton Stewart by way of a large masonry arched bridge that crosses the River Cree near the mid-point of the towns.

The study area for flood mapping and 2D (two-dimensional) modelling extends along the banks of both watercourses from National Grid Reference (240617, 566768) close to Ghyll Crescent on the Cree and (241359, 567078) close to Kirkland on the Penkiln, down to the A75 road bridge, as shown in Figure 1.

As per the Terms of Reference for the study the River Cree upstream of Newton Stewart to Water of Minnoch, and Penkiln Burn upstream of Minnigaff to Auchenleck are modelled as 1D (one-dimensional) routing reaches for the purpose of assessing upstream flood storage options and calculating flood travelling times from the headwaters to Newton Stewart.

1.4 Catchment Description

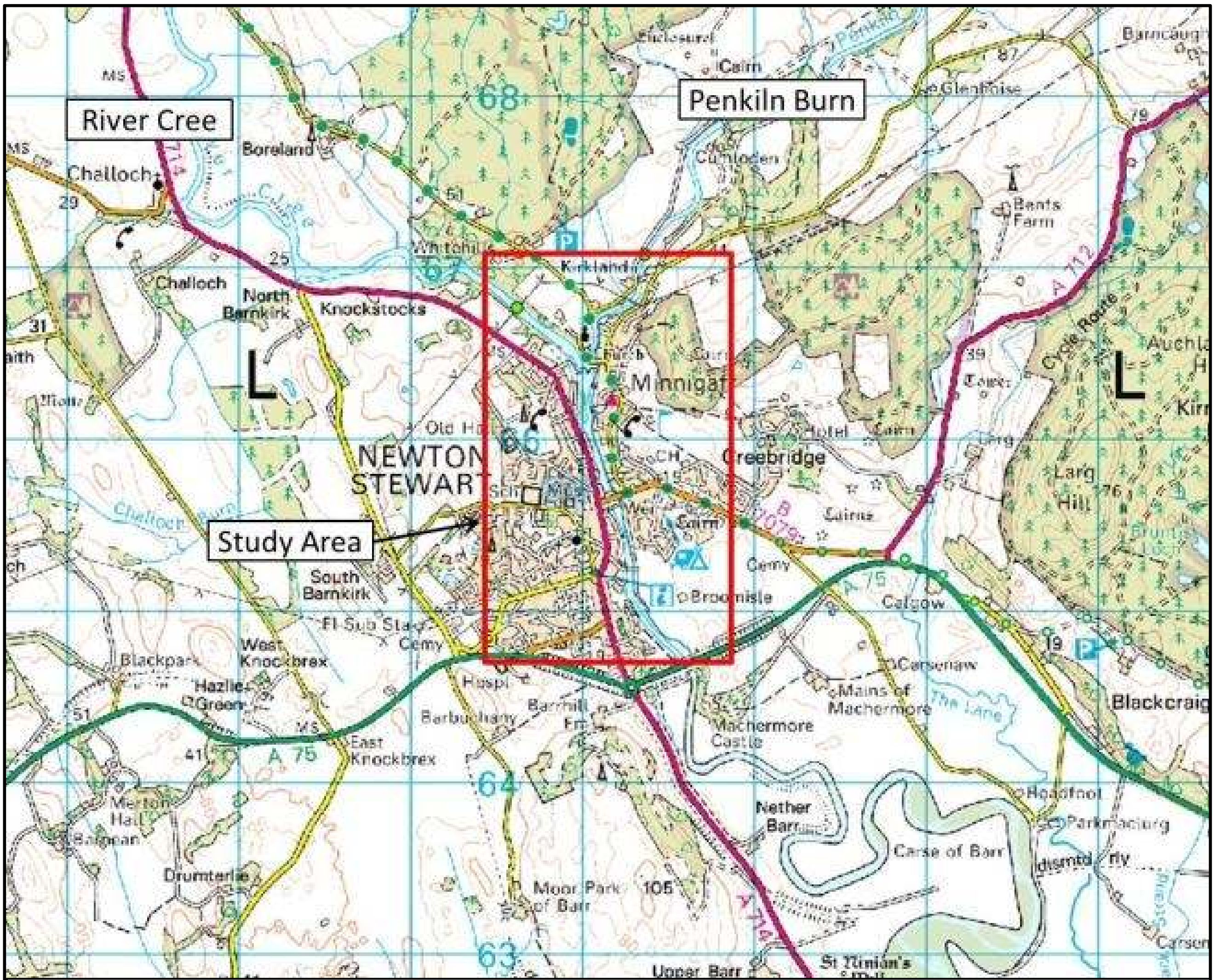
Catchments within the south-west of Scotland experience a relatively warm and wet climate compared to the rest of Scotland. The average annual rainfall for the combined catchment (River Cree and Penkiln Burn) is around 1750 mm based on the Flood Estimation Handbook (FEH). Annual mean temperatures are expected to range from 9.4 to 9.7 °C (Met Office: Regional Climate: Western Scotland.)

In terms of physiography, most of the River Cree catchment is upland forest in character. To the north of the river the headwaters (including Penkiln Burn) are steeper than the headwaters to the south. For example, the Penkiln Burn reaches elevations as high as 550 m AOD and is characterised by steep V-shaped valleys, with numerous small tributaries falling sharply towards the main channel. In contrast to the south gentle U-shaped valleys have formed with large floodplain areas surrounding the channel (upstream of Penninghame, around 35 m AOD).

The Penkiln Burn catchment is significantly smaller than the Cree. At the upstream end of the study area in Figure 1, the Penkiln Burn main channel sits within a deep valley before the valley widens as the burn joins the River Cree at the northern end of Newton Stewart (~10 m AOD).

Downstream of the A75, south of Newton Stewart, the River Cree sits within a tidally influenced plain, with wide open floodplains which have been artificially altered for agricultural drainage, with local agricultural embankments acting as flood defences.

Figure 1: Study area for flood mapping and detailed flood modelling



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In summary, the catchment area can be divided into three main geographic units based on physiography. These are:

- Flat, low lying tidally influenced, deltaic area at the coast;
- Sloping, U-shaped valleys rising upstream of Newton Stewart; and
- Steeper tributaries falling from upland catchments.

2 Historical Flood Information

2.1 Historical Flood Information Received from SEPA

Historical flood records were provided by SEPA in April 2013. These records include events from 1960 to December 2012. The records consist of fluvial flooding records from the River Cree and Penkiln Burb, as well as incidences of road flooding caused by pluvial, surface runoff and drainage failure flooding.

Details of the records are given in Table 2.

2.2 Historical Flood Information Received from Dumfries and Galloway Council

Historical flood records were provided by the council in March 2013. The dataset contained flood records from 2010 to 2012.

Details of the records are given in Table 3.

2.3 Other Historical Flood Information

Kaya Consulting met with representatives of the Cree Valley Flood Action Group (CVFAG) on two occasions. Members of the CVFAG provided additional information related to historical flooding and especially related to the November 2012 event. This information was used in the model set-up and calibration, with model predicted inundation extents compared to the observed flood levels.

2.4 Overview of November 2012 Flood Event

The November 2012 flood event resulted in flooding of substantial areas of Newton Stewart. Discussions were held with the local council, local SEPA flooding officer and local residents regarding the extent of flooding during this event. The results of these discussions are described in more detail in Section 4.3.4, where model predictions are compared to observed flood inundation extents within the town.

Photographs of flooding during the event, including mapped locations and a description of the flood extent are provided in Appendix B.

The November 2012 event reached the highest recorded water level in the River Cree at the SEPA gauging site located at the south end of the town. The gauge has records from 1963 to the present. The flow associated with the event is considered the second highest on record, with a higher flow recorded in 2001, although this was produced by a lower water level. This is likely a result of changes to the river channel morphology over time that has resulted in a different stage / discharge relationship at the SEPA gauge between the two events. SEPA have modified their rating curves which convert recorded water level to flows at the gauge during this time (based on data collected between 2001 and 2012), suggesting either a change In channel form or improved data being available in 2012, compared to the earlier event.

The recorded flow hydrograph at the SEPA gauge for the 2012 event is provided in Figure 2. Based on the hydrological assessment discussed in Section 4, analysis would suggest that the November 2012 event had a return period of around 50 - 85 years.

Figure 2: Recorded flow hydrograph at SEPA gauge in Newton Stewart during November 2012 event

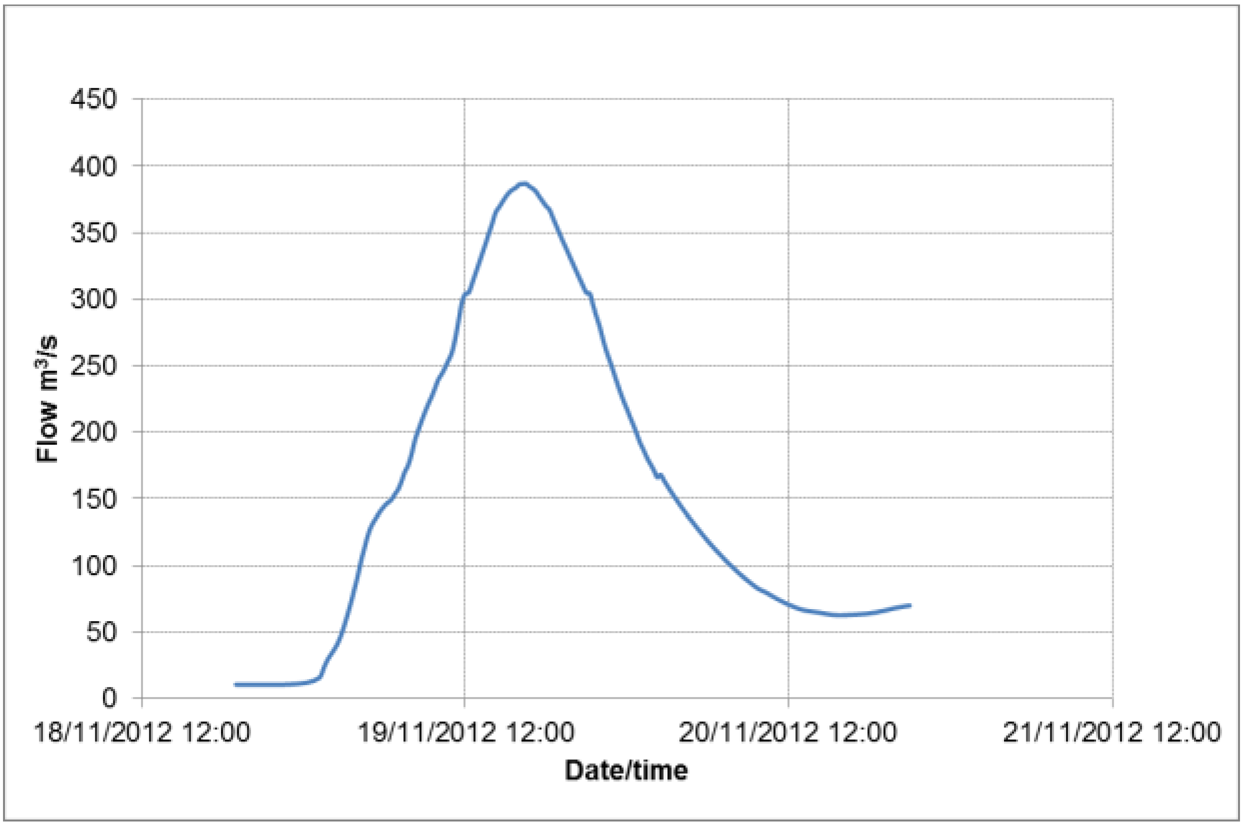


Table 2: Historic flood records provided by SEPA

Record date	Main source	Location	Description
1960	Fluvial	Minnigaff	Flooding from Penkiln Burn that resulted in town level flooding in Minnigaff. Extensive damage including flooded properties and a bridge washed away.
Oct 2000	-	-	Record of flooding. No record of flood level or location details.
Dec 2001	Fluvial	Riverside car park	Surface level flooding from the River Cree. The riverside car park flooded and water entered the door of the public toilet
Jan 2002	Pluvial	A714 and B7079	Surface water was reported across the carriageway at A714 N and B7079 Kirroughtree.
Aug 2002	Pluvial	St Counas Street	Record of pluvial flooding at St Counas Street
Aug 2002	Pluvial	B7079 at Newton Stewart	Surface water was reported across the carriageway
Oct 2004	Assumed surface runoff	B7079 at Cree Bridge	Street level flooding on the B7079 at Cree Bridge. The area around the Minnigaff side of the bridge flooded.
Oct 2005	Sewer	Riverside car park	Extensive flooding of the riverside car park and flooding of the public toilets
Oct 2005	Drainage failure	Victoria Street/St Couans Road A714	Street level flooding at Victoria Street/St Couans Road A714. Shops were in danger of flooding as drainage system could not cope with large volume of water
Dec 2006	-	-	Record of flooding. No record of flood level or location details.
Nov 2006	Drainage failure	St Counas Place	Blocked gullies on St Counas Place.
Oct 2007	Fluvial	Minnigaff	Record of flooding from Penkiln Burn that resulted in flooding in Minnigaff. A request for sandbags was made.
-	-	Cree Bridge	Record of street level flooding at Creebridge. B7079 roadway was partially flooded.
Dec 2007	Drainage failure	York Road	Blocked drainage gullies resulted in street level flooding on York Road.
Dec 2008	-	Riverside car park	Record of flooding at the riverside car park and A714 road.
Jun 2012	-	-	Record of flooding. No record of flood level or location details.
Dec 2012	-	-	Record of flooding. No record of flood level or location details.

Table 3: Historic flood records provided by Dumfries and Galloway Council

Record date	Main source	Location	Description
25/10/2000	Fluvial	55 Arthur Street, Wigtown	Flood water entering premises
07/12/2001	Assumed Fluvial	Riverside car park, Wigtown	Water entering door of public toilet
27/01/2002	Surface Runoff	A714 North and B7079 Kirroughtree, Wigtown	Surface water across width of carriageway
01/02/2002	Fluvial	Ghyll Crescent, Wigtown	Flooding threat. Fire fighters cleared drains and diverted floodwater away from properties
15/02/2002	Surface Runoff	A714 South and B7085 Barnkirk, Wigtown	Surface water across width of carriageway
30/08/2002	Surface Runoff	C50w, St Couans Road, Wigtown	Surface water across carriageway
30/08/2002	Surface Runoff	B7079, Wigtown	Surface water across carriageway
24/10/2004	Other Drainage	B7079 Creebridge, Wigtown	Area around Minnigaff side of bridge flooded.
14/01/2005	Fluvial	B7079 Creebridge (At Minnigaff side of Bridge), Wigtown	Roadway partially flooded
28/10/2005	Other Drainage	A714 Victoria Street / St Couans Road, Wigtown	Shops in danger of flooding, due to drainage system unable to handle large volumes of water.
28/10/2005	Sewer	U357w Riverside car park, Wigtown	Extensive flooding in car park. Public toilet block flooded and sewage coming up toilet pans.
15/11/2006	Other Drainage	St Couans Place, Wigtown	Gully blocked by debris
03/10/2007	Assumed Fluvial	Clauchrie Lodge, Cumloden, Minnigaff, Wigtown	Request for sandbags
18/11/2007	Other Drainage	York Road, Wigtown	Gullies blocked
13/12/2008	Other Drainage	A714 Newton Stewart to Barrhill Road at Penninghame Prison, Wigtown	Road flooded
07/02/2011	Other Drainage	Road	Flooding at the school
08/12/2011	Other Drainage	Road	A75T from B7079 near Newton Stewart hospital to C6W at Knockbrex Toll
23/06/2012	Fluvial	Road	Footbridge and Riverside Walk flooded
19/11/2012	Fluvial	-	Newton Stewart flooded
19/11/2012	Fluvial	-	Newton Stewart flooded - FRMT and Flood Pod in attendance

3 Data Collection and Review

Information from numerous sources has been obtained and analysed for the purpose of this study. Key data obtained for this assessment are described in the following sections.

3.1 Data Received from Dumfries and Galloway Council

3.1.1 LiDAR Data

Filtered LiDAR (Light Detection and Ranging) data were provided by Dumfries and Galloway Council for the town of Newton Stewart and the River Cree Valley upstream of Newton Stewart. LiDAR is a remote sensing technology that produces high quality topographical point data that is converted into a gridded Digital Terrain Model (DTM) that is used in flood modelling studies. A LiDAR device is normally flown under an aircraft and works by illuminating a target with a laser and analysing the reflected light. LiDAR accuracies vary depending on the equipment used and the height of the aircraft. However, data is usually provided with a spatial (horizontal) accuracy of 1 m and a vertical accuracy of the order of 0.15 m.

The raw LiDAR data has been filtered prior to its use in the modelling work, i.e., vegetation, buildings and structures have been removed from the survey data. Hence the DTM used in the modelling work represents “bare earth” elevations. Although the filtering techniques are well tested, there can be errors within dense urban areas, where there may be limited data on ground levels between buildings. In these areas the filtered DTM may be in error. To try and overcome some of these issues a check was made between LiDAR results in Newton Stewart and Minnigaff with ground survey data at nearby locations. The ground survey was obtained for the purpose of this assessment (Section 3.3). The comparisons indicated that potential errors within the LiDAR data in urban areas did not exceed approximately 200 mm. This was deemed acceptable for use within the study and no changes were made to the LiDAR data set.

3.1.2 Other Mapping and GIS Data

Dumfries and Galloway Council provided the following Ordnance Survey mapping information for Newton Stewart and the surrounding study area:

- 1:1,250 detailed maps;
- 1:10,000 maps; and
- 1:25,000 maps.

In addition to the above Dumfries and Galloway also provided Scottish Water infrastructure drawings which covered urban areas within Newton Stewart and Minnigaff.

3.1.3 Historical Flood Information

Historical flooding details were obtained from Dumfries and Galloway Council and are described in Chapter 2.

3.2 Data Received from SEPA

3.2.1 Flood Maps

SEPA publish Indicative River and Coastal Flood Maps for Scotland (2nd Generation) (http://www.sepa.org.uk/flooding/flood_extent_maps.aspx). The maps show the predicted extent of flooding for a 200 year flood event (0.5% chance of occurring in any year) in line with Scottish Planning Policy (SPP). As indicated by their name, these maps are considered Indicative only as they are based on relatively crude topographical data and mathematical modelling. They are considered to have a vertical accuracy of 0.7 m – 1 m and local features such as flood defences and bridge structures have not been included. Therefore, they provide an overview of likely flood extents, but cannot be relied on for detailed assessments. For any specific study area, more accurate flood maps are typically required based on more detailed modelling work and site-specific topographical data.

The flood maps for Newton Stewart and Minnigaff indicate a significant area of flooding on both banks of the River Cree and Penkiln Burn. Notable areas of flooding include a large area on the left bank of the Penkiln Burn, immediately upstream of the confluence, which is predicted to be at risk of flooding. Additionally, a large flood extent on the left hand bank of the Cree immediately downstream of the main Bridge of Cree is predicted to be at risk of flooding during a 1 in 200 year event.

Following submission of the draft report for this study and before issuing this final version, SEPA published their 3rd Generation flood maps, which show a smaller extent of flooding than shown in Figure 3 on the east bank of the river and slightly larger extent on the west bank particularly upstream of the A75. The flood extent in the 3rd Generation maps is more similar to the extent predicted in this study than shown in the 2nd Generation map.

3.2.2 Historical Flooding

Historical flooding data were obtained from SEPA and are described in Chapter 2.

3.2.3 Hydrometric Data

SEPA operates a flow monitoring station within Newton Stewart, located towards the southern (downstream) end of the town. The gauge ‘River Cree @ Newton Stewart’ (station number 81002) is roughly 800 m upstream of the A75 road bridge, which forms the downstream boundary of the study area. The gauge has a long period of record from 1963 to 2012, i.e. 50 years of records, see Figure 6. The catchment area of the River Cree at the gauge is estimated to be around 366 km^2.

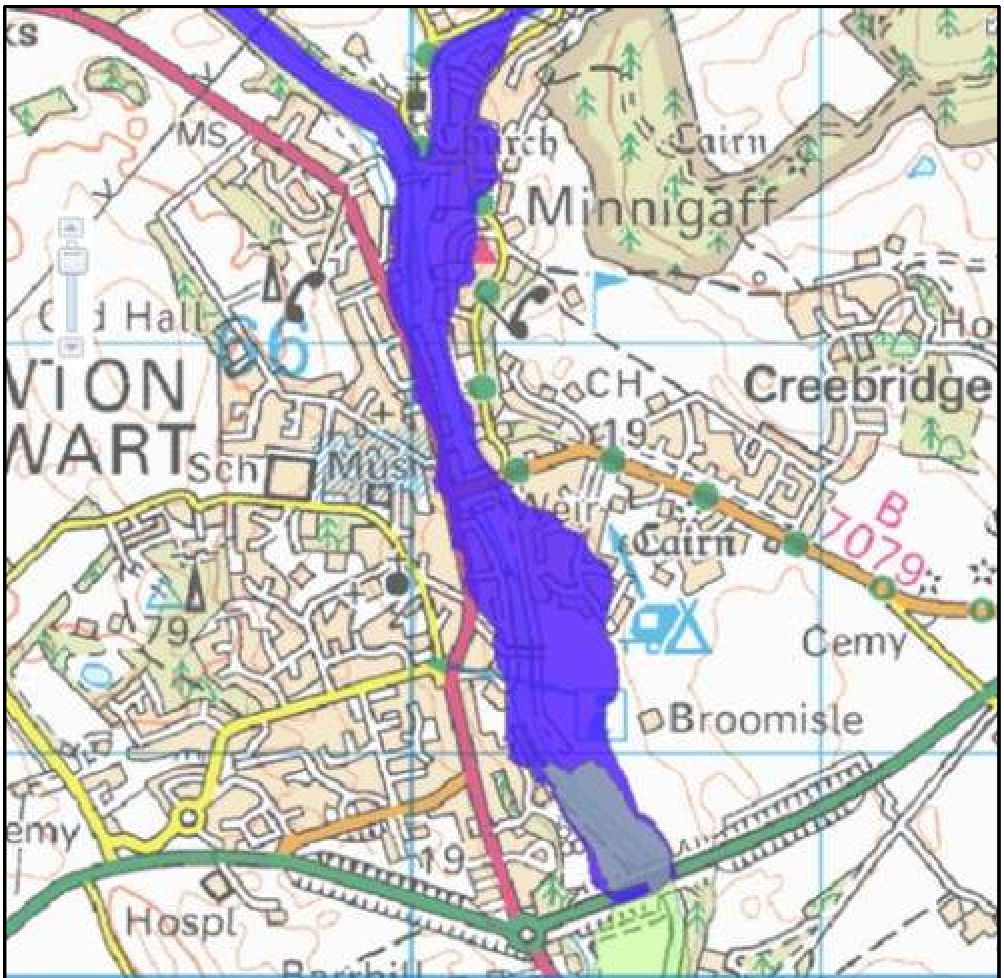
The gauge is a “velocity-area” station, situated on a long, reasonably straight, gravel bedded reach. It has a cableway for gauging high flows. Ratings have been derived with current meter gauging with simple extrapolation above the highest readings.

Data received from SEPA for this gauge included;

- AMAX series data;
- Spot flow measurements used to develop rating curves;
- Details of all rating curves at the site; and

- 15 minute stage recordings for five flood events (i.e., 12/11/2012, 19/06/2012, 13/12/2008, 24/11/2006, 18/10/2000).

Figure 3: SEPA indicative flood map for study area (2nd Generation) – Courtesy of SEPA



A series of discussions were held with SEPA related to the AMAX series and rating curves at the site. Following these discussions the data, methods and design flows for River Cree were agreed with SEPA for use in the present study.

3.2.4 Rainfall Data

Data from a single rain gauge, 'Brigton' (NGR; 236103 574487), located approximately 10 km upstream of the study area, was provided by SEPA. 15-minute rainfall totals were provided for the five river flood events for which water level data were received (Section 3.3.3).

3.3 Ground Survey Data

3.3.1 Cross Sectional Survey Including Structures

Although the study area is covered by LiDAR data, LiDAR is unable to penetrate below deep water (likely around 0.5 m), so LiDAR surveys do not provide details of the channel form under the water level at the time of the survey. Therefore, in order to construct detailed mathematical models of River Cree and Penkiln Burn, a channel cross-section topographical survey was undertaken. In addition, surveys were taken of in-channel structures that could affect the passage of flood waters.

MH Surveyors were commissioned to undertake 54 cross-sections of the River Cree and Penkiln Burn throughout the study area. This work was completed on the 3rd May 2013. Additional cross sections were also surveyed upstream of the study area, to assess potential of upstream storage options (completed 20th August 2013). Bridge structures of both watercourses were surveyed, including Bridge of Cree, the metal footbridge and the A75 road bridge towards the downstream reach of the study area. The large weir structure immediately downstream of the Bridge of Cree was also surveyed. The location of survey sections is discussed in more detail in Chapter 5.

3.3.2 Top of Wall Survey

A small masonry flood defence wall flanks the right bank of the River Cree throughout the built up area of Newton Stewart. The wall is around 350 mm thick and varies in height from 0.75 – 1.0 m. There are a number of gaps within the wall which provide pedestrian access to the river. A full top of wall survey was obtained through the town to identify gaps, low points and any changes in wall height in between surveyed channel sections.

The survey work was undertaken by MH Surveyors and was included in the cross-sectional survey.

3.3.3 Doorstep Threshold Survey

A survey of doorstep threshold levels was undertaken for all properties identified as lying within the floodplain area. These survey levels are key inputs to the cost benefit assessment component of the study.

The work was undertaken by MH Surveyors who carried out the survey on 20th August 2013.

3.4 Kaya Site Visits

A series of site visits were undertaken by members of the Kaya team and sub-contractors. Site visits are listed below.

- 16th April 2013: Michael Stewart undertook walkover survey and gave presentation to local Flood Forum.

- 13th June 2013: Michael Stewart undertook walkover site visit of full study area with Ross Gibson of Dumfries and Galloway Council. The key purpose of the assessment was to finalise locations for model cross-sections, including requirements for channel topographical surveys.
- 28th June 2013: Callum Anderson and Yusuf Kaya, together with Ross Gibson, undertook site survey to review key flood flow pathways. Initial model results were reviewed in the field to ensure that the model results were realistic and that no physical features significantly affecting flow conditions were omitted from the model.
- 30th August 2013: DRM wall survey.

During the site visits the entire reach of the River Cree and Penkiln Burn within the study area was visited and a photographic record of the watercourses was produced. Some photographs of the watercourses and hydraulic structures are presented in Appendix A.

4 Hydrological Assessment

4.1 Objective and Location of Flow Estimate

The objective of the hydrological analyses is to provide estimates of return period peak flood flows within the study area and to provide design flow hydrographs to be used as inputs into the mathematical modelling work described in Chapter 5.

For consistency the flood mapping study was undertaken for flow conditions that produced design flows calculated at the location of the SEPA gauge site in Newton Stewart.

4.2 Catchment Description

The River Cree has a catchment area of approximately 368 km^2 at the downstream end of the study area, based the Flood Estimation Handbook (FEH) and shown in Figure 5. Upstream of Newton Stewart the Cree has a catchment area of 319 km^2, with the Penkiln Burn (46 km^2) joining the Cree at the northern end of the town. Upstream of the Penkiln Burn the two largest tributaries to the River Cree are Water of Minnoch (82.4 km^2) and Water of Trool (47.2 km^2).

The headwaters of the Cree lie in undeveloped upland areas to the north of Newton Stewart and the river flows in a southerly direction towards the Solway Firth. Most of the headwaters of the catchment are in upland forest and open moor.

Catchment characteristics for the River Cree, from the FEH CD-Rom Version 3 are provided in Table 4. The Standard Average Annual Rainfall (SAAR) for the River Cree catchment is approximately 1,754 mm. The Standard Percentage Runoff (SPR) for the catchment is 51%, which is a relatively high value. A review of available geology and soil maps indicated that the underlying geology of the upstream catchment is predominantly comprised of glacial till with some upland peat[2]. Upland peat can act to retain water if unsaturated, but when fully wet peat can be relatively impermeable to additional rainfall. Till deposits can also be relatively impermeable if dominated by finer grained silts and clays. These surficial layers in combination with the relatively steep upland catchment would tend to suggest high runoff rates in response to rainfall, consistent with a high SPR value.

The value of FARL in Table 4 (indicating the effect of water bodies on flood flows) is 0.932, which is low. Values of less than 0.95 typically indicate that flood flows in a catchment could be affected (attenuated) by storage in lochs or reservoirs. A list of the main water bodies and the upstream catchment which they attenuate is provided below;

- Loch Middle (239536,574082) – 0.8 km^2
- Loch Dornal (229120,576150) – 6.3 km^2
- Loch Moan (234852,585645) – 5.4 km^2
- Loch Trool (240636,579795) – 32.7 km^2
- Loch Valley (244535,581694) – 7.0 km^2

[2] British Geological Survey Online GeoIndex Onshore Interactive Viewer

Loch Trool is the largest water body measuring around 0.6 km^2 in surface area and therefore providing a significant influence on downstream flood flows. An indicative map showing the upstream river network and water bodies are provided in Figure 5.

Table 4: Catchment characteristics

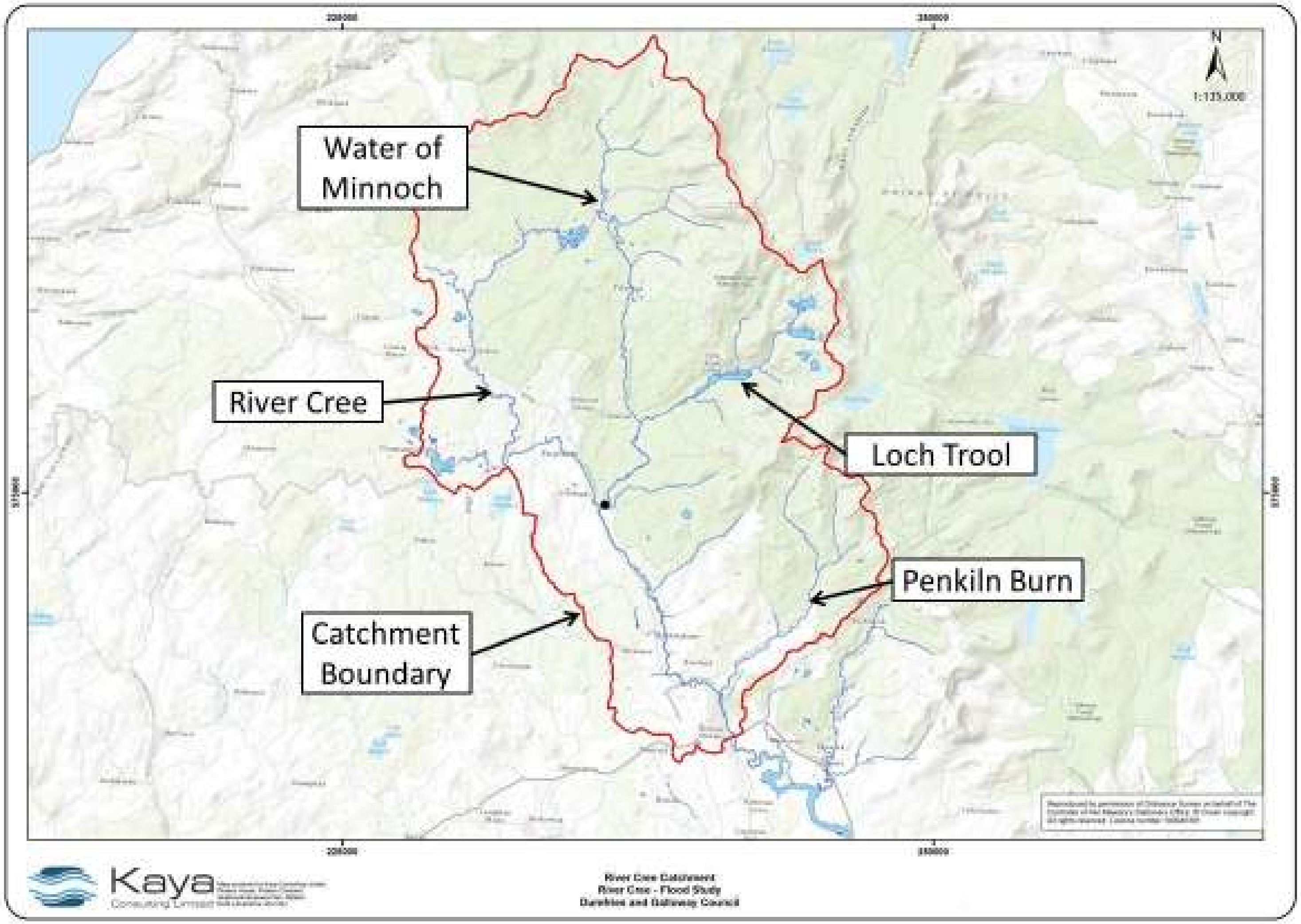
Parameter	Cree at downstream end of study area
Easting (m)	241600
Northing (m)	564700
AREA (km^2)	367.94
ALTBAR (m)	237
ASPBAR (^{o})	204
ASPVAR	0.23
BFIHOST	0.342
DPLBAR (^{o})	24.46
DPSBAR (^{o})	118.5
FARL	0.932
FPEXT	0.07
FPDBAR	0.98
FPLOC	0.909
LDP	46.65
PROPWET	0.69
SAAR (mm)	1754
SAAR4170 (mm)	1710
SPRHOST	50.79
URBCONC1990	0
URBEXT1990	0.0014
URBLOC1990	0
URBCONC2000	0
URBEXT2000	0.0025
URBLOC2000	0

4.3 River Flow Data

SEPA operates a flow monitoring station within Newton Stewart, located towards the southern (downstream) end of the town, Figure 4. The gauge River Cree @ Newton Stewart (station number 81002) is roughly 800 m upstream of the A75 road bridge, which forms the downstream boundary of the study area. The gauge has a long period of record from 1963 to 2012, i.e. 50 years of records. SEPA estimate the catchment area of the River Cree at the gauge to be around 366 km^2.

Figure 4: Location of the SEPA gauge in Newton Stewart

Figure 5: Indicative River Network and Water Bodies



4.4 Design Flow Estimation

4.4.1 Methodology

The approach taken to estimate design flows for River Cree at Newton Stewart was agreed with SEPA following a technical memo sent to Nick Gair of SEPA on the 18 April 2013. The flows agreed at that time were then reviewed following additional data being provided by Alasdair Hardie of SEPA. The approach was then agreed with Alasdair Hardie.

Design flows for River Cree are based on;

- FEH single site analysis of Annual Maximum (AMAX) flows at the Newton Stewart gauge; and
- Enhanced Single Site Analysis (FEH Pooling Group assessment) at the gauge site.

Given the proximity of the SEPA gauge in Newton Stewart to the downstream end of the study area, it was agreed that return period flows calculated for the gauge would be considered equivalent to design flows at the downstream end of the study area.

The results of this analysis are provided below, along with a short discussion of the return period associated with the November 2012 event on the Cree, which produced one of the highest recorded peak flows at the gauged site.

4.4.2 FEH Single Site Analysis

A single site analysis involves the statistical fitting of observed AMAX series data to an appropriate extreme value distribution.

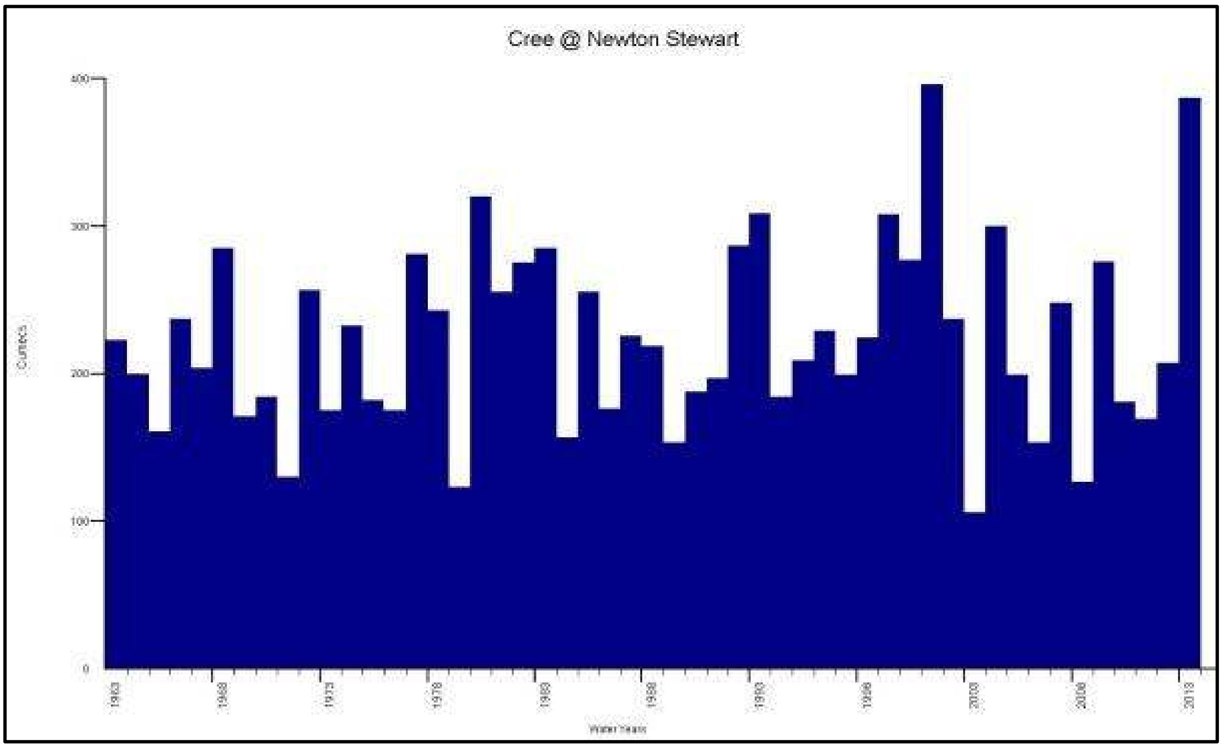
The AMAX series for the Newton Stewart gauge was obtained from SEPA. The first stage of the analysis involved a review of the AMAX series, comparison with data in the HiFlows data set and a review of rating curved at the gauge site.

The SEPA AMAX series data differed from the data within the standard FEH WINFAP Version 3 software package. Based on discussions with SEPA it was agreed that the SEPA data was more appropriate for use in this assessment. Hence, the Hi-Flows data was not used in the analysis.

A series of rating curves were made available by SEPA and in discussions with Alasdair Hardie of SEPA it was concluded that for years 1963 – 2006 the SEPA AMAX series data was to be used. For 2006 onwards the SEPA AMAX data were to be re-calculated based on the peak water data and the most recent rating curve for the site (SEPA Rating curve number R25(2)).

The AMAX series used in this assessment is shown in Figure 6.

Figure 6: Observed AMAX at Newton Stewart (Source SEPA)



Using these data the single site flood frequency curve (Generalised Logistics and GEV distributions) for the Newton Stewart gauge site is shown in Figure 7. The Generalised Logistics distribution is used in this assessment and it provides a generally good fit to the observed data. Return period flow estimates are provided in Table 5 where they have been rounded to the nearest ten cumecs (m^3/s).

Given the period of record at the SEPA gauge (50 years), single site analysis of the flow data would be expected to provide a reasonable estimate of the design flow up to around 1 in 50 years, with poorer quality results for return periods in excess of 50 years. The FEH advises that the FEH Pooling Group method should be used for return periods in excess of the period of record at the gauged site.

Figure 7: Single site flood frequency curve

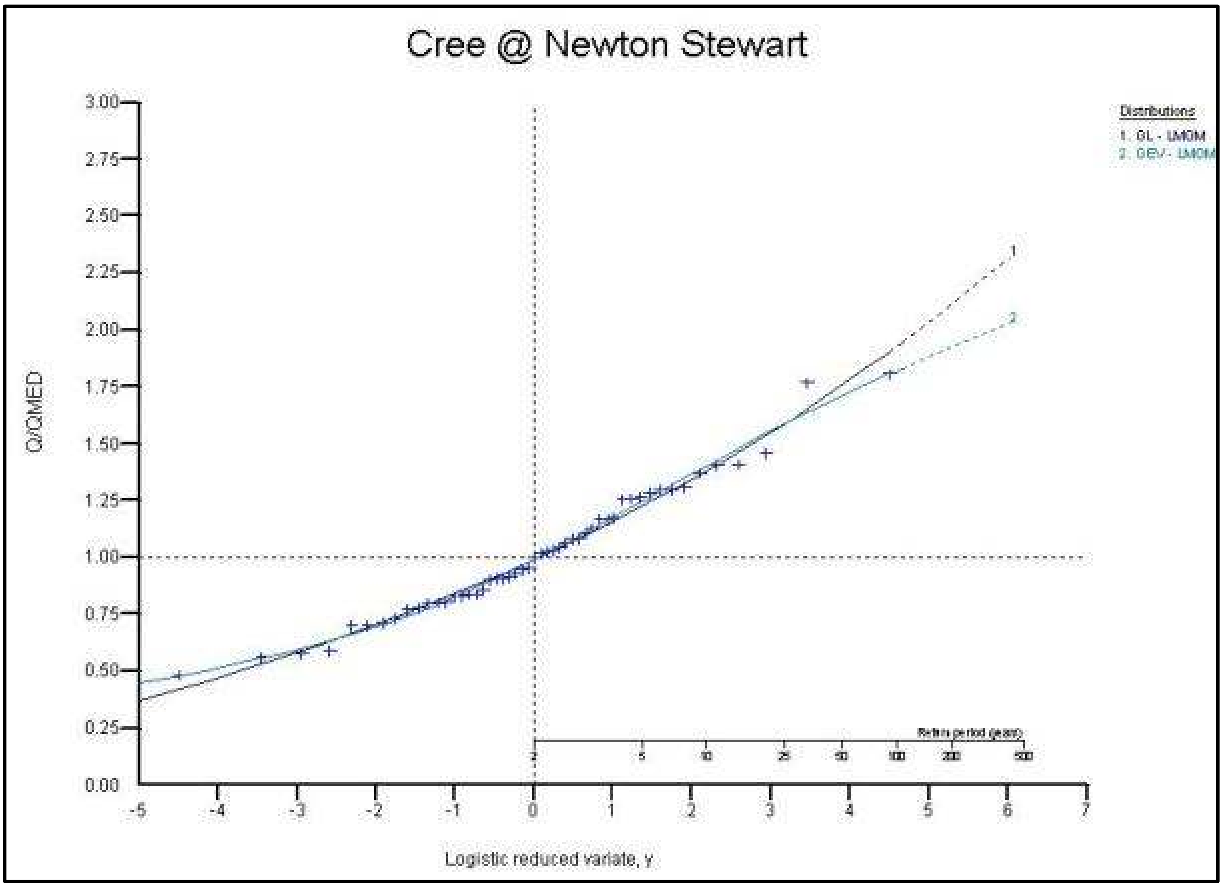


Table 5: Design flows for single site analysis

Return Period	General Logistic (m^3/s)	General Extreme Value (m^3/s)
2	220	220
5	270	270
10	300	310
25	350	350
50	380	370
100	420	400
200	**460**	**420**
500	520	450

4.4.3 FEH Enhanced Single Site

An FEH Enhanced Single Site Analysis was undertaken for the Newton Stewart gauge. This is the most appropriate method (WIN-FAP Version 3) for Pooling Group analyses for gauged sites. Return period flow estimates are summarised in Table 6. The Pooling Group developed for the gauged site is provided in Table 7.

Table 6: Flood frequency analyses for River Cree at Newton Stewart

Return Period	General Logistic (m^3/s)	General Extreme Value (m^3/s)
2	219	220
5	272	280
10	307	310
25	356	360
50	395	390
100	438	420
200	**485**	**450**

It is notable that the design flows produced by the Enhanced Single Site method are similar to those produced using the single site analysis at the gauged site, with the 200 year flow from the Enhanced Single Site Analysis only 5% higher than the flow predicted using the single site analysis.

Based on recommendations with the FEH, the Enhanced Single Site Analysis is considered the most appropriate method for estimation of 200 year design flows at the Newton Stewart gauge. Hence, our best estimate of the 200 year flow at the gauge is 485 m^3/s.

Table 7: Pooling Group for River Cree

Station	Distance	Years of data	Q_{MED} AM	L-CV	L-SKEW	Discordancy
81002 (Cree @ Newton Stewart Edited)	0	51	219	0.159	0.102	1.119
201008 (Derg @ Castlederg)	0.296	33	195.072	0.073	0.055	1.502
3003 (Oykel @ Easter Turnaig)	0.326	28	342.057	0.187	0.243	0.963
76003 (Eamont @ Udford)	0.486	47	179.819	0.183	0.14	0.29
71008 (Hodder @ Hodder Place)	0.596	39	225.574	0.163	0.206	1.041
72006 (Lune @ Kirkby Lonsdale)	0.662	16	441.994	0.143	-0.015	2.443
79006 (Nith @ Drumlanrig)	0.679	39	336.556	0.133	0.132	0.453
27043 (Wharfe @ Addingham)	0.694	35	262.267	0.173	0.089	0.768
60002 (Cothi @ Felin Mynachdy)	0.7	47	177.666	0.22	0.236	1.009
71006 (Ribble @ Henthorn)	0.751	40	219.072	0.15	0.188	0.326
83005 (Irvine @ Shewalton)	0.753	37	193.78	0.191	0.16	0.375
25018 (Tees @ Middleton in Teesdale)	0.756	32	190.903	0.167	0.126	0.976
202001 (Roe @ Ardnargle)	0.76	30	200.492	0.143	0.206	1.381
3002 (Carron @ Sgodachail)	0.765	33	142.802	0.081	0.053	1.355
Total		507				
Weighted means				0.158	0.132	

4.4.4 Penkiln Burn

The flood study focusses on flooding from River Cree, based on peak flows in the Cree. However, in the northern end of the study area in Minnigaff there is the potential that peak flood levels will be produced by high flows in Penkiln Burn. Penkiln Burn design flows were derived using the FEH Pooling Group method, with estimated flows in Table 8.

Table 8: Peak flows for Penkiln Burn

Return Period	Design Flow (m^3/s)	
	Generalised Logistic	Generalised Extreme Value
2 year	52	52
5 year	66	67
10 year	75	77
25 year	88	89
50 year	99	98
100 year	111	106
200 year	124	115

4.4.5 Return Period Assessment for November 2012 Event

The November 2012 flood event has been described in detail in Section 2.4. During the event the peak observed water level at SEPA gauge was 3.82 m AOD, which is converted to a flow of 387 m^3/s, based on the most recent rating curve at the gauge.

Comparing the peak flow during the 2012 event to the return period flows in Table 6, the return period of the 2012 event is estimated at around 40-50 years (45 years). However for comparison a statistical analysis of peak water levels at the gauge was undertaken and this gave a return period for the 2012 event of around 85 years.

Such a discrepancy between flow and water level data is not unexpected, especially at sites where water goes out of bank at high flows. For such sites, increases in flow, once the river is out of bank, produce relatively small changes in water level. This would have the tendency of clustering peak water levels around the overtopping level. Once these data sets are analysed statistically it might result in a flat water level frequency curve, which would tend to under-estimate the return period of any flood event. Hence, based on available data the 2012 event would appear to have a return period of between 40 and 85 years, with the likelihood of the return period being closer to the upper end of the scale.

4.4.6 Impact of Climate Change

SPP (2010) states that:

> *"The design of new development should address the causes of climate change by minimising carbon and other greenhouse gas emissions and should include features that provide effective adaptation to the predicted effects of climate change. The changing climate will increase the risk of damage to buildings and infrastructure by flood, storm, landslip and subsidence. Development*

should therefore normally be avoided in areas with increased vulnerability to the effects of climate change, particularly areas at significant risk of flooding, landslip and coastal erosion and highly exposed sites at significant risk from the impacts of storms."

SEPA currently recommend a 20% increase in peak flow for the 0.5% AEP (1:200) event, in accordance with DEFRA (Department of Environment, Food and Rural Affairs) research. Additional guidance from the Environment Agency[3] has provided regionalised estimates on the effect of river flows based the UKCP09 projections. The published data for the Solway Firth river basin is provided in Table 9.

Table 9: Environment Agency (2011) - Regionalised impact of climate change to river flows within Solway River Basin

Region	Total Potential change for 2020s	Total Potential change for 2050s	Total Potential change for 2080s
Solway			
Upper Range	25%	35%	65%
Best Estimate	15%	20%	25%
Lower Range	0%	5%	15%

It can be seen that estimates for climate change increase for flows in the Solway Firth River Basin detail a 20% and 25% increase in precipitation for the periods 2050 and 2080 respectively. However, more research is required before practical guidance can be provided as to the impact that UKCP2009 will have on extreme rainfall and flooding across Scotland. However, for the current study a 20% increase in peak flows is assumed.

4.5 Design Hydrographs for Model Boundary Conditions

In order to run the mathematical models used to predict flood levels in Newton Stewart, design flow hydrographs need to be defined for the model boundaries on the River Cree upstream of Newton Stewart and Penkiln Burn.

4.5.1 Derivation of Hydrograph Shape

Rather than using synthetic hydrograph shapes produced by rainfall-runoff models, the hydrograph shape for River Cree was based on analysis of observed flow hydrographs at the SEPA gauge in Newton Stewart.

Hydrometric data (15 minute stage recordings) was provided by SEPA for five significant flood events that were observed at the gauge in Newton Stewart. The five events are plotted in Figure 8.

The five events were;

- 12/11/2012
- 19/06/2012
- 13/12/2008
- 24/11/2006

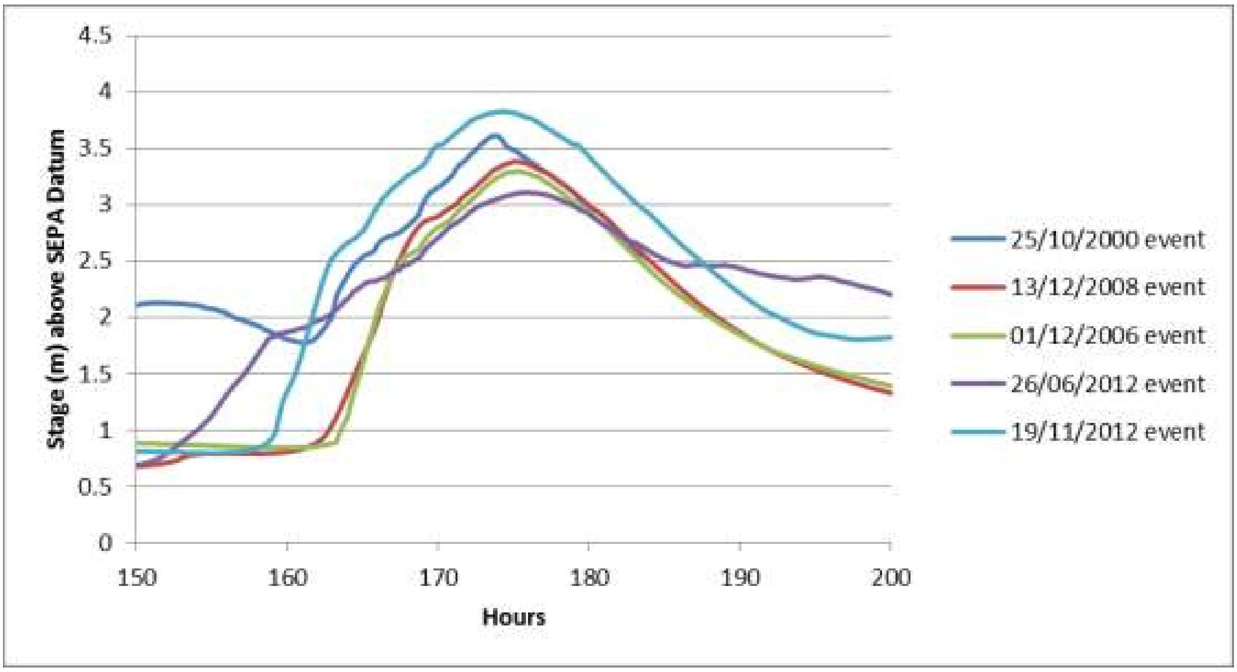
[3] Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities

- 18/10/2000

The hydrograph shape of the 2012 event was chosen to represent the design flow events used in the modelling study. Therefore, for each design flow event the 2012 event hydrograph was scaled to produce the required design flow outlined in Section 4.4.3.

For Penkiln Burn the input hydrograph was based on FEH Rainfall-Runoff model hydrograph scaled to produce the required peak flow outlined Section 4.4.3. In the first instance a FEH Rainfall-Runoff model was run for the full River Cree catchment to identify a storm duration that would produce the hydrograph shape consistent with observed hydrographs in Figure 8 (18.5 hours). The FEH Rainfall-Runoff model was then re-run for the Penkiln Burn for the same duration.

Figure 8: Observed flood hydrograph shape at Newton Stewart SEPA gauging station



4.5.2 Partitioning of Flows between River Cree and Penkiln Burn

Design flows for River Cree were calculated in Section 4.3. For the purpose of flood mapping, the modelled flow at the gauged site had to match with the calculated design flow. However, the model required upstream flow hydrographs for River Cree upstream of Newton Stewart and Penkiln Burn. To achieve the desired flow in Newton Stewart, peak flows in River Cree and Penkiln Burn were scaled based on their relative catchment areas; River Cree has a catchment area of 319 km^2 upstream of the town whereas the Penkiln Burn has a smaller catchment area of approximately 46 km^2. The flows for each of the inflow hydrographs are shown in Table 10.

It was also assumed that peak flows in both channels would coincide to produce the appropriate flow in the town of Newton Stewart. As the two watercourses combine at the upstream end of Newton Stewart uncertainties associated with the spilt of flows between the two watercourses is not expected to affect flood predictions within the town. During flood modelling the predicted peak flow at the gauged site was compared to the required design flow and inflows were scaled appropriately to produce the required flow in the town, if discrepancies were found.

Table 10: Peak inflows for hydraulic model

Return Period	Total Flow (m^3/s)	River Cree (m^3/s)	Penkiln Burn (m^3/s)
2 year	225	192	33
5 year	275	235	40
10 year	310	265	45
25 year	360	308	52
50 year	395	338	57
100 year	440	376	64
200 year	485	415	70
500 year	585	498	84

4.6 Sea Level Assessment

The NTL (Normal Tidal Limit) is located approximately 120 m downstream of the A75 road bridge. Under normal tide conditions water levels within the study area are not thought to be influenced by downstream tidal conditions. However, based on discussions with local residents (Cree FLAG group) and the local council, it is clear that slack water periods can be observed at high tide extending upstream of the A75 road bridge, but not to the SEPA gauge site. Under extreme sea level conditions (e.g., extreme astronomical tides and storm surge) it is possible that water levels in the town could be impacted by tide levels. However, it would be unlikely that such high tide conditions would coincide with the peak of a river flood.

For completeness and to assess potential high tide levels at the downstream end of the model, extreme sea levels have been derived using standard methods as outlined below.

4.6.1 Derivation of Still Water Levels

Extreme sea levels are determined by a combination of astronomical tides and storm surges caused by weather conditions offshore. Astronomical tides are created by the attraction of the moon and are accurately predictable in advance. Storm surges are caused by the meteorological factors (such as winds acting on sea surface and variation in atmospheric pressure), the prediction of which are less accurate.

A recent study by DEFRA, SEPA, and EA (‘Coastal boundary conditions for UK mainland and islands’, EA 2011) gives the predicted still water levels (astronomical tides and storm surges) around the UK coastline. Extreme sea levels (still water levels) for a range of return periods based on DEFRA/EA (2011) at the mouth of the River Cree at Creetown are provided in Table 11.

4.6.2 Impact of Climate Change

There are a number of methods for the estimation of the effect of climate change on sea levels.

DEFRA guidance (2006) provides estimates of the likely effect of climate change on sea water levels over the next century for areas around the UK coastline, Table 12. Between 2010 and 2085 the DEFRA guidance would indicate a sea level rise of around 0.55 m for most of Scotland.

Table 11: SEPA Extreme Sea levels at the mouth of River Cree

Return Period	Extreme Sea level (m AOD)[a]	Confidence Interval (m)
1 year	4.73	0.2
2 year	4.84	0.2
5 year	4.99	0.2
10 year	5.10	0.2
25 year	5.23	0.2
50 year	5.34	0.2
100 year	5.43	0.3
200 year	5.52	0.3
500 year	5.60	0.3

[a.] Values for DEFRA/EA (2011) Point 89 (Chainage 1524)

Table 12: Adjustments due to climate change, DEFRA (2006)

Component	1990 - 2025	2025 - 2055	2055 - 2085	2085 - 2115
New sea level rise (mm / year)	2.5	7.0	10.0	13.0
Increase in extreme wave height	+ 5 %	+ 5 %	+ 10 %	+ 10 %

SNIFFER (2008) provides a review of available research on the effect of climate change on sea levels, storm surges and wave heights around the Scottish coast. The report does not give guidance on estimates to be used for design, but indicates that sea level rise in this part of the Solway Firth might be expected to be of the order of 1.6 – 0.24 m by 2080 depending on the climate change model scenario used.

UKCP09 provide the latest climate change predictions for a range of parameters, including sea level. The UKCP09 provides predictions for a range of emissions scenarios (High, Medium and Low) and provides results as a probability distribution. Predictions nearest to the River Cree at Creetown are provided in Table 13. At present, there is no guidance as to the most appropriate emissions scenario and exceedance percentile to use for flood risk assessments in Scotland.

Table 13: UKCP09 sea level rise estimates (m) at the mouth of River Cree (2008 – 2085)

Emissions Scenario	Net Sea Level Rise (m)		
	5% probability	50% probability	95% probability
Low	0.02	0.14	0.27
Medium	0.03	0.19	0.36
High	0.04	0.25	0.46

Based on raw data output from UKCP09 user interface

Based on the available sea level rise estimates, values ranging from 0 – 0.55 m are available from the three methods. This assessment is based on the most recent UKCP09 model results. Based on these results and to take into account the inherent uncertainty involved in estimating sea level rise, this assessment considers:

- 0.25 m is likely to be the 'best estimate' of sea level rise due to climate change (UKCP09, High Emissions, 50%ile), i.e., the value appropriate for derivation of the 1 in 200 year + climate change flood water level.

4.7 Joint Probability of Extreme Flows and Sea Levels

Joint probability describes the likelihood that two events will occur at the same time. In this study we considered the joint probability of an extreme still water level occurring at the same time as an extreme flood flow within a river. An extreme, 1 in 200 year high tide level has a 0.5 % chance of occurring once during any year and when it occurs it will have a duration of only a couple of hours or so. In a similar way a 1 in 200 year fluvial flow also has a 0.5 % chance of occurring once during any year and will also have a duration of hours only. Hence, the likelihood that both events will occur at the same time is small.

There has been much research into this issue and the estimation of joint probabilities is covered by many publications including Environment Agency (EA 2005). EA (2005) describes a desk study method that predicts joint probability based on the degree of dependency between the two variables. However, initial model results indicated that extreme water levels in the River Cree within the town are not affected by water levels at the downstream boundary which is approximately 9 km away from the study area. As a result, model runs have been undertaken with the conservative assumption that both fluvial flows and downstream water levels are assumed to jointly occur, i.e. 200 year fluvial flows coupled with a 200 year sea level. This is the worst case scenario.

5 Hydraulic Assessment

5.1 Modelling Approach

Due to the overtopping flow pathways within the urban extent of Newton Stewart and the interactions between the floodplain and the river channel, a 1D-2D linked modelling approach using the ISIS 1D/2D mathematical modelling package has been used to predict the flood extent within the study area.

5.1.1 Schematisation

Both river channels (the River Cree and Penkiln Burn) are represented by a 1D model and are defined by river cross-sections. The cross-sections are truncated at bank top locations which are then dynamically liked to the 2D domain through a designated boundary condition. Water levels in the 1D domain, exceeding the bank top levels, are passed into the 2D domain which is constructed based on a DTM of the surrounding floodplain. Flood waters are able to exchange between domains (i.e. river channel and floodplain), with conservation of mass between the domains.

5.2 Topographic Datasets

5.2.1 Survey

MH Surveyors were commissioned to undertake a comprehensive river channel survey of both watercourses. The survey included channel cross-sections and hydraulic structures (i.e. bridges, culverts and weirs) throughout the study area and selected areas located significantly upstream of the study area.

River Cree:

- In total 32 river channel cross sections were surveyed within the wider study area;
- An additional 6 channel cross sections have been surveyed upstream of the study area;
- Three bridge structures have been surveyed including; Bridge of Cree, metal footbridge close to SEPA gauge and A75 road bridge; and
- Masonry weir downstream of Bridge of Cree.

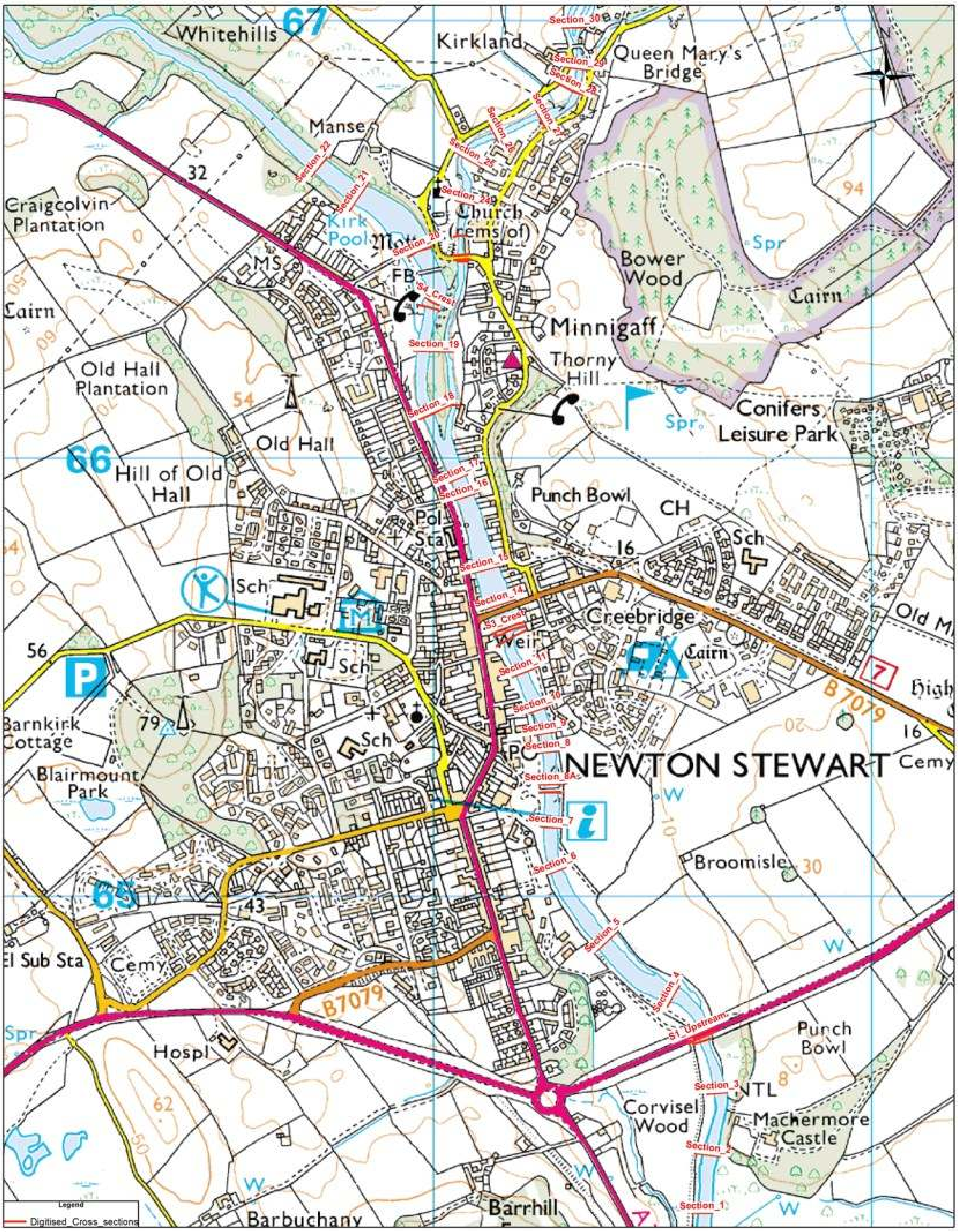
Penkiln Burn:

- 14 channel cross sections of the Penkiln Burn were surveyed in the study area;
- 4 channel cross sections were surveyed in the upper catchment; and
- The concrete bridge structure upstream of the confluence with River Cree.

A footbridge located upstream of the confluence with Penkiln Burn was not included in the model as a field survey indicated that it would have negligible effect on flood levels in the watercourse, as the footbridge rises high above the river level.

The locations of surveyed cross-sections are provided in Figure 9.

Figure 9: Location of surveyed channel cross-sections



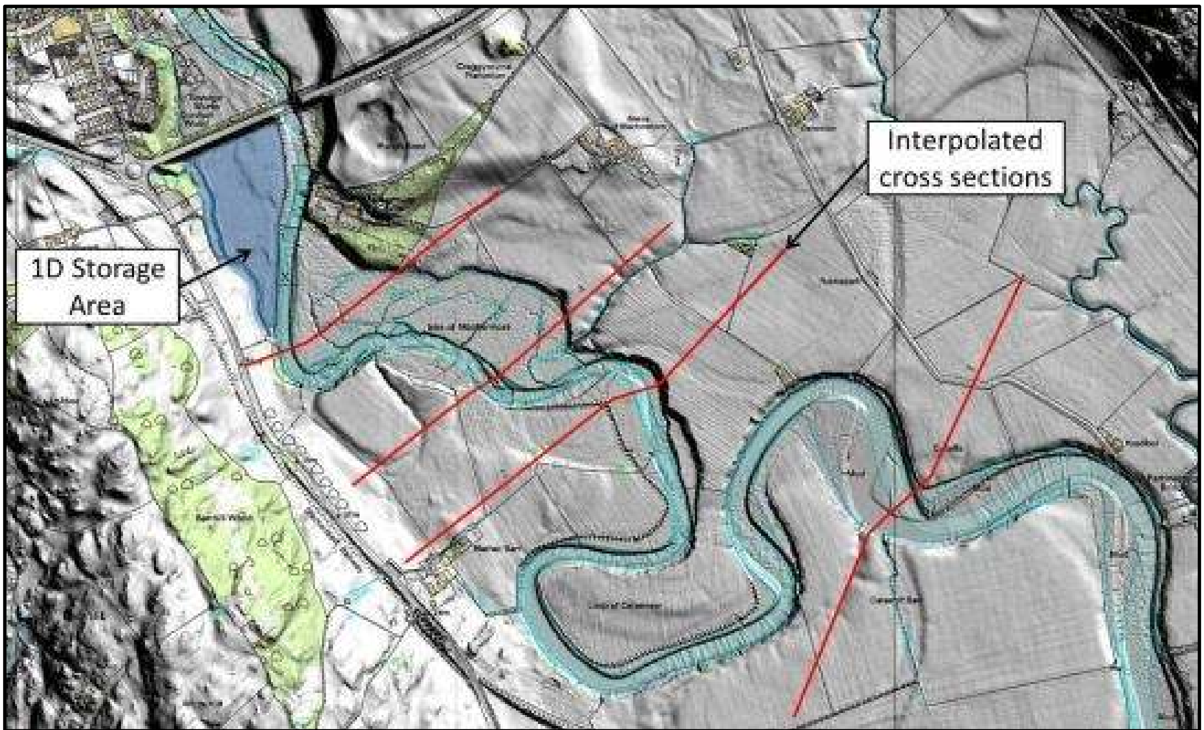
Kaya Consulting Limited Reproduced by permission of Ordnance Survey on behalf of The Controller of Her Majesty's Stationery Office. © Crown copyright. All rights reserved. Licence number 100045301.	**Newton Stewart Flood Study** **Location of Surveyed Cross-sections**	0 50 100 200 300 400 500 600 Meters **Kaya Consulting Ltd.** Maps produced by Kaya Consulting Limited Phoenix House Strathclyde Business Park, Bellshill North Lanarkshire, ML4 3NJ, Scotland, U.K. **Scale** 1:7,750 **www.kayaconsulting.co.uk**

5.2.2 Digital Terrain Model (DTM)

Dumfries and Galloway Council provided a 2 m LiDAR DTM of the study area. The raw LiDAR data has been “filtered” to produce a terrain surface without vegetation and structures. The DTM is used as the topographical data for the 2D model within Newton Stewart and Minnigaff. Model runs within the 2D domain have been undertaken using a 2.5 m regular grid within Newton Stewart and 5 m grid within Minnigaff

The DTM model was also used to extract 4 additional cross sections downstream of the A75 road bridge. Topography within this area is flat, comprising of fields and open ground, hence LiDAR elevations in this area are expected to be sufficiently accurate enough to derive model cross sections, see Figure 10. Floodplain storage downstream of the A75 road bridge was modelled within a 1D ISIS model as flood storage areas. Storage/elevation relationships for the storage areas were extracted from the LiDAR DTM.

Figure 10: Interpolated cross sections and 1D storage area



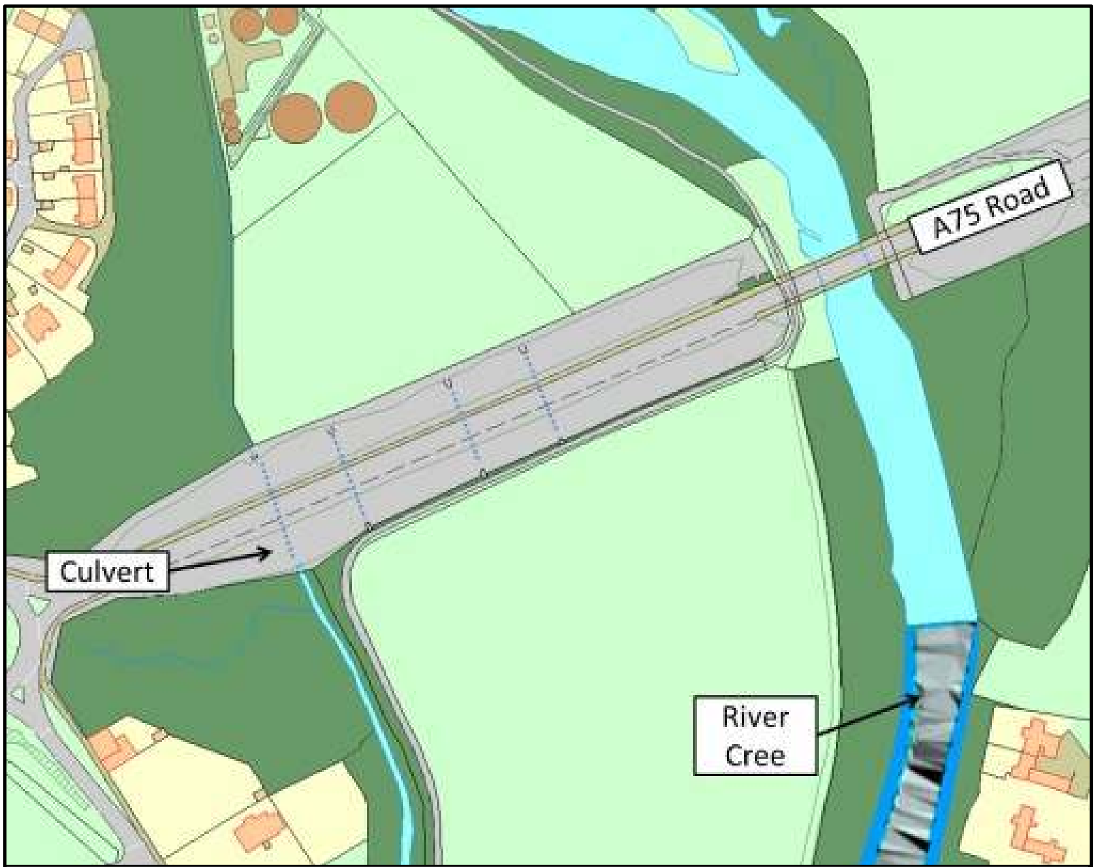
5.3 Key Hydraulic Structures

Throughout the study area there are a number of hydraulic structures affecting flows in River Cree and Penkiln Burn. The key structures have been included in the hydraulic model. A location and description of the surveyed structures are provided below:

- Main Bridge of Cree (241161, 565655) – Old masonry arch bridge approximately 80 m wide comprising of 5 arches approximately 12-15 m wide and 4.6 - 4.7 m high (middle arch);

- Masonry Weir (241170, 565607) – Historical weir spanning the entire width of the channel. The weir is constructed from masonry bricks, which have become displaced within a small area in the centre of the channel;
- Footbridge (241270, 565247) – Metal footbridge spanning the entire width of the channel which is supported by two concrete abutments. The deck of the bridge is around 4 m above the bed of the channel;
- A75 Road Bridge (241614, 564685) – Concrete road bridge spanning the entire channel. The soffit of the bridge is elevated significantly above the bed of the channel and would not cause an impediment to flows within the channel. The bridge is supported by two concrete piers which are approximately 1 m wide and are located a short distance either side of the main channel;
- Penkiln Road Bridge at "Old Minnigaff" (241064, 566458) – Concrete flat bridge which crosses the entire Penkiln Burn channel. The soffit of the bridge is elevated around 7 m above the channel;
- Culverts under A75 – Invert levels for four 1.5 m culverts which pass under the A75 road embankment have also been surveyed. The locations of the four culverts are shown in Figure 11.

Figure 11: Culverts under A75 road embankment



A small suspension footbridge, known as "Penkiln Bridge", is located close to the upstream boundary of the study area. The soffit of the bridge is situated sufficiently above the bed of the channel that it will not affect flood levels. Therefore it has been omitted from the model. In addition, there are two "islands" at the confluence of the Penkiln Burn with River Cree, which have been formed from sediment accumulations over the years. The islands have been included within the 1D model.

5.4 Model Boundaries

5.4.1 Upstream Boundary Condition

Flows entering into the hydraulic model from the River Cree and Penkiln Burn are represented by a flow hydrographs as outlined in Section 4.5.

5.4.2 Downstream Boundary Condition

The downstream boundary is represented by a tidal, head-time boundary unit. The boundary represents water levels against time and is considered industry standard in areas where there is a tidal influence. Water levels used in the model have been derived from the tidal data as described in Section 4.6.

The timing of the inflow hydrographs and tide was set to coincide, so that peak flows occurred at the same time as the given peak tide to produce a worst case combination (resulting in highest water levels within the tidal reaches of the river). This was considered acceptable as initial model runs indicated extreme tides have no impact on extreme water levels within the urban part of the town.

No lateral inflows have been added to the 2D domain. Hence, water can only enter the 2D domain through overtopping from River Cree or Penkiln Burn.

5.5 1D Model Roughness

Roughness values used in the model are summarised in Table 14 below:

Table 14: 1D model friction values

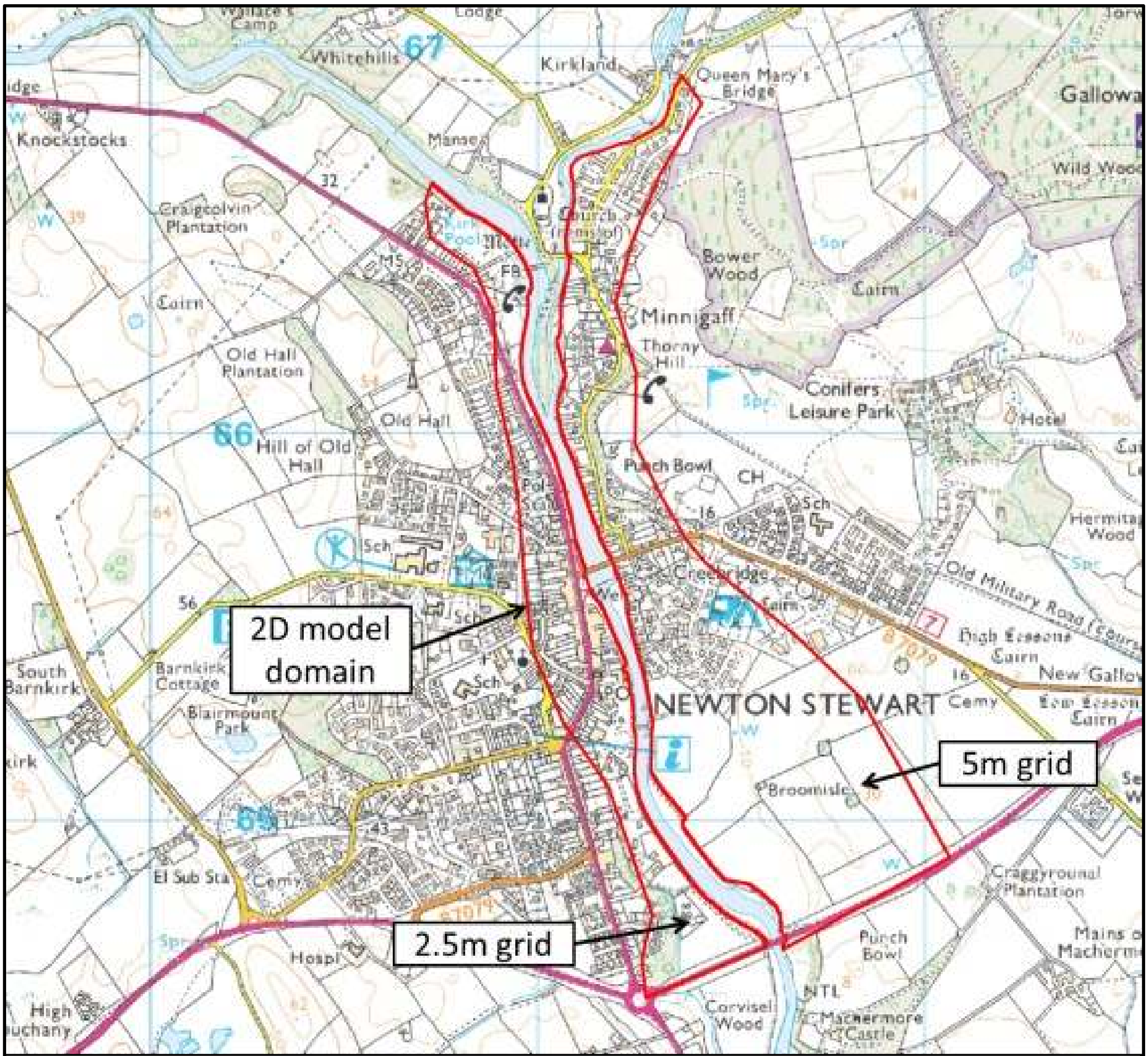
Location	Roughness value (manning's n)
Main Channel	0.045
Channel Banks	0.055
Island in Channel	0.055
Floodplain area downstream of study area	0.055

5.6 2D Model Set-up

The extent of the 2D model domain is shown in Figure 12 and covers two areas east and west of River Cree and Penkiln Burn. The 2D domain was extended far enough away from the river channels so that flood waters in the 2D domain were not constrained by the edges of the 2D models. This allows a natural flood extent to be generated by the model, unaffected by the domain boundaries.

The 2D model domains extend from approximately 0.75 – 1 km upstream of the confluence of River Cree and Penkiln Burn, downstream to the A75 road embankment. This embankment provided a physical barrier to flood waters leaving the 2D domain, hence providing an ideal location for the 2D boundary.

Figure 12: 2D model domain



Within the ISIS 2D software water can pass between the 1D and 2D model areas where the models are connected using "link lines". The over-spill level for each link line was determined based on the available survey data, e.g., at walls, link lines were based on the top of wall level and at locations where there were no obvious embankments the link lines were set to the local ground level based on the topographical survey data. Where link lines represented as walls a spill coefficient of 1.7 was attributed to the line. In areas where there is no recognised overtopping spill, e.g., at grassed embankments, a coefficient of 0.9 was used. These coefficients are less critical when peak water level in the 2D domain approach the water level in the river.

A uniform roughness (Manning's n) of 0.045 was applied to the entire 2D domain. Considering that most land is urban, such a roughness value appears reasonable. Sensitivity runs were also undertaken with a higher roughness value. In addition, “Z lines” have been used to digitize impermeable features within the domain where flood flow paths are known be restricted, i.e. concrete walls etc. All buildings are represented as impermeable polygons within the model domain (bounded by Z lines). This prevented floodplain flows from passing through the buildings. A sensitivity run was also undertaken allowing floodplain flows through buildings to allow comparison with the base case simulation with buildings included.

5.7 Model Calibration

Whenever possible and to increase confidence in model predictions, model calibration needs to be carried out. This involves comparison of model predicted water levels/flows and observed water level and flood data. Calibration is carried out by adjusting physical parameters within the model which have been estimated based on standard methods, i.e., river channel friction values, etc. The model is re-run with different parameters until a reasonable agreement is obtained with recorded water levels and flood extents, making sure that model parameters stay within acceptable limits.

For most modelling studies calibration is not possible given a lack of historical information to compare against model predictions. However, for this study it was possible to compare model flood predictions against data obtained during the November 2012 flood event and against the rating curve (and spot flow measurements) for the SEPA gauge located within the study area.

The key hydraulic calibration parameters used to calibrate the model included:

- Manning’s “n” values for river channel and floodplain (2D domain);
- Coefficients for spill units;
- Discharge co-efficient including weirs;
- Changes to building representation; and
- Changes to downstream boundaries.

5.7.1 2012 Calibration Event

The November 2012 flood event in Newton Stewart occurred during the day on 11th November 2012. The event was well documented and as a result a high volume of information was available to help calibrate the model. Video and photographic information, such as the photos shown in Appendix B, were used in conjunction with the topographic survey information of the study area to identify the approximate extent and level of the floodwaters.

As described in Section 4.4.5 above, and agreed by SEPA, the November 2012 event was considered to be comparable to a 40-85 year return period event and provided a suitable event for calibration purposes. The peak flow rate used in model calibration for this event was 387 m^3/s.

Following initial model calibration based on hydrometric records at the SEPA gauge, the model was further verified against a visual comparison of floodplain depths and recorded extents. This was supplemented by direct comparison of photographs taken during the 2012 event and by anecdotal evidence provided by Mr James McLeod of Dumfries and Galloway Council, who visited the area on the day of the flood.

Table 15: Model results compared against photographic/anecdotal evidence (November 2012)

Location	Ground Level (m AOD)	2012 Flood Level (m AOD)	Predicted Water Level (m AOD)
SEPA Office	12.2	At least 12.0	11.9
Near suspension footbridge	12.7	At least 12.7	12.7
Millcroft Road	9.7	At least 10.0	10.5
Fire Station	10.1	At least 10.3	10.5
LHS Bridge of Cree	9.8	At least 9.8	10.2
Car Park and toilets	7.8	At least 8.6	8.6
Footpath near sewage station	7.5	At least 7.6	7.7

Table 15 indicates that the model provides a good comparison when compared against locally recorded data.

At Bridge of Cree, the predicted water level is approximately 0.6 m above the spring level of the arches (which is 9.6 m AOD). This correlates well with the observed water level.

As well as largely agreeing with the results in Table 15, when reviewing initial flood flow paths originating from overtopping flood waters on Riverside Road, Mr McLeod stated that flooding did not directly extend as far as Victoria Street. Flooding mechanisms and flow paths were reviewed and it was consequently decided that, due to the effect of urban structures were having on flooding in this area, all buildings and significant structures would be rendered impermeable, i.e., flood waters would be blocked by buildings and structures. This change resulted in the model producing more realistic flooding in this area and helped increase the level of confidence in ability of the model to replicate flooding in a critical location within the study area.

Model predictions were also compared to recorded water level and flow measurements at the SEPA gauge in Newton Stewart. Figure 13 shows comparison of recorded and predicted water levels and flows. There was a very good comparison between model predictions and observed data at the gauging site.

Further calibration was also undertaken with regards to the number of properties known to have flooded during the 2012 event. Flood maps generated for the 2012 event indicate that around 20 properties were predicted to have a flood depth in excess of 0.2 m and were likely to have flooded during the event. Council records state that only 12 properties are known to have flooded; however, this is likely to be a lower limit and may not reflect the actual number of flooded properties due to a number of reasons including; property being independently protected, emergency pumping of flood waters, flooding instances not being reported to the council. The ~20 properties predicted to flood were also visually cross-referenced with photographic evidence recorded during the event. This additional check was discussed with the council and it was agreed that the model results produced an acceptable level of agreement with observed data.

5.7.2 Rating Curve Comparison

The model results were also compared with the rating curve for the SEPA gauge in Newton Stewart. The highest spot gauging records were also requested from SEPA to aid the comparison. The final model rating curve is presented against the SEPA rating curve along with the highest spot gauge recordings, see Figure 14.

Figure 13: Comparison of recorded and predicted water level and flow at SEPA gauge (81002) during 2012 event

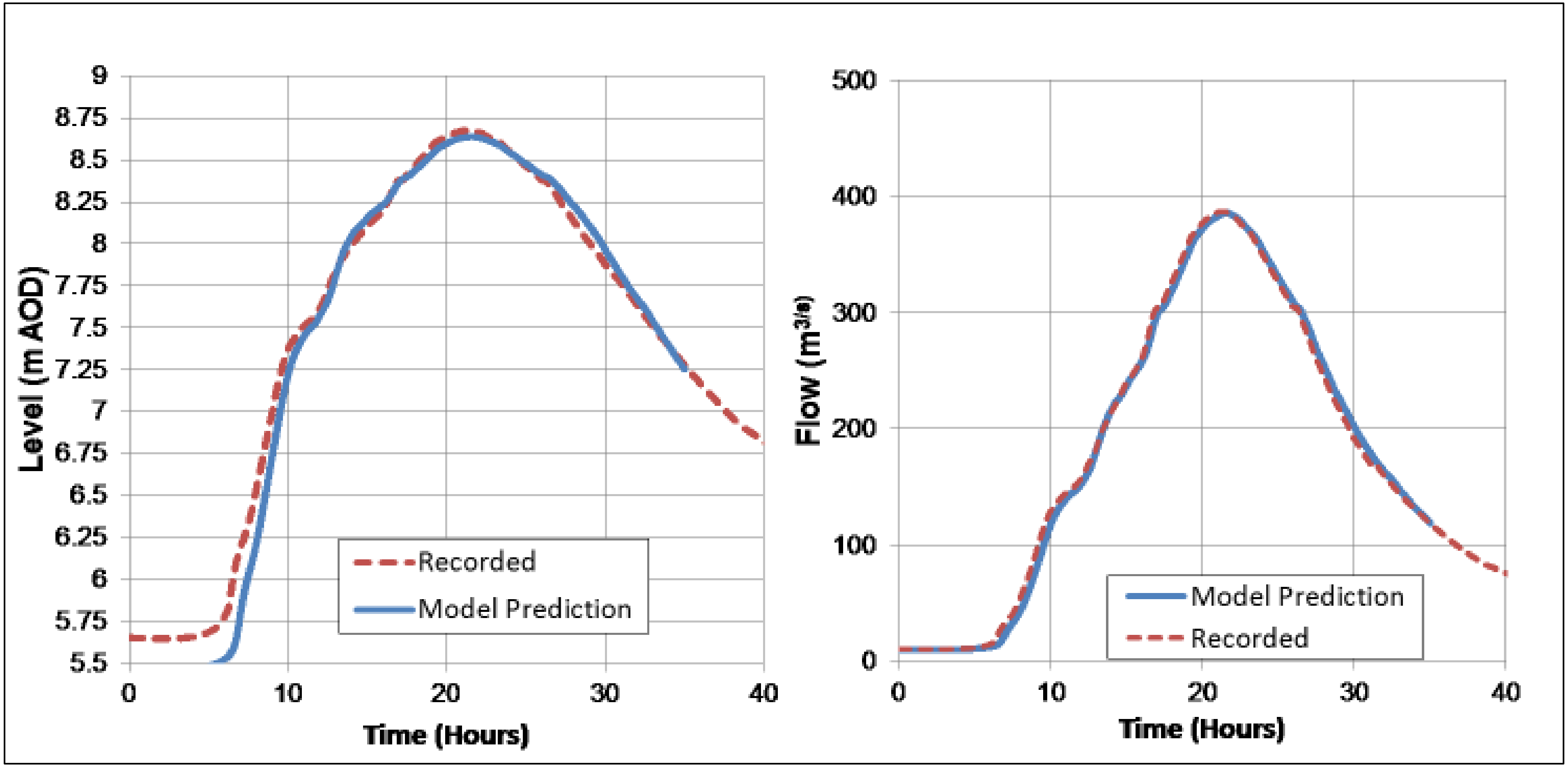
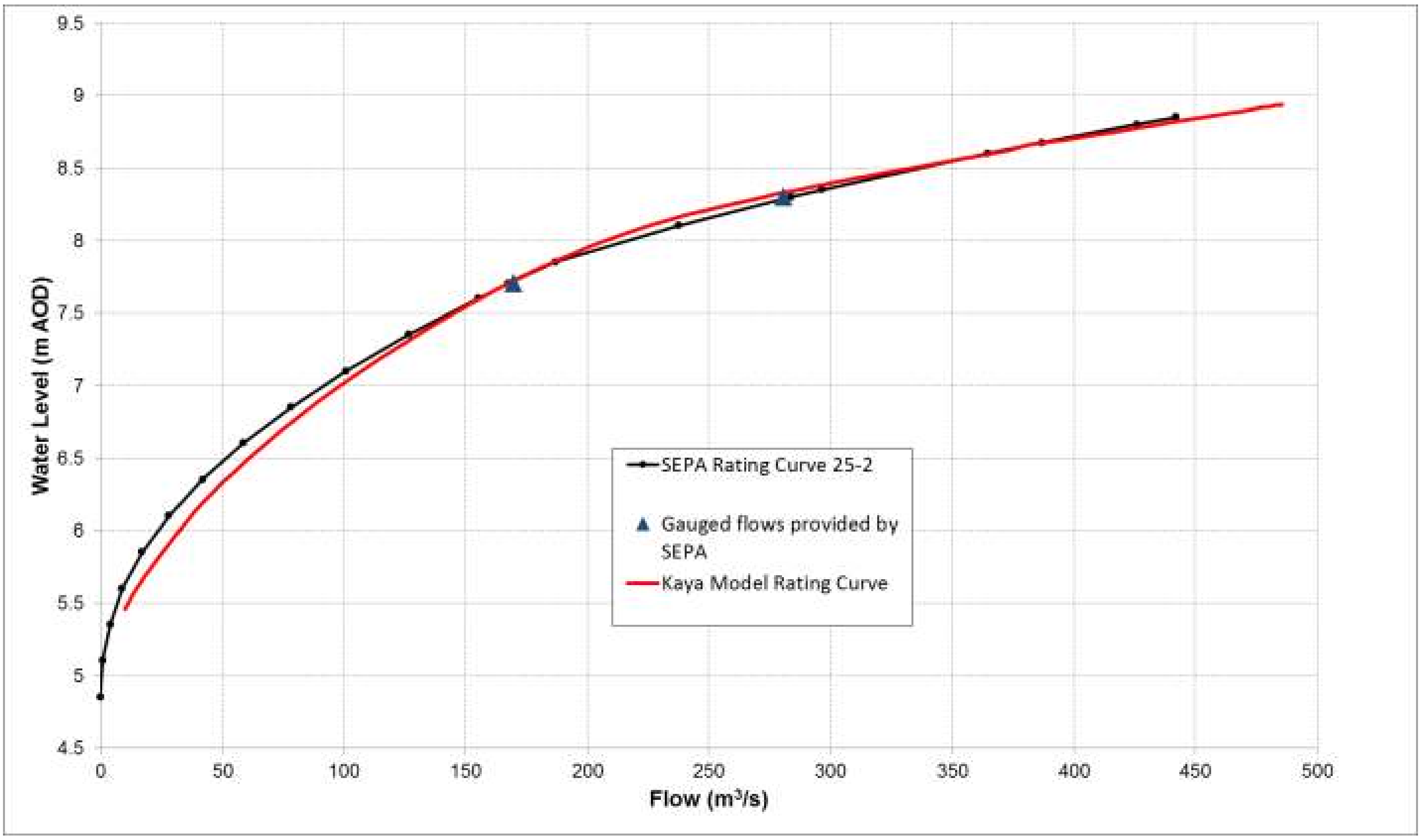


Figure 14: SEPA Rating Curve for 81002 plotted against modelled results



Initial baseline model results indicated that the model was over-predicting flows for higher return periods, thus friction values for floodplains and channel banks were decreased in both model domains. Final results indicated that the model compares favourably with the SEPA rating curve with a comparison table provided in Table 16.

Table 16: Model results compared against SEPA gauged recordings

Flow (m^3/s)	SEPA recorded stage (m AOD)	Difference with model prediction (mm)
170	0.045	22
281	0.055	29
387 (SEPA 2012 estimation)	0.055	-5

5.7.3 Calibration Summary

The model calibration process focused on the simulation of flow and flood behaviour for the November 2012 flood event and comparison with SEPA data from the Newton Stewart gauge.

The model was shown to provide a goof fit with observed flood levels and flood extent information from the November 2012 event.

Comparison of modelled and observed stage/flow output indicated that the model produced water levels slightly higher than recorded at high flows. The maximum differences in the modelled reach are 22 mm and 29 mm for SEPA estimates for gauged flows of 170 m^3/s and 281 m^3/s, respectively. Such small differences are regarded as acceptable for the purpose of assessing flood levels throughout the study area.

5.8 Model Runs

The calibrated 1D-2D linked model of the River Cree and Penkiln Burn was run for a range of flow combinations, with and without the effects of climate change, and with potential flood mitigation options. Model runs undertaken and peak flows assumed for each run are summarised in Table 17.

Table 17: Modelled scenarios

Run No.	Scenario	Peak flow in River Cree (m^3/s)	Peak Flow in Penkiln Burn (m^3/s)	Downstream Water Level (m AOD)
1	Q200 + CC	498	84	5.60
2	Q200	415	70	5.52
3	Q100	376	64	5.43
4	Q50	338	57	5.34
5	Q25	308	52	5.23
6	Q10	265	45	5.10
7	Q5	235	40	4.99
8	Q200 Defended-200 year defence	415	70	5.52
9	Q200 Defended-50 year defence	415	70	5.52
10	Q200 Defended-10 year defence	415	70	5.52
12	Q200-Gravel berm removed	415	70	5.52
13	Q200-Increased flow area through A75	415	70	5.52
14	Q200-Increase soffit level of metal footbridge	415	70	5.52

(Q200: 200 year flow; CC: climate change)

Scenarios 1 to 7 were run assuming a base case condition (i.e., the existing case using the surveyed data with no proposed flood defences), for a range of standard design events; 5, 10, 25, 50, 100, 200 years and 200 year plus climate change. The climate change condition assumes a 20% increase over the 200 year design flow.

The defended case refers to model runs with flooding of River Cree being prevented by a wall on both banks of the channel to a location approximately 140 m downstream of the metal footbridge.

5.9 Model Results – Base Case Condition

The model results at selected cross-sections and for all return periods, are provided in Table 18 and Table 19 below, whilst detailed flood maps are provided in Appendix C. A detailed description of flood mechanisms and location of flooding including flood maps, for the 1 in 200 year event is described below.

Table 18: Model results for River Cree; Base Case

River Cree Cross-section	Run 1: 200 year plus CC	Run 2: 200 year	Run 3: 100 year	Run 4: 50 year	Run 5: 25 year	Run 6: 10 year	Run 7: 5 year
Section 22	14.00	13.65	13.52	13.36	13.22	13.01	12.85
Section 21	13.86	13.51	13.39	13.23	13.10	12.90	12.74
Section 20	13.45	13.13	13.02	12.88	12.76	12.59	12.45
S4_Crest	13.09	12.75	12.60	12.45	12.34	12.16	12.03
Section 19	12.06	11.74	11.59	11.43	11.30	11.11	10.96
Section 18	11.89	11.50	11.34	11.12	10.94	10.68	10.49
Section 17	11.30	11.02	10.89	10.71	10.56	10.34	10.17
Section 16	11.43	11.03	10.89	10.67	10.51	10.27	10.11
Section 15	11.18	10.80	10.64	10.42	10.25	10.00	9.84
Section 14	11.06	10.64	10.49	10.26	10.08	9.82	9.65
s3_Crest	11.09	10.66	10.49	10.26	10.08	9.81	9.64
Section 12	10.34	9.96	9.83	9.63	9.47	9.23	9.05
Section 11	9.88	9.56	9.41	9.23	9.10	8.89	8.74
Section 10	9.43	9.12	8.98	8.91	8.79	8.61	8.46
Section 9	9.20	8.98	8.86	8.76	8.66	8.50	8.38
Section 8	9.11	8.89	8.77	8.67	8.57	8.42	8.30
Section 8A	9.48	9.13	8.96	8.78	8.62	8.39	8.22
Section 7	8.66	8.30	8.16	8.02	7.92	7.77	7.66
Section 6	8.61	8.19	8.02	7.80	7.63	7.41	7.28
Section 5	8.47	8.03	7.85	7.60	7.41	7.14	6.95
Section 4	8.29	7.87	7.66	7.40	7.20	6.91	6.68
S1_Upstream	7.13	6.91	6.80	6.66	6.54	6.36	6.22
Section 3	7.19	6.94	6.82	6.67	6.54	6.37	6.23
Section 2	7.17	6.93	6.80	6.65	6.52	6.34	6.20
Section 1	7.17	6.91	6.79	6.63	6.50	6.32	6.18

Table 19: Model results for Penkiln Burn; Base Case

Penkiln Burn Cross-section	Run 1: 200 year plus CC	Run 2: 200 year	Run 3: 100 year	Run 4: 50 year	Run 5: 25 year	Run 6: 10 year	Run 7: 5 year
Section 27	17.55	17.39	17.31	17.23	17.17	17.07	17.00
Section 26	15.83	15.66	15.57	15.47	15.40	15.29	15.20
Section 25	14.74	14.58	14.51	14.42	14.35	14.26	14.15
Section 24	12.85	12.59	12.46	12.32	12.20	12.04	11.92
Section 23	12.62	12.32	12.17	12.01	11.88	11.68	11.53
S5_Upstream	12.62	12.30	12.14	11.97	11.82	11.61	11.44
Section 19	12.04	11.70	11.55	11.38	11.23	11.01	10.85
Section 18	11.63	11.27	11.13	10.92	10.76	10.52	10.35

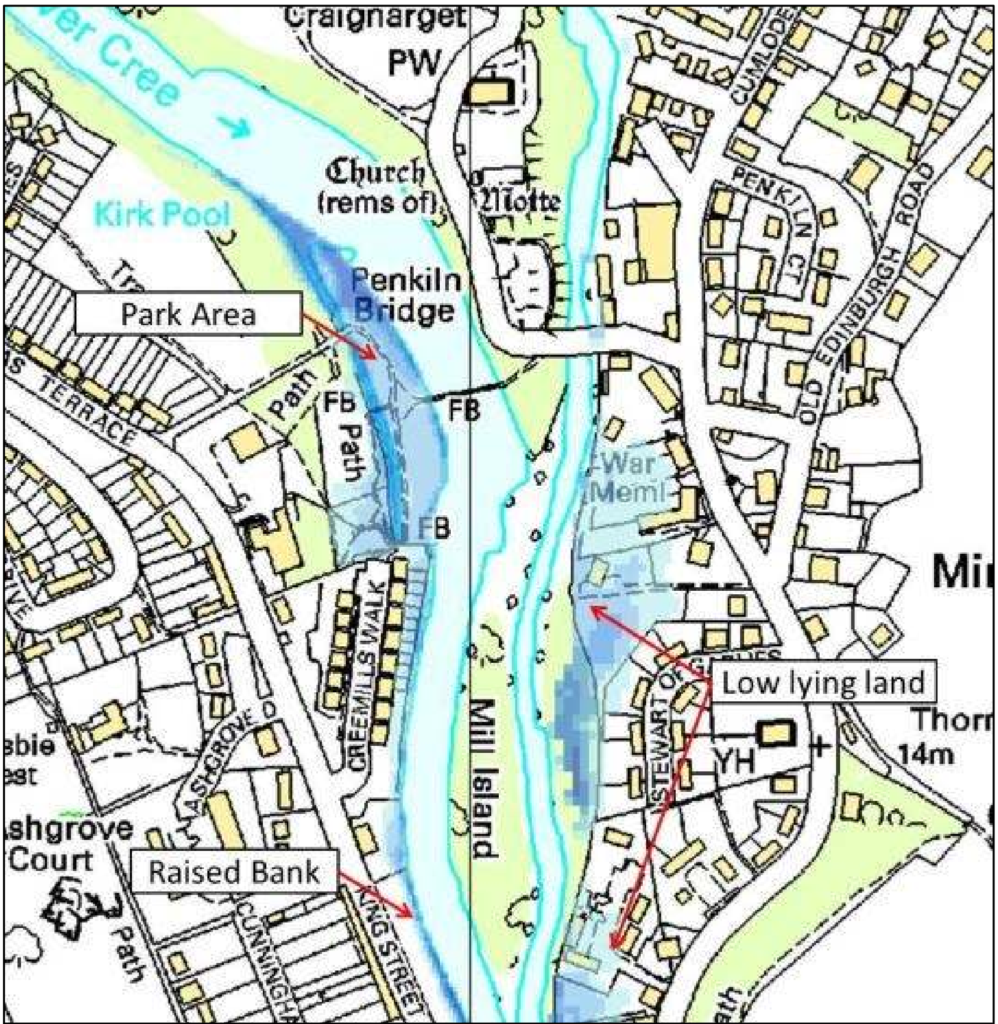
5.9.1 Flood Mechanisms

Flooding mechanisms throughout study area are described in the following sections;

5.9.1.1 Upstream End of Study Area

The upstream end of the 2D model domain is located close to the "Kirk Pool" on the River Cree and just upstream to Queen Mary Bridge on the Penkiln Burn. The modelled flood extent for the 200 year event in this area is provided in Figure 15.

Figure 15: Flooded areas at upstream extent of study area



Modelling indicates that a large area of low lying park land, at the north of Creemills Walk is predicted to be inundated by River Cree during a 1 in 200 year event. Ground levels are relatively low and flooding occurs at lower return periods in this location. Peak flood depths in this area could reach approximately 0.7 – 0.8 m during a 200 year event. It should be noted that properties on Creemills

Walk have been raised and flood waters are not expected to overtop the road and flood adjacent properties.

The Penkiln Burn main channel is located within a deep valley and significant flooding along the burn within the study area is not predicted. However, ground levels and the east banks are low immediately upstream of the Penkiln Bridge. Model predictions in this area show a small volume of water could overtop the channel and flood low lying areas of ground upstream of the bridge. The bridge is raised significantly above the water line and is not though to produce significant backwatering effects.

Ground levels on the east bank are low downstream of the bridge and close to the war memorial and a large area of low lying land is predicted to flood (see Figure 15). This area is predicted to flood for return periods as low as 1 in 10 years.

Adjacent to the confluence with the River Cree, flooding is predicted on the east bank of the burn. Low lying properties in this area have been affected by flooding in the past.

In this reach River Cree is relatively steep and fat-flowing. On the western bank of the River Cree ground levels on Arthur Street (western bank) are raised approximately 3.5 m above the channel.

5.9.1.2 Area Upstream of Bridge of Cree

Ground levels on the west bank of the River Cree, immediately downstream of the confluence with Penkiln Burn, are high and flooding is not predicted in this area. Ground levels in this area reduce in height towards the Bridge of Cree; however, a small masonry wall protects properties and low lying land. Flooding in this area is predicted to occur in excess of 5 year events. Flood waters overtopping onto Arthur Street are predicted to pond in a low area, close to the Police and Fire Station, before flowing south towards Victoria Street.

Water levels upstream of Bridge of Cree are expected to reach approximately 10.7 m AOD for the 1 in 200 year event and overtopping of the bridge is not expected to occur for any event considered, see Figure 16. The predicted 200 year water level is approximately 1 m below the soffit level of the highest middle arch (Figure 16) and 1.1 m higher than the spring level of the arches. This water level is 0.5 m higher than the predicted peak water level for the November 2012 event.

Ground levels adjacent to the eastern bank (Minnigaff) are low and a large area of land is predicted to flood. This area is well known for flooding and mainly comprises back gardens and undeveloped land.

5.9.1.3 Riverside Road

Downstream of the Bridge of Cree, flood waters are predicted to overtop both banks and inundate areas of low lying ground. Riverside Road is located on the western bank and is defended from flooding by a small masonry wall. The wall varies in height between 0.6 – 0.9 m (above existing ground level) and discontinues at the end of the road, close to the public car park. In this area the car park is defended by a small grassed embankment. Overtopping of Riverside Road is predicted to occur for at least the 5 year return period. Overtopping flood waters for larger events are predicted to combine with flood waters passing south, down Arthur Street (north of Main Cree Bridge) before inundating Victoria Street, see Figure 17.

Figure 16: Predicted 200 year water level upstream face of Cree Bridge

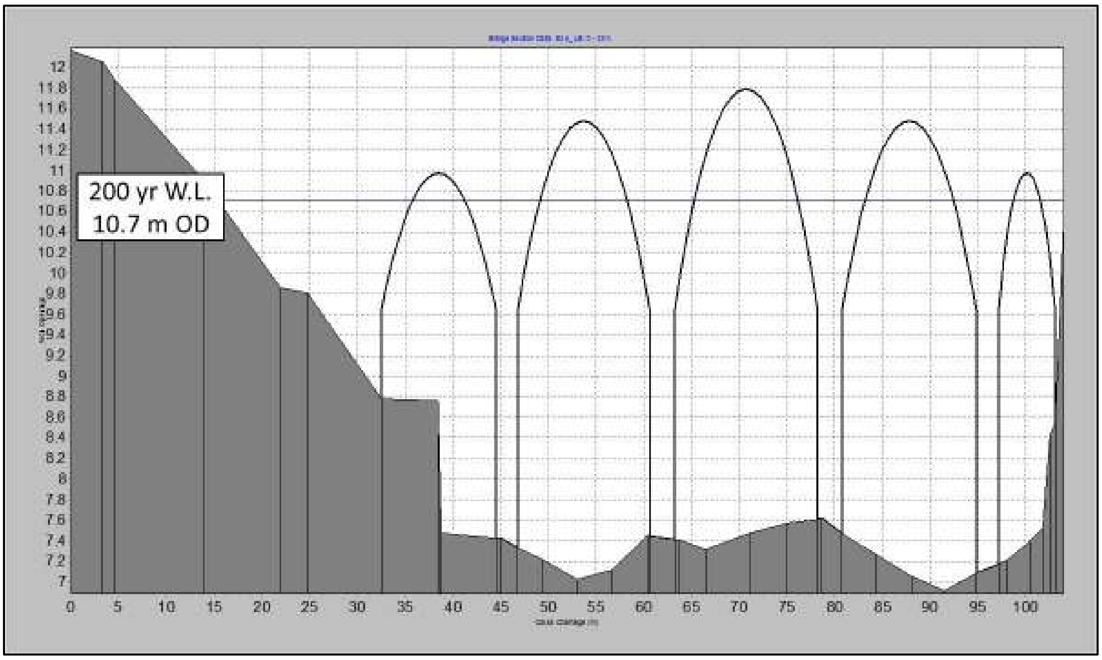
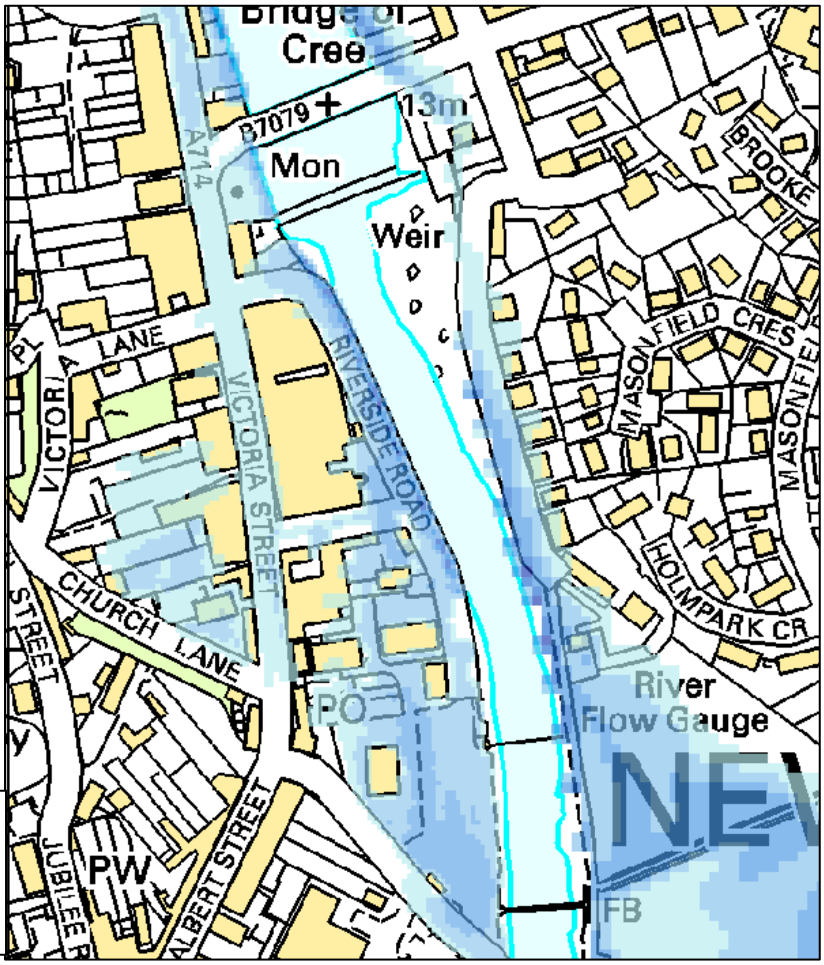


Figure 17: 200 year flooding at Riverside Road



The grassed embankment is also predicted to be overtopped with flood waters reaching around 1.1 – 1.3 m on the car park.

Flooding is also predicted to occur on the eastern bank. Flood waters in this area are predicted to overtop a small road between the river and houses to the east. Overtopping of the eastern bank also occurs upstream and downstream of the metal footbridge. Unlike the western bank, ground levels do not rise up away from the channel, instead a large area of undeveloped land is predicted to flood, with peak water depths in this area reaching approximately 0.6 – 1 m in the field close to the metal footbridge.

5.9.1.4 Downstream of Riverside Road

Ground levels on the western bank within the industrial estate remain high; however, immediately upstream of the A75 road embankment, bank overtopping levels drop which results in flooding of a large area of low lying land. Flood waters overtopping the channel are unable to flow south (overland) due to being partially blocked by the A75 embankment. The embankment has four 1.5 m diameter culverts which have been included in the model. All four culverts are predicted to operate during a 200 year flood.

Ground levels on the eastern bank immediately upstream of the A75 bridge are high and as a result, no flooding is predicted to occur. However, a significant area of land on the east bank is predicted to flood, due to flood waters overtopping the east bank further upstream (close to the metal footbridge) and flowing downstream, see Figure 18.

Flooding in these two areas is likely to be a regular occurrence with both areas flooding for return periods of around 1 in 5 years.

A long profile of the River Cree, including the predicted 200 year water level, is provided in Figure 19.

5.10 Model Sensitivity Analysis

A model sensitivity analysis provides an illustration of the effects of changing key model parameters on the important model outputs (in our case flood levels). By re-running the model, changing one input parameter at a time, the effect of that input on the model results can be isolated. Repeating this process to account for several model parameters of interest within the range of their possible input values, gives a sensitivity analysis that, when compared with the model assumptions and knowledge of realistic inputs, can provide an indication of the uncertainty associated with the model predictions.

The sensitivity analysis considers changes in Manning’s n roughness coefficient, removal of gravel berm, increasing the deck of the metal footbridge (deck raised by 0.5 m), increasing conveyance through the A75 road bridge and embankment, as well as considering a blockage to both the Bridge of Cree and the metal footbridge. The sensitivity runs undertaken are summarised in Table 20. Results from these runs were compared to the ‘base case’ 200 year flow model run (Scenario 2) and are presented in Table 21.

Figure 18: 200 year flooding upstream of A75 embankment

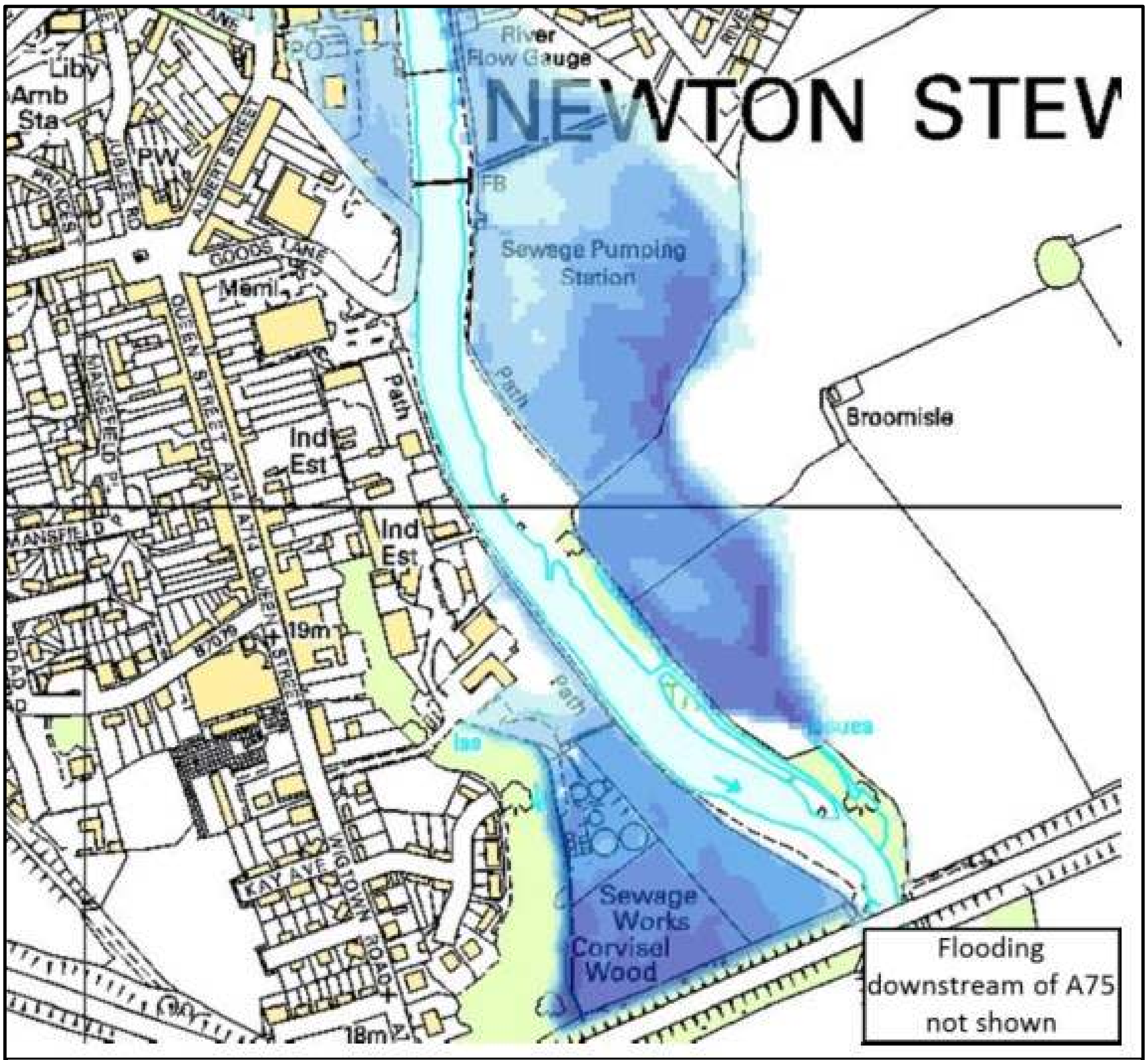


Table 20: Sensitivity analysis scenarios

Scenario no.	Change to model
S1	Manning's n increased by 20%
S2	Gravel Berm removed
S3	Increase deck height of footbridge above flood level (0.5 m)
S4	Increase A75 conveyance
S5	Pier and deck blockage (increase pier and deck widths by 1 m)

Figure 19: Longitudinal profile of River Cree (Penkiln Burn not shown) (Red triangle shows soffit level of middle arch, red dot shows crest level of masonry weir and green dot shows soffit of footbridge)

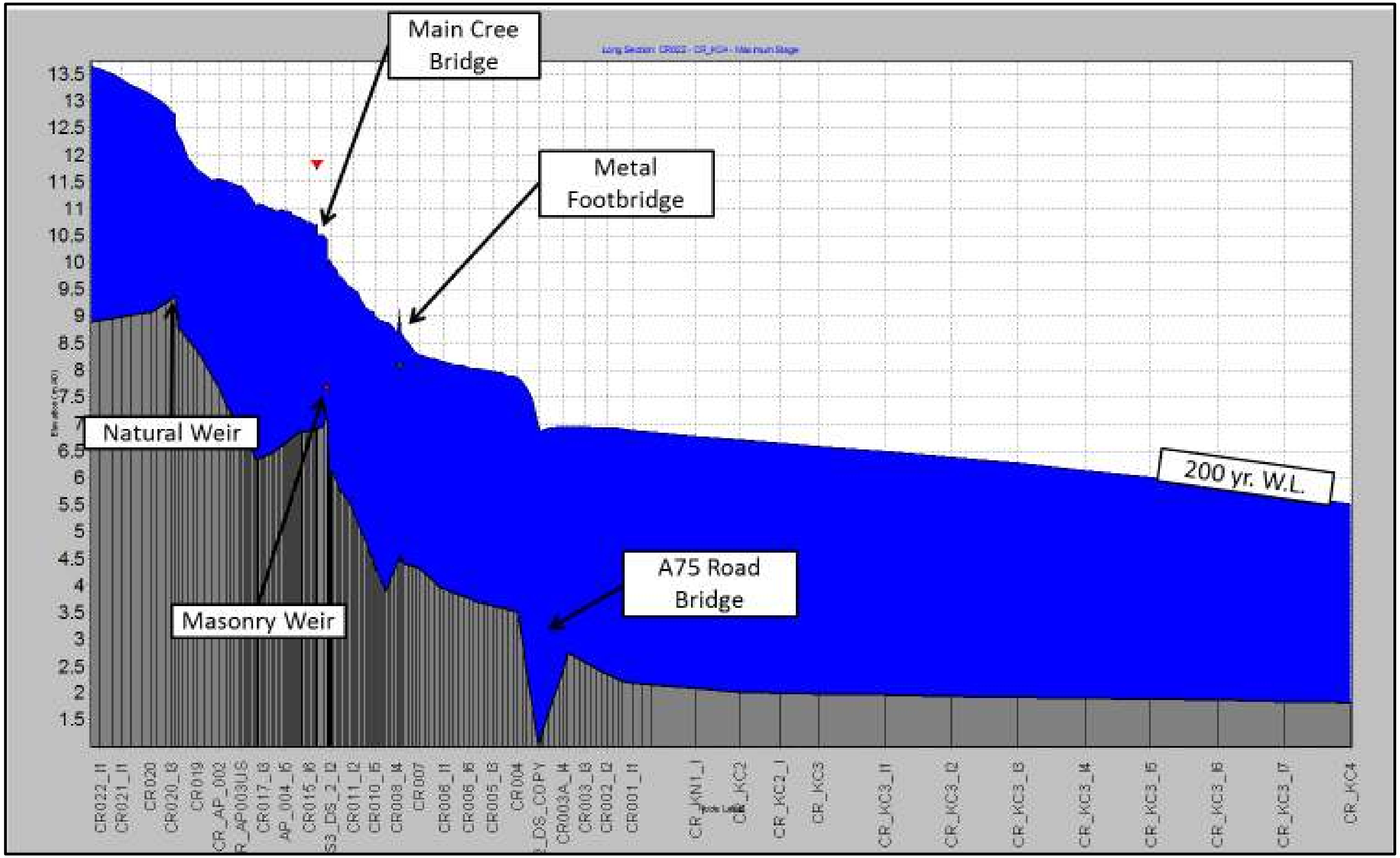


Table 21: Sensitivity results at selected cross-sections

ID	Approximate Location	Cross-section	Base 200 year level (m AOD)	Difference from Base Case (m)				
				Scenario S1	Scenario S2	Scenario S3	Scenario S4	Scenario S5
1	Stewart of Gairles	PN019	11.70	+0.29	-0.01	0.0	0.0	+0.02
2	Meal Mill	PN018_I1	11.36	+0.31	-0.02	0.0	0.0	+0.06
3	Reid Terrace	AP_004_I1	10.95	+0.27	-0.03	0.0	0.0	+0.08
4	Opposite Windsor Road	CR015	8.98	+0.23	-0.04	0.0	0.0	+0.09
5	SEPA Gauge - Carpark	CR009	7.87	+0.14	0.00	-0.18	-0.0	+0.01
6	Upstream A75	CR004	11.70	+0.23	0.00	+0.02	-0.30	+0.02

Model roughness (as Manning's roughness coefficient) was raised in the 1D model by 20%. The increase in Manning's friction value resulted in a water depth increase of up to around 0.2 - 0.3 m throughout the 1D model. This indicates that model results are somewhat sensitive to roughness.

A run was undertaken assuming that the gravel berm downstream of the weir was removed from the main channel. Results indicated that the effect on water levels is local to the berm with the water level reduction not generally exceeding 50 mm throughout the model.

The level of the deck of the metal footbridge was increased above the flood level. Results indicated that the effect on water levels is local to cross-sections close to the bridge, which show a slight decrease in water levels (0.45m immediately upstream of the bridge, reducing to 0.18m at SEPA gauge).

The conveyance through the A75 road bridge was also increased, and results showed water levels decreased by 0.3m immediately upstream of the bridge, reducing to nil at metal footbridge (see Table 21).

Bridge of Cree and the metal footbridge were tested for their sensitivity to blockage. Both bridges were blocked by reducing the flow area within the bridge openings within the ISIS 1D software. The increase in pier widths equates to approximately 5% decrease in flow area compared to the conservative base case. Model results indicated that water levels increased by as much as 300 mm upstream of the Bridge of Cree and up to 0.1 m adjacent to the metal footbridge; water level increases did not exceed 0.1 m elsewhere in the model.

The sensitivity results indicate that uncertainties associated with the model set-up have a relatively small impact on peak flood levels throughout the study area. Although, increasing roughness within the model resulted in the maximum increase in water levels, the roughness used in the model is the best estimate and has been shown to calibrate to known flood events.

Given the relatively steep gradient of the channel throughout the study area and the capacity in the main channel, these potential uncertainties in flood level have a relatively small impact on the 200 year flood extent.

6 Flood Mitigation Options

Flooding of properties in Newton Stewart and Minnigaff occur due to flood waters overtopping the banks of the River Cree and Penkiln Burn. It was also shown that high tides and storm surges in the sea have little effect on flooding risk within Newton Stewart.

An initial assessment has been carried out to identify possible flood mitigation options. In developing these options consideration has been given to discussions with Cree Valley Flood Action Group (CVFAG) and Dumfries and Galloway Council officials. Based on these, a total of five flood mitigation options have been identified and assessed.

The potential flood mitigation options identified are:

1) Upstream storage;
2) Direct defences;
3) Removal of gravel berm;
4) Increased conveyance through the A75 Bridge;
5) Raised metal footbridge; and

An assessment of each of these options has been undertaken and their effect on flows and water levels through Newton Stewart has been quantified using the calibrated mathematical model of the river and its floodplains.

6.1 Level of Protection

In Scotland, the standard level of protection against flooding is 1 in 200 year (i.e., a flood which has an annual probability of exceedance of 0.5%). This is the level of protection for most type of development including residential and commercial/industrial, except for sensitive infrastructure for which a higher level of protection is required (i.e., 1 in 1000 year).

Although the 1 in 200 year flood would be the ideal level of protection for residential and commercial areas, sometimes this may not be cost effective or indeed acceptable to local residents. For example, if the most effective option of flood mitigation is direct defences and the required defence heights are such that it would cut-off views of the river from the surrounding areas, a lower level of defence providing a lower level of protection may be more acceptable. Hence, in this assessment, both the 200 year level of protection and lower levels of protection have been considered.

Model runs carried for this assessment indicate that threshold level of flooding (i.e., return period of a flood at which flooding of properties would commence) is approximately 1 in 2 years. It was predicted that during a 2 year flood only 3 properties were predicted to flood. This does not mean that these three properties would flood every other year. Taken over a long period of time, say 50 years, these properties would be expected to flood of the order of 25 times.

The main flood defence within the town of Newton Stewart is comprised of low masonry walls upstream and downstream of the Bridge of Cree and a small earth embankment adjacent to the metal footbridge. These defences provide a standard level of protection that has been estimated to be approximately 1 in 10 years.

It should be noted that the above threshold return periods refer to flooding of properties. Flooding of land occurs more frequently.

6.2 Freeboard Allowance

For the flood defences considered, a standard freeboard allowance of 0.3 m has been considered. This is normally what is applied for hard defences (i.e., walls). For soft defences (i.e., earth embankments) a higher freeboard allowance of the order of 0.6 m is usually considered. In modelling overtopping of defences providing protection for less than 200 year, these freeboard allowances have been added to the defence heights.

6.3 Option 1 – Upstream Flood Storage

A well-recognised method of sustainable flood management (and Natural Flood Management) is to attenuate flood flows in the upper catchment to reduce peak flows arriving in urban areas. There are numerous social and environmental benefits of providing upstream storage including additional benefits if constructed in-line with other flood defences. If sufficient attenuation of peak flows could be provided in the upper catchment, no or limited flood defences may be required in the urban areas to provide an acceptable level of protection.

The aim of the assessment was to identify any areas in the upper catchment where the natural flood storage in the catchment could be augmented by removal of existing agricultural development or through the engineering of new storage areas. This form of flood mitigation is in line with the requirements of the Flood Risk Management (Scotland) Act (2009) which sets out that Natural Flood Management should be considered as part of any local flood risk management plan.

6.3.1 Potential Areas for Upstream Flood Storage

A desktop investigation was undertaken to identify areas which could be suitable for use for additional flood storage during extreme events. The review considered local features such as roads and the location of properties, as well as topographical information from LiDAR data.

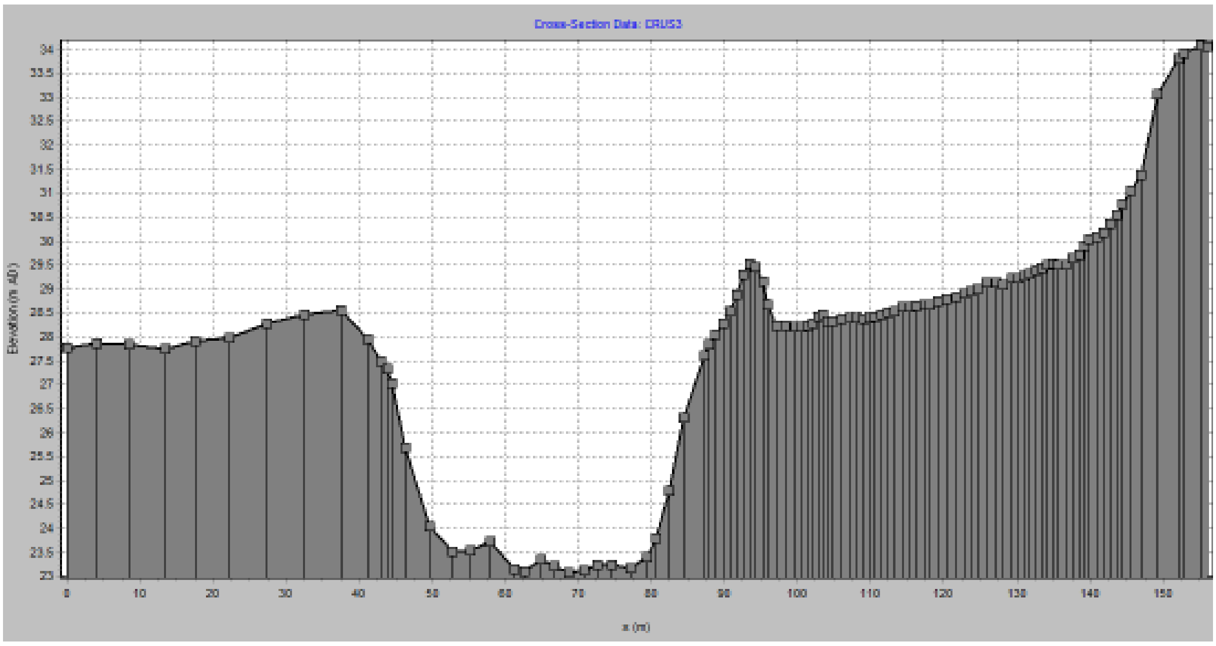
Loch Trool is one of the largest water bodies within the catchment and therefore an obvious location for the utilisation of additional flood storage. However, it was found that the catchment draining to the loch is not large enough to provide a significant attenuating effect on peak flows downstream. In addition, the current spillway appeared relatively small indicating that flows passing downstream are already limited by the spillway and potential for additional storage within the reservoir is likely to be low.

No significant natural floodplains, where additional flood attenuation could be provided, were identified within the headwaters upstream of Water of Minnoch. The catchment in this area is relatively steep and in such catchments it is not practical to provide significant flow attenuation without constructing large man-made structures, such as dams. Without a large structure any flood attenuation scheme constructed in such a steep catchment would be limited to retaining a small volume of water only. The assessment indicated that significant headwater storage solutions on the tributaries of the River Cree could not be achieved without the construction of significant water retaining embankments.

Further downstream from the headwaters, within the main River Cree valley (downstream of confluence with Minnoch Water), the average bed slope of the channel reduces and there are large natural floodplains. This is evidenced by three small ponded areas which are thought to be directly linked to the river. This area was further investigated for the possibility of increasing the existing flood storage potential within the natural floodplain. The area investigated for additional flood storage extends from Penninghame House (238340, 570240) to Cordorcan (236870, 572540) over a length of the river of some 2.8 km and is illustrated in Figure 21.

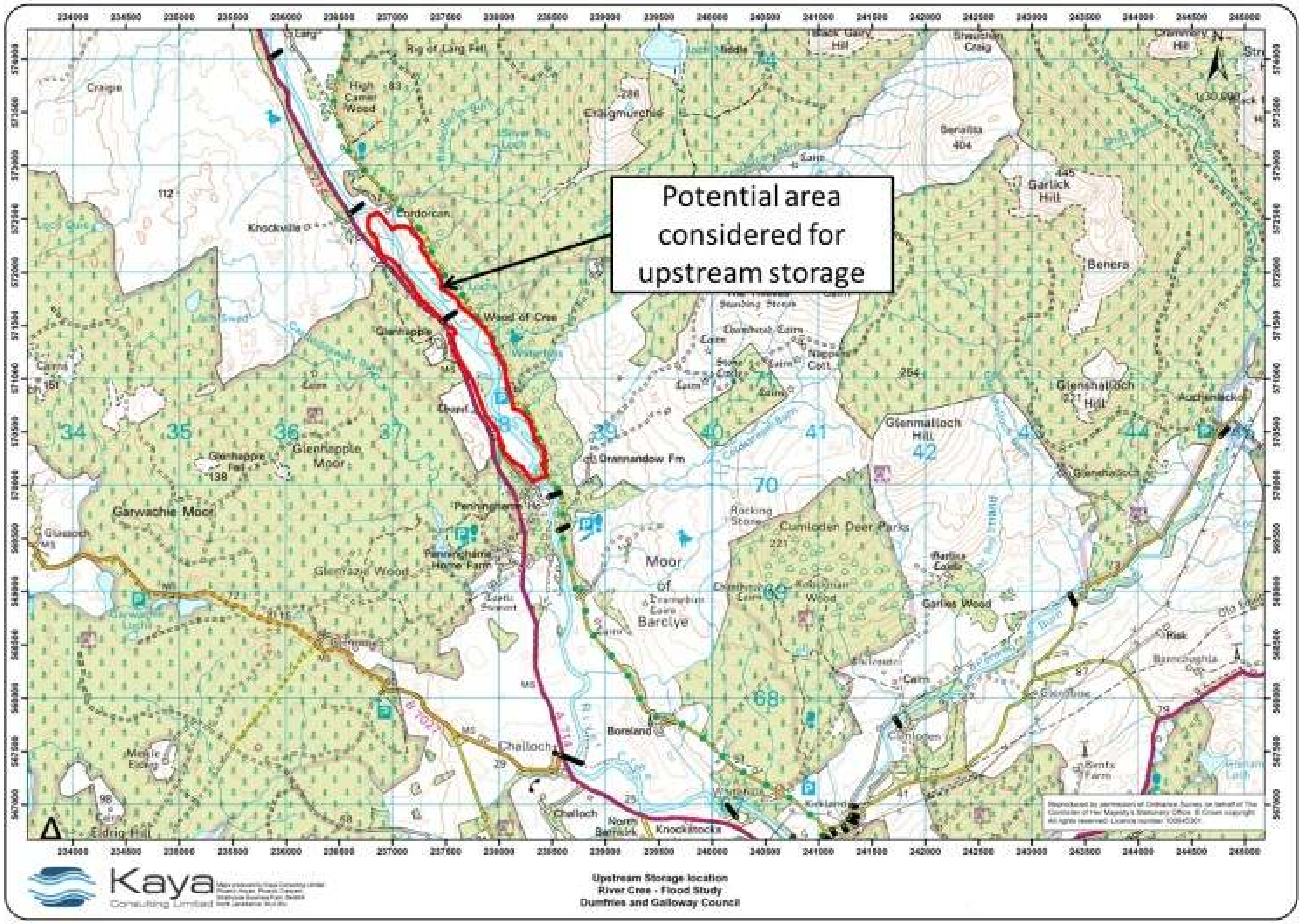
The large floodplain in this area measures approximately 0.7-0.8 km^2 in plan, and around 77% of the Cree catchment drains through this area. LiDAR topographical information suggests that the adjacent floodplain areas are around 200 – 300 m wide (including the channel). Elevation data and site observations indicate that small agricultural embankments currently exist along stretches of the eastern banks. The embankments are small measuring around 1 – 2 m in height; see Figure 20 which shows a cross-section taken close to Glenhapple (approximately half way along the floodplain area).

Figure 20: Cross-section topographical survey taken close to Glenhapple



Ground levels within the floodplain area gently rise away from the channel before reaching two roads which run parallel to the floodplain area on either side; i.e., the A714 to the west, and a local access track to the east. Lowest road levels in this area were found to be approximately 31 m AOD based on LiDAR data. This presence of existing infrastructure appears to be the main limiting factor with respect to raising water levels in the area.

Figure 21: Potential area considered for upstream storage



At the downstream (south) end of the area the river channel bifurcates around a large island feature located within the channel. In this area the width of the river valley reduces significantly to around 100m, measured following the 30 m AOD contour, see Figure 22.

It should be noted that this area already provides some attenuation to peak flows passing downstream. The assessment carried out here is to determine if the storage capacity within such a large natural floodplain area could be increased to provide a significant reduction in peak flows passing downstream.

There are no known hydraulic structures located within the proposed storage area.

The current ownership of the floodplain area is not known; however, it is assumed that the land is privately owned. Access conditions have not been assessed, although the possible location of a water retaining structure (i.e., dam), is close to the roads on both banks of the river.

Figure 22: Detailed map of potential storage location

6.3.2 Methodology and Assumptions

Upstream flood storage, by definition, refers to holding excess flows temporarily in the upper catchment thereby reducing flows passing downstream. Stored flood waters are then released at a reduced flow rate once flood waters subside.

In order to store more water in the area than at present, a dam structure would need to be constructed across the full width of the channel at the downstream end of the area considered for storage. Such structures are usually built using suitable earth (ideally locally sourced), but can also be made of concrete. A water retaining structure will generally raise the normal water level on the upstream side of the structure. However, alternative designs may also be possible to limit the storage volume upstream of the structure during normal flows. A dam structure will have a spillweir over which normal and flood flows pass downstream. During normal flows the water level is just above spillweir level. As flood flows increase, the water level on the spillweir rises, causing water to pond within the storage area.

In addition to a spillweir, it may also be possible to have other outfall from the storage area in the form of pipes/culverts/orifices, Hydro-Brakes©, etc. If such outfall devices were set at a lower level than the spillweir, the normal water level in the storage area could be lowered below the spillweir level. However, such designs will be more expensive to build and maintain than a simple dam and spillweir arrangement. There will also be an increased risk of such outfall devices becoming blocked during floods.

Other than outflow arrangements from a large storage basin, there are a number of other factors which will need to be assessed in detail and taken into account. These include geotechnical investigations to determine if the soil structure in the area is suitable for water storage and a water retaining structure, environmental studies to assess the potential impact of such a scheme, public perception (associated with large volumes of water held in the upper catchment above natural ground level), dambreak analysis to determine the likely impact if dam failure occurs, ongoing maintenance issues, etc. In this assessment none of these issues have been considered. The assessment solely focuses on hydraulic effects (i.e., how much additional flood storage could be achieved in this area and what reduction in flood flows could be achieved at Newton Stewart).

If this initial high level assessment shows that a significant reduction in flooding risk at Newton Stewart could be achieved at an acceptable cost, then more detailed studies and investigations will need to be carried out to develop the scheme further.

6.3.2.1 Hydraulic Modelling

An ISIS 1D hydraulic model was constructed to assess the attenuating effect of the potential storage area on peak flows at Newton Stewart. The River Cree main channel was represented in the model by cross-sections and floodplain storage has been accounted for using an online flood “storage area” unit. This unit is commonly used in ISIS models to represent storage within river reaches.

As part of the topographical survey work undertaken for this study, 6 channel cross sections were surveyed of the River Cree close to the potential storage area. These cross sections have been used in the model. Available storage volumes within the storage area were obtained from the LiDAR elevation data, with the surface area of the storage basin calculated for a range of elevation slices.

The lowest bed elevation at the downstream end of the storage area is approximately 25 m AOD. Flood waters could reach approximately 31 m AOD before overtopping the main A714 (running along the western boundary). This corresponds to a maximum volume of approximately 2 million cubic meters presently available within the storage area without flooding the A714.

Outflow from the storage basin was represented as a culvert. Please note that culvert is used in the model as a means of limiting downstream flows. If upstream storage option is taken forward a different structure which provides a better flow control may be required. In addition, for the largest culvert dimensions, the predicted flow velocity through the culvert is in excess of 10 m/s. This is unrealistic and impractical, but the culvert acts like an orifice. As indicated before, as the main aim of this exercise is to calculate what the peak water level would be in the basin if flows passing downstream were restricted to a given flow rate, such an approach was considered reasonable at this stage of the study. The type of flow control structure suitable for such requirements has not been investigated in any detail.

At this stage only the 200 year flood has been modelled.

A number of different culvert sizes were modelled and this resulted in a range of downstream flows and corresponding water level in the storage basin. No overtopping of dam structure (culvert) has been allowed. It should be noted that the floodplain storage area is considered to be dry at the beginning of the flood event .The results are shown in Table 22.

Table 22: Hydraulic results for upstream storage area options

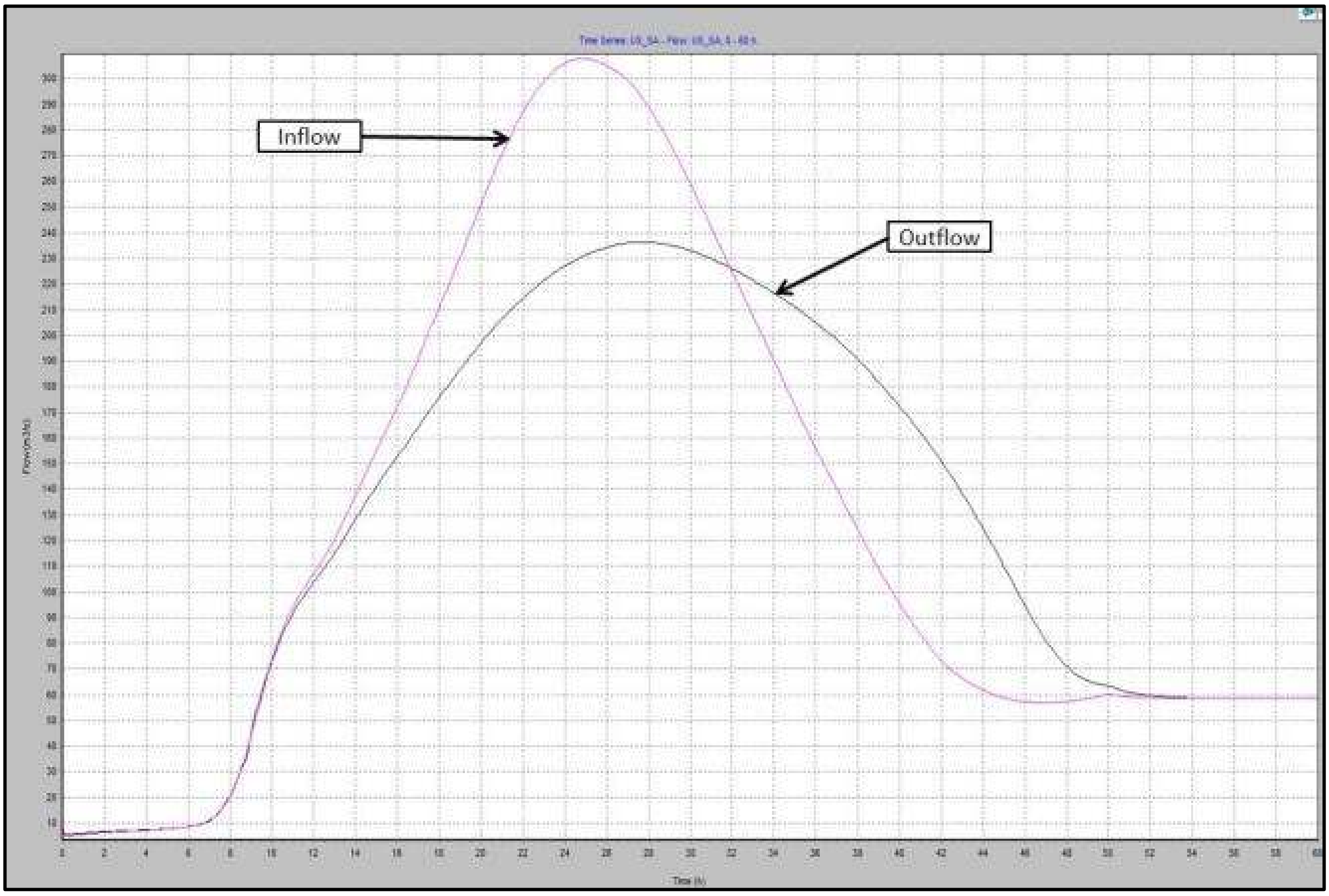
Outflow Arrangement	Basin water level (m AOD)	Peak flow downstream storage area (m^3/s)	Approximate Equivalent Return Period (Years) in Newton Stewart
(8m x 2m) rectangular culvert	31.0	414	70 - 75
(8m x 2.3m) rectangular culvert	30.5	437	95 - 100
(8m x 2.65m) rectangular culvert	30.0	456	130 - 140
(9m x 3m) rectangular culvert	29.5	472	170 - 180
(11m x 3.5m) rectangular culvert	29.0	478	185 - 190
No Culvert	N/A	485	200

The results indicate that if flows passing downstream were reduced by approximately 70 m^3/s (to approximately 414 m^3/s), this would increase peak water level in the basin to 31 m AOD level. The 31 m AOD is the threshold level above which the A714 would flood.

The predicted flow hydrographs upstream and downstream of the storage basin are shown in Figure 23. The reduced flow corresponds to a return period of the order of 1 in 70-75 years in Newton Stewart. If peak flow passing downstream were to be reduced below 414 m^3/s, the corresponding water level in the basin would be higher than 31 m AOD and flood the A714 road.

As indicated above, a culvert unit was used in the model to control flows passing downstream. Model runs indicated that for the most extreme case where downstream flows were reduced by 70 m^3/s, the culvert required would be approximately 8 m wide and 2 m high. The predicted flow velocity through such a culvert was in excess of 20 m/s which would be unrealistic and impractical. Model results indicate that this size of culvert would only be able to pass a flow of approximately 62 m^3/s before flood waters start to rise within the basin. This would have an impact on the local environment as some areas would flood more frequently than at present. Hence, different type of flow control structure would likely be required. If a Hydro-Brakes© were used to control downstream flows, a large number of them would likely be required with a significant associated cost. The largest Hydro-Brake© has recently been installed on the White Cart Water in Glasgow and this only pass a flow of 11 m^3/s. In order to pass, say 414 m^3/s, 38 of such structures would be required. This would not be a practical or cost effective solution.

Figure 23: Inflow/Outflow hydrograph comparison



If a fixed spillweir were used, say 20 m wide, the depth of water above spillweir crest required to pass 414 m^3/s would be approximately 5.7 m. This would need to be set at an elevation of approximately 25.3 m AOD so that flood waters do not reach to 31 m AOD level within the basin. The bed level of the river at the potential location of the dam is approximately 25 m AOD. No freeboard is included in the above levels.

A fixed spillweir structure would not necessarily provide a similar level of reduction for lower flow events. If the weir were designed to provide optimum discharge and basin water level, say for a 200 year flood, as flood flows reduce its effectiveness will also reduce. Therefore a two-stage weir may be more effective for a range flow rates. This indicates that if the upstream storage option is taken forward, the fixed weir option as well as alternative options will need to be assessed in more detail. Additional flood storage (in excess of 31 m AOD) has not been considered.

The results of this assessment indicate that there is the potential to increase upstream flood storage to improve attenuation of large flood events.   Additional modelling work would be required to determine the optimum design. However, it is suggested that before such work is undertaken consideration is given to some of the other critical issues such as land ownership, ground conditions to see if upstream storage is a feasible option to take forward and likely costing.

Model results presented in Section 5 indicate that flooding occurs for flows with a one in 2 year return period and higher. This indicates that, even utilising the upstream storage potential to its maximum, resultant downstream flows (with a return period of the order of 75 years) would still cause widespread flooding in Newton Stewart. Upstream storage option has to be combined with other flood mitigation options to provide the required level of protection (i.e., 200 years). However, such a storage scheme would provide a significant reduction in peak flows in Newton Stewart.

It should be noted that the outflow control structure from the storage basin could also be designed to provide optimum effect for smaller and more frequent flood events, (i.e., say 10 to 50 years), and less or no effect for higher flows. In this way, the start of flooding could be delayed, with little or no benefit gained for higher flows. This could be considered further if the scheme is considered economically feasible and taken forward (see Section 8).

6.4 Option 2 – Direct Defences

The conventional way of flood mitigation is by way of direct defences involving the construction of flood walls and earth embankments to protect flood risk areas. In some cases this may be the most effective way of reducing flood risk to an acceptable level. However, containment of the entire flood flow within the main channel would increase flood levels both upstream and downstream of the area protected. Any increase in water level due to direct defences will need to be included in the design.

6.4.1 Methodology

The baseline model representing the existing river channel and floodplains was modified to include defences along the banks to stop flooding of properties. Indicative lines of defences along both banks of the river are shown in Figure 24.

The modelling has been undertaken assuming that the entire western bank of Penkiln Burn and both banks of River Cree, within the study area, will be defended. It should be noted that this may not be possible or desirable as some areas (e.g., low lying areas of back gardens) may be left unprotected assuming flood waters cannot reach properties. Such issues would need to be looked at during detailed design.

Areas on both banks upstream of the A75 road bridge have not been defended and these areas have been allowed to flood as at present.

6.4.1.1 Hydraulic Modelling

The model was modified to prevent flood waters spilling outside the defence lines shown in Figure 24. The model was then run for the estimated 200 year flow and water levels along the river at model cross sections were compared with those obtained from the model run for the base case (i.e. no defences). The results are summarised in Table 23 alongside Riverside Road and Table 24 for other areas. Locations of cross sections along Riverside Road are shown in Figure 25.

Table 23: Model results for defended and undefended cases at Riverside Road

Model Section	Road Level (m AOD)	Top of Wall (m AOD)	200yr Existing Water Level (m AOD)		Existing Defence Height (m)	Required Defence Height above road (m)[a]
			Existing	Defended		
CR012	9.0	9.7	10.0	10.0	0.7	1.3
AP0_008	8.6	9.5	9.8	9.9	0.9	1.6
CR011	8.5	9.4	9.6	9.7	0.9	1.5
CR011_I2	8.1	8.6	9.5	9.6	0.5	1.8
CR010	8.2	8.9	9.1	9.2	0.7	1.3
CR009	8.3	8.4	9.0	9.0	0.1	1.0
CR008	8.3	8.4	8.9	8.8	0.1	0.8

[a-] includes 300mm freeboard

The differences in peak water levels along Riverside Road between the existing and defended cases are of the order of 0.1 m. However, differences of up to 0.2 m were predicted in other parts of the town. These indicate that the effect of floodplain storage on peak water levels in the river through the town is limited.

Figure 24: Location of proposed direct defences (indicative)

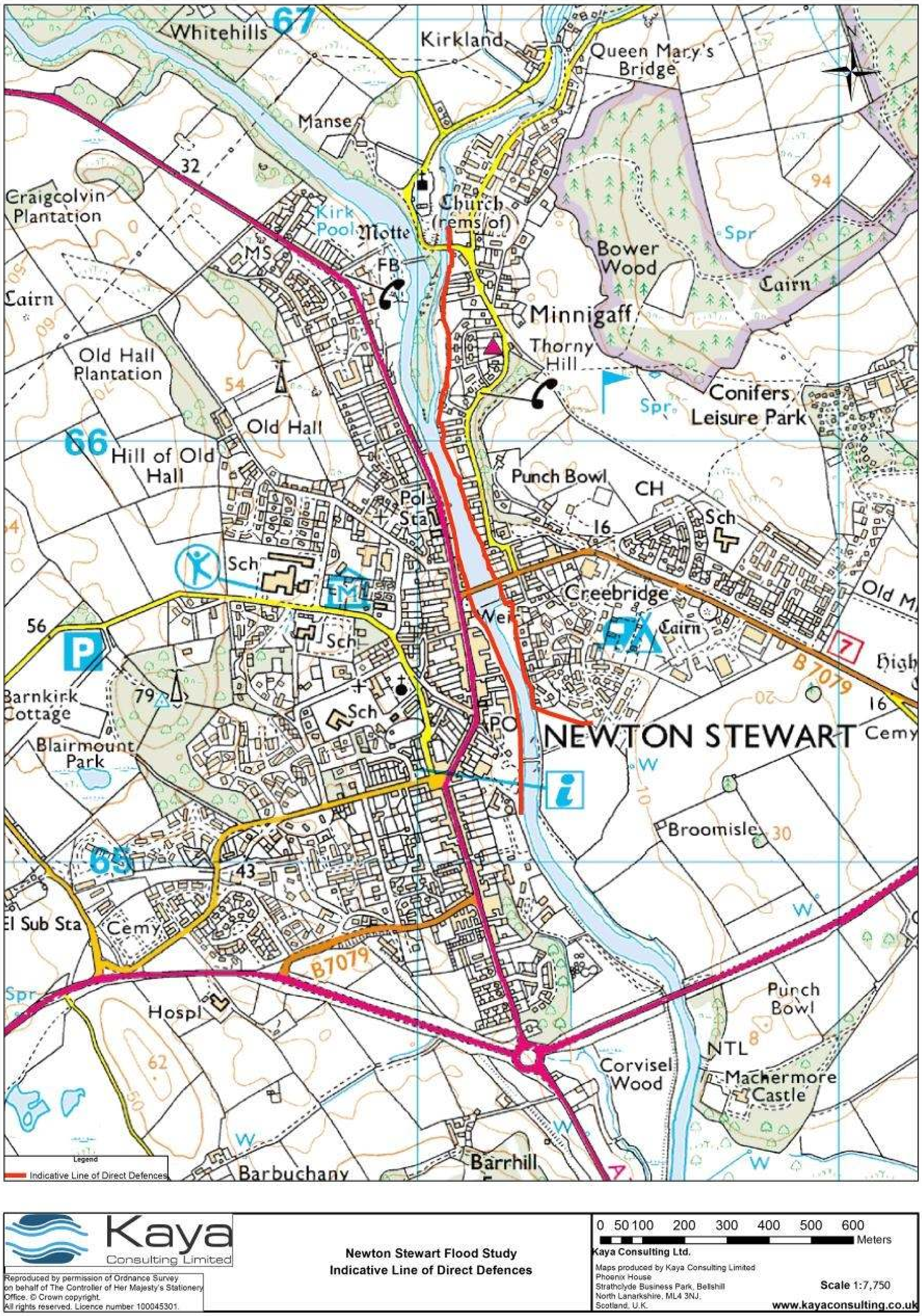
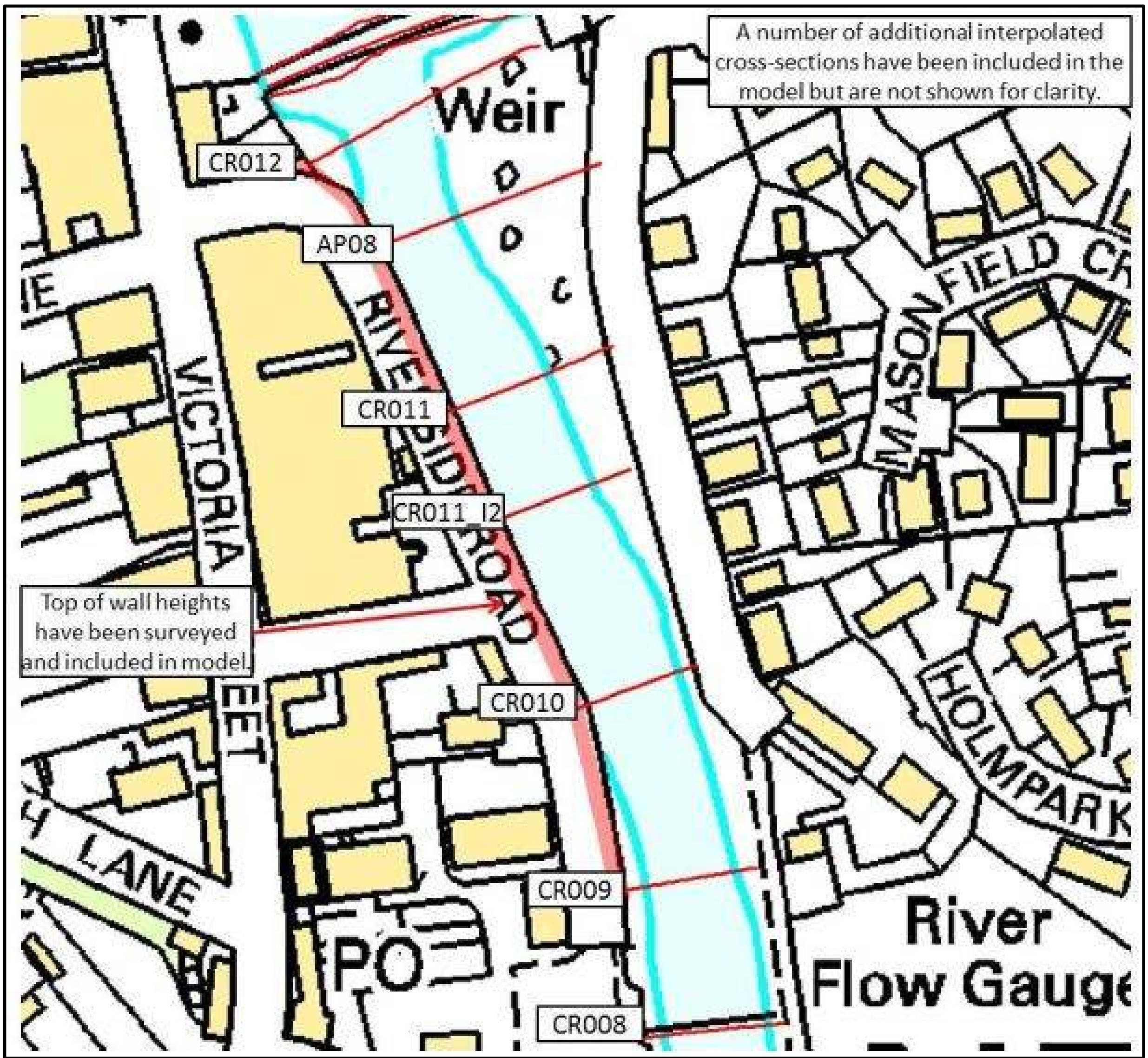


Figure 25: Location of cross-sections along Riverside Road (Refer to Table 23)



It is shown above that defence heights (above existing road level) of up to 1.8 m would be required to protect Riverside Road from being flooded during a 200 year flood. This includes a freeboard allowance of 0.3 m.

Along the other reaches of the river through the town, defence heights up to 1.8 m were predicted to be required, except at Reid Terrace where a defence height of the order of 2.3 m was predicted. However, this is taken from the bottom of the garden. Similarly, the required defence height at the Fire Station is measured from above the river bank and not the main road, which is higher.

The height of defences required in other places are summarised in Table 24. The locations of defences listed in Table 25 are shown in Figure 26.

Table 24: Required defence heights at selected locations (outside Riverside Road) - 200 year

ID	Approximate Location	Cross-section	Ground Level (m AOD)	Defended Water Level (m AOD)	[a]Required Defence Height at Bank (m)
1	Stewart of Gairles	PN019	11.5	11.8	0.6
2	Meal Mill	PN018_I1	10.3	11.5	1.5
3	Reid Terrace	AP_004_I1	9.5	11.0	1.8
4	Fire Station	AP_004_I2	9.5	11.0	1.8
5	Opposite Windsor Road	CR015	9.6	10.9	1.6
6	SEPA Gauge - Carpark	CR009	8.3	9.0	1.0

[a-]includes 300mm freeboard

An assessment was also carried out for defences to provide 1 in 50 and 1 in 10 year protection. The required average defence heights along Riverside Road are shown in Table 25. It is shown that approximately 1 m high defences would provide 1 in 10 year protection.

Table 25: Estimated average defence heights for 200, 50 and 10 year protection

Location	Existing Average Ground Level (m AOD)	[a]Required Defence Height at Bank (m)		
		200 year	50 year	10 year
Arthur Street (North)	10.4-10.0	1.1	1.0	0.8
Arthur Street (South)	10-9.5	1.4	1.2	1.0
Riverside Road (North)	9.0-8.5	1.6	1.4	1.2
Riverside Road (South)	8.5-8.3	1.2	1.0	0.9
Car Park	8.3-8.0	1.2	1.0	0.9
Stewart of Gairles	11.5-11.0	0.5	0.4	0.2
Millcroft Road	9.8-9.5	1.6	1.5	1.2
Holmpark (North)	9.7-9.0	1.0	0.8	0.6
Holmpark (South)	9.0-7.5	0.2	0.1	n/a

[a-]includes 300mm freeboard

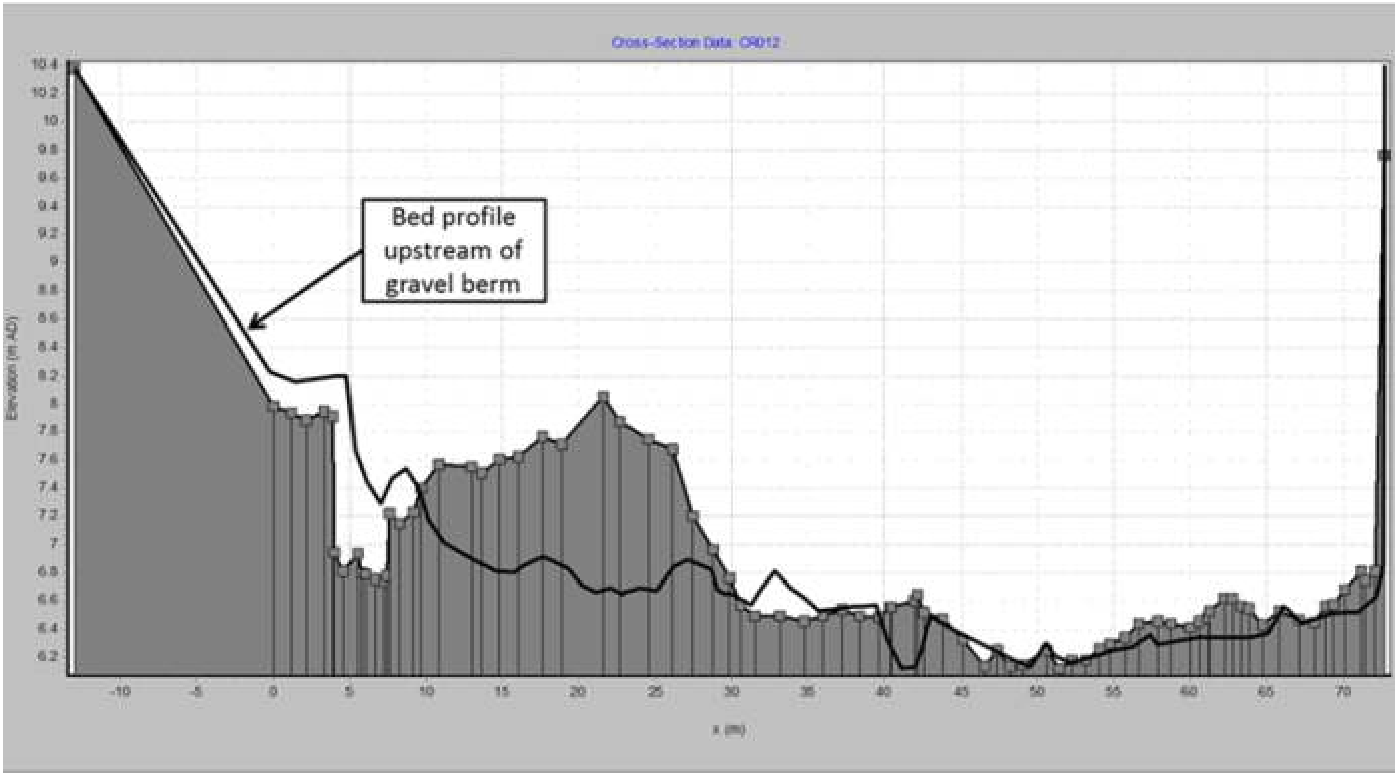
Figure 26: Location of defences referred to in Table 24

6.5 Option 3 – Removal of Gravel Berm

There are gravel berms flanking the eastern and western sides of the main channel, just downstream of the weir in the centre of the town, Photo 7 in Appendix A. The largest berm is to the eastern side of the channel, but there is a smaller feature attached to the western bank. Upstream of the weir the channel is more than 60 m wide, while downstream of the weir the channel becomes much narrower due to the berms, see Figure 27. The berms are vegetated gravel bars that, although inundated during high flows, may restrict the flow capacity of the channel downstream of the weir.

The existing baseline model has been amended to remove the berms from the cross section representing the river in this area. The model was then run and results were compared to the base case model run. However, prior to this a review of historical maps was undertaken to assess the longevity of the berms.

Figure 27: Channel cross-sections through berm and upstream



6.5.1 Review of Historical Maps

A review of available historical maps of Newton Stewart was undertaken accessing the online National Library of Scotland map database (http://maps.nls.uk/geo/find/). Maps dating back to 1846 were reviewed and images of the historical maps are provided in Appendix D. A discussion of the morphological changes in the channel downstream of the weir is provided below.

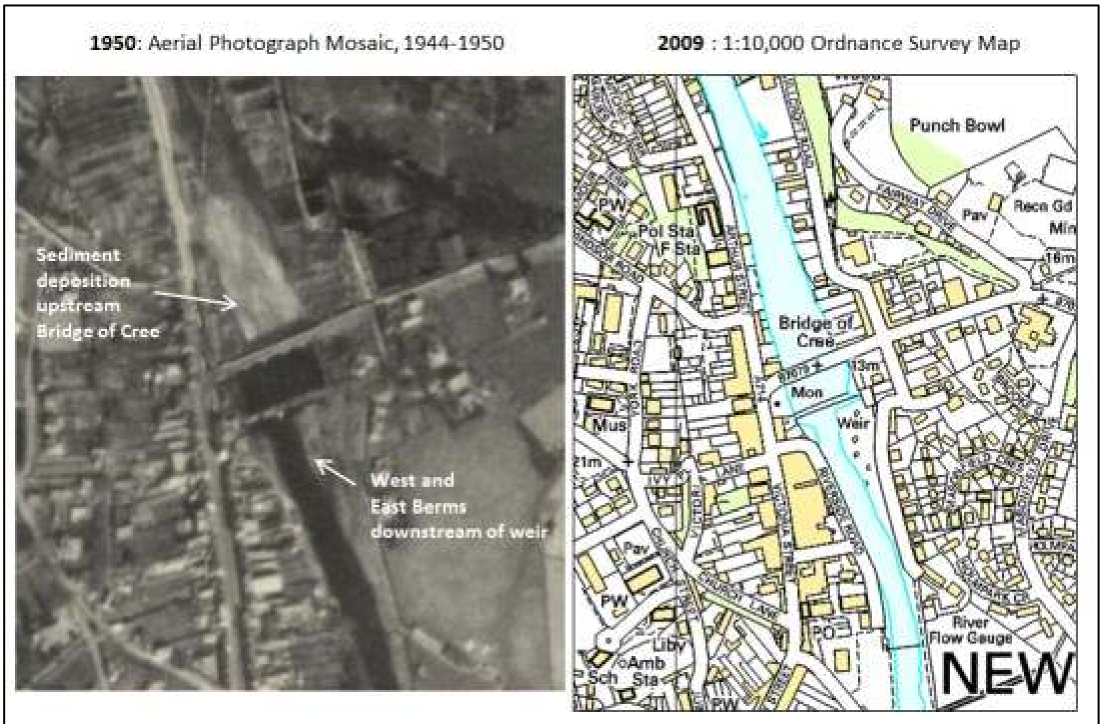
The 1846 mapping shows that the berm on the eastern side of the channel was present at that time, with a distinct open channel to the west of the berm. The open channel between the berm and the west bank appears much wider in 1846 than what is present today. The berm on the western side of the channel is not shown clearly in the 1846 mapping.

Ordnance Survey mapping in 1894 indicates that the channel geomorphology has not significantly changed over the previous 48 years. The gravel berm is vegetated and again the channel between the berm and the east bank is thought to be significantly larger than at present. The small berm on the western side of the channel has still not formed.

Mapping in 1907 and 1938 shows little change in morphology (see Appendix D) with the size of the berm on the eastern side of the channel remaining approximately the same and still no feature on the west bank is shown on the maps.

Aerial photographs taken in 1950 (Figure 28) show a more accurate visualisation of sedimentation patterns in the River Cree, in the vicinity of the Bridge of Cree and weir. The aerial photo is likely to have been taken during the summer where flows within the channel are at their lowest. The photo shows a large volume of sediment has been deposited upstream of the Bridge of Cree. Downstream of the weir, the gravel berm is a feature on the western side of the channel, although it is not as large as the berm on the eastern side. In addition, during these low flow conditions, the channel between the main eastern berm and the east bank is much smaller.

Figure 28: Aerial Photograph (1950) and 1:10,000 Ordnance Survey Map (2009) showing sedimentation patterns at Bridge of Cree and weir



Later mapping in 1953 shows only the eastern berm, although it is likely the western berm is still present as a feature that is only visible during low flows. Again the 1953 mapping shows a larger channel between the eastern berm and the east bank than is present today (see Appendix D).

The latest 1:10,000 Ordnance Survey maps (approx. 2009) show two areas of sediment deposition attached to both the eastern and western banks; again the eastern berm is the larger (see Figure 28). The pattern of sedimentation downstream of the weir shown on the current 1:10,000 mapping is very similar in plan form to that shown in the aerial photographs of 1950, indicating little has changed in plan form over the last 59 years. However, it is possible that vertical changes in the features may have occurred over this time.

In summary, the review has shown that the eastern berm downstream of the weir has been present in approximately the same position since 1846. It appears that since then there has been additional sedimentation, as the open channel between the berm and the east bank is significantly smaller now than shown in early mapping. In addition, sediment has accumulated on the western bank and this has been present since at least 1950.

It is likely that the sediment features on the east and west banks downstream of the weir are the result of the weir shape focussing the main flow and higher velocities in the centre of the channel, with lower velocities at either side, resulting in sediment deposition over time. Depending on the dynamics and availability of sediment supply from upstream sources, it is likely that if the berms were to be removed as part of a flood management strategy, they would gradually reform over time if the weir remains in place and there is abundant sediment supply from upstream.

6.5.2 Hydraulic Modelling

In order to simulate the effect of removal of the gravel berm on peak water levels in the river, three cross-sections (Sections 12, AP8 and 11), which are located through the gravel berm, were modified as shown in Figure 29 with the level of the berm lowered to approximately adjacent river bed levels.

The model was then re-run with berm removal and compared to base case results with the berm in place. The results are shown in Figure 30.

Model results for the 200 year flow indicate that by removing the gravel berm immediately downstream of the masonry weir, flood waters at cross-section 12 (at the upstream edge of the berm) are reduced by approximately 0.1 m, indicating that removing the gravel bed would not have a significant effect on peak water levels in the river. At high flows the berms do not appear to affect significantly the conveyance within the main river channel.

If the berms are to be removed, it would be important to assess the rate at which the berms would be expected to re-form and to assess the impact of removal on channel geomorphology, bank stability and the weir.

Figure 29: Cross section 12 showing area of gravel berm removed

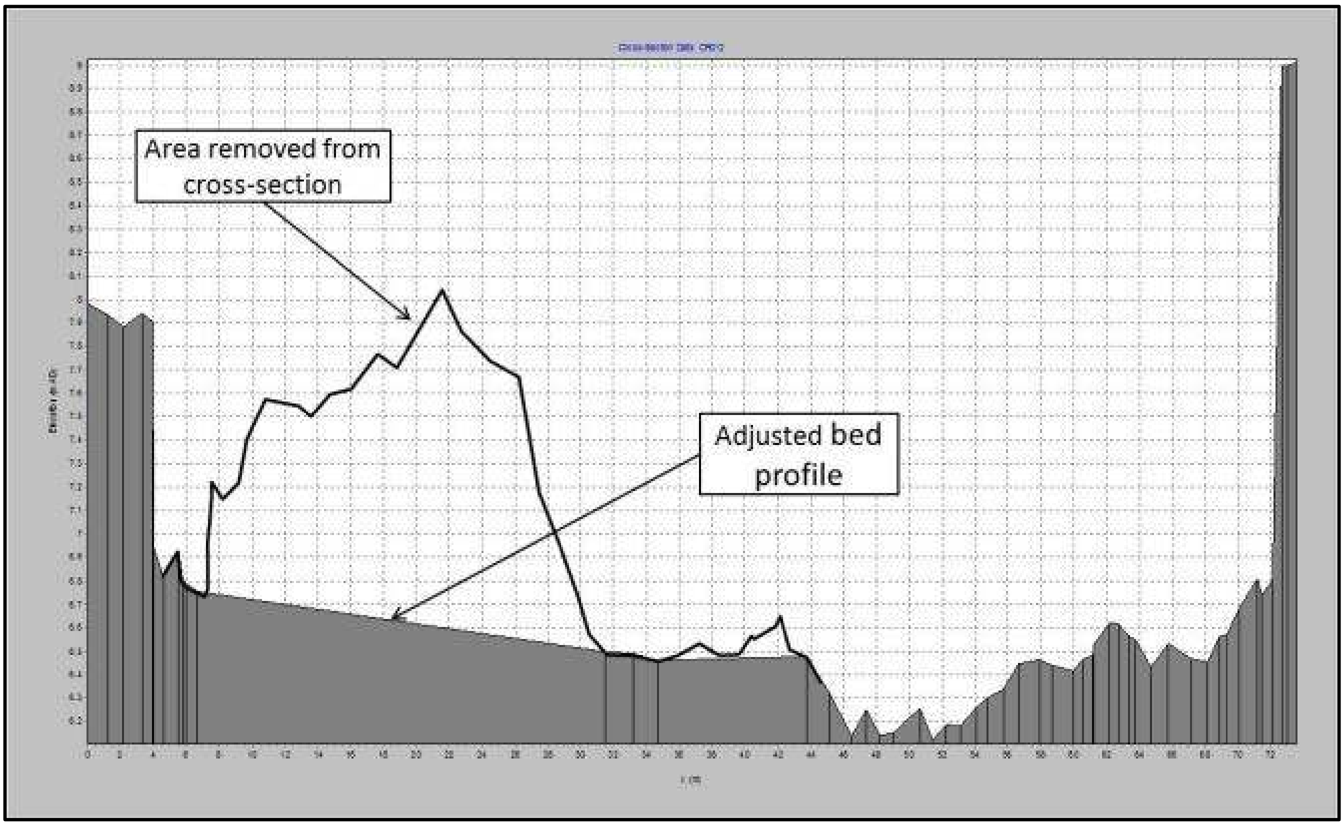
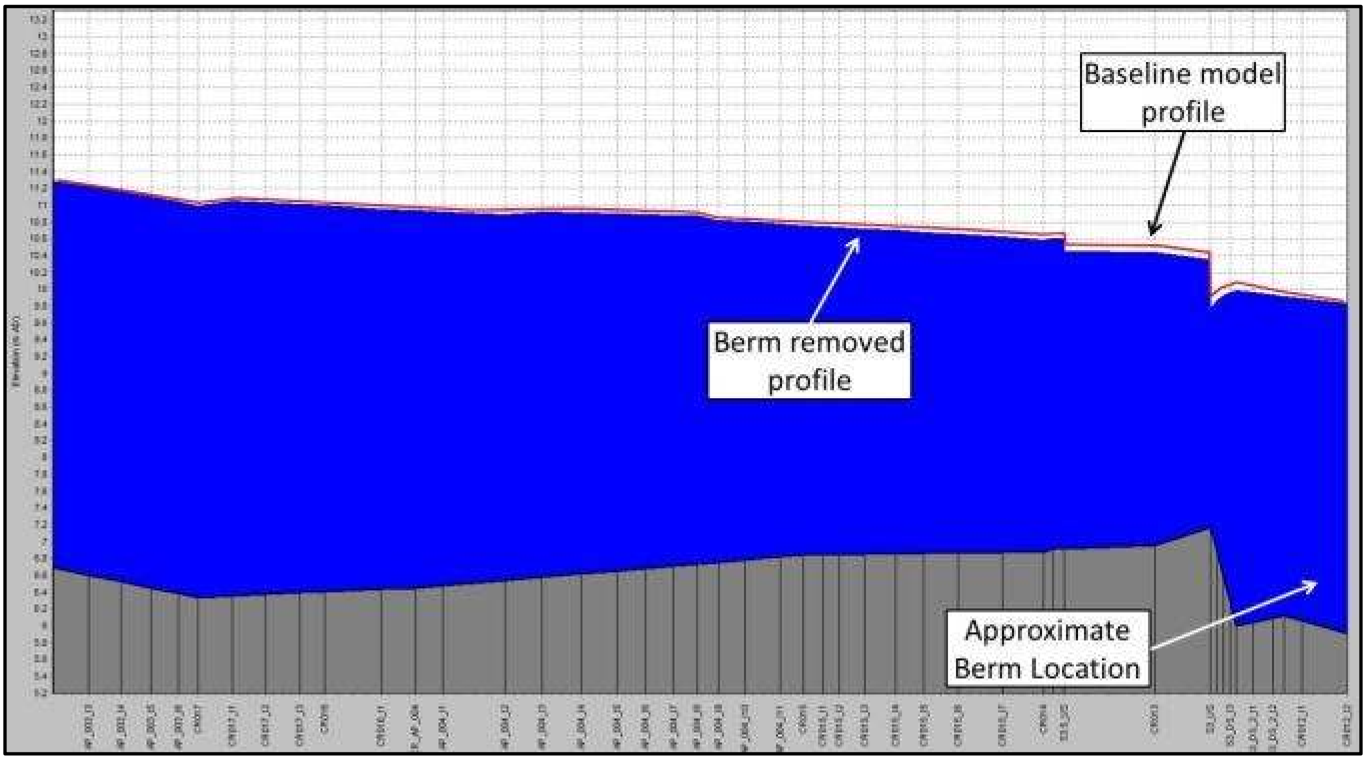


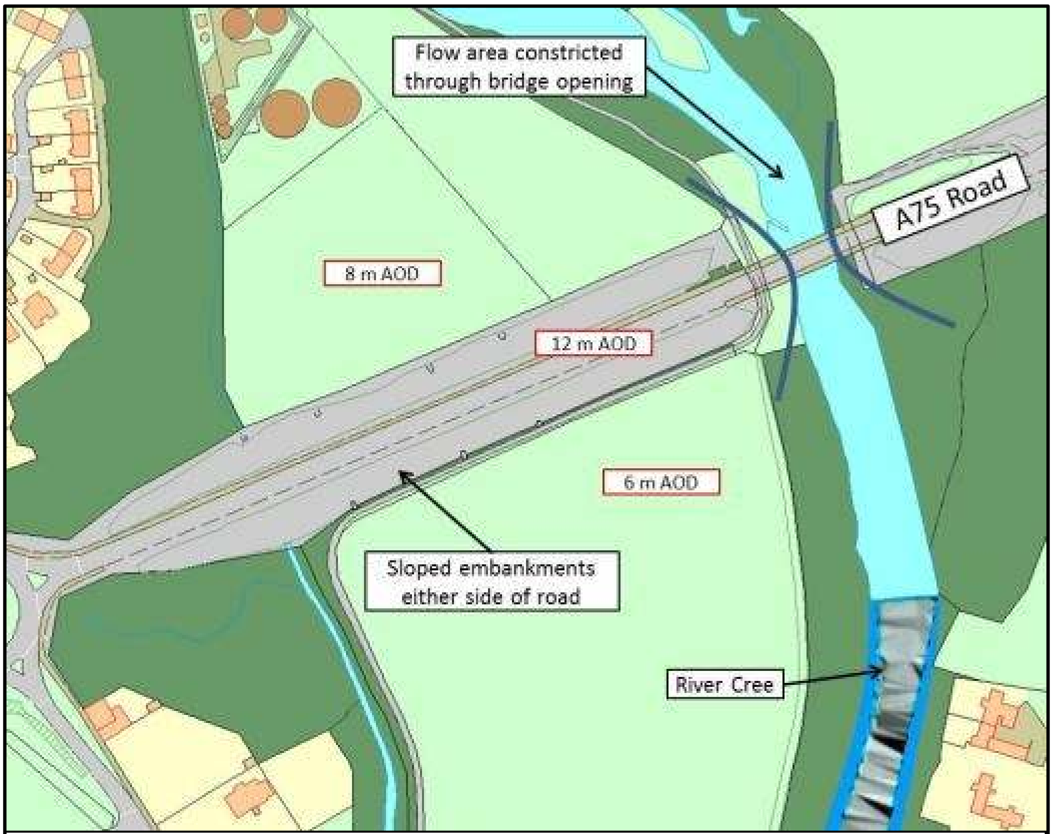
Figure 30: Longitudinal peak water level profile (before and after berm removal)



6.6 Option 4 – Increase Flow Area through A75 Bridge

The A75 road bridge is a large, relatively new single span crossing. The bridge is supported by two large concrete piers, which allow the road to span across the river channel and connect to raised embankments on either side, see Figure 31 and Photo 15 in Appendix A.

Figure 31: Model schematic of A75 Road Bridge and embankment



The land on the east side of the bridge is high, but the land on the west side is low both upstream and downstream of the bridge. The road embankment is approximately 4 m higher than the land on the west bank immediately upstream of the bridge, Figure 31.

There are four 1.5 m diameter culverts through the road embankment on the west side of the river. These culverts operate at times of high flows when the field immediately north of the bridge is flooded.

6.6.1 Increasing Conveyance

As shown in Figure 31 the bridge is at a bend of the river channel and is not streamlined. Therefore, there is a degree of head loss due to the bridge opening being at an angle from the main flow direction.

The bridge structure itself is a single span crossing which causes almost no obstruction to flows within the main channel for smaller flood events. However for larger events, the main channel is overtopped on the west bank and the supporting embankment provides a constraint to flows passing through the floodplain.

There is a public footpath passing under the bridge on the west bank, Photo 1. The footpath is raised under the bridge and is lower on either side of the bridge. It would be possible to lower the footpath to create additional flow area under the bridge. The existing cross-sectional profile and cross-sectional profile with footpath lowered are shown in Figure 32. An area of about 8 m wide could be lowered by up to 2 m to create an additional flow area of approximately 15 m^2.

It is assumed that the area considered for lowering is owned by Transport Scotland as part of the road embankment. The lower sections of the public footpath on both upstream and downstream of the bridge are predicted to flood during extreme events. Therefore lowering the footpath under the bridge would not have a significant effect on the use of the footpath during extreme events.

Figure 32: Cross-section at A75 (existing and considered modification)

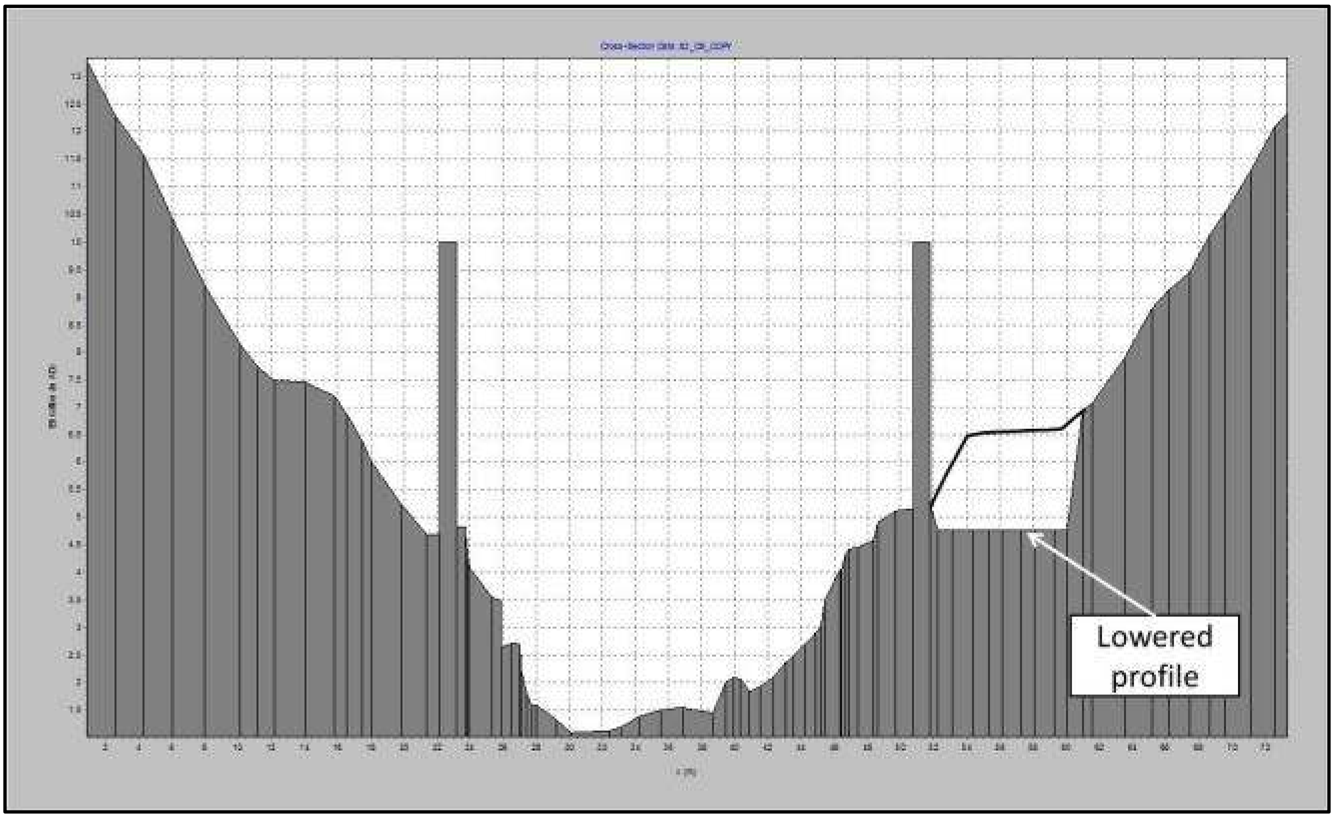


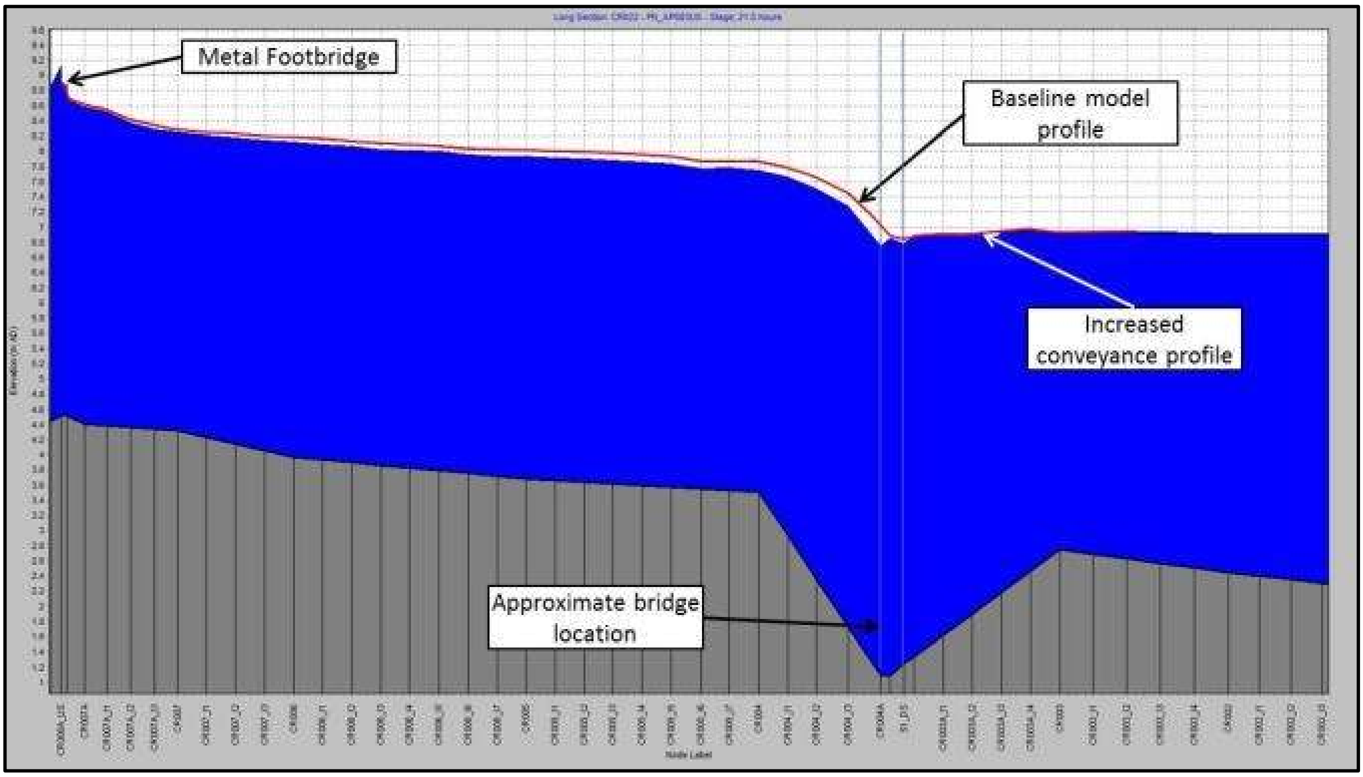
Photo 1: Existing Footpath under A75 (looking north – upstream)

6.6.1.1 Hydraulic Modelling

The baseline model was modified to include the adjusted cross-section profile as shown in Figure 32. The modified model was then run and the predicted peak water levels were compared to those from the base case model. Model results are shown in Figure 33.

The model results indicated that peak water levels immediately upstream of the bridge would be reduced by up to 0.3 m. However, this reduces further upstream and disappears by the time the metal footbridge by the car park is reached. No changes in peak water levels were predicted upstream of the metal footbridge. Hence, the increased cross-section flow area under the A75 bridge did not make any difference to flood levels in the town where flooding of properties is predicted.

Figure 33: Longitudinal water level profile (baseline and with increased flow area)



6.7 Option 5 – Raised Bridges

There are five bridge structures within the modelled reach of the river:

- Suspension footbridge (241000,566425) is elevated significantly above the flood level and the deck structure does not have a significant effect on peak water levels;
- Penkiln Road bridge at "Old Minnigaff" (241064, 566458) is a concrete flat-topped bridge which crosses the Penkiln Burn channel. The soffit of the bridge is elevated around 7 m above the channel;
- Bridge of Cree (241161,565655) is an old masonry arch bridge comprising of 5 arches approximately 7 – 15 m wide, with the middle being approximately 4.6 - 4.7 m high (above bed level);
- Metal footbridge (241270, 565247) is a single span bridge supported by two concrete abutments. The deck of the bridge is around 4 m above the bed of the channel;
- A75 road bridge (241614, 564685) is a concrete road bridge. The soffit of the bridge is elevated significantly above the bed of the channel and would not cause an impediment to flows within the channel. The bridge is supported by two concrete piers which are approximately 1 m wide and are located a short distance either side of the main channel;

The deck structures of the suspension footbridge, Penkiln Road bridge and A75 road bridge are all sufficiently above the flood level in the river that they will not cause a significant effect on flows in the river.

Bridge of Cree is a significant historical structure and as a result removal or raising the bridge soffit has not been considered. However, a model run was carried out to determine the effect of the Bridge of Cree on peak water levels in the river. Although the bridge is not likely to be removed we have modelled the removal as a purely academic exercise. Model results showed that by removing the bridge or raising the bridge structure above the 200 year water level, flood waters upstream of the bridge are reduced by a maximum of 150 mm immediately upstream of the bridge, gradually reducing to zero at the confluence with the Penkiln Burn. Flow velocities within the channel were predicted to increase by approximately 0.1 m/s at the bridge location. This indicates that Bridge of Cree has a limited effect on flows in the river up to 200 year.

Historical flood photographs taken during the November 2012 indicated that the deck structure of the metal footbridge, located close to Riverside Road by the car park, was overtopped, see Photo 2. This was confirmed by mathematical modelling that indicated that the river level would reach the bridge deck for flows in excess of a 1 in 2 year flood (or thereabouts). The bridge deck was also surcharged during the December 2013 event. As shown in Photo 2, flows hitting the bridge deck can back up and increase water levels upstream. Hence consideration was given to raising the deck level of the bridge above the predicted peak flood level.

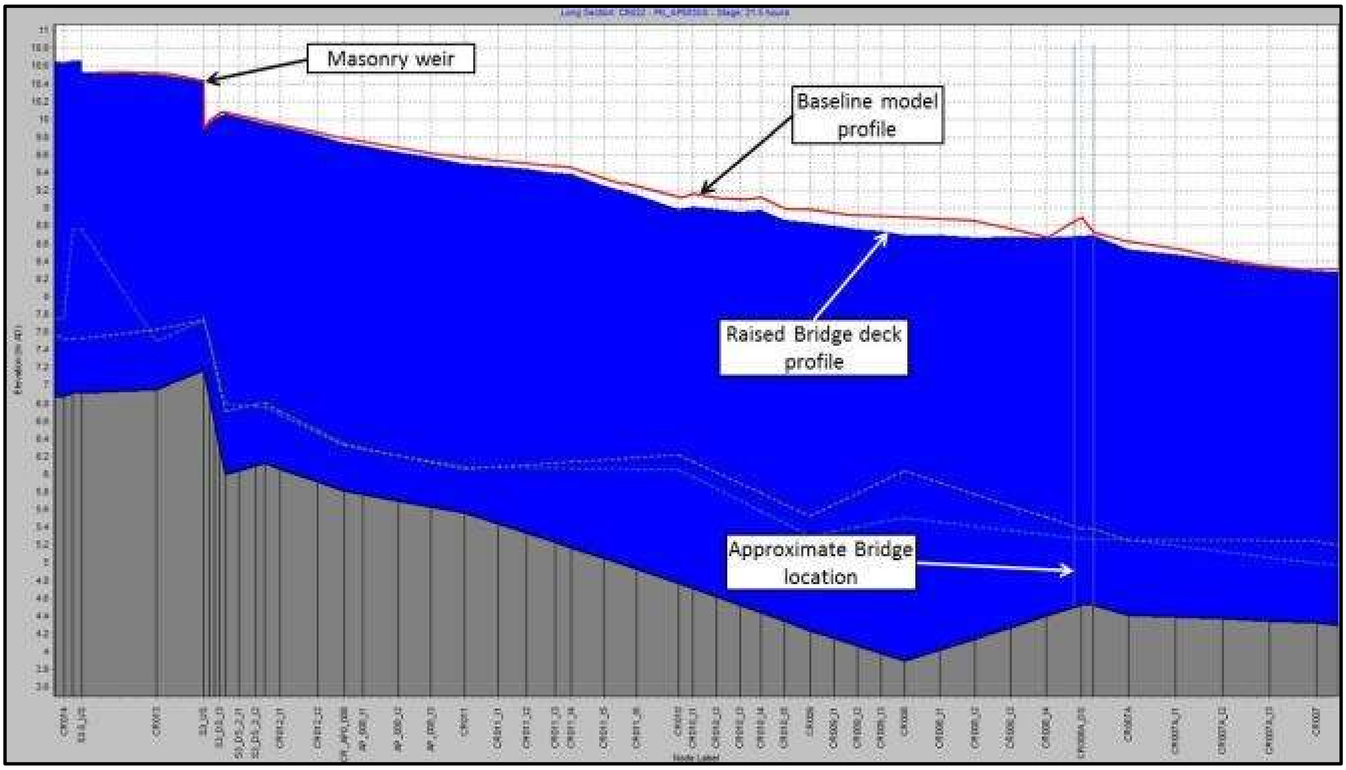
Photo 2: Metal footbridge during November 2012 event

The bridge is accessed by a short ramp hence bridge alteration proposals include raising the bridge deck sufficiently above the peak water level in the channel only and options for access to the bridge other than installing steps or increasing the height of the ramp have not been considered.

6.7.1 Hydraulic Modelling

The baseline model was amended with the bridge deck raised above the predicted 200 year water level. Model results indicated that the peak water level immediately upstream of the bridge decreases by approximately 0.45 m (to 8.7 m AOD). This reduces to 0.2 m at cross-section 8 (about 100 m upstream of the bridge) and there is no reduction predicted about 350 m upstream of the bridge (about 28 m downstream of the weir). The predicted longitudinal water level profile is shown in Figure 34.

Figure 34: Longitudinal water level profile (baseline and raised bridge deck)



6.8 Channel Maintenance and Siltation

It has been reported by local residents that siltation has been taking place within the river channel upstream of Bridge of Cree and this may have resulted in an increase in the risk of flooding from the river. The effect of removal of the sediment berm downstream of the weir (some 50 m downstream of Bridge of Cree) was assessed in Section 6.5. This section concentrates on the effect of possible siltation upstream of the weir, and in particular upstream of the bridge, on peak water levels in the river.

The information shown in Figure 19 is tabulated and summarised in Table 26 in graphical form. Crest level of the masonry weir, which is some 40 m downstream of Bridge of Cree, is approximately 7.7 m AOD. Bed level upstream of the weir and at the location of Bridge of Cree is approximately 6.9 m AOD, some 0.8 m lower than crest level of the weir. As shown in Figure 19, bed level of the river gently falls upstream of the bridge to approximately 6.3 m AOD, some 360 m upstream of the bridge), before rising to 7.7 m AOD some 190 m further upstream (approximately 150 m upstream of the confluence with Penkiln Burn). This indicates that immediately downstream of the confluence of the two watercourses, the bed level is approximately 1.4 m lower than the crest of the weir, and this is likely due to high flow velocities resulting from combined flows from both watercourses discouraging sediment deposition in this area or perhaps encouraging erosion. Bed level gently rises to 6.9 – 7.0 m AOD at the bridge. It is not known if this is the level of scour protection work at the bridge.

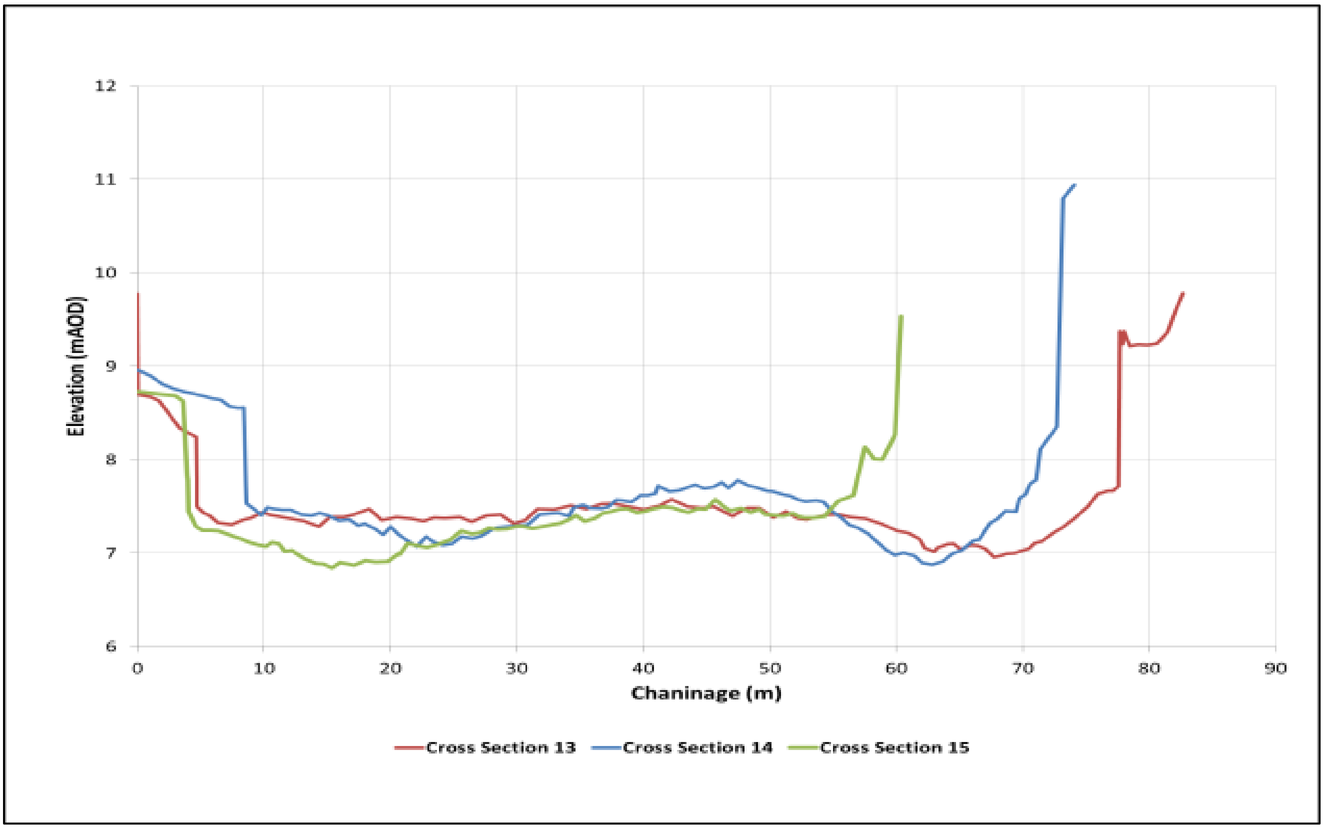
Table 26: Key levels in the vicinity of Bridge of Cree

Location	Level (m AOD)	Distance to Bridge of Cree
Crest level of Masonry Weir	7.7	40 m downstream
Minimum bed level between weir and Bridge of Cree	6.9	20 m downstream
Bridge of Cree Soffit Level (middle arch)	11.8	0
Bed level at bridge	6.9 – 7.0	0
Minimum bed level upstream of bridge	6.3	360 m upstream
Bed level same as weir crest level	7.7	550 m upstream of bridge (150 m upstream of confluence with Penkiln Burn)

The above indicates that almost 600 m length of the river bed upstream of the weir is lower than the crest level of the weir. Along this length of the river, peak water levels would largely be controlled by the weir; and to a certain extent by the bridge.

An additional model run was also undertaken to assess the effect of lower bed level upstream of Bridge of Cree on peak water levels in the area. As discussed and agreed with Dumfries and Galloway Council, bed level of the river was lowered by 300 mm upstream of the bridge over a length of some 100 m. Model results indicated that there would be no significant decrease in water levels upstream of the bridge for the 200 year event as a result of lowered bed level. This is not surprising as peak water levels in this part of the river are largely controlled by the weir. This indicates that even if some siltation has taken place upstream of the bridge, the effect of this on peak water levels in the river would be negligible. However, if siltation were to continue and bed level of the river were to approach the crest level of the weir, this would start affecting peak water levels in the area.

It is possible that some siltation may have taken place in some areas at the banks of the river and this may show seasonal variations. This could be as part of natural processes where fine sediment deposits during normal and low flow periods and this is washed away during high flows. The figure below shows three surveyed channel cross sections - one between the bridge and the weir (red) and two upstream of the bridge (blue and green respectively). These sections extend over a length of the river of some 120 m. It can be seen that the bed level at these sections is similar but width of the channel gets narrower upstream (i.e. Section 15 is narrower than the others). There is no evidence of significant siltation in this area obvious from this figure.



6.9 Summary of Potential Mitigation Options

Upstream Storage: A potential online storage area was identified in the River Cree valley upstream of Newton Stewart. Analysis indicated that a maximum storage volume of approximately 2 million cubic metres may be available in the area. With an appropriate flow control structure, the maximum attenuation this area could provide was calculated to be of the order of 70 m^3/s, which could reduce the theoretical 200 year flow to approximately 75 year flow in Newton Stewart. A 1 in 75 year flood would still cause extensive flooding in Newton Stewart; hence, flood storage by itself would not alleviate flood risk in the town. Flood storage would need to be considered in combination with other mitigation options to provide protection against a 200 year flood. A flood storage scheme could be designed to attenuate flows for return periods less than 1 in 200 years, but such a scheme would still require the construction of a large storage area and might provide little or no benefit for higher return period flows.

There are a number of factors which would affect the feasibility of such a storage scheme; such as ground conditions, land ownership, access, environmental impact, public perception and cost. These factors will need to be assessed before more detailed hydraulic analysis is carried out to optimise the scheme.

Direct Defences: Direct defences in the form of flood walls and earth embankments could be used to provide the desired level of protection (i.e., 1 in 200 year). However, in some areas the predicted height of defences would be up to 2.3 m from existing ground levels. This may not be acceptable to riparian owners and the general public. Hence, consideration has been given to a lower level of protection and with defences of around 1 m required to protect against a 1 in 10 year event.

Removal of Gravel Berm: Removal of the gravel berm downstream of the weir would make very little difference in flood risk, with local effects on flood levels only. The gravel berm has been there for at least 100 years and if removed it will likely re-form over time.

Increased Flow Area under A75 bridge: Increasing the available flow area under the A75 bridge reduces peak water levels by up to 0.3 m immediately upstream of the bridge, reducing to nil at the metal footbridge. This option would not produce a significant benefit for flood risk areas upstream of the footbridge.

Raised deck of metal footbridge: Raising the deck level of the metal footbridge above the 200 year flood level reduces peak water levels immediately upstream of the bridge by up to 0.45 m. This reduces to 0.2 m at Cross-section 8 (100 m upstream of the bridge) and zero at a point 28 m downstream of the weir (350 m upstream of the bridge). This provides some benefit in reducing the depth of flooding in the car park and along Riverside Road.

7 Flooding Risk from Sewer System

If flood defences were implemented and these included direct defences to contain flood waters in the main channel of the river, the water level in the river during extreme events would be higher than the existing ground levels on the land side of the defences. In such cases, flows from the river could back up into the local drainage system if there was a connection between the drainage system and the river (i.e., through outfalls). Flood waters backing up the drainage system would be able to come out of manholes within the urban area of Newton Stewart and Minnigaff and cause flooding.

In addition to flows backing up from the river during high flows, surface water from any rainfall during such times would not be able to flow into the river and would cause or exacerbate flooding behind the defences. There is also a risk of seepage of river water under the defences

When developing flood mitigation measures these sources of flooding will need to be considered.

The locations of existing manholes on both the Scottish Water combined and surface water sewer system located within the predicted 200 year flood extent are shown in Figure 35. This information was extracted from Scottish Water service drawings. If the area were defended by flood walls, high flows in the river could back up and come out of the manholes highlighted, causing secondary flooding. A detailed assessment of this secondary flooding has not been undertaken as part of this study. If a flood mitigation scheme is taken forward, a more detailed assessment of secondary flooding from the existing drainage system will need to be carried out.

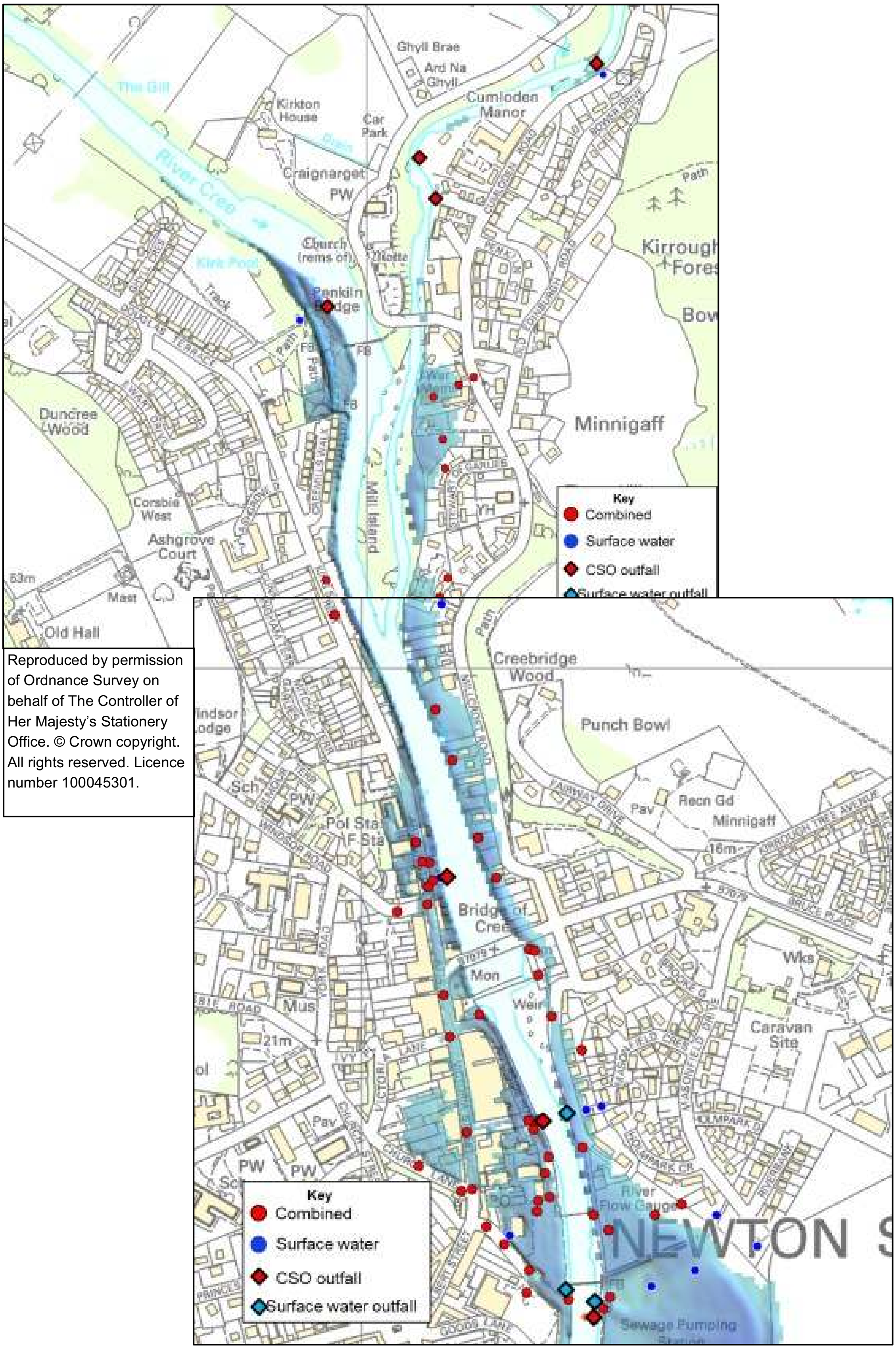
Historical Flooding

Scottish Water report entitled “Newton Stewart Drainage Area Study, Phase 2: Model Preparation, Verification and System Performance Assessment Report”, October 2002 by Hyder Consulting states that:

> *Internal property flooding was highlighted during the Inception Workshop at the Coach House, Church Lane, Church Street and King Street.*
>
> *External flooding was also highlighted at Queen Street and Victoria Street. The drainage systems along Arthur Street and Victoria Street are known to have sediment problems and a number of conduits have had sediment accumulation affecting the capacity of the system.*

The study concluded that there were 14 areas with structural deficiencies, 4 areas with hydraulic deficiencies, 6 areas with environmental deficiencies, and 5 areas with operational deficiencies. Off these, hydraulic deficiencies highlighted relate to flooding and these affect the Coach House, Church Lane, Church Street, Riverside Car Park, Torlundy Rooftops 6 and Wigtown Road. The predicted level of service for flooding was 1 in 1 year. This indicates that on average flooding from the sewer system would occur once every year.

Figure 35: Location of existing manholes and outfalls within flood extent



8 Economic Appraisal

The cost benefit analysis has been undertaken using the UK standard methodology based on;

- 'Flood and Coastal Erosion Risk Management Appraisal Guidance', 2010 published by Environment Agency; and
- 'The Benefits of Flood and Coastal Risk Management: A Handbook of Assessment Techniques – 2010', published by Flood Hazard Research Centre for DEFRA (Department of Environment, Food and Rural Affairs) and Environment Agency (EA). This report is also known as the Multi-Coloured Manual 2010.

The cost-benefit analysis undertaken in this chapter has been carried out based on conceptual designs of possible flood defence options. As a result, there is a high degree of uncertainty associated with the estimated costs. In order to account for such uncertainties, a standard 60% bias adjustment is made to the estimated scheme costs as part of cost-benefit analysis. Flood damage calculations are based on detailed mathematical modelling and surveyed finished floor levels for each property, and are more accurate. A number of assumptions have been made in this outline cost-benefit analysis and these are listed in Section 8.2.4.

8.1 Comments on Outline Design

The potential flood mitigation options are based on a high level conceptual design with costing being based on similar projects undertaken elsewhere in the UK, input from a qualified Quantity Surveyor with experience of construction works and national guidance. No design drawings have been prepared or no account has been taken of factors such as condition of existing defences, ground conditions, utility services, environmental aspects including contaminated land, site investigations, planning requirements, etc. Although the standard adjustment factor of 60% may cover such factors, the present outline analysis is the first stage in the development of the scheme and if the scheme were to be taken forward a more detailed assessment will need to be carried out as and when more detailed information becomes available.

8.2 Outline Cost Benefit Analysis

8.2.1 Properties at Risk of Flooding

A list of address point data for all properties in Newton Stewart was provided by Dumfries and Galloway Council. This provided geo-referenced property information including property type, location, etc. These were ground truthed in the field. Based on the modelled flood extents for each return period event, a list of all properties lying within each return period flood extent was compiled. A summary of the number of residential and non-residential properties affected is given in Table 27.

The total numbers of properties predicted to flood during a 200 year plus climate change flood event were 163. Of these, 90 are residential and 73 are non-residential. For the 200 year event, the total number of properties predicted to flood was 134 (67 residential and 67 non-residential). It should be noted that these are significantly less than those estimated by SEPA (2011) and the council's Strategic

Flood Risk Assessment (SFRA) report of 2007. Both these reports estimated a total of 352 properties within the 200 year floodplain. Of these, 306 are residential.
The discrepancy in the number of properties between the SEPA/SFRA studies and the present study are believed to be due to:-

a) The present study is based on detailed mathematical modelling of the river and its floodplains, whereas the previous studies were based on crude modelling techniques suitable for application to the whole country; and
b) It is understood that SEPA/SFRA figures include first and second floor flats and businesses as they refer to number of properties within the flood extent, irrespective of the floor level of the property. In the present study properties at risk are based on those predicted to flood based on measured doorstep levels. This would exclude upper floor flats which would not be expected to suffer direct flood damage.

For a 10 year flood event, 24 properties were predicted to flood (19 residential and 6 non-residential), while for a 2 year flood event only 3 properties were predicted to flood (all residential). The three residential properties predicted to flood during a 2 year flood are all at Reid Terrace (Location 3 in Figure 26). It should be noted that the Sewage Pumping Station on the east bank of the River Cree was predicted to flood (even during a 2 year flood), but it was considered that it would remain operational during flooding and is not included in the damage cost analysis. This should be reviewed during the detailed design stage, if the scheme is to be taken forward.

Table 27: Predicted number of properties at risk in Newton Stewart

(a)

Property Type	Existing Case					
	200+ cc	200	100	50	10	2
Residential	90	67	53	44	19	3
Non-Residential	73	67	43	15	6	0

(b)

Property Type	200 Year Defended						50 Year Defended		10 Year Defended		
	200+ cc	200	100	50	10	2	200	100	200	100	50
Residential	89	0	0	0	0	0	63	20	69	57	44
Non-Residential	73	0	0	0	0	0	68	58	66	61	18

Table 27 (b) shows the predicted number of properties at risk of flood damage with 200 year, 50 year and 10 year level of protection in place. It should be noted that these defences do not include a freeboard allowance. In practice, with a reasonable freeboard allowance included in the design the actual number of properties affected would be less.

8.2.2 Depth of Flooding

A door step ground survey was carried out as part of this study to obtain finished floor levels of all properties considered at risk of flooding.

Within the damage calculations the depth of flooding for each property was obtained by subtracting the surveyed floor level of the property from the predicted water level at the property. Water level predictions were extracted from the 2D model.

It should be noted that number of properties at risk of flood damage (those shown in Table 27) includes those with the predicted water level within 0.3 m of their finished floor levels (as it is assumed that foundation of such properties would be affected even though water level is below finished floor level). The number of properties affected by flood damage are not the same as the number of properties at risk of flooding, with the latter being smaller as it only includes properties where water level reach and exceed finished floor level.

8.2.3 Flood Damage Data

Flood damage data for each property was extracted from the Multi-Coloured Manual 2010 excel spreadsheets. An Excel Macro programme was developed to extract flood damage data from the appropriate section of the Multi-Coloured Manual based on property type, property age and depth of flooding.

Flood damage data extracted from the Multi-Coloured Manual is given per square metres of property. These values were then multiplied by plan area of each property to obtain the total damage for each property for the given depth of flood.

8.2.4 Assumptions

A number of assumptions have been made in this outline cost benefit analysis and these are summarised below:

a) Plan areas of each property calculated from 1: 1250 Ordnance Survey maps were considered sufficiently accurate for this assessment.
b) Age of each property was estimated based on visual appearance of the property. As the property bands within the Multi-Coloured Manual 2010 are quite broad it is likely that reasonably robust estimates of property age have been made.
c) Property values were estimated based on type and size of property. There is likely to be a high degree of error in these values as they have not been reviewed by experienced surveyors or estate agents. A sensitivity analysis was carried out to determine the effect of property values on the final results. However, more information would be required if the assessment is taken forward. In addition, within the Multi-Coloured Manual 2010 methodology there are assumptions related to the damage costs associated with properties of different age and value and there is likely to be a high degree of uncertainty associated with these assumptions of damage costs.

It should be noted that the assumed property value is important in cases of extreme flooding when the estimated flood damage cost for the property exceeds its value. For those cases the flood damage costs cannot exceed the property value. Hence, if the property value is over-estimated damage costs could be too high and conversely if the property value is under-estimated damage costs could be too low. Property values are also important in estimating the write-off value of properties. Therefore, property values can have a significant impact on the 'Do Nothing' scenario to which other options are compared.

It is suggested that more reliable estimates of property values are made for future detailed cost-benefit analysis, particularly those properties with large estimated flood damage.

8.2.5 Outline Costing of Flood Management Measures

Flood management measures considered for this study have been developed to a conceptual stage only.

For the type of defences considered, average construction cost figures were used based on similar work elsewhere and national guidelines. This work was carried out by a qualified Quantity Surveyor (Jones Associates). No design drawings were prepared or quantities of materials assessed for costing of the defences. A standard unit cost appropriate for the type of defence and particular location was used. This approach was considered appropriate for the purposes of this study. Should the scheme be taken forward, the cost-benefit analysis will need to be refined as and when more accurate information on the design becomes available.

A number of items have not been included in the outline costing and these include the following:

- Civil/geotechnical investigations, asbestos and contamination surveys;
- Ground remediation or the removal of contaminated or deleterious materials, if applicable;
- Road and public footpath remedial works or upgrades;
- Remedial or upgrade works to existing public realm or private property;
- Land acquisition, legal and financing charges;
- Statutory charges; and
- Cost of environmental surveys.

Allowances have been made for:

- Professional fees (including design, tendering, supervision, etc.);
- Utility diversion costs; and
- Emergency service costs.

Outline costings have been prepared for the following flood mitigation options:

1) Upstream flood storage; and
2) Direct defences to provide protection for 1 in 200 year, 1 in 50 year and 1 in 10 year events.

Details of these options are provided in Section 6.4.

Other options, such as removal of the gravel berm, increasing flow conveyance through the A75 bridge, and raising of the deck of the metal footbridge provided limited local benefits only in terms of reducing flood risk and were not considered in the cost-benefit analysis. These options could be considered in the future, perhaps in combination with one or both of the two main options considered above. However, these options have not been costed.

The outline costing of the upstream flood storage option was estimated based on the cost of the recently constructed flood storage basins for the White Cart Flood Prevention Scheme in Glasgow.

Direct defences would largely be in the form of flood walls, due to limited space available between properties at risk and the main channel of the river. A detailed structural survey of existing flood defences has not been carried out, although a visual inspection has been carried out of defences along Riverside Road and Arthur Street. Without knowledge of the conditions of the existing defences, particularly below the normal water level, it was assumed that all required flood defence walls would be sheet piled (i.e., foundations extending some depth below the river bed). This is likely to be a conservative assumption (i.e., actual costs will be lower than considered in this assessment) as in some areas it may be possible to provide alternative wall types which may be more cost effective without sacrificing structural integrity.

The defences were considered to extend from a point approximately 70 m upstream of Penkiln Road Bridge to the southern end of Mansfield Drive on the east bank and from a point in line with Garlies Crescent to Goods Lane on the west bank. Continuous flood defences were assumed along these lengths.

The estimated indicative cost of flood defences to provide 200 year level of protection is given in Table 28.

Table 28: Estimated indicative flood defence construction costs only

Flood Mitigation Feature	Approximate Length (m)	[a]Approximate Average Height (m)	Estimated Indicative Cost (£M)
Flood Wall (west bank)	900	10	5.3
Flood Wall (east bank)	750	9	4.6
Earth Embankment (east bank)	250	3	0.6
Storage Dam Structure	200	10	4
Flow Control Structures	-	-	1-10
Total	1.9 km (excluding dam)		£10.5M Direct Defences £5 – £14M Upstream Storage

[a] *Sheet piled wall assumed to extend at least 4-5 m below river bed*.

The above indicated costs do not include the cost of temporary works, preliminaries, contingencies, service diversion costs, design and supervision costs, and land acquisition costs. However, the estimated indicative total cost of the scheme is presented in Table 29.

It should be noted that although the upstream storage and direct defence options are presented as two separate options in the above tables, the upstream storage option does not provide the required level of defence in Newton Stewart (i.e., the peak flows passing downstream are only reduced to about a 1 in 75 year flow and this will still cause extensive flooding in the town). Therefore, the upstream storage option would need to be combined with direct defences to provide a 1 in 200 year level of flood protection. The average difference in water level between the 200 year and 75 year flows is of the order of 0.2 – 0.3 m. If the upstream storage option is combined with direct defences to provide a

200 year level of protection, direct defence heights would be reduced by 0.2 – 0.3 m, compared to those for the 1 in 200 year event with no upstream storage. By lowering the defence heights by this amount the cost of direct defences would not necessarily be significantly reduced, perhaps of the order of 10%.

Table 29: Estimated total scheme cost

Defence Option	Construction Cost (£M)	Preliminaries (£M) (10%)	Contingencies (£M) (20%)	Utility Diversion (£M) (10%)	Design & Supervision (£M) (15%)	Total (£M)
Direct Defences	10.5	1	2.1	1	1.5	[a]16.1
[b]Upstream Storage 1 (Lower)	5	0.5	1	-	0.8	[a]7.3
[b]Upstream Storage 2 (Upper)	14	1.4	2.8	-	2.1	[a]20.3

a *Does not include the standard optimism bias of 60%. With 60% optimism bias, total cost of Direct Defence option becomes **£25.8M**. This is automatically added in the cost-benefit analysis.*
b Does not include land acquisition costs.

For mitigation schemes costed at an initial stage, the DEFRA/EA methodology suggests a factor of safety of 60% to account for unforeseen costs including difficulties which might be encountered during further detailed investigations. As uncertainties are eliminated through detailed investigations, the cost of any scheme will be refined and the 60% factor will be reduced. However, for the present cost-benefit analysis a standard 60% optimism bias has been assumed. This increases the cost of the scheme for direct defences to £25.8M and this is automatically taken into account in the cost-benefit analysis.

If the upstream storage option were to be combined with direct defences to provide 200 year level of protection, the estimated total cost of the scheme would be of the order of £21.8M (compared to £16.1M based on direct defences only) if the flow control structure from the storage area is based on simple structures such as weirs. If better controls on downstream flows are required and Hydro-Brakes© are used, the cost could be as much as £34.8M, without the 60% bias factor.

8.2.6 Outline Cost-Benefit Analysis

The national standard approach for cost-benefit analysis of flood schemes was applied to five options;

- Option 1: Do Nothing.
- Option 2: Maintenance only.
- Option 3: 200 year protection.
- Option 4: 50 year protection.
- Option 5: 10 year protection.

Direct flood damage costs only have been included in the analysis at this stage and intangibles, environmental and social impacts have not been included.

A standard 100 year analysis period was assumed in the calculations.

The assessment was undertaken for two different estimates of property costs. A base case scenario with conservative (low) property costs and another with 50% higher costs. This was done as there was a high degree of uncertainty associated with property costs and it was felt important to illustrate the effect of this uncertainty on the final benefit-cost ratios.

Option 1 - Do Nothing

The “Do Nothing” scenario sets a baseline for comparison. The scenario is based on a number of standard assumptions:

- Once a flood event occurs, no repairs are made to the damaged properties.
- Although each property suffers damage each time it floods, the total damage value of each property is limited to the present value of the property.
- It is assumed that only one breach (of defences) occurs in the analysis period (100 years).
- The probabality of a flood event occurring is increased over time due to lack of maintainance of exisiting defences.

The average annual flood damage of £134,000 was calculated from the likely damages for a range of floods of different return periods. The total damage in the analysis period is equal to and cannot exceed the total property value at risk of flooding (between £18.6M and £27.9M), which is the estimated write off value of all properties within the 1 in 200 year floodplain.

This scenario is unrealistic as the Council has duties to carry out clearance and repair works (as per the Flood Risk Management (Scotland) Act 2009), but this ‘Do Nothing’ option is a standard used in all such cost-benefit analyses and it is used as a baseline for comparison purposes.

Option 2 - Maintaining Existing Defences and Properties

Option 2 has the following general assumptions:

- Over the analysis period each property will be subject to a number of floods of different magnitudes. The total flood damage is the summation of damage each time the property floods.
- Damages to defences and properties are assumed to be repaired. The total damage cost can therefore be higher than the present value of the properties over the analysis period of 100 years.
- It is assumed that a flood event can occur in any year and in each year there is the same probability of a flood occurring.
- Investment is made annually to maintain the flood defences in their current state. The risk of flooding remains constant throughout the analysis period.
- This option can also be described as the cost of maintaining all defences and properties at their present state.
- This is similar to present day scenario where Council maintains watercourses and existing defences. Also riparian owners have a duty to maintain the defences in their ownership.

An annual average maintenance cost of £40,000 was assumed (incurred by all responsible for maintaining existing defences, i.e., the total amount for the Council and private owners), see Table 30. This also includes the cost of replacing defences when their design life expires and they are no longer capable of providing the same level of defence as at present. Over the 100 year analysis period this is equivalent to £1.2M at present day value (i.e., if £1.2M were invested today it would provide funding for maintenance for the full analysis period of 100 years). This cost was assumed to maintain the current risk of flooding for existing defences without any improvements. It should be noted that most of the existing flood defences (walls) are in third party ownership. Hence, it is difficult to envisage a mechanism whereby Option 2 could be made to work. However, it is presented here for comparison as it is considered to be a more realistic scenario than Option 1 "Do Nothing".

Table 30: Estimated annual maintenance cost

Maintenance Item	Approximate Length (m)	[a]Estimated Indicative Cost (£)
Replace Existing Defences as New		
Flood Walls (1 m)	8	5,000
Flood Wall (3m)	3	10,000
Earth Embankment	10	5,000
Maintenance and Contingency		
Channel Clearance		5,000
General Maintenance		5,000
Contingencies and Emergencies		10,000
Total		40,000

a Estimated annual maintenance cost incurred by all responsible for maintaining existing defences to provide the same level of defence as at present, i.e. by Council and private owners

A Bias Adjustment Factor is applied to the costs as discussed above. The DEFRA/EA method recommends a value for Bias Adjustment of 60% for feasibility level studies. This means that an additional 60% is added to the costs of maintenance.

The average annual flood damage for this case is the same as the ‘Do Nothing’ option.

Option 3 - Full Flood Mitigation Scheme (200 Year Level of Protection)

Option 3 has the following general assumptions:

- Damages to properties are assumed to be repaired. The total damage cost can therefore be higher than the present value of the properties over the analysis period.
- It is assumed that a flood event can occur in any year and in each year there is the same probability of a flood occurring.
- A capital investment for the construction of the flood management scheme is made in the first two analysis years followed by an annual maintenance each year. The risk of flooding is greatly reduced by the flood management scheme and as a result the damage to property is greatly reduced. Although the properties will be protected against flows up to the design flow (i.e., 200 year), there is still a residual risk, albeit very small, that a flood greater than the design flood occurring during the analysis period of 100 years.

It is assumed that the full flood mitigation scheme, which includes 1.65 km of flood walls and 0.25 km of earth embankments, is implemented, see Table 28.

A time varying annual maintenance cost is considered. It is assumed that for the first 20 years maintenance of the new flood alleviation scheme should be small (£20,000 per year), with maintenance increasing over time up to a value of £100,000 per year around 50 years after the construction of the scheme. Over the 100 year analysis period this is equivalent to £1.2M (£1.9M with optimism bias) at present day value.

A Bias Adjustment Factor is applied to the costs. As outlined above the DEFRA/EA method recommends a value for Bias Adjustment of 60% for feasibility level studies. This means that an additional 60% is added to the costs of constructing the scheme and to maintenance. This factor can be reduced at detailed design stage as confidence in cost estimates is increased. As uncertainties associated with the scheme are eliminated during the design stages (through undertaking site investigations, environmental assessment, consultation with stakeholders, and changing market conditions), the estimated cost of the scheme will be improved and bias adjustment factor will be reduced.

As the frequency of flood events would be considerably reduced with the scheme in place, the calculated average annual damages figure is reduced to £13,000 for this option. Due to the large cost of initial construction a large bias adjustment is calculated by the DEFRA/EA methodology (£10.1M). Total flood damages are reduced to a value of £0.5M over the analysis period of 100 years due to the implementation of the scheme, with the remaining damage costs reflecting the residual risk during the 100 year period of an event in excess of the design condition for the defences.

Results of the analysis are summarised in Table 31. The values in the table refer to the costs for a scheme with direct defences only. During the development of the cost-benefit analysis calculations were undertaken for schemes that combined direct defences with upstream storage (with simple and complex outflow control structures on the flood storage area). However, the cost of the flood storage area resulted in significantly lower benefit-cost ratios than produced through considering direct defences only. For the option with protection up to a 200 year event the combination of direct defences and flood storage area (simple outflow control) reduced the benefit-cost ratio by 30%. For an option with a flood storage area and a complex outflow structure (e.g., Hydro-Brakes©) the benefit-cost ratio was calculated to be 50% less than for the case with direct defences only. These results are not presented in detail as the benefit-cost ratios are significantly less than one.

<u>Option 4 – 50 Year Level of protection</u>

Option 4 has the same general assumptions as Option 3.

It is assumed that flood defences are provided up to the 50 year flood level including 0.3 m freeboard. The total length of defences was assumed to be similar to Option 3, only defence heights are lower. As before, a 60% bias factor is applied to the estimated cost of the scheme. Annual maintenance costs assumed to be similar to Option 3.

The results of the analysis is summarised in Table 31. As for Option 3 calculations were undertaken which considered the development of a flood storage area in tandem with direct defences. However, the results showed low benefit-cost ratios and detailed calculations are not presented.

<u>Option 5 – 10 Year Level of Protection</u>

Option 5 has the same general assumptions as Option 3.

It is assumed that flood defences are provided up to 10 year flood level including 0.3 m freeboard. The total length of defences was assumed to be similar to Option 3, only defence heights are lower. As before, a 60% bias factor is applied to the estimated cost of the scheme. Annual maintenance costs assumed to be similar to Option 3.

The results of the analysis is summarised in Table 31. As for Options 3 and 4 calculations were undertaken which considered the development of a flood storage area in tandem with direct defences. However, the results showed low benefit-cost ratios and detailed calculations are not presented.

8.2.6.1 Summary Cost-Benefit Analysis

Summary results for the cost-benefit analysis are shown in Table 31.

The cost of the flood defence schemes assessed in this study vary between £7.5M for the 1 in 10 year defence (made up of £3.4 construction cost, £1.3M maintenance over 100 years, and £2.8M optimism bias adjustment) to £27M for the 1 in 200 year protection (which could increase to £60M if Hydro-Brakes© or similar structures were used to control outflow from storage basin).

Total flood damage over the 100 year analysis period was estimated to be between £16.3M and £24.5M, depending on value of properties at risk of flooding.

Table 31: Summary results of cost-benefit analysis

Cost-Benefit Summary	Option 1 Do Nothing	Option 2 Maintenance	Option 3 200 Year Protection	Option 4 50 Year Protection	Option 5 10 Year Protection
Cost	£M	£M	£M	£M	£M
Total Capital Cost	0	0	15.6	10.9	3.4
Total Maintenance Cost	0	[a]1.2	[b]1.3	1.3	1.3
Bias Adjustment (60%)	0	[c]0.7	[c]10.1	7.3	2.8
Total Cost	**0**	**1.9**	**27.0**	**19.5**	**7.5**
Benefit					
Average Annual Damages	0.13	0.13	0.01	0.05	0.05
Total Flood Damages	16.9 (25.0)	9.1	0.5	1.9	4.2
[d]Total Flood Damages Avoided	**-**	**[g]7.8 (15.9)**	**[g]16.3 (25.6)**	**[g]14.6 (22.8)**	**[g]12.3 (20.5)**
[e]Net Present Value of Benefits(NPV)	-	-	[g]-10.7 (-1.4)	[g]-4.9 (-3.3)	[g]4.8 (13.0)
[f]Average Benefit/Cost Ratio (BCR)	-	**-**	**[g]0.60 (0.94)**	**[g]0.75 (1.17)**	**[g]1.64 (2.73)**

[a] *Annual maintenance = £0.04 Million. The value in the table is sum of money invested at present day value that would provide funding for future maintenance*

[b] *Time varying annual maintenance, with low maintenance soon after construction of new scheme rising to £0.1M per year after around 50 years. The value in the table is sum of money invested at present day value that would provide funding for future maintenance*

[c] *Capital and maintenance costs are increased by 60% to account for optimism bias in initial estimates of costs. The 60% factor is recommended by DEFRA/EA as appropriate for feasibility level studies.*

[d] *Total Flood Damages of Option 1 (Do Nothing) - Total Flood Damages of Option*

Numbers in brackets refer to 50% higher property values

[e] *Total Flood Damages avoided - Total Cost*

[f] *Total Flood Damaged Avoided / Total NPV (including bias adjustment)*
IBCR= (NPV benefits 3 – NPV benefits 2)/(NPV costs 3 – NPV cost2)

[g] *Value in brackets considers property values 50% higher than those used to calculate value outside of brackets (see Section 8.2.6)*

In simple terms the above would indicate that a scheme costing more than £16.3M would not give a positive benefit-cost ratio. However, it should be noted that the current analysis is based on direct damage and no account has been taken of other factors such as environmental and social impacts, intangibles, etc. With such factors included, the final benefit-cost ratio will likely to be higher.

Based on the analysis summarised above, it appears that 200 year level of protection is unlikely to be economically feasible as it would not (even with environmental and social benefits included) produce a benefit-cost ratio sufficiently above unity.

The 10 year scheme considered appears to produce the highest benefit-cost ratio. Although benefits accrued with this scheme would be limited compared to the present day situation, the cost of the scheme will likely to be significantly less as flood walls may not require sheet piles at most locations. The present day cost of this scheme including maintenance and optimism bias is of the order of £7.5M (compared to £27M for the 200 year level of protection).

The analysis presented in this Chapter has to be regarded as outline at this stage and will need to be refined as and when more detailed and reliable information becomes available.

9 Flood Warning System

Two separate modelling systems have been set up as part of this study. An integrated 1D/2D ISIS model was set up of the River Cree and Penkiln Burn within the urban areas. A separate 1D ISIS model was set up for the upper reaches of the River Cree from Crungie Wood (a short distance downstream of the confluence with Water of Minnoch) to a point a short distance downstream of the confluence with Penkiln Burn, over a length of some 10.8 km. This model was based on limited surveyed cross sections and was used for flood routing purposes only.

It is understood that SEPA may wish to use the existing model to provide flood warning services to Newton Stewart. The model is suitable for such use. However, any amendments could also be made to the model to suit SEPA's final requirements.

Model work undertaken indicated that the peak flood would take between 30 minutes and one hour to travel from Minnoch Bridge to Newton Stewart, depending on the flow rate and to a certain extent the shape of the flood hydrograph. The additional time for flow routing from the headwaters of the catchment to Minnoch Bridge is estimated to be a further 1 hour.

It is normally accepted that a minimum lead time of the order of 3 hours would be required for a real-time flood warning system. This is the time before the arrival of a flood peak that a flood warning is given to allow actions to be taken by emergency services, flood response teams and property owners to mitigate flood risk. A lead time of the order of 30 minutes to one hour would not provide sufficient time for the relevant people to act, particularly if flooding occurred during the hours of darkness.

Flood forecasts and flood warnings can typically be based on gauged river flows upstream of a site of interest, gauged rainfall within the catchment upstream of a site of interest and/or forecast rainfall from a rainfall radar or meteorological model. Flood forecasts based on observed data (especially observed river flows) are likely to be more accurate than those based on rainfall forecasts.

Based on the analysis outlined above there would not appear to be sufficient lead time upstream of Newton Stewart for a forecasts to be based on gauged river flows, i.e., the approximate time of travel of a flood wave from a gauged site to the area at risk of flooding is too low.

An alternative approach would be to monitor rainfall over the upper catchment or use rainfall radar predictions and set up a catchment wide hydrological model to predict flows that could arrive at Newton Stewart, based on rainfall inputs. This will provide a longer lead time, but an initial assessment would suggest a lead time from the peak of extreme rainfall would still not exceed 3 hours for some events. In this case flood warnings would need to be based primarily on forecast rainfall produced by weather radar and meteorological models. Such an approach could provide warnings with sufficient lead time for Newton Stewart; however, warnings based on forecasts would have a higher degree of uncertainty than those based on observed data (rainfall or river flow).

Irrespective of their application for real-time flood forecasting, additional rain or flow gauges within the headwaters of the River Cree would be useful in terms of providing data to calibrate forecasting models and they would provide some degree of warning for flood events produced by long-duration rainstorms.

Giving the large catchment area, strategically located rain gauges in the upper catchment as well as at least one flow gauge upstream of the town would be required. If this is the preferred option by SEPA, a catchment specific study will need to be carried out to determine the optimal number of rain gauges and their location. Once the system is set up and rainfall records become available in tandem with recorded flows, a catchment model could be calibrated for use as a flood forecasting tool. SEPA would normally undertake such work and specific requirements for Newton Stewart will need to be discussed with SEPA. Since the submission of the original report, it is understood that SEPA has been assessing viability of a Flood Warning System for Newton Stewart.

10 Visual Inspection of Existing Flood Walls

The following assessment is based on visual observations made by David R Murray, Structural Engineers, during a walkover survey and should be regarded as an initial assessment of the existing walls on Riverside Road and Arthur Street. A more detailed assessment of the walls and river banks will need to be carried out if any significant engineering works involving these walls and river banks are required.

10.1 Riverside Road - Observations

The wall on Riverside Road appears to be constructed of random rubble whinstone and granite with a lime mortar. The wall has been constructed to a slight batter.

The stone wall provides retention to the public road and footpath with the river to the low side. The level of retention varies from 1.5m to 2.5m.

The wall stem extends upward to form a parapet and varies between 500 – 900 mm above the footpath level. The wall stem width above ground level is generally 500 mm; however, some sections of the wall reduce to 400 mm wide.

The wall has generally been capped with a rough sand/cement mix. This coping has cracked and broken up in many areas and is likely allowing water to penetrate the wall structure.

The wall appears to have been maintained in the past, however a number of areas of the wall require re-pointing or other maintenance work. Light vegetation was seen to be growing over the majority of the wall.

The last 30-40 m of the wall to the northern end appears to have been re-built in the past and a large cope added. The wall stem over this section appears to have been reduced to approximately 400 mm in width.

The wall does not show any significant signs of movement or distress due to the retention of the public road or footpath.

10.2 Arthur Street - Observations

Similar to the wall on Riverside Road, the wall on Arthur Street appears to be constructed of random rubble whinstone and granite with a lime mortar. The wall has also been constructed to a slight batter.

The stone wall provides retention to the public park with the river to the low side. The level of retention varies from 1.2 m to 2.8 m

The wall stem extends upward to form a parapet and varies in height between 600 – 900 mm above the park ground level. The wall stem width above ground level is generally around 600 mm wide.

The wall is broken up into sections by private properties and return walls providing access steps down to the river.

In two sections, one immediately south and one immediately north of the Fire Station, the wall appears to be of a poor condition. A number of large cracks, missing stones and mortar were noticed and significant vegetation and roots are growing over and through the wall. These sections of wall do not appear to have been maintained for a number of years.

The remaining part of the wall appears to be of a reasonable condition and has been maintained in the past. Light vegetation was noted over the full extent of the wall and was more severe on the low side.

The wall does not show any significant signs of movement or distress due to the retention of the public park.

10.3 Comments of Existing Conditions – Riverside Road & Arthur Street

Based on visual observations made along two sections of the wall (immediate north and south of the Fire Station) the wall was observed to be in a poor condition and is unsuitable for use as a flood retention wall.

The remaining section of wall to Arthur Street, and the wall to Riverside Road will require maintenance works (noted below); however, based on our initial calculations, walls with a stem width of 500 mm (or greater) are adequate to retain water levels up to 900 mm above finished ground levels (high side). Wall stems with a width of 400 mm or greater are adequate to retain water levels up to 700 mm above finished ground levels. Walls where the wall height is less than noted above could be increased in height to the maximum height appropriate to its base width using suitable stone and mortar.

The calculations carried out are based on assumed values that were derived based on our inspection. Adequate testing (likely intrusive tests) would need to be carried out to confirm these assumptions. The water loading applied to the wall has been assumed as static and no account of load from wave or water fluctuations have been taken into account; however, it is likely that any additional load from these actions would be dealt with by the factors of safety in any design.

The calculations have been carried out in accordance with the appropriate codes of practice and relevant factors of safety. It is possible that these factors of safety could be reduced (having the effect of increasing the potential retained height) as the flood load could be classed as an accidental condition. However, due to the consequences of the wall failing during a flood condition, it may not be appropriate to apply reduced factors of safety.

10.4 Initial Recommendations - Riverside Road & Arthur Street

The initial calculations carried out and summarised above assume the wall is in a very good state of repair. To ensure the walls are suitable to act as flood walls and to ensure the longevity of the walls, regular maintenance works will be required.

We would recommend that the following maintenance works be carried out:

- All vegetation growing through or over the wall should be removed or appropriately treated to stop further growth. Vegetation growing through a wall can cause significant damage and allow water and dampness to penetrate the wall, both of which can significantly reduce the working life of the wall;
- All areas of eroded or damaged mortar or stone work should be repaired or replaced with appropriate materials. A specialist stonework contractor should be consulted for the extent of works required;
- A suitable cope should be added to the full extent of the wall to stop water penetration to the wall structure; and
- The section of wall identified that is not adequate to act as a flood wall should be removed and replaced with a new suitably designed wall. As the existing wall does not show any significant signs of distress or movement from the retention, the rebuilt section of the wall could likely be limited to the upper stem. The wall replacement would likely take the form of a concrete foundation cast 300-500 mm below finished ground levels (on the high side) with a suitably designed stone wall constructed above.

Suitable intrusive investigations should be carried out to confirm material strength and assumptions made.

11 Selected Options for Further Consideration

11.1 Options Summary

A number of potential flood mitigation options have been assessed for their effectiveness for reducing flooding risk in Newton Stewart. The options considered were:

a) Provision of upstream storage to attenuate peak flows at Newton Stewart;
b) Direct defences to protect flood risk areas;
c) Removal of gravel berm (island);
d) Increasing flow conveyance through the A75 road bridge; and
e) Raising deck level of the metal footbridge.

A potential flood storage area in the upper catchment was identified and its effectiveness was assessed using a mathematical model specifically set up for this exercise. The assessment indicated that by constructing a water retaining structure across the River Cree valley together with appropriate flow control structures, it might be possible to reduce the peak flow of a 200 year flood passing downstream by approximately 70 m^3/s. This would reduce the 200 year flood flow to a 75 year flood in Newton Stewart. A 75 year flood would still cause significant flooding in Newton Stewart. Hence, upstream storage would not, on its own, be able to provide protection against a 200 year flood event and would need to be combined with other flood mitigation options (i.e., direct defences). In addition, the assessment considered the feasibility of providing flood storage upstream of Newton Stewart only and did not consider potential negative environmental and other social impacts of such an option.

Direct defences in the form of flood walls and flood embankments can provide an effective mitigation option. However, in some places the height of defences required to provide protection up to the 1 in 200 year event could be as high as 2.3 m. The average height of defences along Riverside Road would be 1.5 m for a 1 in 200 year event. Lower defence heights would provide protection for return period events less than 1 in 200 years, e.g., up to 1 m high defences would be able to provide approximately 10 year protection.

Removal of the gravel berm (island) in the middle of Newton Stewart would reduce peak water levels immediately upstream by up to 0.1 m, but this is not significant when compared to flows in the river and the rise in water level at the location during an extreme event.

Increasing flow conveyance through the A75 road bridge would locally lower the water immediately upstream of the bridge, but would have little effect upstream of the metal footbridge where most of the flooding affecting properties occur.

Raising the deck of the metal footbridge would reduce peak water levels upstream for flood events in excess of 5 year return period. The peak water level immediately upstream of the bridge could be reduced by up to 0.45 m during a 200 year event, reducing to zero at the weir (350 m upstream of the bridge). This would provide some benefit in reducing the depth of flooding in the car park and along Riverside Road.

11.2 Preferred Option

The high level assessment of possible flood mitigation options considered in the study indicated that the direct defence option would be able to provide 200 year level of protection, but with a lower benefit-cost ratio than unity. Although the upstream storage option would significantly reduce peak flows at Newton Stewart, it has a lower benefit-cost ratio than direct defences and it does not provide the desired level of protection without being combined with direct defences. With such low benefit-cost ratios, these two schemes would unlikely attract grant funding from the Scottish Government.

A positive benefit-cost ratio was obtained for a direct defence scheme providing approximately 10 year level of protection. This benefit-cost ratio could be further increased by including intangibles, environmental benefits, and social and health benefits in the analysis and may be suitable for attracting grant aid from the Scottish Government.

Based on the above, it appears that a flood management scheme providing lower than 200 year level of service, perhaps of the order of 10 year, combined with property level protection appears to be the most economically viable option and merits further consideration. A scheme designed for say 10 year level of service and including a reasonable freeboard would provide protection against floods with the order of 50 year return period (without freeboard).

12 Conclusions and Recommendations

This report presents the results of a detailed flood study of Newton Stewart and Minnigaff that considers flooding risk from River Cree and Penkiln Burn. A Strategic Flood Risk Assessment undertaken by Dumfries and Galloway Council in 2007 identified Newton Stewart and Minnigaff as a priority area in Dumfries and Galloway in terms of the number of properties potentially at risk of flooding from a 1 in 200 year flood event. Following this report a flood event in November 2012 in Newton Stewart and Minnigaff was the largest flood in recent memory. A small flood event also affected the towns in winter 2013.

The flood study undertook a detailed hydrological assessment for the River Cree, developed a linked 1D/2D flood model of the river through the two settlements, produced flood inundation maps for a range of return period flood events, assessed a range of possible flood alleviation measures and presented an initial cost-benefit analysis for the preferred flood alleviation options.

The 1D/2D mathematical models of River Cree and Penkiln Burn were calibrated against recorded flood level and flood extent information from the November 2012 event. The modelled flood extent matched well with the observed data. The River Cree model was also calibrated against observed SEPA data from their gauged site within Newton Stewart. Again the model provided a good fit with the observed data.

The calibrated model was used to simulate inundation during floods with a range of return periods (2, 5, 10, 25, 50, 100, 200, 200 plus climate change, and 1000 year return periods). Flood maps were prepared for each event.

The model results predicted that 142 properties would be affected during a 200 year flood, almost equally split between residential and non-residential properties. For a 2 year flood event, 3 residential properties were predicted to flood. The number of properties predicted to flood is significantly smaller than indicated in previous studies, due to the improved methods and datasets used in the current assessment.

A number of flood mitigation options were considered, including; flood storage upstream of Newton Stewart; direct defences where flood risk areas could be protected by flood walls and embankments; removal of a gravel berm (island) just downstream of Bridge of Cree; increasing the flow passing capacity of the A75 road bridge; and raising the deck of the metal footbridge towards the south end of Newton Stewart.

Modelling work indicated that the removal of gravel berm, increasing flow capacity of the A75 bridge and raising the deck level of the metal footbridge had local effects on peak water levels only. Either individually or collectively these flood alleviation options would not significantly reduce flooding risk to urban areas. However, they could be considered as part of a wider scheme.

A potential large upstream flood storage area was identified in the River Cree valley and its effectiveness was assessed using the mathematical model. The storage area was shown to be technically able to reduce peak flows passing downstream by up to 70 m^3/s, resulting in a 200 year flow being reduced to 75 year flow in Newton Stewart. However, the storage area option was not predicted to be able to reduce 200 year flows sufficiently to prevent widespread flooding in the urban areas of Minnigaff and Newton Stewart. This option, if considered, would need to be combined with direct defence option to provide protection against a 200 year flood.

It was calculated that a total 1.65 km of flood walls and 0.25 km of flood embankments would be required to protect all the flood risk areas in Newton Stewart and Minnigaff for a 200 year flood. Wall heights would generally be up to 1.5 - 2 m high (above existing ground level), except at Reid Place where the required height including freeboard would be approximately 2.3 m. Options were also considered for defence schemes that provided lower levels of protection. In comparison, a scheme which would provide 10 year level of protection would require defence heights on average up to 1m only.

An initial cost-benefit analysis was undertaken, based on the model results and conceptual level flood alleviation options. Hence, the cost-benefit analysis should be considered as initial only, with a high degree of uncertainty. A bias factor of 60% was added to cost estimates for the flood defence schemes as per standard practise for initial cost-benefit analyses.

The conclusions of the cost-benefit analysis were that the benefit-cost ratios for schemes with flood storage areas and with flood defence walls protecting properties up to the 1 in 200 year or 1 in 50 year level would be less than 1. This indicates that these schemes would not be economically viable. However, for a scheme with flood walls to provide a 1 in 10 year level of protection the estimated benefit-cost ratio was greater than 1. This indicates that such a scheme would be economically feasible and may be suitable to attract grant aid from Scottish Government.

Based on the outline cost-benefit analysis undertaken, a scheme consisting of direct defences (combined with property level protection where appropriate) and providing an approximately 1 in 10 year level of protection would appear technically and economically feasible and worth further consideration.

Appendix A – Site Photographs

Photo – 3: River Cree – Short distance upstream of study area

Photo – 4: River Cree – Downstream view taken from suspension footbridge

Photo – 5: River Cree – Downstream view taken a short distance upstream of the Main Cree Bridge

Photo – 6: River Cree – Upstream view of Main Bridge over Cree and masonry weir including western

gravel berm.

Photo – 7: River Cree – View of masonry weir and gravel berm taken from Main Cree Bridge.

Photo – 8: River Cree – Upstream view of gravel berm taken from eastern bank of main channel.

Photo – 9: River Cree – Downstream view taken a short distance upstream of the Main Cree Bridge.

Photo – 10: River Cree – Upstream view of Main Bridge over Cree and masonry weir.

Photo – 11: Riverside Road – Upstream view taken a short distance upstream of the metal footbridge.

Photo – 12: River Cree – Upstream view taken a short distance upstream of the metal footbridge.

Photo – 13: River Cree – Upstream view of metal footbridge

Photo – 14: River Cree – Upstream of A75 bridge

Photo – 15: River Cree – Upstream view of A75 road bridge

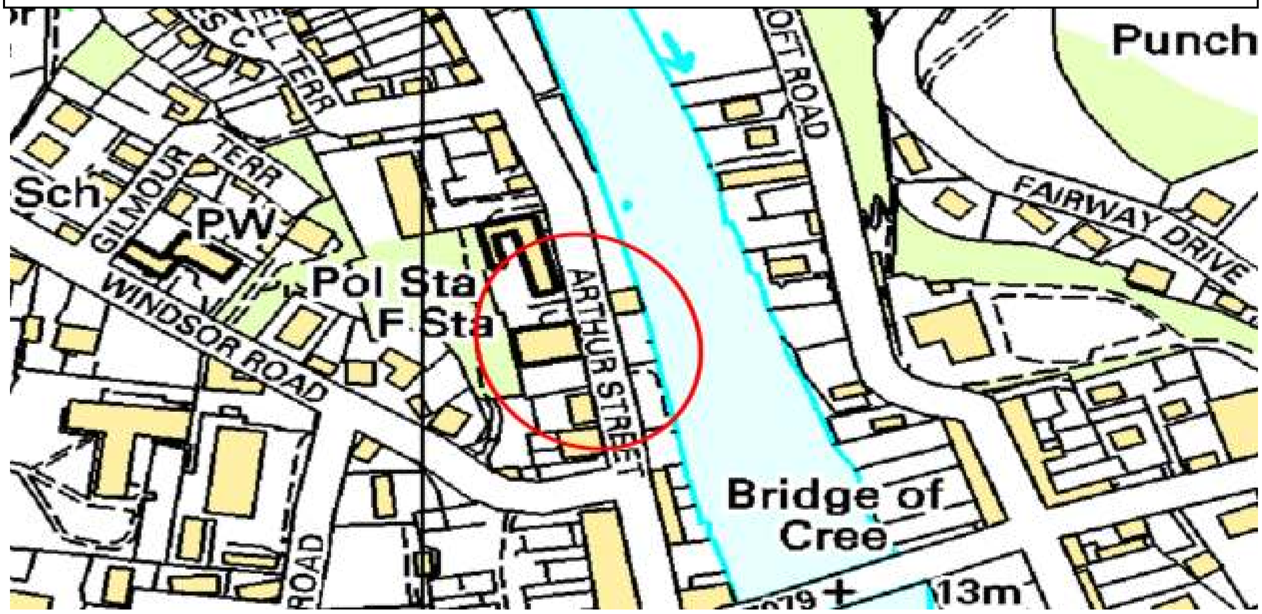
Photo – 16: Penkiln Burn – Upstream of concrete road bridge

Photo – 17: Penkiln Burn – Concrete bridge

Photo – 18: River Cree – Confluence with Penkiln Burn

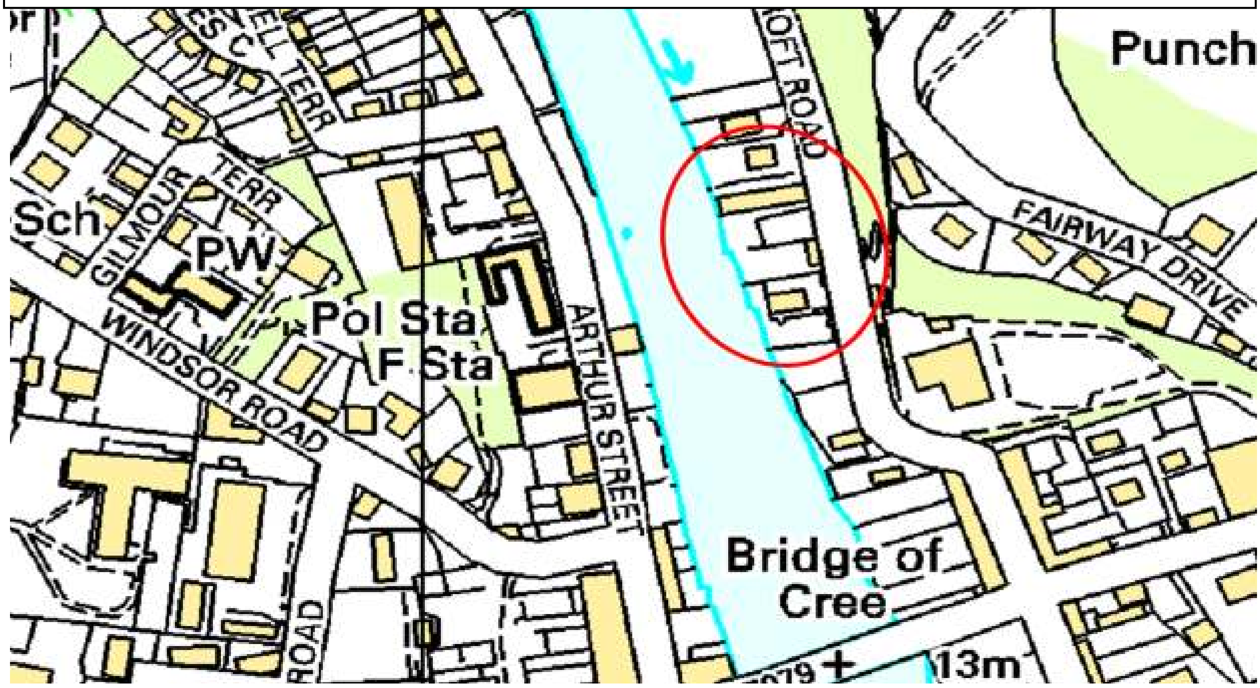
Appendix B – Photographs from Nov 2012 event

Photo – 19: Arthur Street Flooding (source of photo unknown)



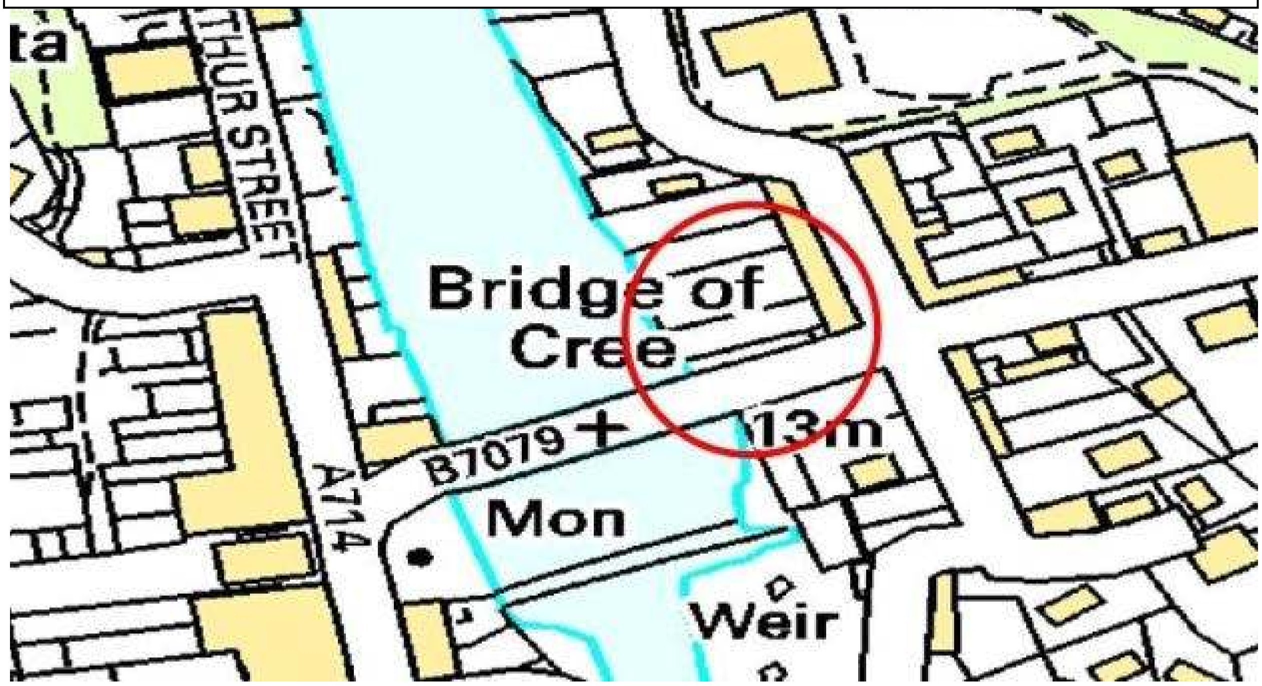
River Cree – Upstream of Bridge of Cree

Photo – 20: Millcroft Road Flooding (Courtesy of Lewis McCallum sourced from www.itv.com)



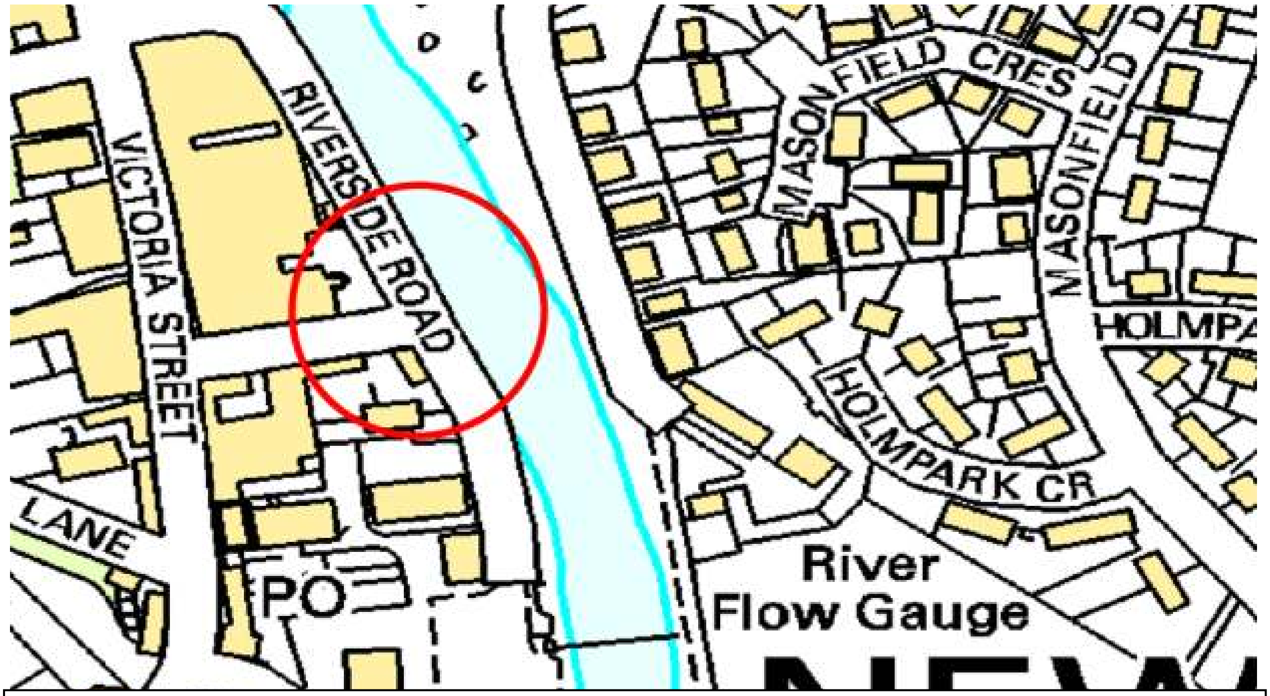
River Cree – Confluence with Penkiln Burn

Photo – 21: Flooding upstream of Main Bridge of Cree (Source of photo unknown)



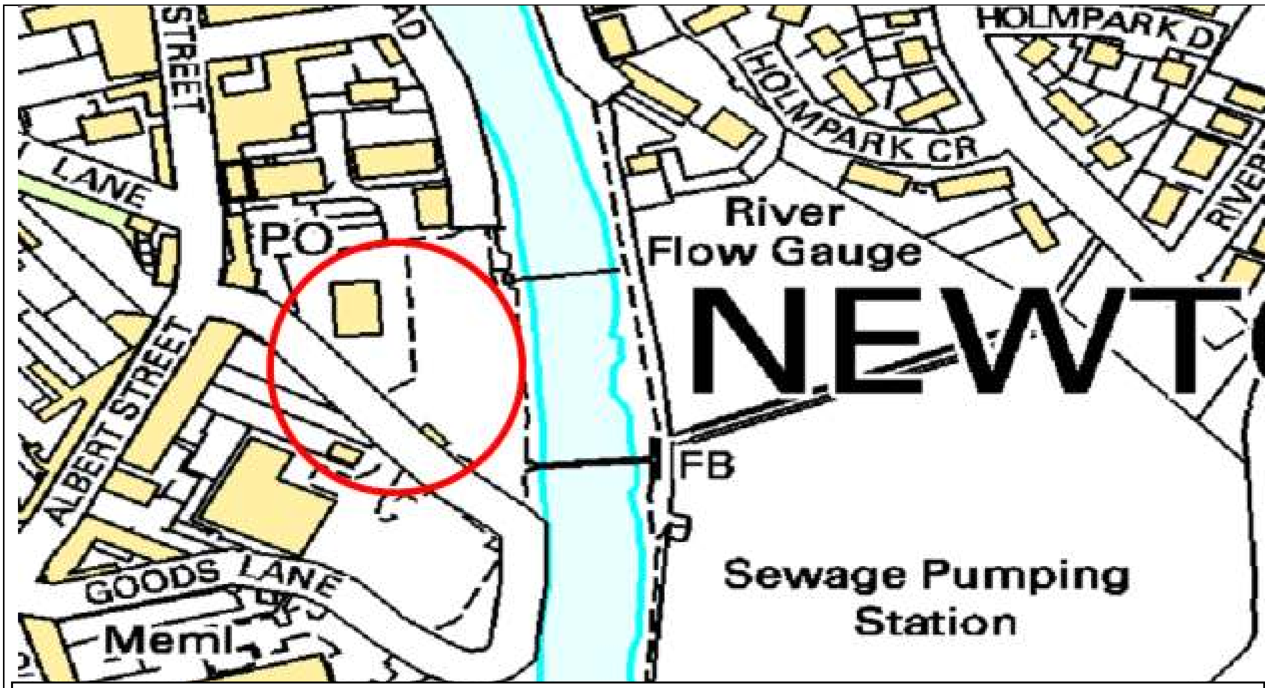
River Cree – Upstream of Bridge of Cree

Photo – 22: Flooding at Riverside Road (Courtesy of “jimzvidsz1” youtube.com)



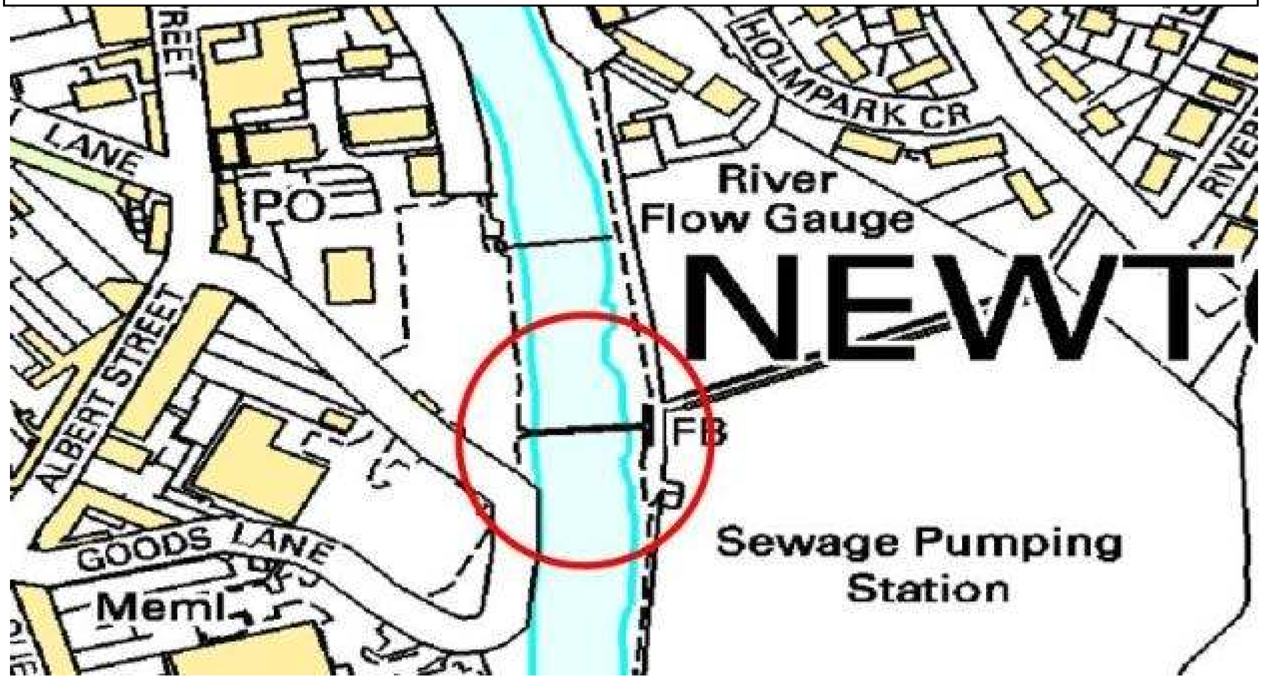
River Cree – Riverside Road

Photo – 23: Flooding at Riverside Road Car Park (Source of photo unknown)



River Cree – Riverside Road Car Park

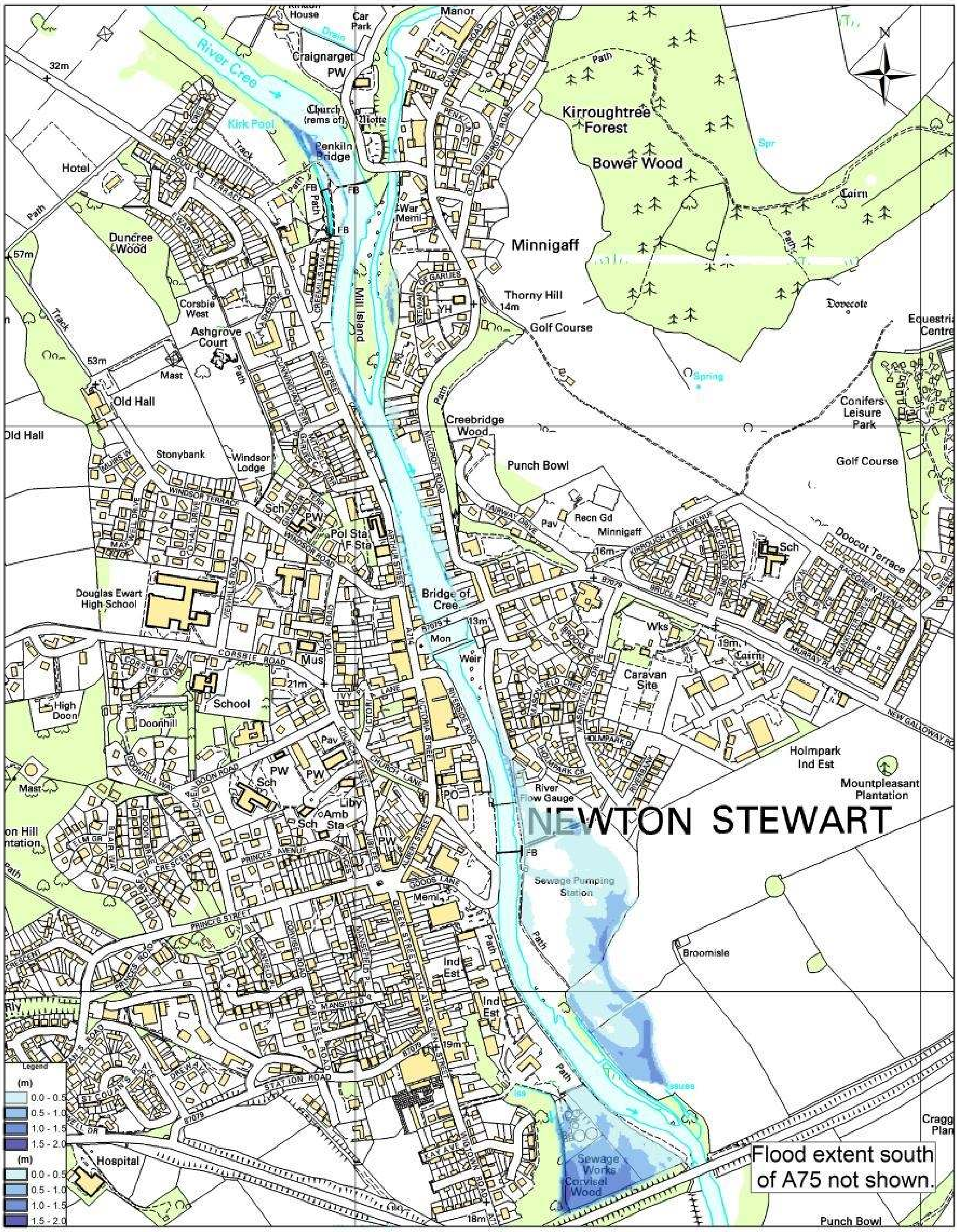
Photo – 24: Flooding at metal footbridge (Courtesy of Mark McKie via flikr.com)



River Cree – Riverside Road Car park

Appendix C – Flood Maps

Figure 36: 2 Year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
2 Year Return Period

0 50 100 200 300 400 500 Meters

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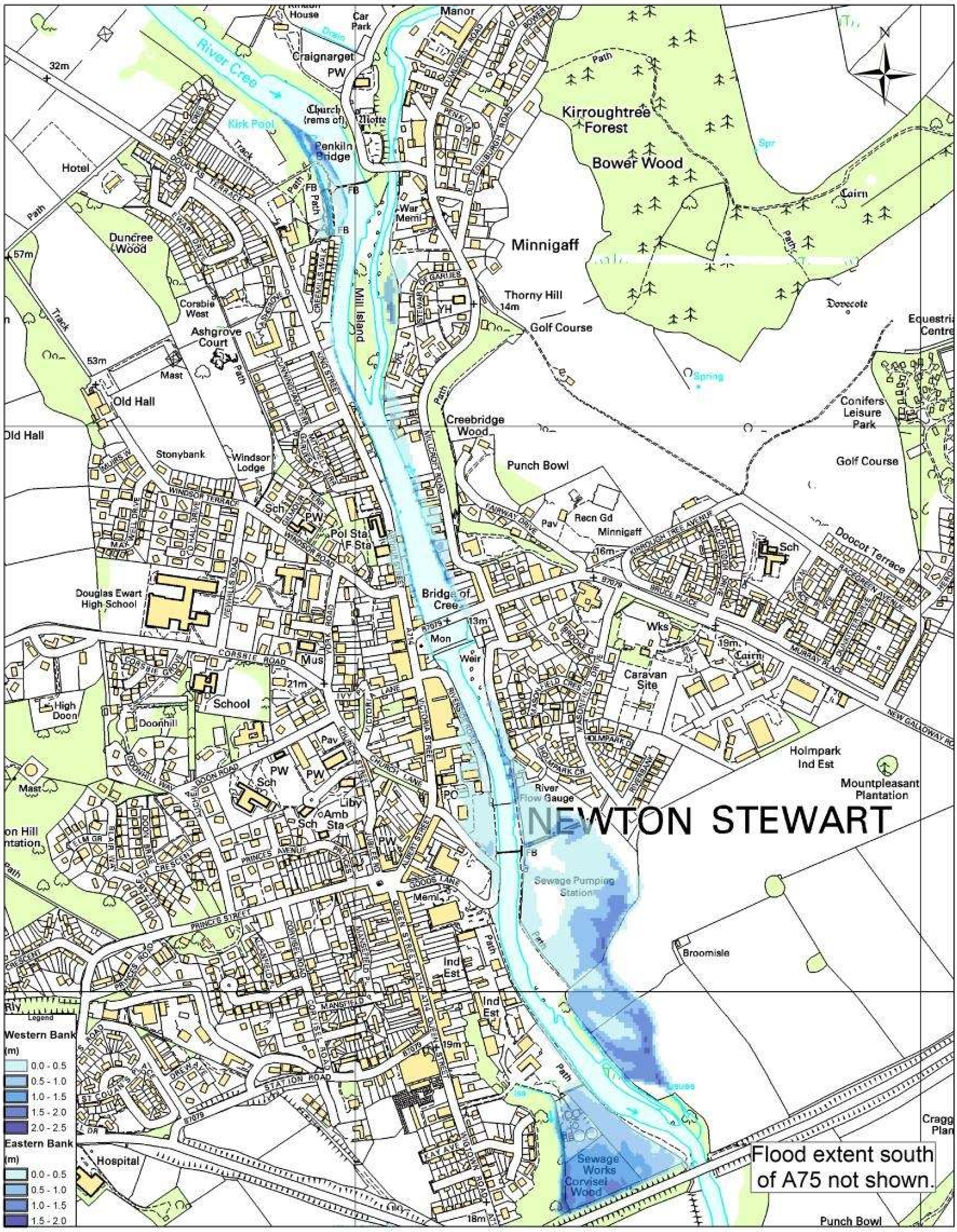
Strathclyde Business Park, Bellshill
North Lanarkshire, ML4 3NJ.
Scotland, U.K.

Scale 1:6,000

www.kayaconsulting.co.uk

Figure 37: 5 Year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
5 Year Return Period

0 50 100 200 300 400 500 Meters

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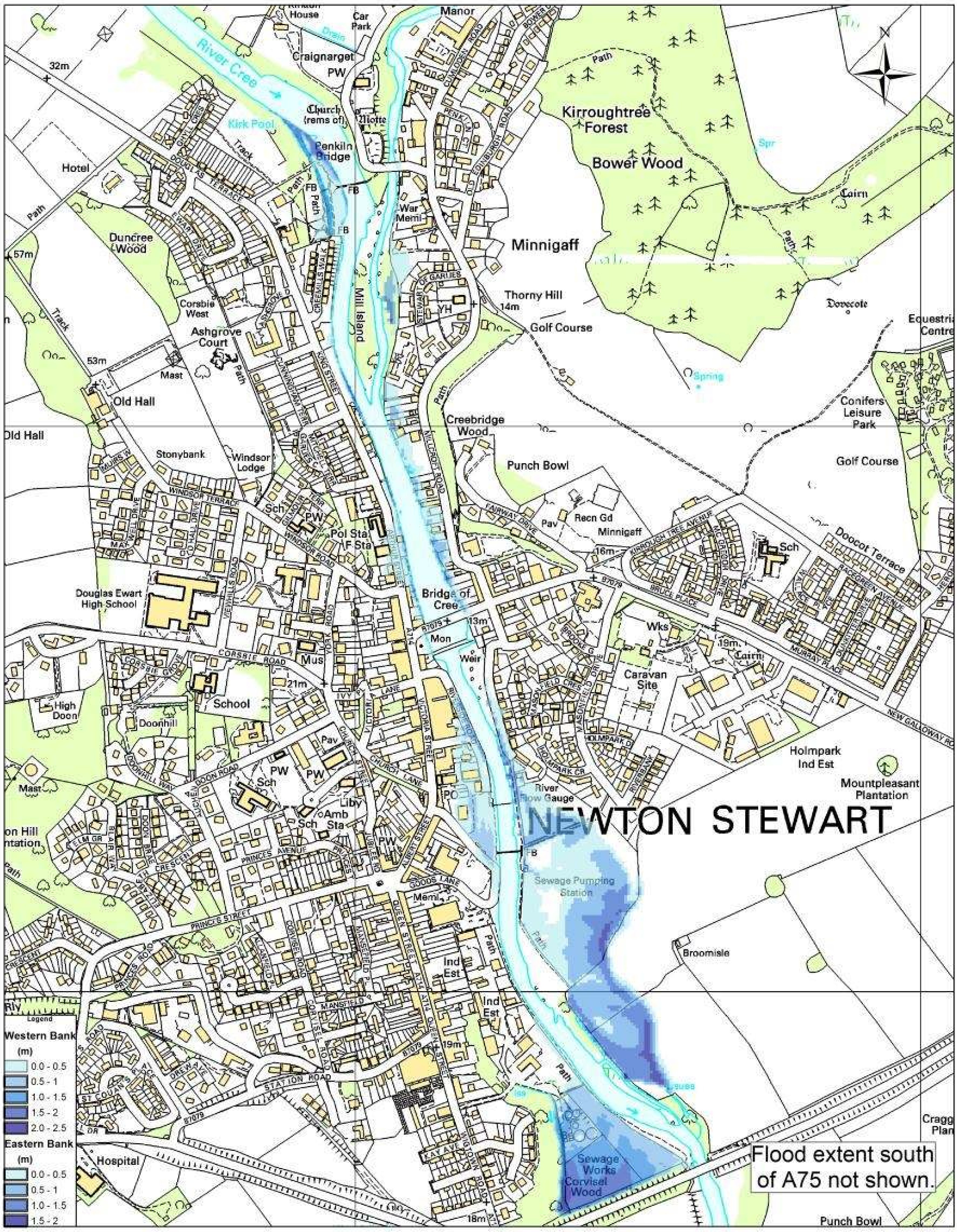
Figure 38: 10 year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
10 year Return Period

0 50 100 200 300 400 500 Meters

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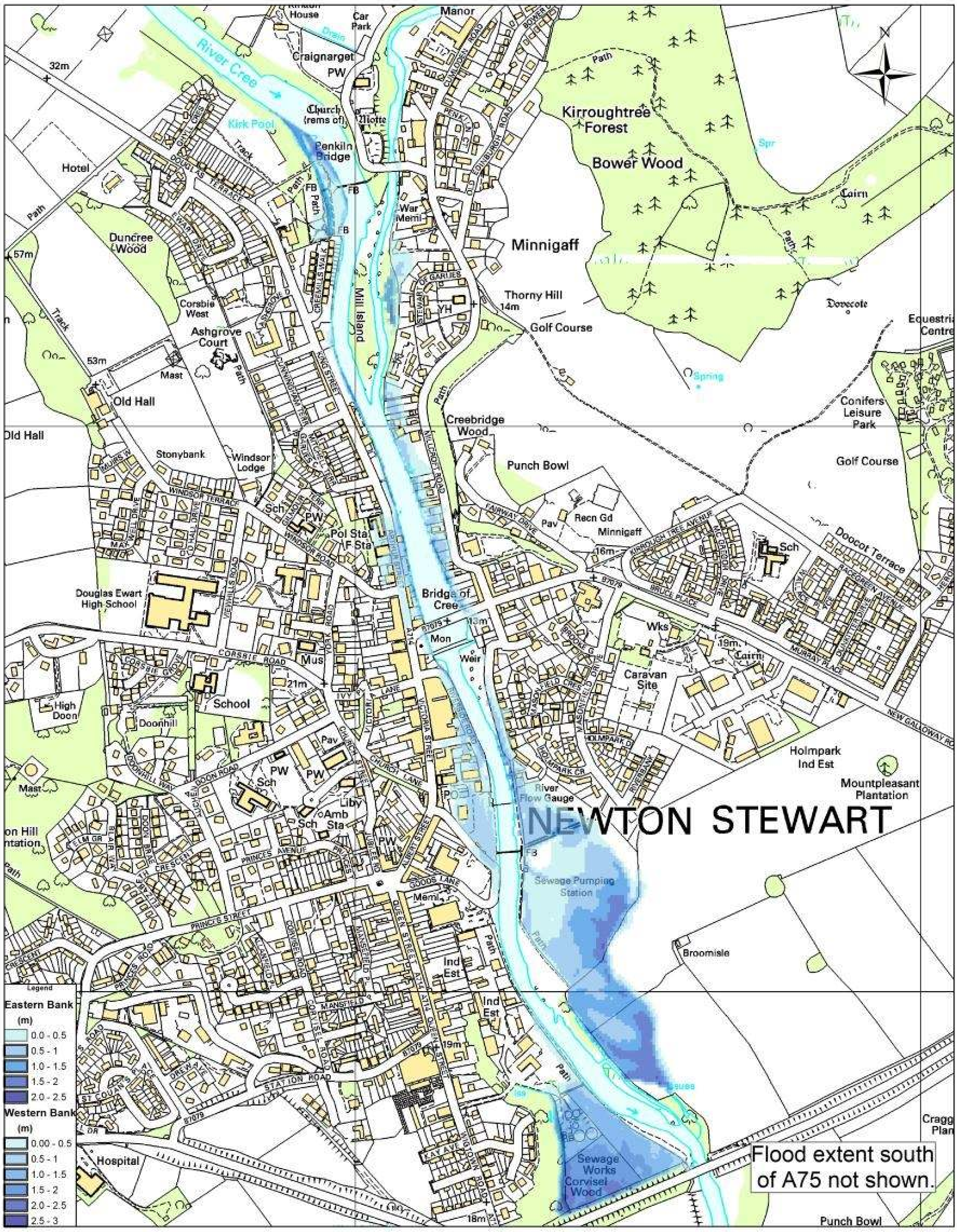
Figure 39: 25 year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
25 Year Return Period

0 50 100 200 300 400 500 Meters

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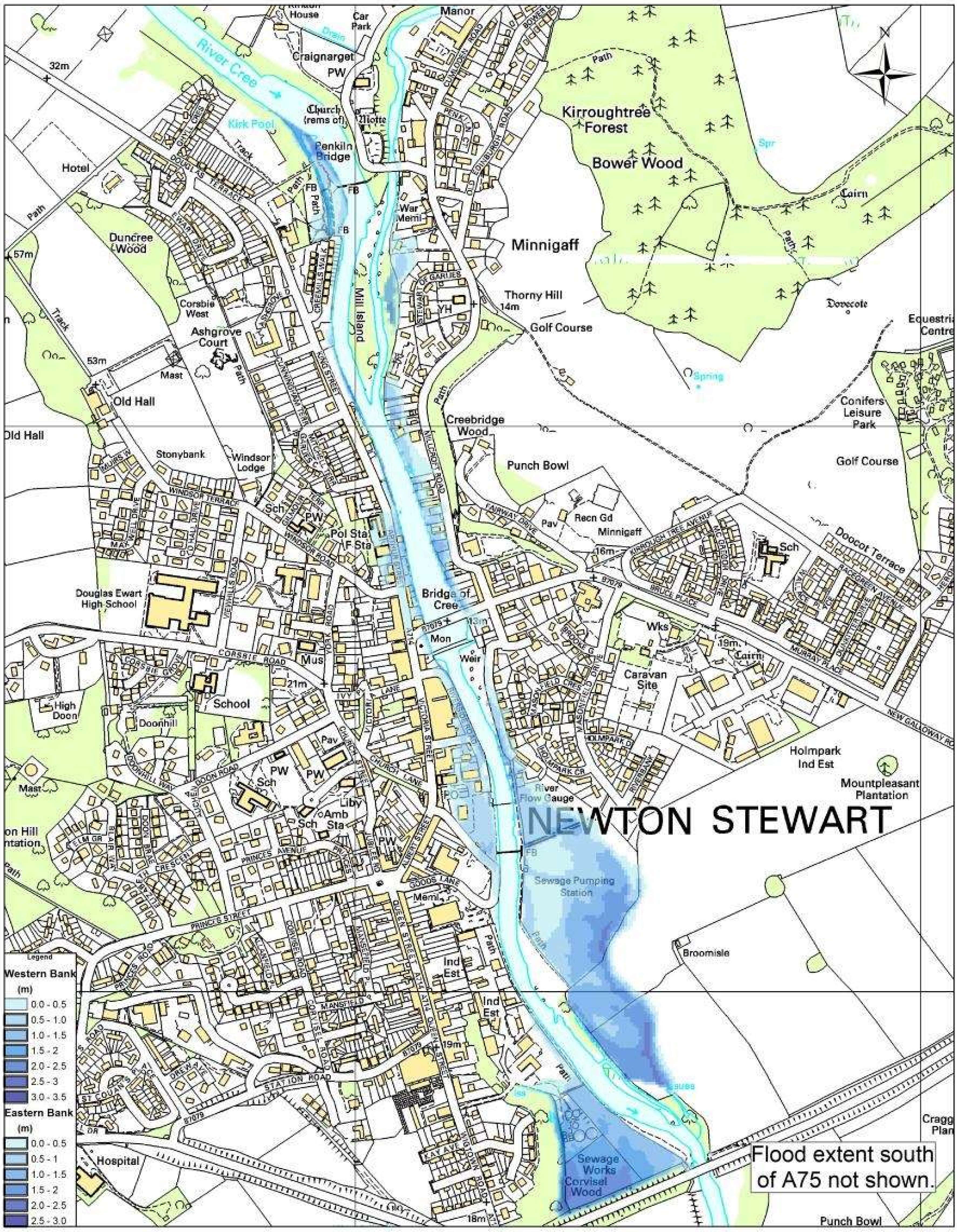
Figure 40: 50 year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
50 Year Return Period

0 50 100 200 300 400 500 Meters

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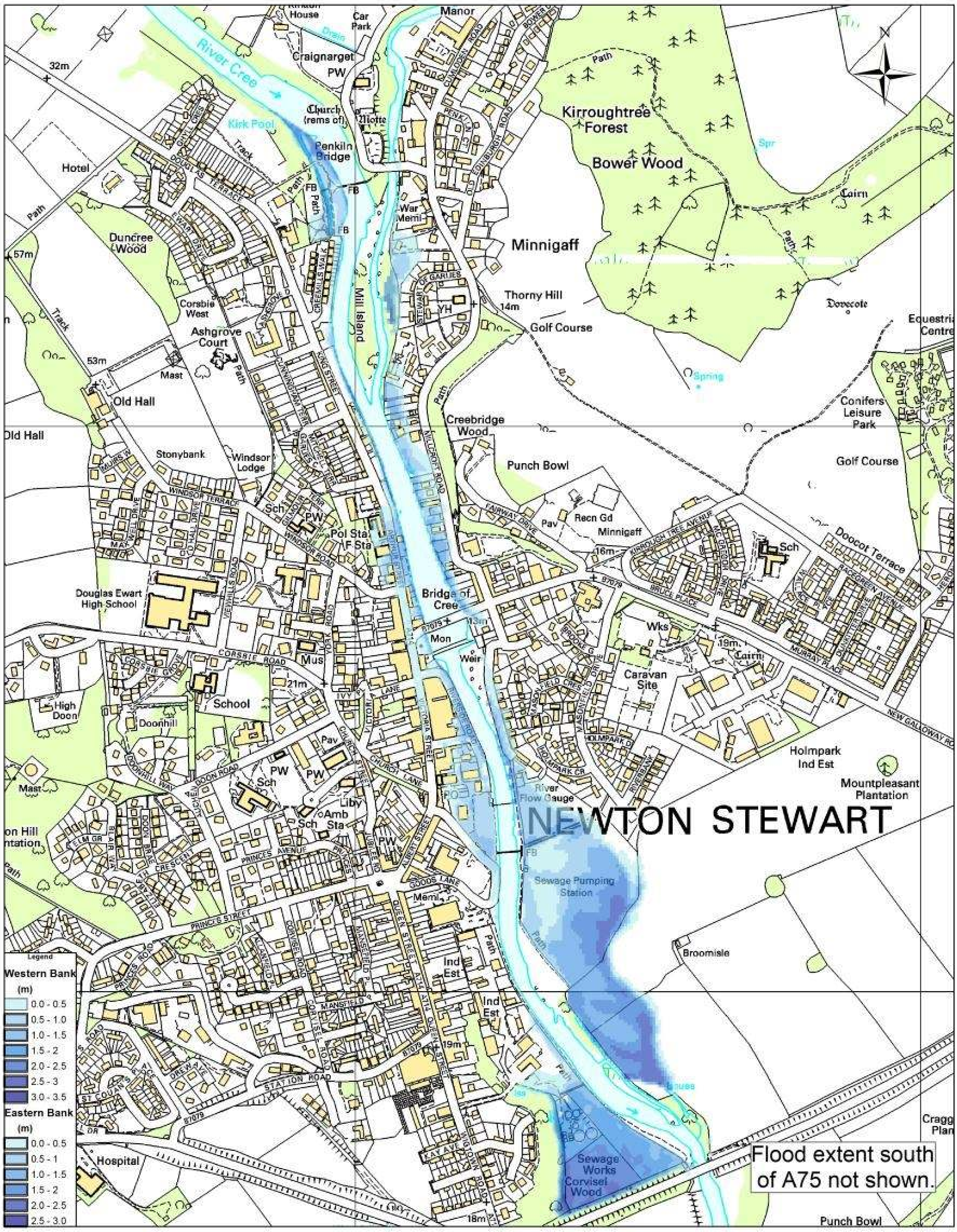
Figure 41: 100 year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
100 Year Return Period

0 50 100 200 300 400 500 Meters

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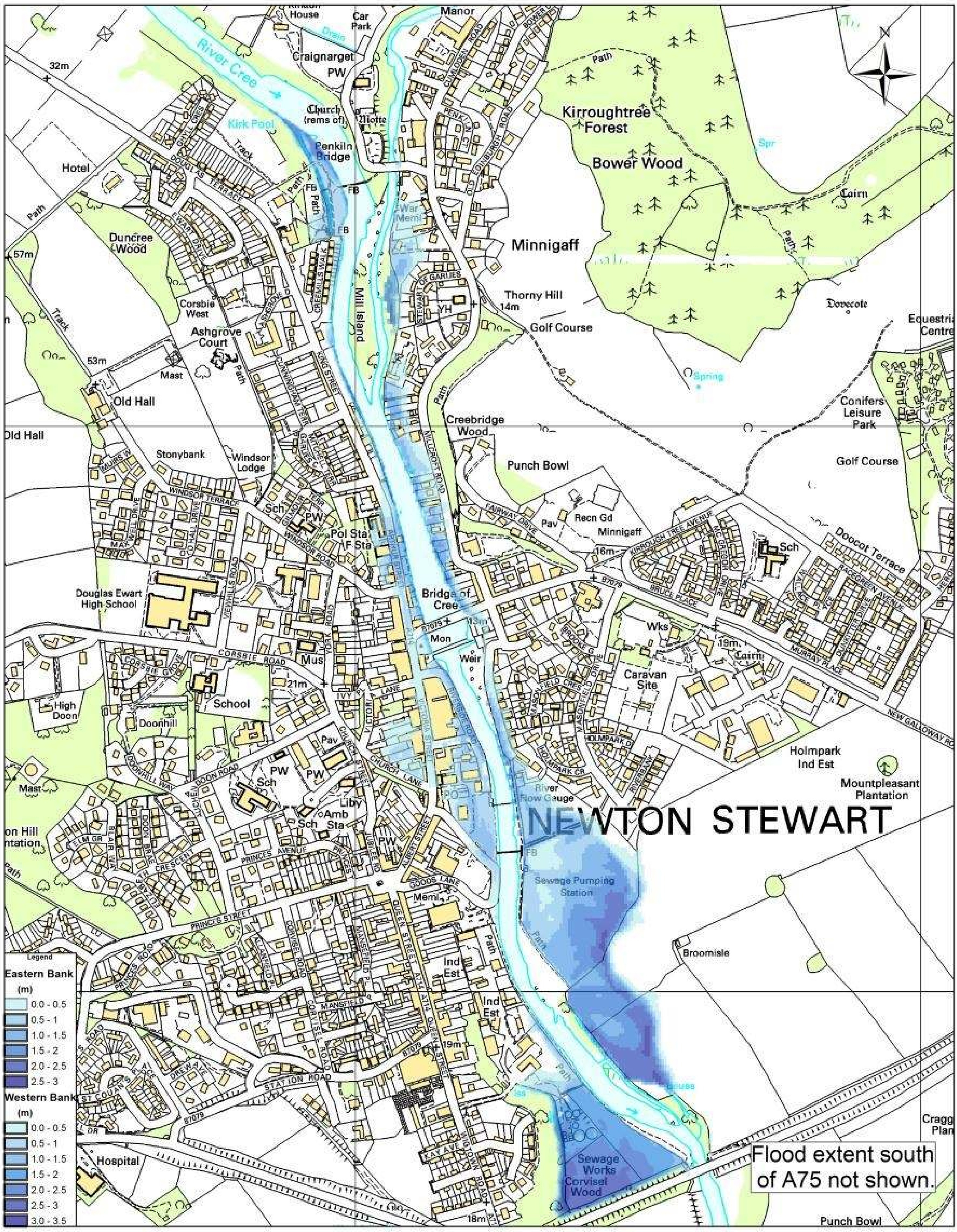
Figure 42: 200 year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
200 Year Return Period

0 50 100 200 300 400 500 Meters

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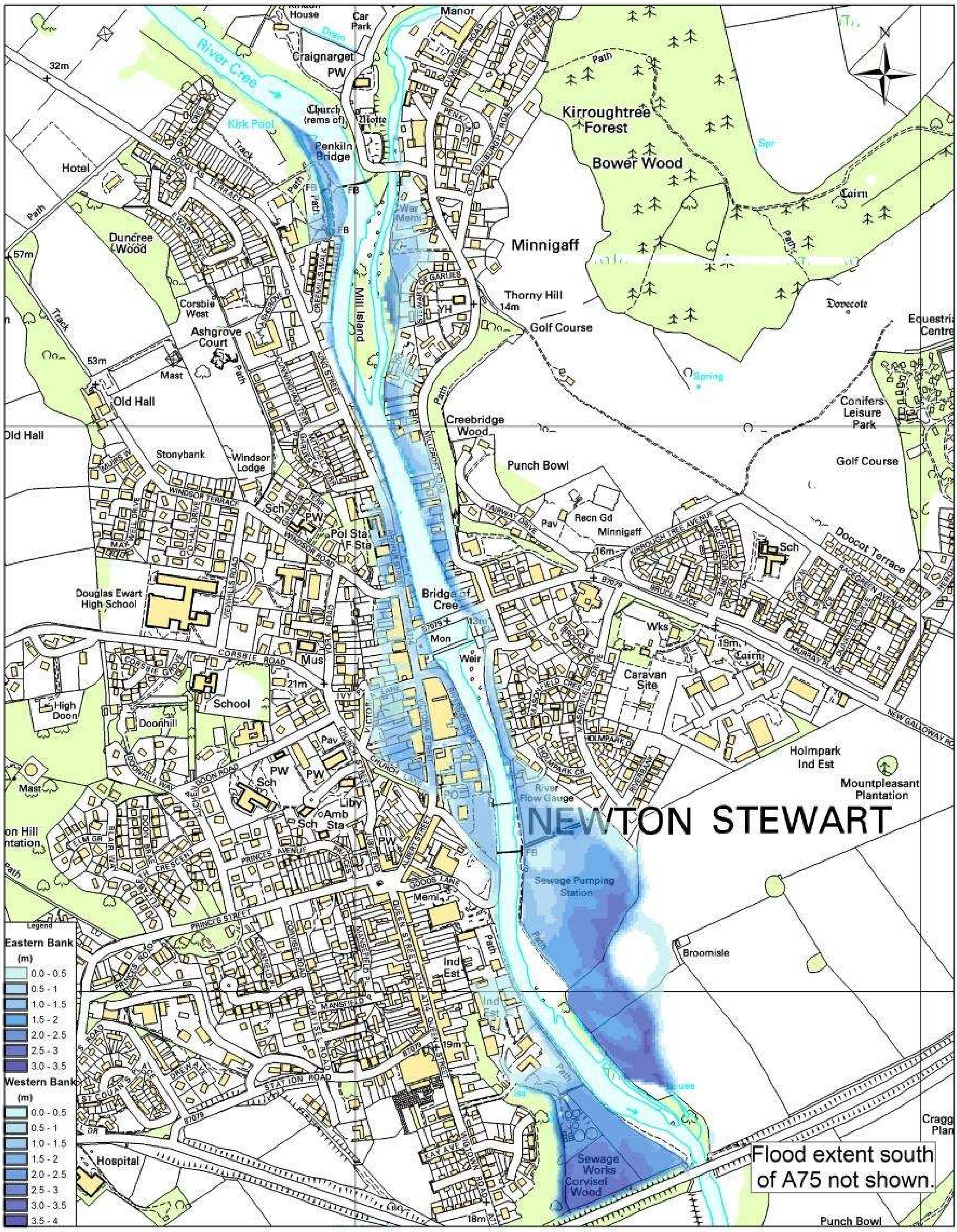
Figure 43: 200 year plus Climate Change Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
200 year plus Climate Change Return Period

0 50 100 200 300 400 500 Meters

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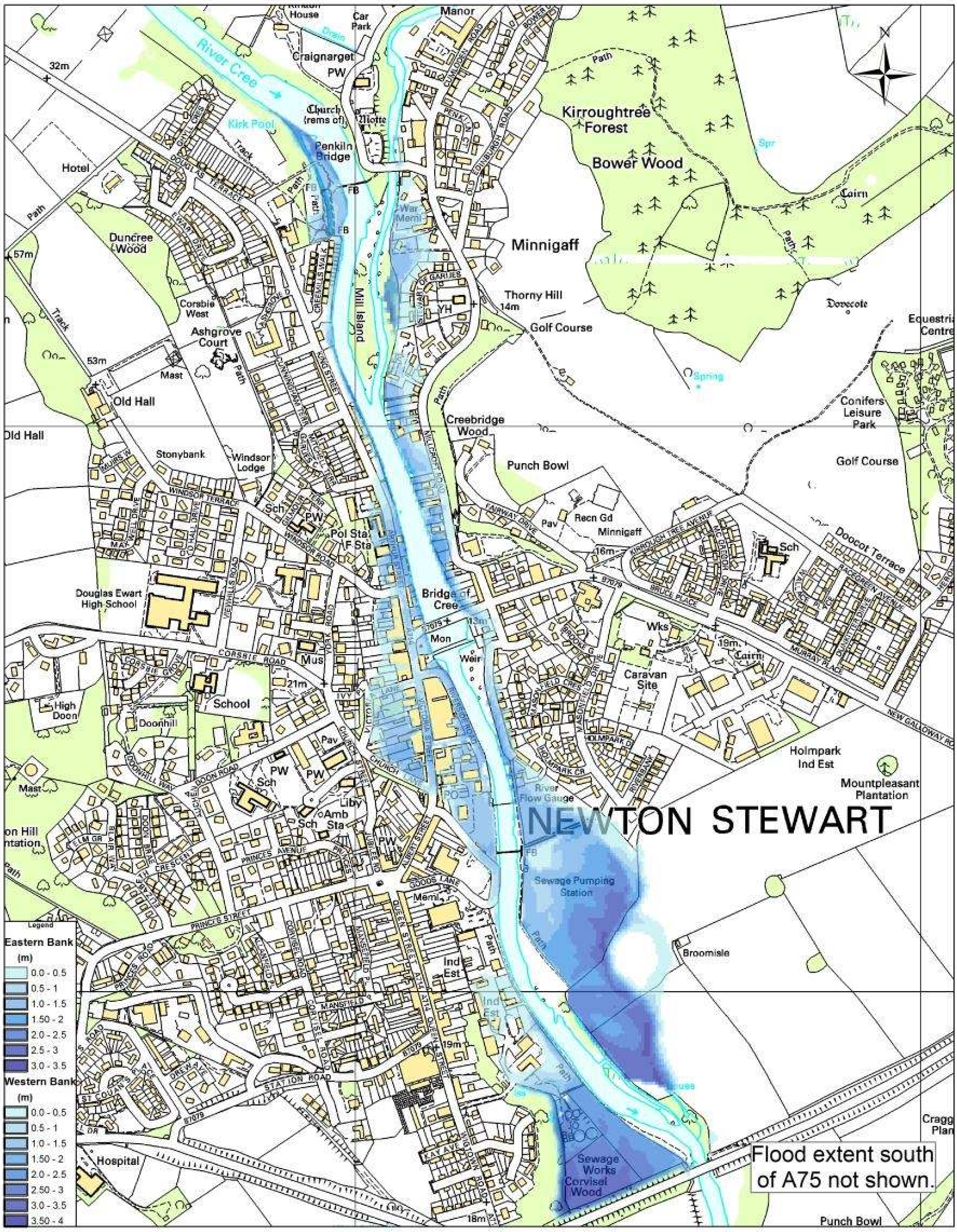
Figure 44: 1000 Year Flood Map



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Newton Stewart Flood Study
Flood Inundation Mapping
1000 Year Return Period

0 50 100 200 300 400 500 Meters

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Scale 1:6,000

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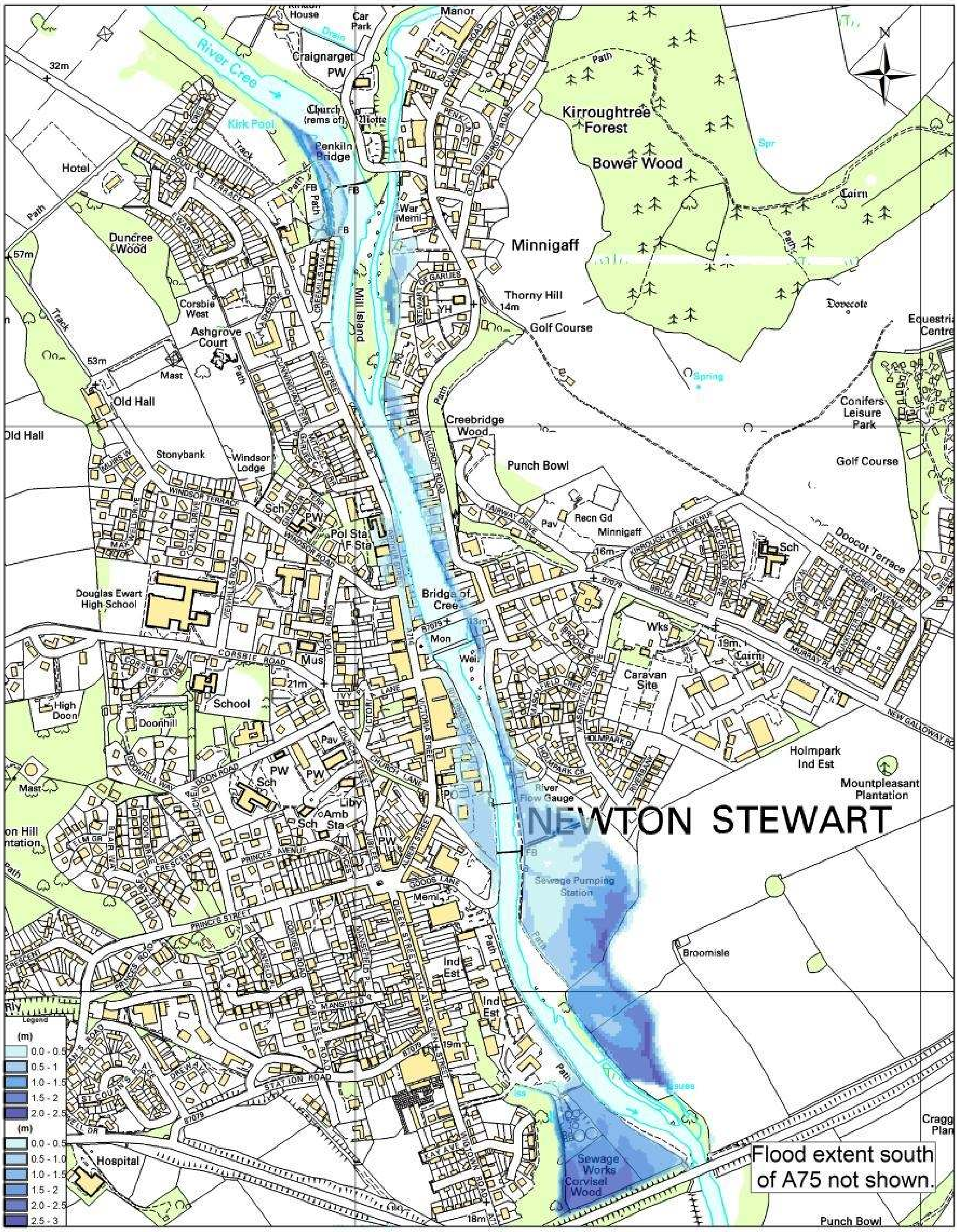
Figure 45: Predicted 2012 Event Flood Extent



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Newton Stewart Flood Study
Flood Inundation Mapping
Estimated 2012 Event

0 50 100 200 300 400 500 Meters

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Scotland, U.K.

Scale 1:6,000

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Appendix D – Review of Historical Mapping at Gravel Berm Location

Figure 46: 1846: 6 inch to mile 1st edition, 1843-1882

Figure 47: 1894: 25 inch to mile 2nd edition, 1892-1949

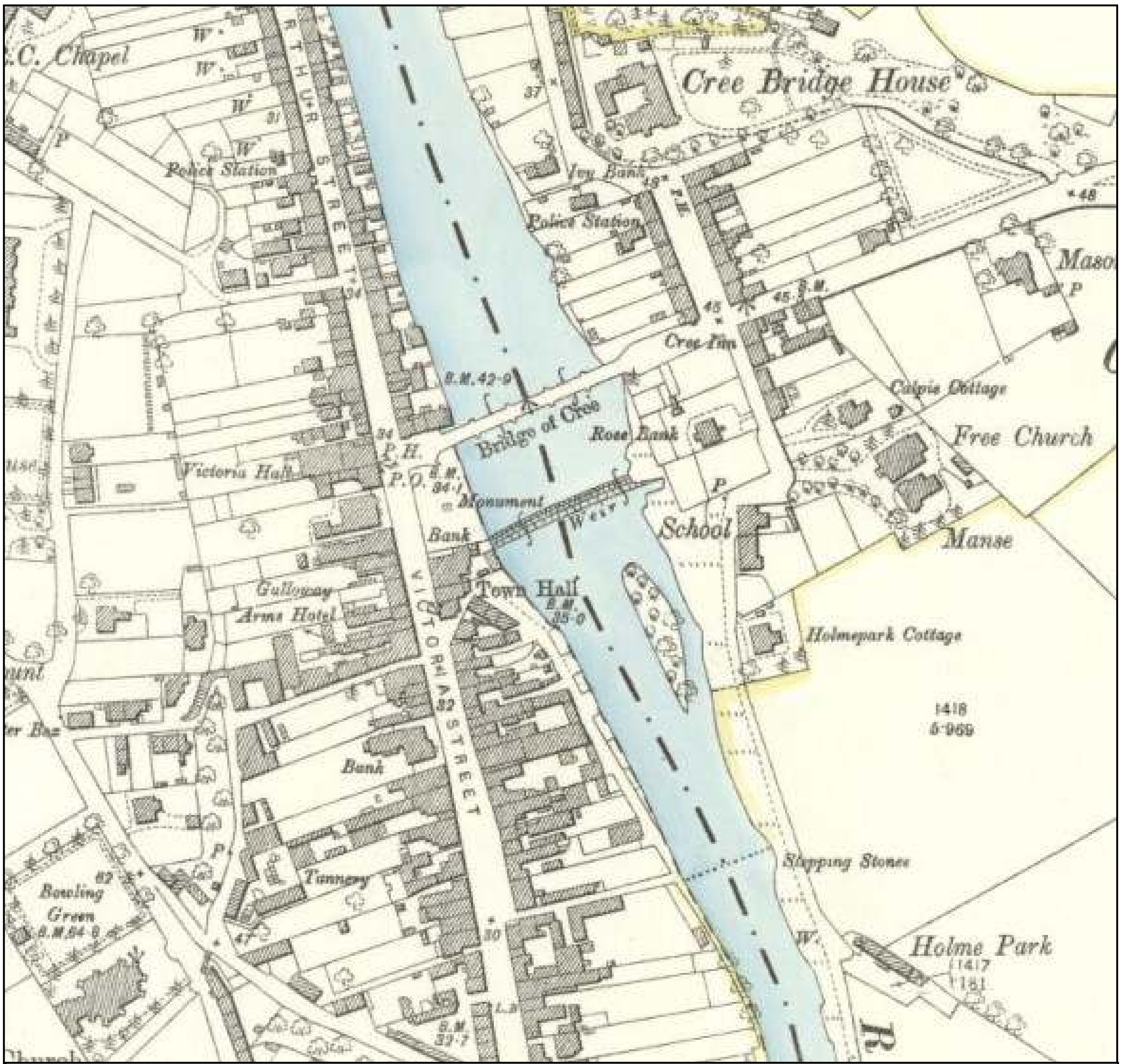

Figure 48: 1907: 6 inch to mile 2nd edition, 1892-1960

Figure 49: 1938: 6 inch to mile 2nd edition, 1892-1960

Figure 50: 1950: Aerial Photograph Mosaic, 1944-1950

Figure 51: 1953: 1:25,000 Map 1937-1961

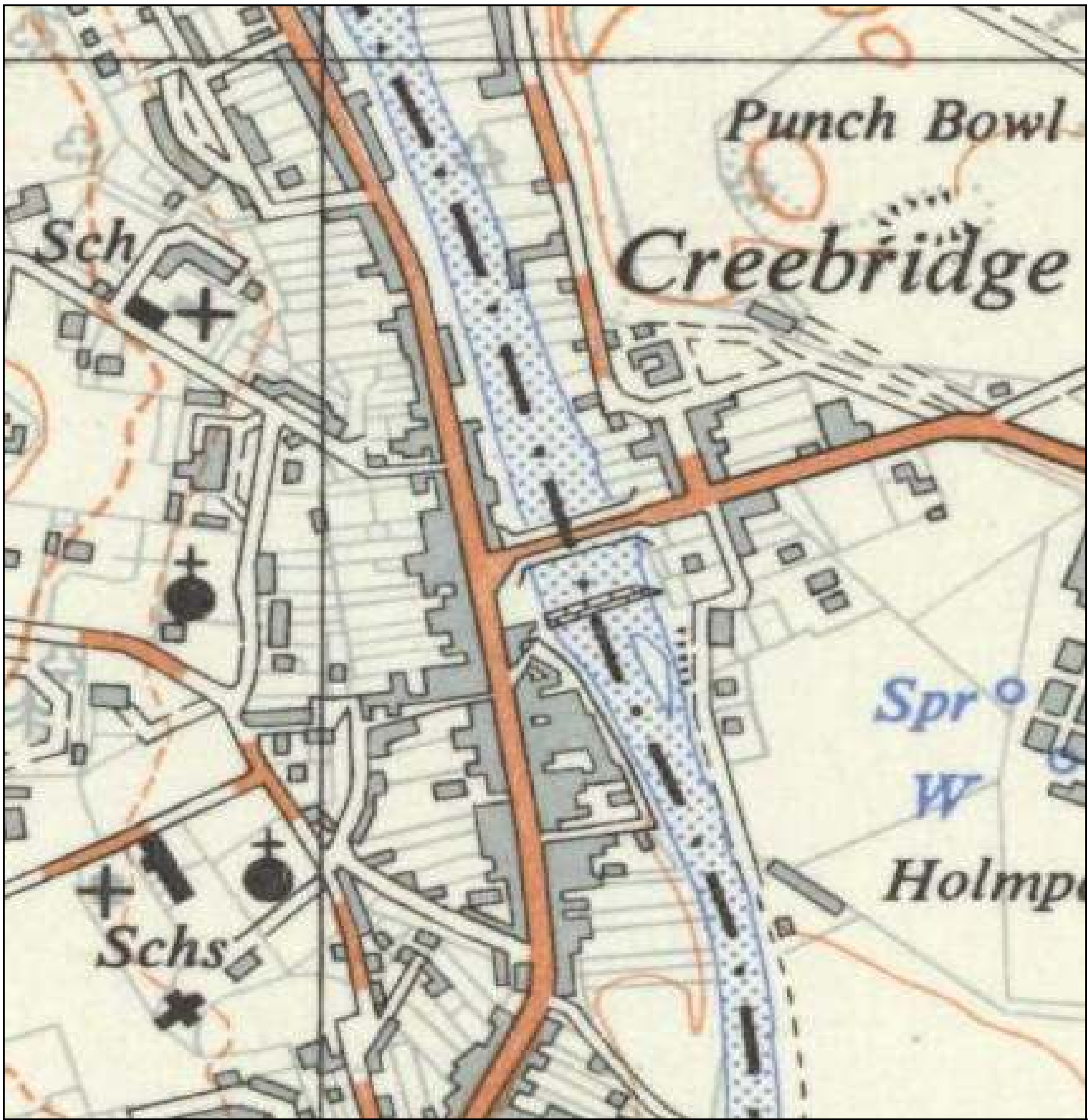

Figure 52: 2009 : 1:10,000 Ordnance Survey Map