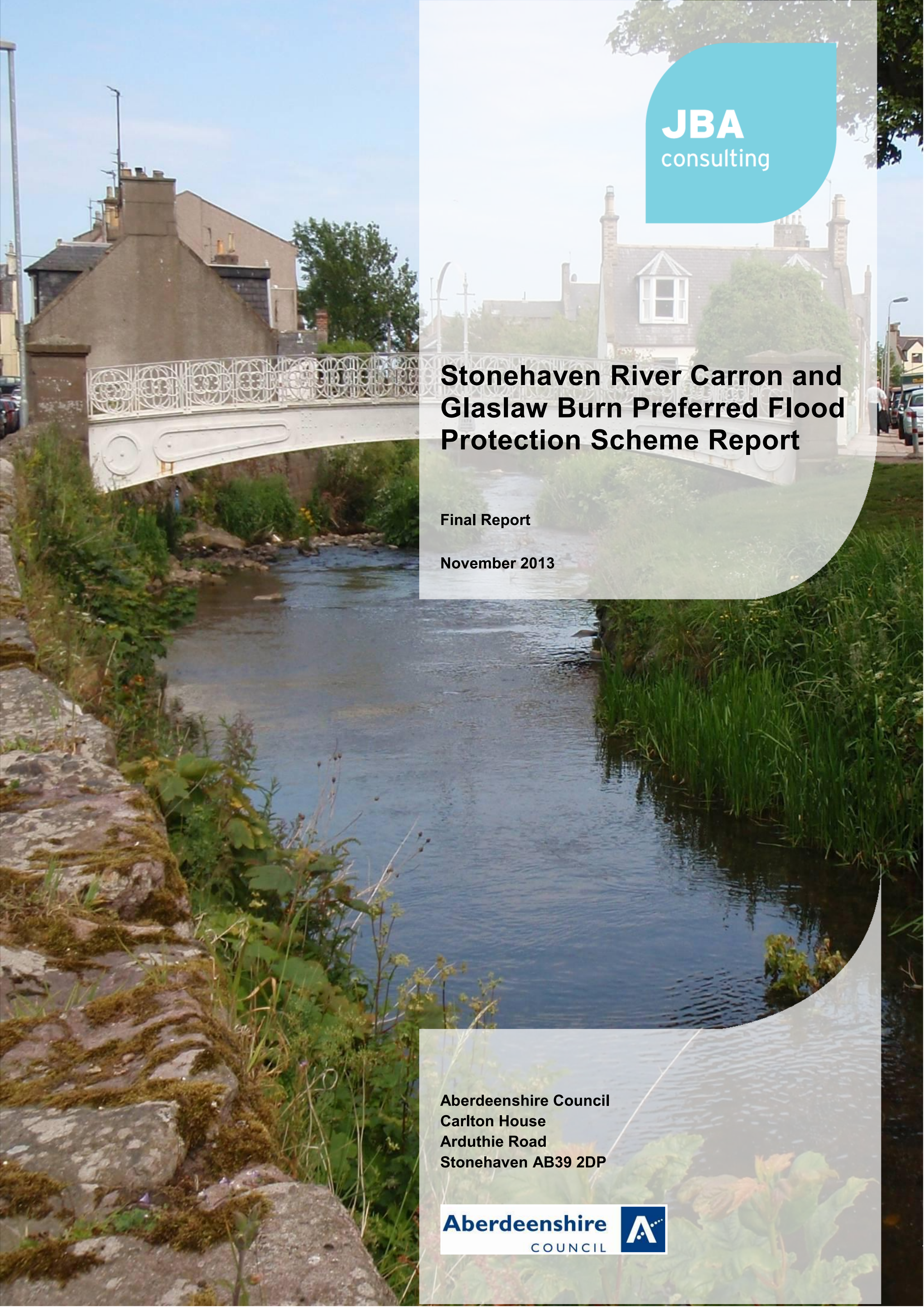

JBA consulting

Stonehaven River Carron and Glaslaw Burn Preferred Flood Protection Scheme Report

Final Report

November 2013

Aberdeenshire Council
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Revision History

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Draft 10 July 2013		Rachel Kennedy
Final 25 November 2013	General corrections Revision to sections regarding upstream storage Addition of feasibility of upstream storage on Glaslaw Burn Revision to costs Inclusion of Option 3: Combined storage and direct defences	Rachel Kennedy
Final 20 February 2014	Revision to section 4.3.4 to better explain the analysis undertaken	Rachel Kennedy

Contract

This report describes work commissioned by Willie Murdoch, on behalf of Aberdeenshire Council, by a letter dated 14 March 2013. Aberdeenshire Council's representative for the contract was Rachel Kennedy. Mark McMillan Angus Pettit and Caroline Anderton of JBA Consulting carried out this work.

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Purpose

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Acknowledgements

We would like to thank Una Thom of SEPA for supplying updated hydrometric data for the local gauges.

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Executive Summary

Serious flooding in 2009 and 2012 has caused significant damage to low lying areas of Stonehaven. In 2009 water left the River Carron and flooded areas including the High Street and Town Square. In 2012 water from the River Carron and Glaslaw Burn caused similar areas to flood with more significant damages in the High Street area. Water also flowed overland from the Bervie Braes increasing the depth of flooding to the south of the River Carron. In light of the recent flooding Aberdeenshire Council have proposed the construction of a Flood Protection Scheme which will alleviate flooding from the River Carron and Glaslaw Burn. The purpose of this report is to consider the various options for the scheme and present a preferred scheme based on hydraulic modelling and cost analysis. In 2010 a range of flood protection options were presented to the public. The information gathered from the public consultation indicated strongest support for direct defences (walls and flood embankments) through the town or storage upstream of the town. This report considers these options and linked combinations to determine a preferred scheme. The options considered for the scheme are as follows:

- Direct flood defences (Walls & Embankments)
- Direct flood defences (Walls & Embankments) with channel modifications
- Direct flood defences (Walls & Embankments) with channel modifications and bridge raising
- Upstream attenuation of flows and provision of storage
- Combination of upstream attenuation and direct defences (Walls & Embankments)

As the scheme is to be designed with a long life, the impacts of climate change (predicted by the UK Climate Predictions (UKCP09) on flood flows have been considered. An assessment was conducted to investigate the regionalised impacts of climate change in Scotland for the next century and an allowance of 33% was applied to current flow predictions to represent flood flows in 2080.

The previous analysis of flood storage related to the capacity of the upstream floodplain. This study has reviewed the proposed floodplain zones and undertaken detailed hydrological and hydraulic modelling to test the applicability of these areas. Only the area upstream of Fetteresso Bridge has an adequate area and volume to provide a significant reduction in flood flows in Stonehaven.

By implementing upstream storage the 200 year flow can be reduced to the maximum capacity of the River Carron channel upstream of the Green Bridge, however, flows cannot be reduced sufficiently to prevent out of bank flow through the lower reaches and this option would need to be augmented with direct flood defences from the White Bridge to Bridgefield Bridge. Additionally there is no viable option for providing flood storage on the Glaslaw Burn which contributes significant flows to the River Carron during flood events. Other constraints that impact on this option include the height of the impounding structure, the cost of providing this storage, the inability of this option to cope with flood flows from the Glaslaw Burn and the anticipated impacts of climate change.

The findings of this study promote the following preferred scheme in Stonehaven:

- Direct flood defences on both banks of the River Carron from immediately upstream of the Red Bridge NGR OS 386915 785636 to the coast at NGR OS 387522 785732.
- Modifications to the River Carron channel between the Green Bridge and Bridgefield Bridge removing approximately 1200m^2 of material from the channel. This will also involve the reduction of the weir downstream of Green Bridge at NGR OS 387052 785636.
- Raising of the Red Bridge by 1m from its current soffit of 9.01 mAOD to reduce its hydraulic impact and provide sufficient freeboard.
- Direct flood defences on the Glaslaw Burn from upstream of Carron Gardens to the upstream face of Low Wood Road

- Raising and relocation of the Green Bridge from its current soffit of 7.71 mAOD by 1.2m to reduce its hydraulic impact and provide sufficient freeboard.
- Raising of the White Bridge by 1.00 m to remove its hydraulic impact.
- Replacing the culvert under the Woodview Court Bridge with a box culvert with a natural bed with dimensions of approximately 4m by 2m with an invert level of 6.72 mAOD.
- Provision of pumping stations in low lying areas to alleviate surface water flooding from overland flows.
- Infilling of the parapets on Bridgefield Bridge to provide freeboard

The whole life (present value) cost of the preferred option is £16.2 million. This includes all the aspects of the preferred scheme including the necessary pumping station costs for the Arbuthnott drain and an optimism bias of 60%. The optimism bias will reduce through the detailed design stage as more understanding of the defences and more detailed cost estimates are derived.

An economic appraisal of the options has been revised to take account of the variation in costs. The preferred scheme has a benefit cost ratio of 1.25. This shows that the scheme is cost effective and economically worthwhile.

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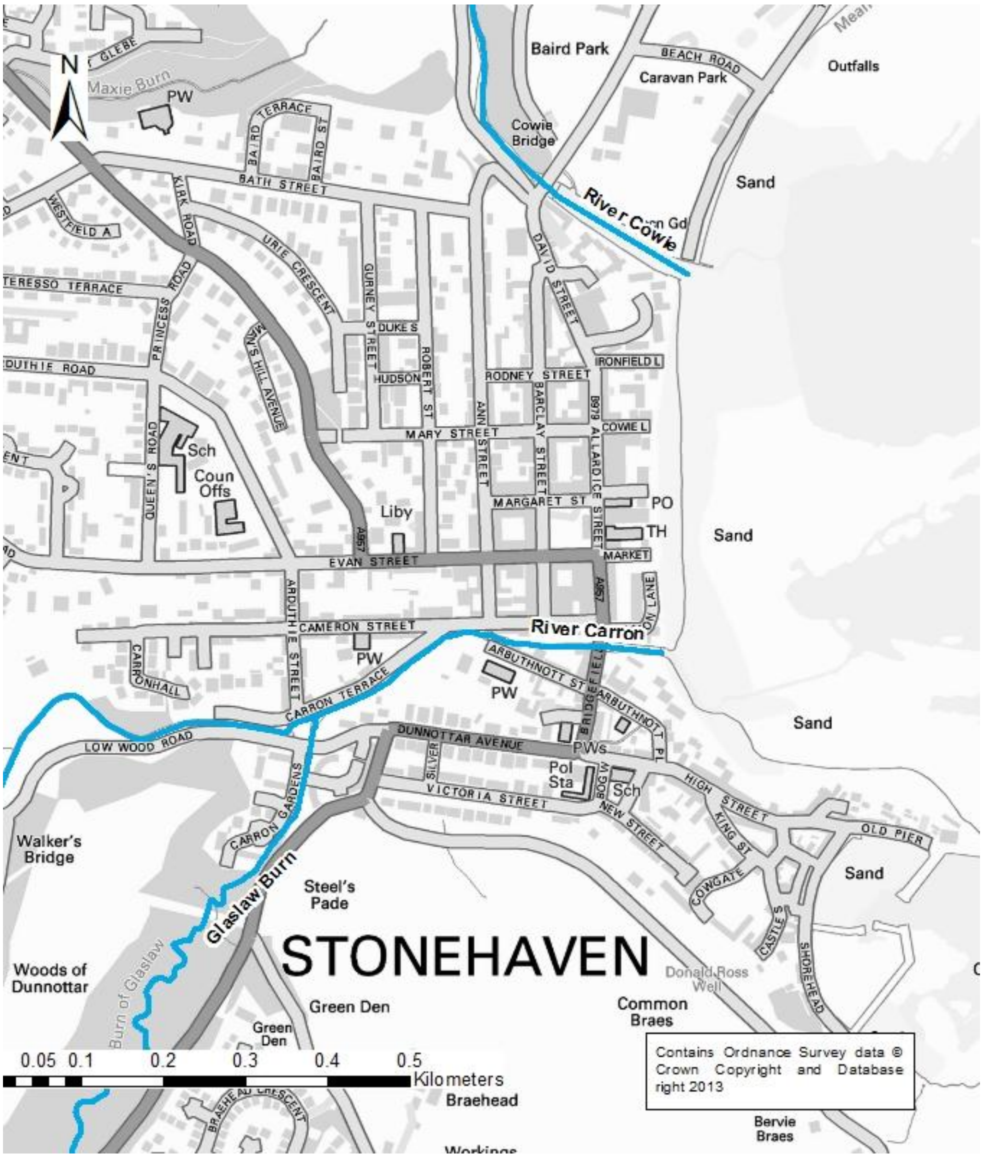
Abbreviations

AMAX .... Annual Maximum
AP .... Annual Probability
FPS .... Flood Protection Scheme
LiDAR .... Light Detecting and Ranging
RP .... Return Period
FEH .... Flood Estimation Handbook
CEH .... Centre for Ecology and Hydrology
UKCP09 .... UK Climate Projections 2009
FCERM .... Flood and Coastal Erosion Risk Management
DEFRA .... Department for Environmental Flood and Rural Affairs
SEPA .... Scottish Environmental Protection Agency
EA .... Environment Agency

1 Introduction

Stonehaven is located in the northeast of Scotland in Aberdeenshire. The coastal town is potentially at risk of surface water flooding from the River Cowie, River Carron and the Glaslaw Burn which are shown in Figure 1-1 and surface runoff particularly from the Bervie Braes. Parts of the town are also at risk of coastal (tidal wave) flooding.

Figure 1-1: Sources of Surface Water Flooding to Stonehaven.



Following extensive flooding from the River Carron in November 2009, Aberdeenshire Council commissioned JBA Consulting to carry out a feasibility assessment with respect to seeking flood alleviation options for the River Carron and Glaslaw Burn. The study considered feasible options could consist of the following:

1. Construction of direct defences as a stand alone option.
2. Construction of direct defences combined with modifications to the channel and bridges (where applicable).
3. Provision of upstream storage as a stand alone option.
4. Construction of direct defences combined with upstream storage.

Stonehaven was inundated again in December 2012, reinvigorating the proposal for a Flood Protection Scheme to be constructed to alleviate flooding from this source. JBA Consulting has been commissioned to undertake the design of the proposed River Carron and Glaslaw Burn Flood Protection Scheme (referred to hereafter as the Stonehaven Flood Protection Scheme).

This report consists of the following:

- An update of the existing hydrological analysis of the River Carron and the Glaslaw Burn following the 2012 flood event and the estimated impact of climate change
- A review of the existing flood risk to Stonehaven from the River Carron and Glaslaw Burn
- An analysis of the potential short list options to be incorporated in the final scheme and the development of a "preferred scheme"
- An initial estimation of the costs associated with the preferred scheme
- An economic appraisal of the preferred scheme.

2 Hydrology

A detailed hydrological analysis of the Carron Catchment was undertaken as part of the flood alleviation study in 2009[1]. After the December 2012 flood event this hydrology was reviewed, using the extended flood record to improve flood estimates. The hydrology of the Glaslaw Burn was also reviewed following the December 2012 event as a result of the high flows observed in the burn during the event. The updated hydrology to be used in the design of the scheme is explained in detail in the review of the December 2012 flood event report.[2]

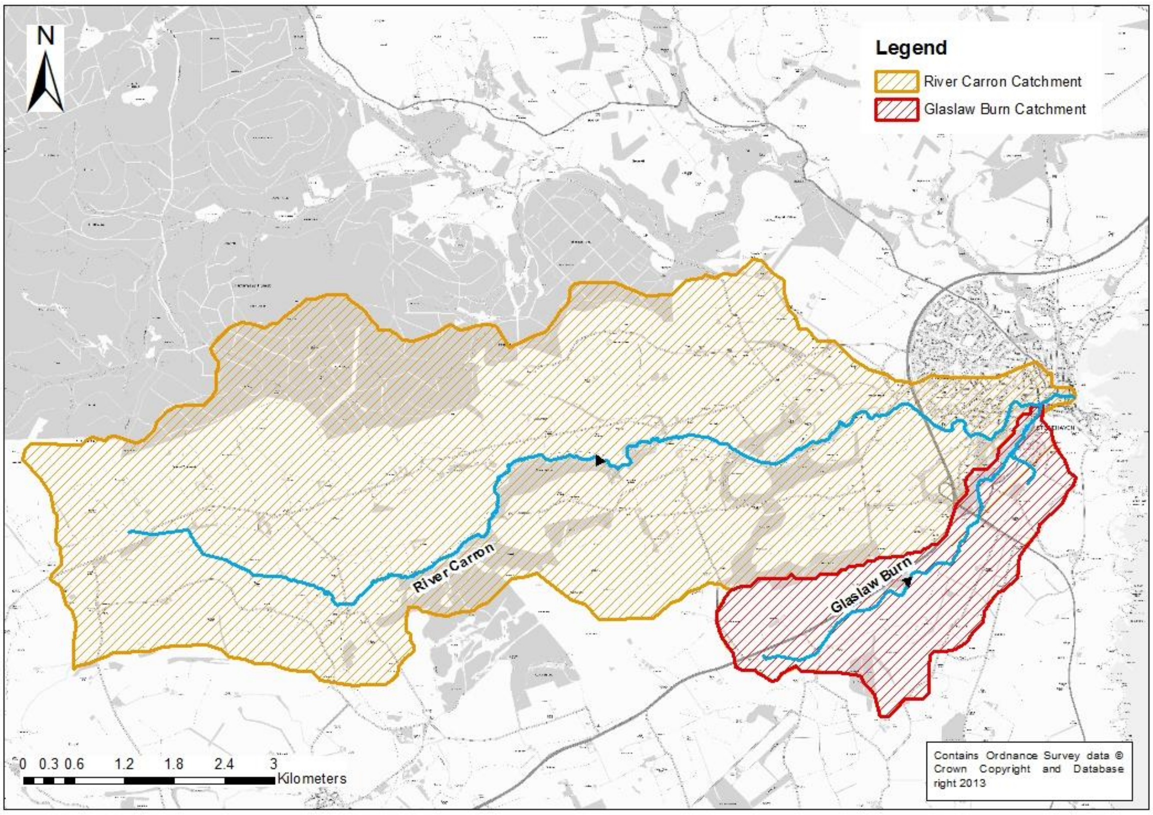
2.1 Overview of Hydrology for the Stonehaven FPS

The Stonehaven Flood Protection Scheme will alleviate flooding from the River Carron and the Glaslaw Burn and reduce the impact of surface water flows from other sources. Figure 2-1 illustrates the route of the River Carron and the Glaslaw Burn and their catchments.

The catchment of the River Carron to the SEPA gauge at the Red Bridge has an area of approximately 43km^2. The river flows from its source in the Brae of Glen Bervie in an easterly direction until it passes under the A90 at the western boundary of Stonehaven. The Carron then passes along the southern periphery of the town centre where it merges with the Glaslaw Burn and thence onto its River Mouth where it joins the sea.

The Glaslaw Burn has a catchment area of approximately 5.7km^2 from its confluence with the River Carron to its source at Upper Criggie. The burn flows in a north easterly direction and enters Stonehaven through the wooded valley between the Woods of Dunnottar and Breahead housing development.

Figure 2-1 Catchments of the River Carron and Glaslaw Burn



[1] Stonehaven River Carron Flood Alleviation Study, JBA Consulting, July 2012

[2] Stonehaven December 2012 Flood Event Review, JBA Consulting, June 2013

2.2 Peak Flow Estimation

2.2.1 River Carron

Flood estimations for catchments of this size and nature are undertaken using the Flood Estimation Handbook (FEH). For the River Carron the FEH statistical method was deemed the most appropriate due to the size of the catchment and availability of gauged data from the SEPA gauge located between the Red Bridge and the Green Bridge. The statistical method combines an estimation of the index flood (median annual flood (QMED)) at the subject site with a growth curve derived from one of the following methods;

- Single site analysis of a nearby gauge.
- A pooling group of gauged catchments which are considered hydrologically similar.
- A combination of the two through an enhanced pooling group

Estimation of QMED

The gauge on the River Carron has a relatively short record of less than 14 years. However the River Bervie at Inverbervie has gauged data extending back to 1979. The annual maximum (AMAX) data for both gauges was compared for the overlapping period (2003 - 2010) and a reasonable correlation was found to exist. Based on this correlation a regression analysis was carried out to allow AMAX flows on the Carron to be made from the Bervie series.

QMED for the River Carron was then estimated to be 12.6m^3/s based on the extended data series.

Flood Growth Curve

Flood growth curves were derived for the River Carron using an extended AMAX dataset for the Carron as part on an enhanced pooling group analysis. The enhanced pooling utilises a pooling group of gauged catchments which are hydrologically similar to the subject catchment, but which gives higher weighting to the subject site's dataset.

2.2.2 Glaslaw Burn

Peak flows for the Glaslaw Burn have been estimated using the FEH Rainfall Runoff methodology. Current FEH guidance deems this method appropriate given the size and nature of the catchment and the lack of gauged data for the burn.

The FEH Rainfall Runoff methodology combines design rainfall with a unit hydrograph for the subject site to estimate flows at a range of return periods. The catchment descriptors for the Glaslaw Burn catchment are included in table 2-1.

Table 2-1: Catchment Descriptors for the Glaslaw Burn

Parameter	Value
Area (km2)	5.69
FARL (unitless)	1
PROPWET (unitless)	0.37
ALTBAR (unitless)	104
BFI HOST (unitless)	0.585
DPLBAR (unitless)	3.46
SPRHOST (unitless)	40.81

2.3 Climate Change

The impact of climate change on flood flows is a key risk in the design of a Flood Protection Scheme. Typically for flood studies the potential effects of climate change are considered by upscaling by a factor of 20%, as recommended by SEPA most recent guidance for flood risk assessment.[3]

Recent work in England and Wales has provided regionalised estimates of how climate change will impact upon river flows through the next century based on UKCP09 (UK Climate Predictions 09) projections. Although this research does not include Scottish catchments, it does indicate

[3] Strategic Flood Risk Assessment- SEPA technical guidance to Support Development Planning, SEPA,

that an increase of flows of between 25 and 30% by 2080 for Scottish catchments may be a realistic figure.

Data from UKCP09 was analysed and the impact on flood flows was estimated for a number of intervals until 2080. The figures are consistent with those produced for England and Wales and the methodology is presented in Appendix A. Table 2-2 shows a list of potential climate changes for the Stonehaven region for each decade available under UKCP09.

Table 2-2: Potential Impact of Climate Change on Flood Flows for Stonehaven

Estimate	Total Potential Climate Change for						
	2020s (2010 to 2029)	2030s (2020 to 2039)	2040s (2030 to 2049)	2050s (2040 to 2059)	2060s (2050 to 2069)	2070s (2060 to 2079)	2080s (2070 to 2089)
	Percentage increase in flows						
90 percentile	19%	22%	30%	37%	46%	55%	67%
80 percentile	14%	17%	23%	29%	37%	44%	53%
Best Estimate	5%	7%	12%	17%	22%	27%	**33%**
20 percentile	-3%	-1%	3%	6%	10%	13%	17%
10 percentile	-7%	-5%	-2%	1%	4%	6%	10%

For the purpose of this study, the best estimate of climate change for 2080 (33%) will be adopted for design.

2.3.1 Peak Flows

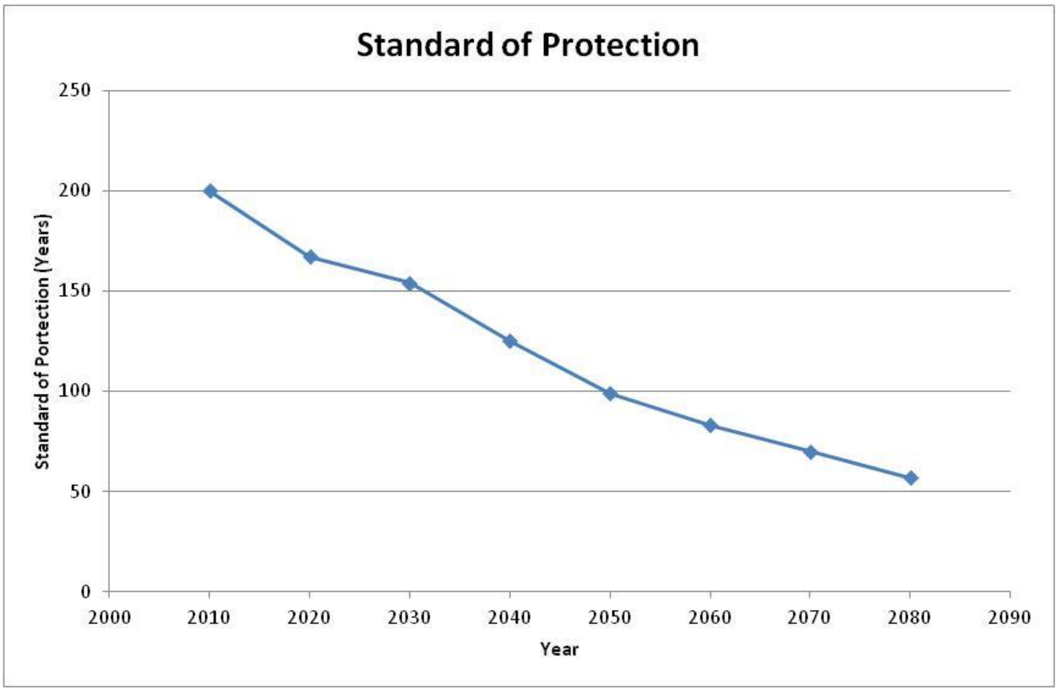
The above methodologies have been used to estimate peak flows in the River Carron and Glaslaw Burn for a range of return periods. The results are displayed in Table 2-3.

Table 2-3: Peak Flow for the River Carron and the Glaslaw Burn

Return Period (years)	AP (%)	River Carron Peak Flows (m^3/s)	Glaslaw Burn Peak Flows (m^3/s)
2	50	14.5	2.5
5	20	20.5	3.4
10	10	24.9	4.2
25	4	31.3	4.6
50	2	36.9	6.2
75	1.33	40.4	6.7
100	1	43.2	7.1
200	0.5	50.4	8.2
200+CC	**0.5+CC**	**67.0**	**10.9**
1000	0.1	71.8	11.7

The Flood Protection Scheme will be designed to offer a standard of protection of 200 years (0.5% AP) for the lifetime of the scheme (approximately 100 years). The impacts of climate change will erode the standard of protection of the scheme as the magnitude and frequency of flood flows increase. Figure 2-2 illustrates how the standard of protection would likely be reduced by increasing flows during the lifetime of the scheme if allowances for climate change were not included within the design.

Figure 2-2: Impact of climate change on standard of protection



Therefore the best estimate for the impact of climate change on 200 year (0.5% AP) flood flows for 2080 will be adopted as the design flows. The design peak flows for this study will be **67.0m^3/s** for the River Carron and **10.9m^3/s** for the Glaslaw Burn. The scheme will therefore provide flood protection for the current 200 year flood with an allowance for climate change.

3 Hydraulic Model

A 1D hydraulic model of the River Carron was constructed as part of the River Carron Channel Capacity Study in 2010 and was enhanced to a 1D-2D hydraulic model as part of the River Carron Flood Alleviation Study in 2011. The model extends along the River Carron from String Brae (OS NGR 385071 785501) to the coastal river mouth (OS NGR 378606 785657). It also incorporates the Glaslaw Burn from Braehead Crescent (OS NGR 386767 785091). A summary of the hydraulic model is included within Appendix B.

3.1 Flood Risk to Stonehaven

Flooding in Stonehaven from the River Carron is likely to begin during a flood event with a return period of between 2 and 5 years (50% and 20% AP respectively). Flood water will leave the River Carron immediately upstream of the Green Bridge (NGR OS 387043, 785641) at a flow of approximately 16m^3/s.

Downstream of the Green Bridge and to the river mouth the River Carron is in a relatively narrow corridor constrained by the development of Stonehaven resulting in numerous locations where the river will overtop its banks and inundate properties. Table 3-1 displays a summary of the River Carron's existing capacity and the return periods at which the banks are likely to be overtopped and figure 3-1 illustrates the location of each cross section.

The Glaslaw Burn is also a source of flood risk to Stonehaven. In December 2012 the Glaslaw Burn overtopped its banks in the vicinity of Carron Gardens and Woodview Court. Water was observed flowing from Woodview Court and onto Dunnottar Avenue. The Glaslaw Burn is likely to overtop its banks upstream of Carron Gardens when flows in the burn exceed 5.7m^3/s. As the burn passes between Carron Gardens and Woodview Court it passes through two short culverts before merging with the River Carron. These are relatively small culverts which are at high risk of blockage during a flood event. The model cross sections for the Glaslaw Burn are displayed in figure 3-2.

Table 3-1: River Carron Channel Capacity

Location	Section	Peak Water Level (mAOD)	Left Bank Level (mAOD)	Right Bank Level (mAOD)	Channel Capacity (m3/s)	Return Period (years)
Upstream of Red Bridge	CAR_929	10.33	10.91	13.73	59	Between 200yr and 200+CCyr
	CAR_866	10.08	12.4	13.26	60	Between 200yr and 200+CCyr
	CAR_812	9.89	11.97	11.88	61	Between 200yr and 200+CCyr
	CAR_768	9.6	10.62	10.1	60	Between 200yr and 200+CCyr
Red Bridge	CAR_763	9.64	11.74	9.93	60	Between 200yr and 200+CCyr
	CAR_757	9.22	10.40	9.51	60	Between 200yr and 200+CCyr
Between Red Bridge and Green Bridge	CAR_734	9.14	9.88	8.92	40	Between 50yr and 75yr
	CAR_733	9.13	9.88	8.93	20	Between 2yr and 5yr
	CAR_710	8.98	8.66	8.66	27	Between 10yr and 25yr
	CAR_671	9	8.41	8.19	17	Between 2yr and 5yr
	CAR_637	9.2	8.49	8.01	16	Between 2yr and 5yr
Green Bridge	CAR_635	9.2	8.54	8	16	Between 2yr and 5yr
	CAR_631	8.66	8.5	8.1	18	Between 2yr and 5yr
Between Green Bridge and White Bridge	CAR_627	8.63	8.24	8.11	18	Between 2yr and 5yr
	CAR_625	7.41	8.13	8.11	30	Between 10yr and 25yr
	CAR_624	7.34	7.86	8.13	32	Between 25yr and 50yr
	CAR_617	7.23	6.99	8.13	29	Between 10yr and 25yr
	CAR_606	7.04	6.99	8.32	34	Between 25yr and 50yr
	CAR_605	7.01	6.99	8.32	41	Between 75yr and 100yr
	CAR_573	6.43	6.17	6.88	43	Between 75yr and 100yr
	CAR_572	6.43	6.03	6.88	41	Between 75yr and 100yr
	CAR_567	6.35	6.3	6.88	57	Between 200yr and 200+CCyr
	CAR_521	6.07	5.60	6.88	33	Between 25yr and 50yr
	CAR_477	5.79	5.26	6.2	35	Between 25yr and 50yr
	CAR_421	5.3	5	4.82	29	Between 10yr and 25yr
	CAR_381	5.22	5.32	4.82	36	Between 25yr and 50yr
	CAR_357	5.04	5.16	4.81	44	Between 100yr and 200yr
	CAR_347	5.19	5.26	4.61	39	Between 50yr and 75yr
White Bridge	CAR_346	5.19	5.26	4.61	39	Between 50yr and 75yr
	CAR_343	5.08	5.03	5.67	52	Between 200yr and 200+CCyr
Between White Bridge and Bridgefield Bridge	CAR_334	4.96	5.03	5.67	61	Between 200yr and 200+CCyr
	CAR_295	4.92	5.7	5.66	61	Between 200yr and 200+CCyr
	CAR_236	4.59	3.45	5.22	19	Between 2yr and 5yr
	CAR_221	4.48	3.48	6.83	22	Between 5yr and 10yr
Bridgefield Bridge	CAR_214	4.49	4.84	4.85	34	Between 25yr and 50yr
	CAR_200	4.38	6.23	6.24	56	Between 200yr and 200+CCyr
DS of Bridgefield Bridge	CAR_198	4.37	6.23	6.24	56	Between 200yr and 200+CCyr
	CAR_196	4.25	4.62	5.86	40	Between 50yr and 75yr
	CAR_169	4.13	3.27	5.54	27	Between 10yr and 25yr
	CAR_132	4.07	3.29	5.76	35	Between 25yr and 50yr
	CAR_126	4.06	3.73	3.72	41	Between 75yr and 100yr
	CAR_122	3.9	3.73	3.72	46	Between 100yr and 200yr
	CAR_117	3.81	4.23	4.28	60	Between 200yr and 200+CCyr

Figure 3-1: Location of Hydraulic Model Sections on River Carron

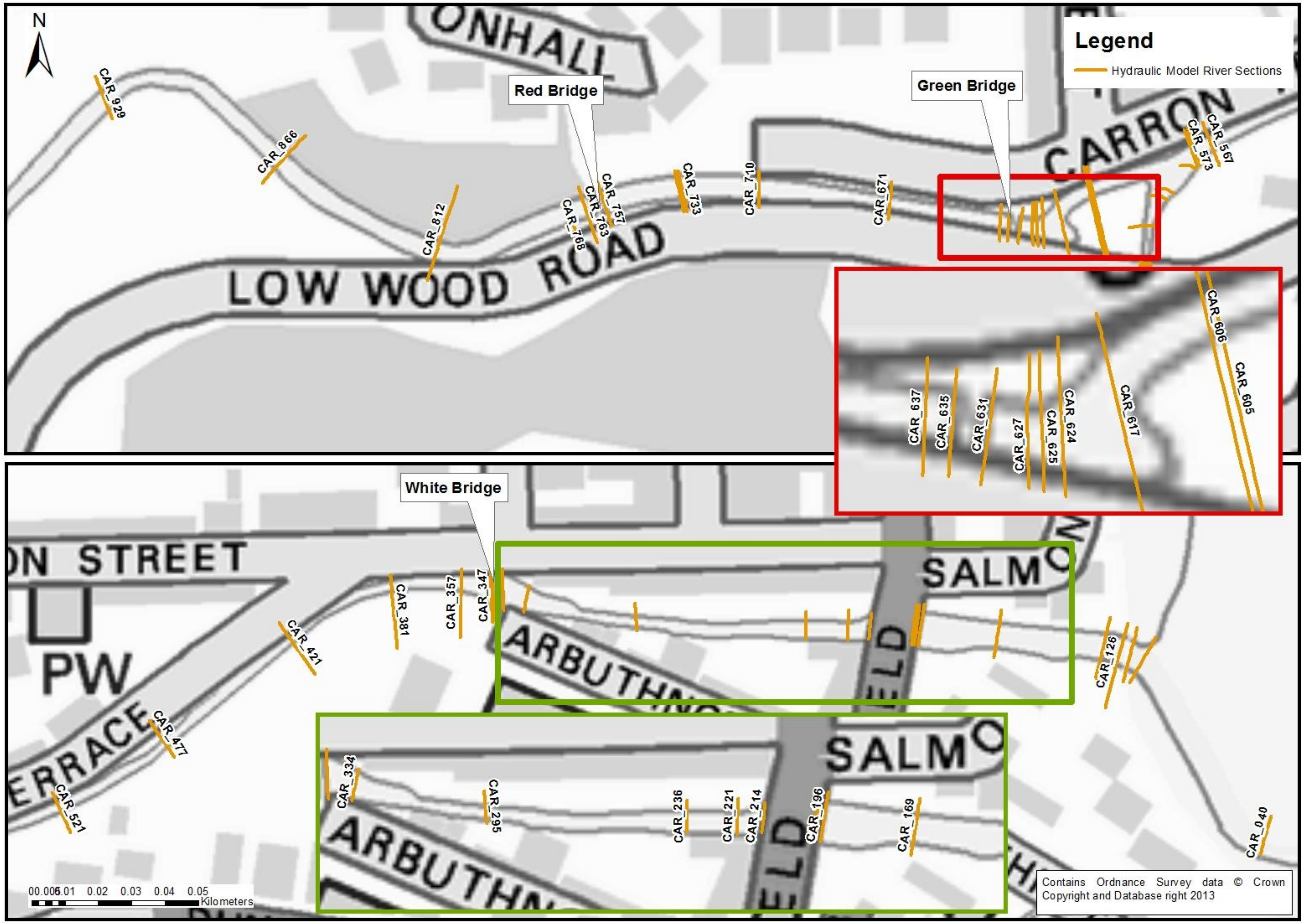


Figure 3-2 Hydraulic Model Sections on the Glaslaw Burn

4 Flood Protection Scheme Options Appraisal

The previous study alleviation study for Stonehaven identified two options that were potentially feasible to provide the required standard of protection. These were as follows:

- Attenuation of flows and flood storage in the mid to lower catchment.
- Construction of flood walls along the river channel throughout Stonehaven.

Since the previous study was carried out, more data has been collected including flood events that affect the hydrology for the River Carron and the Glaslaw Burn, as described in Section 2. As a result the estimation of peak flows have increased. In addition it was decided to include an allowance for climate change to ensure the standard of protection could be extended for the life the scheme

4.1 Standard of Protection

The flood protection scheme at Stonehaven aims to not only alleviate surface water flooding, but have a positive economic impact on the town, allowing for future development within the area protected by the scheme. Therefore the scheme should be designed to provide a 200 year standard of protection as this is the current standard required for planning purposes and the threshold for unacceptable flood risk.

To achieve the required standard of protection a freeboard is added to the estimated peak water level to give a high level of confidence that the scheme will protect to the standard intended. The freeboard should take into account physical processes such as waves as well as a safety margin to allow uncertainties to the estimation of peak flows and the prediction of peak water levels.

To estimate the appropriate level of freeboard for the proposed flood protection scheme the methodology outlined in the EA's Fluvial Freeboard Guidance Note was applied and consisted of the following

1. The physical processes that affect the defence performance hat are not allowed for in the design water. These can include wave overtopping, settlement of the defence and super-elevation at bends.
2. Quantification of the uncertainty used to predict the design water levels in the hydraulic and hydrological procedures. The multi-attribute methodology was used to determine this.

4.2 Option 1: Flood Walls

Flood walls are a highly effective and visible means of mitigating flood risk and protecting properties. They increase channel capacity by accommodating flows to a higher level and reducing the risk to the river being able to overtop its banks. They can be designed to provide the required standard of protection which includes an allowance for freeboard. For this scheme defences will be designed to provide a standard of protection against flows of 67 m^3/s with a freeboard allowance of 450 mm.

4.2.1 River Carron

Direct defences can be detrimental to the aesthetic nature and amenity of the watercourse if they prevent public access or obstruct views of the river. As this is undesirable it is important to consider direct defences in conjunction with other options that will minimise the height of the flood walls.

These options include modifying the river channel to increase its hydraulic capacity as well as raising bridges on the watercourse that cause water to back up. The options that have been considered in conjunction with flood walls are included in Table 4-1 and the impact on peak water levels within the river channel for each scenario is included in table 4-2.

Table 4-1: Direct Defence Options for the River Carron

Option	Description	Scenario ID
Channel Modifications	Modifications to the channel involve removing material from the river channel to improve channel hydraulics. This option involves modifications in the river channel from upstream of the Green Bridge at NGR OS 387046, 785641 to Bridgefield Bridge at NGR OS387446, 785744. It also involves the reduction of the weir immediately downstream of the Green Bridge and modification to the existing island downstream of the Green Bridge to improve local hydraulics.	FW+CM
Raising of Green Bridge and White Bridge	The Green Bridge has been highlighted as a key source of flood risk in Stonehaven. Flooding from the River Carron in 2009 and 2012 was partially attributed to flows backing up behind the Green Bridge. The White Bridge has been close to overtopping in previous floods. Additionally it causes a restriction at a critical point along the river in terms of flood risk to properties. It may not be possible to entirely remove the hydraulic impact of the White Bridge but it may be possible to raise it by approximately 1.04m.	FW+CM+BR1
Raising of Green Bridge, White Bridge and Red Bridge	This option involves raising the Red Bridge as well as the White Bridge and Green Bridge. For the design flood event, the Red Bridge causes water to back up and would force flood water to overtop onto Low Wood Road. The road would convey water through Stonehaven, which would be unable to return to the river if flood defences were present.	FW+CM+BR2

Table 4-2: Impact of Direct Defence Options on Peak Water Levels.

Section		Peak Water Level (mAOD)				
		200yr+CC Existing	200yr+CC Flood Walls	200yr+CC FW+CM	200yr+CC FW+CM+BR1	200yr+CC FW+CM+BR2
Upstream of Red Bridge	CAR_929	10.33	11.75	11.75	10.43	10.35
	CAR_866	10.08	11.78	11.78	10.29	10.00
	CAR_812	9.89	11.78	11.78	10.22	9.79
	CAR_768	9.60	11.74	11.74	10.04	9.48
Red Bridge	CAR_763	9.64	11.75	11.75	10.09	9.50
	CAR_757	9.22	11.06	11.05	9.43	9.43
Between Red Bridge and Green Bridge	CAR_734	9.14	11.05	11.03	9.38	9.38
	CAR_733	9.13	11.05	11.03	9.38	9.38
	CAR_710	8.98	11.03	11.00	9.20	9.20
	CAR_671	9.00	11.02	11.00	9.09	9.09
	CAR_637	9.20	11.02	10.98	8.98	8.98
Green Bridge	CAR_635	9.20	10.98	10.97	8.91	8.91
	CAR_631	8.66	9.42	8.87	8.87	8.88
Between Green Bridge and White Bridge	CAR_627	8.63	9.41	9.05	9.05	9.05
	CAR_625	7.41	7.60	7.28	7.28	7.28
	CAR_624	7.34	7.41	7.39	7.38	7.38
	CAR_617	7.23	7.25	7.25	7.24	7.24
	CAR_606	7.04	6.99	7.02	7.01	7.01
	CAR_605	7.01	6.97	7.00	6.99	6.99
	CAR_573	6.43	6.45	6.53	6.51	6.51
	CAR_572	6.43	6.45	6.53	6.51	6.51
	CAR_567	6.35	6.40	6.48	6.45	6.45
	CAR_521	6.07	6.07	6.10	6.05	6.05
	CAR_477	5.79	5.93	6.05	5.98	5.98
	CAR_421	5.30	5.83	5.96	5.87	5.87
	CAR_381	5.22	5.79	5.93	5.84	5.84
	CAR_357	5.04	5.72	5.88	5.77	5.77
	CAR_347	5.19	5.73	5.88	5.77	5.78
White Bridge	CAR_346	5.19	5.72	5.87	5.77	5.77
	CAR_343	5.08	5.54	5.69	5.69	5.69
Between White Bridge and Bridgefield Bridge	CAR_334	4.96	5.37	5.48	5.48	5.48
	CAR_295	4.92	5.29	5.43	5.43	5.43
	CAR_236	4.59	4.98	5.09	5.09	5.09
	CAR_221	4.48	4.92	5.06	5.06	5.06
Bridgefield Bridge	CAR_214	4.49	4.76	4.92	4.92	4.93
	CAR_200	4.38	4.57	4.77	4.77	4.77
DS of Bridgefield Bridge	CAR_198	4.37	4.56	4.76	4.77	4.77
	CAR_196	4.25	4.43	4.62	4.63	4.63
	CAR_169	4.13	4.32	4.49	4.50	4.50
	CAR_132	4.07	4.26	4.45	4.45	4.46
	CAR_126	4.06	4.27	4.47	4.47	4.47
	CAR_122	3.90	4.04	4.16	4.16	4.16
	CAR_117	3.81	3.95	4.06	4.07	4.07
	CAR_040	2.73	2.82	2.82	2.83	2.83

- The introduction of flood walls significantly raises peak water levels along the reach. This is expected as water is no longer able to leave the channel.
- The modifications to the channel show a significant reduction in peak water levels in the vicinity of the sewer downstream of the Green Bridge. This has the result of increasing peak water levels downstream of the weir. The weir currently acts as a flow control. When removed more flow is able to pass downstream resulting in an increase in peak water levels.
- There is a significant reduction in peak water levels upstream of the Red Bridge when it is raised. This will reduce the extent of the required flood defences.
- Raising of the Green Bridge significantly reduces peak water level upstream of the bridge at this key area of flood risk (10.97 mAOD to 8.91 mAOD).
- Raising of the White Bridge causes a reduction of approximately 100mm upstream of the bridge. This area of the Carron is a narrow corridor with properties adjacent to the river. A reduction of 100mm is considered significant in terms of maintaining the amenity of the watercourse with flood defences.

The attenuation of flows upstream of Stonehaven and the provision of storage could potentially reduce the height and extent of the required flood walls by reducing the peak flow in River Carron during design flood events. The potential to provide upstream storage as a means of reducing the height of flood defences has been explored in the assessment of Option 2.

4.2.2 Glaslaw Burn

This option also includes direct defences on the Glaslaw Burn. Defences on the burn would be in conjunction with a new box culvert under the Woodview Court bridge which currently is at a high risk of blockage during a flood event. The peak water levels in the Glaslaw Burn with direct flood defences are included in Table 4-3.

Table 4-3: Peak Water Levels for Glaslaw Burn with Direct Flood Defences

Location	Section ID	Peak Water Level (mAOD)	
		200yr+CC Existing	200yr+CC FW+CM+BR2
Upstream of Carron Gardens	GLA_801	22.15	22.05
	GLA_653	19.16	19.19
	GLA_496	16.00	15.96
	GLA_413	14.14	14.18
	GLA_315	12.24	12.28
Between Carron Gardens and Woodview Court Bridge	GLA_222	10.71	10.78
	GLA_179	10.02	10.06
	GLA_147	9.41	9.45
	GLA_116	8.85	9.28
Woodview Court Bridge	GLA_089	8.75	9.31
	GLA_070	8.39	8.72
Low Wood Road	GLA_044	8.36	8.57
	GLA_033	7.98	8.00
Between Low Wood Road and Confluence with River Carron (at section CAR_573)	GLA_032	7.92	8.02
	GLA_030	6.72	6.72
	GLA_020	6.71	6.77
	GLA_011	6.63	6.67
	GLA_009	6.52	6.53
	GLA_000	6.43	6.51

4.2.3 Required flood defences

The extent and height of required flood walls will impact on the feasibility of this option. If the flood walls are considered too high and as a result have a severe detrimental impact on the town by ruining river aesthetics, removing trees and affecting third party properties then the option would be considered unfeasible. Table 4-4 highlights the maximum wall height and required length for the proposed flood walls at each modelled section for the 200yr+CC FW+CM+BR2 scenario, the preferred scenario for Option 1, which has the most positive impact on peak water levels. Figure 4-1 provides a graphical representation of Option 1.

Table 4-4: Height of Direct Flood Defences

Section	Maximum Left Bank Wall Height (m)	Left Bank Length (m)	Maximum Right Bank Wall Height (m)	Right Bank Wall Length (m)
River Carron				
CAR_768 - CAR_763	-	-	0.17	5.00
CAR_763 - CAR_757	-	6.00	0.52	7.00
CAR_757 - CAR_734	-	24.00	1.05	22
CAR_734 - CAR_733	1.36	1.00	1.05	23.00
CAR_733 - CAR_710	1.36	24.00	1.15	
CAR_710 - CAR_671	1.96	40.00	1.5	39.00
CAR_671 - CAR_637	2.09	17.00	1.57	34.00
CAR_637 - CAR_635	2.09	3.00	1.57	2.00
CAR_635 - CAR_631	**Green Bridge**			
CAR_631 - CAR_627	2.18	3.00	1.54	5.00
CAR_627 - CAR_625	2.18	1.00	-	-
CAR_625 - CAR_624	0.61	2.00	-	-
CAR_624 - CAR_617	0.70	4.00	-	-
CAR_617 - CAR_606	0.70	11.00	-	-
CAR_606 - CAR_605	0.70	1.00	-	-
CAR_605 - CAR_573	0.47	31.00	-	-
CAR_573 - CAR_572	0.79	1.00	-	-
CAR_572 - CAR_567	0.93	51.00	1.97	6.00
CAR_567 - CAR_521	0.90		0.71	47.00
CAR_521 - CAR_477	1.17	42.00	0.71	43.00
CAR_477 - CAR_421	1.32*	56.00	1.23	57.00
CAR_421 - CAR_381	1.69*	42.00	1.52	30.00
CAR_381 - CAR_357	1.79*	25.00	1.57	22.00
CAR_357 - CAR_347	1.79*	10.00	1.76	14.00
CAR_347 - CAR_346	**The White Bridge**			
CAR_346 - CAR_343	1.79*	3.00	1.61	3.00
CAR_343 - CAR_334	1.78*	11.00	2.07	7.00
CAR_334 - CAR_295	2.56*	38.00	3.28	40.00
CAR_295 - CAR_236	2.56*	60.00	3.47	59.00
CAR_236 - CAR_221	2.69*	15.00	3.47	14.00
CAR_221 - CAR_214	2.69*	8.00	3.38	7.000
CAR_214 - CAR_200	**Bridgefield Bridge**			
CAR_198 - CAR_196	0.71	2.00	1.43	2.00
CAR_196 - CAR_169	1.83	27.00	1.87	27.00
CAR_169 - CAR_132	1.83	38.00	1.87	36.00
CAR_132 - CAR_126	1.77	6.00	1.35	9.00
Glaslaw Burn				
GLA_222 - GLA_179	0.83	67.00	-	-
GLA_179 - GLA_147	1.13	33.00	-	-
GLA_147 - GLA_116	0.77	29.00	0.97	12.00
GLA_116 - GLA_089	1.44	31.00	1.44	34.00
GLA_089 - GLA_070	**Woodview Court Culvert/Bridge**			
GLA_070 - GLA_044	1.38	26.00	1.19	26.00
GLA_044 - GLA_033	**Low Wood Road**			

* Maximum height when automatic flood barriers have been deployed. Visible height under normal conditions may be reduced by up to 1m.

Figure 4-1: Option 1 Flood Protection Scheme

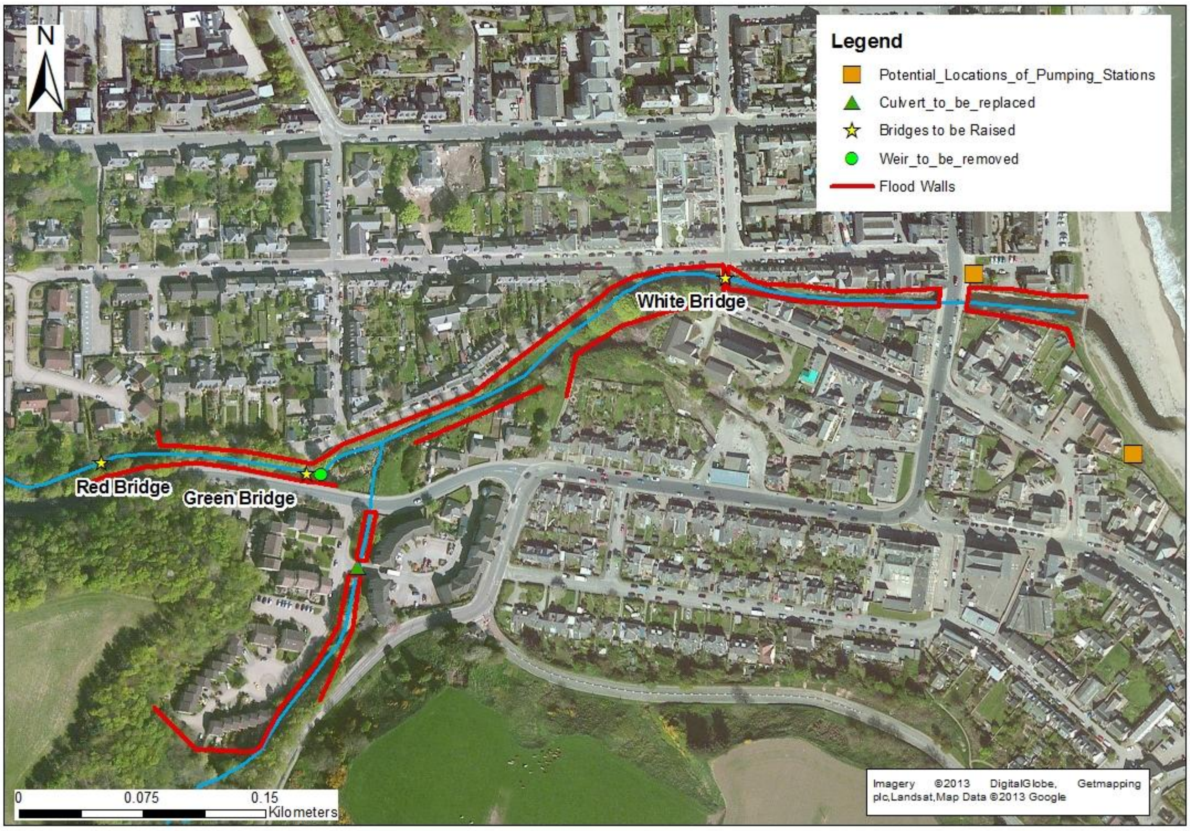


Table 4-4 shows that some of the required flood walls would be in excess of 2m which could obstruct views of the river and have a detrimental impact on the aesthetic nature of Stonehaven. Automatic flood defences in areas with high walls would provide permanent flood defences whilst retaining views and a sense of openness. Automatic flood defences would require lower permanent wall height (approximately 1 - 1.5 m) but would also require a thicker wall base. An internal flood barrier is lifted with rising water levels in the river channel to provide the required standard of protection. The barriers are considered reliable if maintained correctly however they would introduce a low risk of failure to the scheme.

4.3 Option 2: Attenuated Flows and Upstream Storage

Flooding may be alleviated by attenuating flows upstream and providing sufficient storage. The previous flood alleviation study for Stonehaven[4] reviewed the options for flood mitigation via flood storage in the mid to lower catchment. This assessed a number of locations based on available storage volumes derived from topographical information although no specific modelling was undertaken to review and confirm that the required storage volumes were available and that peak flows could be attenuated sufficiently.

4.3.1 Flows

To alleviate flooding from the River Carron using upstream storage, flows would need to be adequately attenuated to ensure that flow through the town would not overtop its banks. The Glaslaw Burn joins the River Carron on its course through Stonehaven adding a considerable flow to the River Carron, which must be accounted for downstream of the confluence as attenuation of flows in the River Carron would have no impact on flows from the Glaslaw Burn. Therefore the feasibility of storage on the Glaslaw Burn was also investigated. To estimate the maximum permissible flow in the channel that would provide the required standard of protection the following analysis was undertaken:

- The threshold of flooding for each section in the hydraulic model was derived.
- 300 mm was subtracted from the threshold to achieve a minimum freeboard. For a storage option a freeboard of 300 mm is considered acceptable as flows are attenuated upstream; there is less uncertainty when estimating peak water levels within the channel.
- A rating curve for each section was extracted from the hydraulic model and the minimum flow at which the threshold of flooding occurred was recorded for each section.
- The flow at which a threshold of flooding occurred (including freeboard) was taken as the maximum flow permissible in the channel.

When attenuating flows within a catchment it is inadvisable to attenuate peak flows to less than the mean annual 2 year flood flow. The volume of water to be stored when attenuating flood flows can be affected by uncertainties in peak flow estimation, critical storm duration, inefficiencies of flow control units, antecedent and pre event flow conditions and inaccuracies in topographical data available. Therefore there can be a large uncertainty when estimating storage volumes which is more likely when attempting to attenuate larger flows. Attenuating flows below the 2 year return period would increase the risk of failure for the scheme. The mean annual flows for the River Carron and Glaslaw Burn are 14.5 m^3/s and 2.5 m^3/s respectively.

Table 3-1 shows that section CAR 637 and CAR_635, in River Carron both have the lowest channel capacity of 16 m^3/s. However, these sections are located upstream of the Green Bridge and the river's confluence with the Glaslaw Burn. Downstream of the Glaslaw Burn, section CAR_236, upstream of Bridgefield Bridge has a channel capacity of approximately 19.4 m^3/s. The Glaslaw Burn could contribute 10.9 m^3/s of flow to the River Carron during a design flood event. This means that the flow from the River Carron that could cause flooding from this section would be approximately 8.5 m^3/s.

As the Glaslaw Burn is likely to have a large impact on the feasibility of storage as an option for the Flood Protection Scheme, storage has been considered as follows

- Storage on the River Carron only
- Storage on the River Carron and Glaslaw Burn

Table 4-5 shows the required attenuation for each of the scenarios assuming that the Glaslaw Burn may be attenuated to the mean annual flow of 2.5 m^3/s.

[4] Stonehaven River Carron Flood Alleviation Study, JBA Consulting, July 2012

Table 4-5: Maximum permissible flows in River Carron to provide 200yr+CC standard of protection.

Scenario	Minimum Bank Level (mAOD)	Maximum Peak Water for Standard of Protection (mAOD)	Flow at which Standard of Protection is Exceeded (m^{3}/s)	Maximum Permissible Flow in River Carron for Standard of Protection (m^3/s)
Storage on River Carron Only	3.45	3.15	13.06	2.16
Storage in River Carron and Glaslaw Burn	3.45	3.15	13.06	10.56

4.3.2 Analysis of Locations to Provide Storage

A number of sites were identified in the previous storage analysis as having potential to provide flood storage for attenuated flows. These sites have been re-assessed using two modelling approaches as detailed below

1. The first was a review of sites identified in the previous flood alleviation study using hydraulic modelling to determine the availability of storage and the potential for attenuation. This has been undertaken to quickly identify those that are suitable (they provide significant attenuation of flood flows on the Carron) and to focus detailed modelling on the areas that are most likely to provide a viable solution.
2. The second phase of modelling was to test those flood storage options that are deemed likely to be suitable. This includes consideration of the critical design storm duration for the catchment with allowances for the storage impact and sub catchment inflows.

A number of constraints and limitations have been assumed which include the following:

- Storage must be located as near to Stonehaven as possible to ensure that the majority of the catchment inflows can be stored and attenuated. Whilst locating storage further up the catchment may be possible, the impact on flow reduction is reduced further upstream.
- A single storage area on each watercourse is preferred. Whilst combination options may be feasible, the cost of constructing multiple dams on the same watercourse would not be cost beneficial

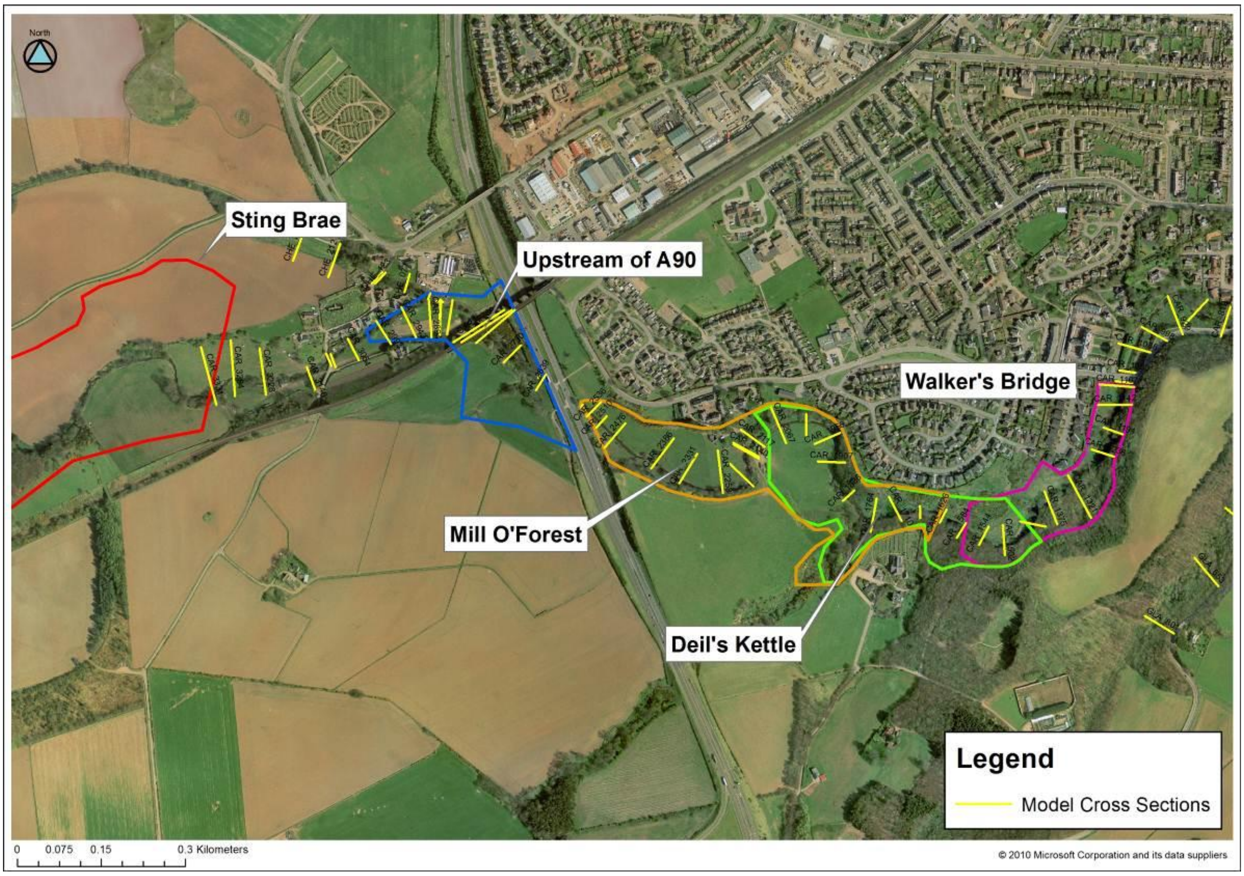
The results and detailed methodology are provided in Appendices C, D and E. A summary of the findings is provided below.

4.3.3 Assessment of potential storage areas

The review of potential storage areas showed that 5 areas upstream of Stonehaven had the potential to provide flood storage and attenuation. The areas assessed are displayed in Figure 4-2 and are as follows:

1. Walker's Bridge. Between Walker's Bridge and Mill O'Forest, the Carron Water flows through a steep sided floodplain suitable for storage.
2. Deil's Kettle. As above, but locating a dam between Riverside Drive and Murray Place would provide additional storage in the floodplain upstream.
3. Mill O'Forest. This assumes the same areas as above, but with a dam further upstream to make the most of the floodplain storage.
4. Upstream of A90. Much of the area available already floods and there is not much additional storage above existing flood levels to attenuate flood flows.
5. Sting Brae. A single storage area was tested at this location to maximise the available land and available storage within the floodplain. The land upstream includes land on the Fetteresso Castle estate and early consultation with third parties would be essential.

Figure 4-2: Storage areas on the River Carron



Of these five zones assessed, only site 5, the area upstream of Fetteresso in the Sting Brae area, has the potential floodplain volume to attenuate flows sufficiently. This area is compromised as it is located upstream of the Cheyne Burn and the Glaslaw Burn and will require additional attenuation will be required as the Cheyne Burn and the Glaslaw Burn are not attenuated.

4.3.4 Assessment of Sting Brae storage area

In order to test the Sting Brae storage location a catchment routing model was constructed to incorporate the unattenuated flood flows from other tributaries within the catchment. LAG analysis was also required to ensure that the impact of flood storage on catchment durations is fully considered. A LAG analysis is required o fully ascertain the volume required for storage. This will change depending on the storm duration; the critic event for storage volume will not necessarily generate the critical flows within the river.

Initial testing with a catchment critical design storm duration of 30.5 hours (incorporating the additional lag associated with the storage, the 200 year peak flow is reduced from 50.4m^3/s to 17.4m^3/s at the gauge located between the Red Bridge and the Green Bridge for the critical storm duration. During a design event the flow at the gauge may be higher. This suggests that whilst significant storage can be achieved, the area upstream of Fetteresso cannot attenuate the 200 year flood flow to the required minimum flow on the River Carron (14.5m^3/s) to provide the required standard of protection in the town. Even with these constraints the design would necessitate a 15m high embankment (plus freeboard) which is unlikely to be aesthetically acceptable to the community of Kirkton of Fetteresso. A summary of key parameters and impacts is provided in Table 4-6 below.

A further test was carried out on the climate change option in order to be consistent with the direct defence option. This shows that a reduced attenuation is achieved due to the higher outflow and higher flood volumes to be stored.

Table 4-6: Storage parameters

Parameter	200 year test	200 year + climate change test
Storage inflow (m^3/s)	43.6	57.9
Storage outflow (m^3/s)	14.4	26.6
Attenuation (m^3/s)	29.2	31.3
Flow at gauge (m^3/s)	17.4	28.8
Flow at outlet (includes Glaslaw) (m^3/s)	25.2	39.9
Maximum level in storage (mAOD)	47.0	47.0
Embankment height (m) (based on ground level of 32.1)	14.9	14.9
Storage volume at maximum level (m^3)	1,132,800	1,132,800

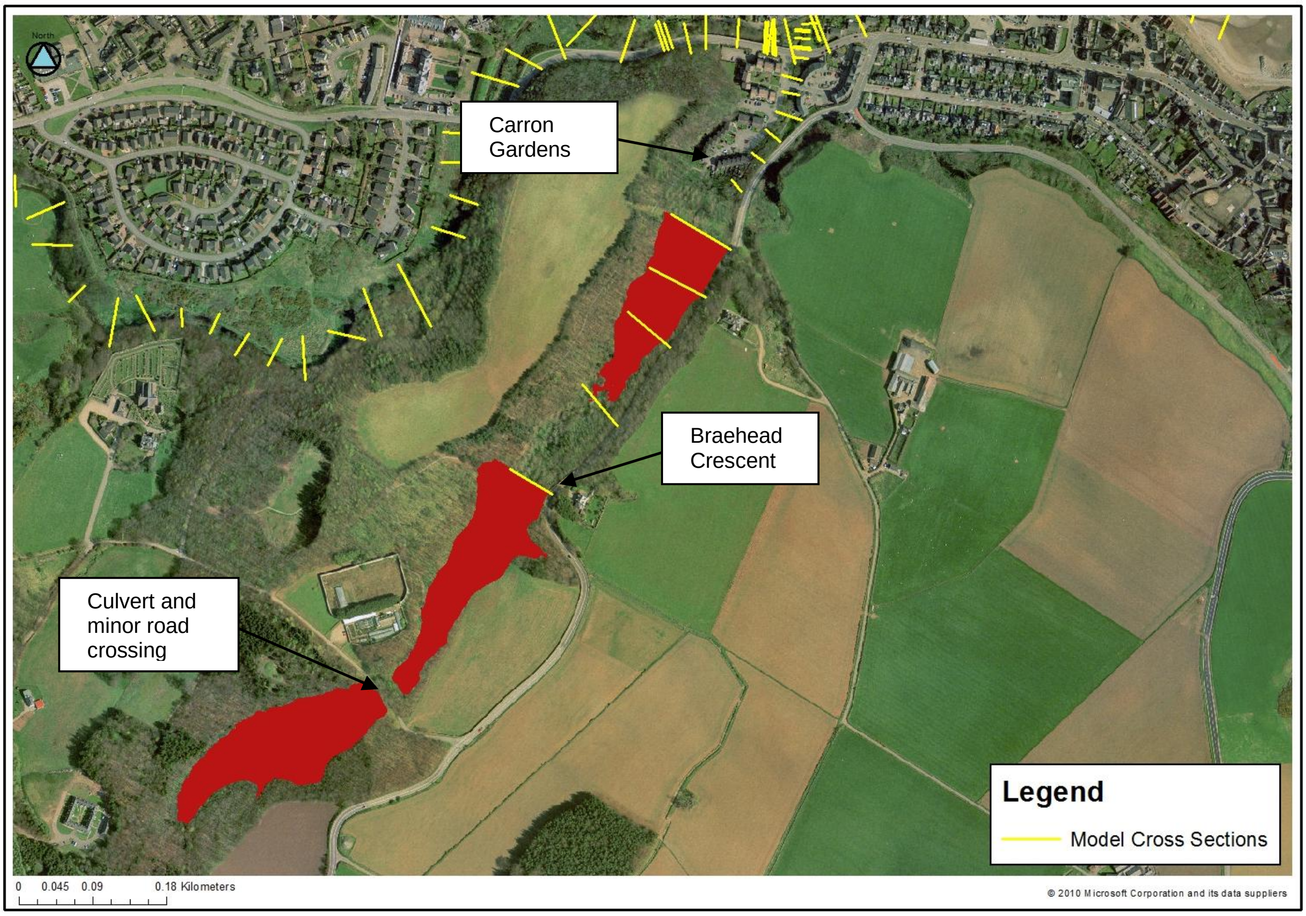
The above tests indicate that insufficient flood storage is achievable within the Carron Water catchment to alleviate flood risk in the town without additional works.

4.3.5 Assessment of storage on Glaslaw Burn

The potential for flood storage on the Glaslaw Burn was not previously assessed. In order to assess the feasibility of storage on this burn a number of potential locations were selected for testing. Three sites have been selected and a simple model constructed to test the impact of flood storage at each. These locations area displayed in Figure 4-3 are as follows:

- Immediately upstream of Carron Gardens opposite Braehead Crescent
- The Woods of Dunnottar upstream of Braehead Crescent
- The Woods of Dunnottar upstream of the culvert and minor road crossing

Figure 4-3: Storage areas on the Glaslaw Burn



The analysis suggests that no one single storage area provides sufficient storage to attenuate flood flows on the Glaslaw Burn from the 200 year flood with an allowance for climate change to the 2 year flood flow necessary to ameliorate flood risk in Stonehaven. It is possible that a combined storage option could provide the storage required, but this has not been assessed on the basis that the cost of this would be too high to make any storage scheme economically viable.

All three locations are not ideal for flood storage due to the relatively steep catchment (and thus high embankments for the storage required), woodland location, and highly mobile bed and floodplain deposits.

Based on the above analysis, it is not possible to physically store sufficient volume of flood water within a single storage area locally upstream of Stonehaven on both the Carron Water and the Glaslaw Burn to completely to provide the sufficient standard of protection required by the Flood Protection Scheme. Upstream storage cannot be considered a technically viable stand alone option, but may be considered in conjunction with direct defences in the town. The provision of storage may reduce the required height of flood walls within the town to warrant further investigation. As such Table 4-7 shows the height of direct defences required to achieve the required standard of protection in conjunction with the maximum volume of storage achievable upstream.

Table 4-7: Height of Direct Defences with Maximum Upstream Storage

Section	Maximum Left Bank Wall Height (m)	Left Bank Length (m)	Maximum Right Bank Wall Height (m)	Right Bank Wall Length (m)
River Carron				
CAR_757 - CAR_734	-	-	-	-
CAR_734 - CAR_733	-	-	-	
CAR_733 - CAR_710	-	-	-	
CAR_710 - CAR_671	0.41	40.00	-	-
CAR_671 - CAR_637	0.43	17.00	-	-
CAR_637 - CAR_635	0.43	3.00	-	-
CAR_635 - CAR_631	**Green Bridge**			
CAR_631 - CAR_627	0.39	3.00	-	-
CAR_627 - CAR_625	0.39	1.00	-	-
CAR_625 - CAR_624	-	-	-	-
CAR_624 - CAR_617	-	-	-	-
CAR_617 - CAR_606	-	-	-	-
CAR_606 - CAR_605	-	-	-	-
CAR_605 - CAR_573	-	-	-	-
CAR_573 - CAR_572	-	-	-	-
CAR_572 - CAR_567	-	-	0.58	6.00
CAR_567 - CAR_521	-	-	-	-
CAR_521 - CAR_477	-	-	-	-
CAR_477 - CAR_421	-	-	-	-
CAR_421 - CAR_381	-	-	-	-
CAR_381 - CAR_357	-	-	-	-
CAR_357 - CAR_347	-	-	-	-
CAR_347 - CAR_346	-	-	-	-
CAR_346 - CAR_343	**The White Bridge**			
CAR_343 - CAR_334	0.05	0.00		
CAR_334 - CAR_295	0.07	11.00	0.8	40.00
CAR_295 - CAR_236	0.83	38.00	1.42	59.00
CAR_236 - CAR_221	0.83	60.00	1.42	15.00
CAR_221 - CAR_214	0.89	15.00	0.36	7.00
CAR_214 - CAR_200	**Bridgefield Bridge**			
CAR_198 - CAR_196	-	-	-	-
CAR_196 - CAR_169	-	-	0.26	27.00
CAR_169 - CAR_132	-	-	0.26	36.00
CAR_132 - CAR_126	--	-	-	-
Glaslaw Burn				
GLA_222 - GLA_179	0.83	67.00	-	-
GLA_179 - GLA_147	1.13	33.00	-	-
GLA_147 - GLA_116	0.77	29.00	0.97	12.00
GLA_116 - GLA_089	1.44	31.00	1.44	34.00
GLA_089 - GLA_070	**Woodview Court Culvert/Bridge**			
GLA_070 - GLA_044	1.38	26.00	1.19	26.00
GLA_044 - GLA_033	**Low Wood Road**			

4.4 Option 3: Combined storage and direct defences

This third option assessed using upstream storage and attenuation to limit the direct defences to a maximum wall height of 1.4 m throughout Stonehaven. A 1.4 m wall was considered to be an appropriate height to provide flood protection without impacting on the aesthetic nature of Stonehaven and therefore would require the minimum attenuation upstream.

Assuming a limit of 1.4 m high walls within Stonehaven, the maximum flood flow channel capacity equates to 43.5 m^3/s. Thus, storage would be required to reduce the 200 year flood flow with an allowance for climate change from 67 m^3/s to 43.5 m^3/s. Based on this revised flow constraint, the flood storage analysis has been reassessed. The results are provided in Table 4-8 below.

Table 4-8: Storage parameters

Parameter	200 year + climate change test
Storage inflow (m^3/s)	58.0
Storage outflow (m^3/s)	30.2
Attenuation (m^3/s)	27.8
Flow at gauge (m^3/s)	33.3
Flow at outlet (includes Glaslaw) (m^3/s)	43.2
Maximum level in storage (mAOD)	46.2
Embankment height (m) (based on ground level of 32.1)	14.1
Storage volume at maximum level (m^3)	970,000

The above analysis suggests that the primary flood storage area upstream of Fetteresso can provide sufficient storage to attenuate the 200 year plus climate change peak flow in the Carron Water to the point where flows would be retained within direct defences in the town with a maximum height of 1.4m.

Whilst these options are technically feasible, there are a number of additional constraints. It should also be noted that the embankment required to attenuate flows upstream of Kirkton of Fetteresso is a significant structure and unlikely to be aesthetically agreeable to the local community. It would also require significant environmental mitigation works to ensure that environmental aspects and fisheries are not adversely impacted.

There may also be breach risks and concerns to the local community. A reservoir of this size would have implications for management under the Reservoirs Act and longer term operation and maintenance costs associated with regular inspections and preparation of breach inundation flood plans.

Inundation during flood events would flood large areas of the upstream catchment and grounds of the Fetteresso Castle. There may be archaeological implications for this and the dam construction.

A review of geology maps from the British Geological Survey indicates that the proposed storage areas are mostly superficial deposits of diamicton till, alluvium and isolated deposits of sand and gravel on a bedrock of sandstone. For large storage areas suitable ground is required to ensure the stability of any retaining structure and that the storage area can be made watertight with minimal seepage. Although this may be achievable, it is unlikely to be cost beneficial based on the initial geological data.

An alternative would be to provide some attenuation on the Glaslaw Burn to reduce the degree of attenuation required on the Carron Water. This option is however unlikely to be viable as insufficient storage is available on the Glaslaw Burn to materially reduce the volume of storage needed on the Carron Water. This option has therefore not been tested.

4.5 Preferred Stonehaven Flood Protection Scheme

From the above analysis the preferred Stonehaven Flood Protection Scheme is as follows:

- Direct flood defences on the River Carron from immediately upstream of the Red Bridge to the coast.
- Modifications to the River Carron channel between the Green Bridge and Bridgefield Bridge (a distance of 435 m) removing approximately 1200m^2 of made-ground 'naturalising' the channel. This will also involve the reduction of the weir/sewer downstream of Green Bridge.
- Infilling of Bridgefield Bridge parapet to provide suitable freeboard
- Direct flood defences on the Glaslaw Burn from upstream of Carron Gardens to the upstream face of Low Wood Road.
- Raising of the Green Bridge from its current soffit of 7.71 mAOD by 1.2 m to reduce its hydraulic impact and provide sufficient freeboard.
- Raising of the White Bridge by 1.04 m to reduce its hydraulic impact.
- Raising of the Red Bridge by 1m from is soffit of 9.01 mAOD to remove its hydraulic impact and provide sufficient freeboard.
- Replacing the culvert under the Woodview Court Bridge with a box culvert with dimensions of approximately 4 m by 2 m with an invert level of 6.72 mAOD.

The Flood Protection Scheme will also require secondary defences to alleviate surface water flooding from overland flow. Stonehaven has been affected by overland flow from the Bervie Braes as a result of intense rainfall. The flood defence scheme will prevent these flows from entering the river. It is likely that they will pond behind the defences and in low lying areas. The previous flood alleviation study[5] concluded that the key areas of risk in Stonehaven include Cameron Street near the junction with Barclay Street, Barclay Street around the junction with Margaret Street and the area around Arbuthnott Place/ High Street. These would be potential locations for pumping stations that could alleviate surface water flooding. The preferred flood defence scheme is displayed in Figure 4-1 and the required flood defence heights are tabulated in Table 4-4.

[5] Stonehaven River Carron Flood Alleviation Study, JBA Consulting, July 2012

5 Cost Analysis

5.1 Introduction

Cost estimates for the scheme were previously estimated as part the original River Carron Flood Alleviation Study. These costs have been updated to take into account the revised modelling and proposed options undertaken for the purpose of this study.

Whole life costs including all enabling, capital and long term inspection and maintenance costs are required for each option. Indicative scheme costs for the design options have been determined; a summary of what is included is provided below.

5.2 Methodology

Costs have been derived from a number of sources suitable for this level of assessment, together with unit costs from previous studies and general guidance. Information gathered by the Environment Agency on more has 450 capital projects with a value of over £500 million which include contractors' direct construction costs, overheads and profits and elemental costs associated with construction. An uplift of 3% per year was applied to the gathered costs to account for inflation.

5.2.1 Flood wall costs

Unit rates for flood walls were taken from the Environment Agency Flood Risk Management Estimating Guide[6]. Costs used vary between £1,000/m^3 for higher and longer retaining walls where economies of scale apply and up to £3,000/m^3 for more complex, shorter walls in areas of restricted access.

Carron Water

Flood walls have been defined for the following three key reaches:

- Downstream of Bridgefield Bridge
- Bridgefield Road to the White Bridge
- White Bridge to the Green Bridge
- Green Bridge to the Red Bridge

In addition to the above an allowance for infilling the parapet walls on Bridgefield Bridge has been taken into account.

Glaslaw Burn

Flood walls have been defined for the following three key reaches:

- Upstream of Woodview Court
- Woodview Court to Low Wood Road

In addition to the above, an allowance for bridge parapet walls have been taken into account on Woodview Court and Low Wood Road to tie into the left and right bank defence elevations.

5.2.2 Bridge raising costs

Bridge raising costs are uncertain and ideally require contractor involvement in costing for the detailed design. However, we have assumed a cost of £80,000 to cover raising (crane hire) and abutment/footpath works for each bridge. Costs have been assumed to be the same for each bridge due to the similar construction and access.

5.2.3 Culvert upgrades (Glaslaw Burn)

Culvert upgrades are required for the Woodview Court culvert. This culvert is approximately 10m long and a £11,900/m length cost of upgrading this culvert has been estimated based on the Environment Agency Flood Risk Management Estimating Guide.

[6] Environment Agency (2010). Flood Risk Management Estimating Guide. (Unit Cost Database).

5.2.4 Channel modification costs

Channel modification costs have been estimated and built up from typical rates for all temporary works and earthworks required. This includes site set up and clearance works, traffic management, excavation and disposal costs, re-profiling and landscaping costs. Volumes of sediment have been estimated from the modelling and assumed to be in the order of 1,200m^3.

5.2.5 Pumping station costs

Pumping stations have already been noted as a requirement as part of the Arbuthnott Drain Improvement. The capital costs associated with these works in the region of £750,000.

Additional surface water pumping stations may also be required. At this stage, prior to detailed design, we have assumed that 2 additional pumping stations are required at a cost of £250,000 per unit.

5.2.6 Flood storage costs

A number of recent flood storage basins have been constructed in Scotland and in the rest of the UK. Whilst final construction costs are not always available for these, costs for a number of recent schemes are available which provide an indicative assessment of likely costs based on a volumetric approach. These include:

- Lhanbryde FPS 2004 (Moray)[7], 140,000m^3 at a cost of £17/m^3.
- Kittoch Bridge – White Cart FPS (Glasgow)[8], 665,000m^3 at a cost of £9.6/m^3.
- Blackhouse – White Cart FPS (Glasgow)[8], 806,000m^3 at a cost of £8.8/m^3.
- River Gaunless Flood Alleviation (England)[9], 1,000,000m^3 at a cost of £9.5/m^3.
- Kirkland Bridge – White Cart FPS (Glasgow)[8], 1,080,000m^3 at a cost of £4.5/m^3.
- Forres (Burn of Mosset) FPS 2005 (Moray)[7], 4,269,000m^3 at a cost of £2.7/m^3.

As can be seen from the above examples there is large variation in price per cubic metre cost for online storage capacity. Whilst there is a is a correlation in terms of costs per cubic metre and total volume a suitable value for the purposes Stonehaven should approximately £5 - £10 per cubic metre of volume stored.

The estimated costs for upstream storage would need to include all aspects of construction including concrete works, steelwork, earthworks, access arrangements, mechanical and electrical items and all habitat and landscaping works.

When considering final costs the following would also need to be taken into consideration:

- Land purchase costs have been excluded at this stage as these may not be required. Land valuations are in the order of £16,055/hectare for arable land in Aberdeenshire[10].
- Plant protection and service diversions (estimated to be in the order of £200k).
- Ancillary works to paving/roads (estimated to be in the order of £500k).

5.2.7 Other general cost allowances

In addition to the construction costs the following items were added:

- Professional fees (10% of civil works);
- Site investigation (1.5% of civil works; 1.5% for the storage options);
- Statutory fees (2.5% of civil works);
- Optimism bias (see below).

5.2.8 Optimism Bias

An optimism bias of 60% has been applied and is representative of a scheme at the design stage of development.

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[7] Scottish Flood Defence Asset Database
[8] Glasgow City Council (2004). White Cart Water Flood Prevention Scheme 2004. Economic Appraisal.
[9] Environment Agency (2010b). Current (Magazine for the Environment Agency, NEECA and NCF Partners). Issue 13, 2010.
[10] Valuation Office Agency (2011). Property Market Report 2011. The annual guide to the property market across England, Wales and Scotland.

5.2.9 Operation and maintenance costs

The maintenance and operation costs for each option have been estimated separately. Annual operation and maintenance costs have been estimated and the assumptions listed in Table 5-1.

Table 5-1: Annual maintenance costs and assumptions

Option	Annual maintenance cost assumption	Annual cost (£k)
'Do minimum'	No costs assumed	0
Option 1: Direct defences	1) £800 per year per km based on Environment Agency guidance for concrete wall annual maintenance costs km. 2) Inspection costs for bridges and culverts at £432 per annum. 3) Channel modification monitoring and repeat works. Monitoring assumes 2 days survey every 5 years. Repeat works assume the capital costs every 20 years (annualised). 4) Annual pumping station costs for inspection, servicing and running costs of £2,250 per pumping station.	Total = £11,637 per annum
Option 2: Flood storage	1) 0.2% of the capital costs for structural maintenance to include all statutory inspections and flood plan maintenance. 2) Annual maintenance of structures and Glaslaw Burn defences as per Option 1. 3) Annual pumping station costs for inspection, servicing and running costs of £2,250 per pumping station. 1) £800 per year per km based on Environment Agency guidance for concrete wall annual maintenance costs km. 2) Inspection costs for bridges and culverts at £432 per annum. 3) Channel modification monitoring and repeat works. Monitoring assumes 2 days survey every 5 years. Repeat works assume the capital costs every 20 years (annualised). 4) Annual pumping station costs for inspection, servicing and running costs of £2,250 per pumping station.	Total = £30,793 per annum
Option 3: Flood Storage and Direct Defences	1) 0.2% of the capital costs for structural maintenance to include all statutory inspections and flood plan maintenance. 2) Annual maintenance of structures and Glaslaw Burn defences as per Option 1. 3) Annual pumping station costs for inspection, servicing and running costs of £2,250 per pumping station. 1) £800 per year per km based on Environment Agency guidance for concrete wall annual maintenance costs km. 2) Inspection costs for bridges and culverts at £432 per annum. 3) Channel modification monitoring and repeat works. Monitoring assumes 2 days survey every 5 years. Repeat works assume the capital costs every 20 years (annualised). 4) Annual pumping station costs for inspection, servicing and running costs of £2,250 per pumping station.	Total = £33,996

5.3 Total costs

A summary of the total costs is given in the table below.

Table 5-2: Annual maintenance costs and assumptions

Option	PV capital costs (£k)	PV operation & maintenance costs (£k)	Total PV costs (£k)	Total PV costs + Optimism Bias (£k)
Option 1: Direct defences	9,998,083	£344,242	10,332,325	16,531,721
Option 2: Maximum flood storage with minimum direct defences	16,871,402	£1,013,533	17,884,936	28,615,897
Option 3: Minimum flood storage with 1.4 m high direct defences	13,753,754	£918,046	16,666,095	26,665752

6 Economic Appraisal

An economic appraisal was undertaken as part of the original River Carron Flood Alleviation Study. This appraisal has been updated based on the revised modelling, options and costs undertaken for the purpose of this study.

No changes to the flood damages have been undertaken. It is anticipated that additional appraisal will be undertaken on the preferred approach during the design stage once detailed design has been undertaken and a more thorough assessment of all costs has been undertaken.

6.1 Options assessed

The previous analysis identified 5 options within the economic appraisal. Of these, the direct defence option was deemed to be the most economically viable option. However, the storage option was also viable, but with a reduced benefit-cost ratio.

These two primary options have been revised to assess the economic viability based on the updated modelling. The following options have been reviewed and presented as part of this report:

- Baseline (Do Minimum) option - This represents the current existing situation within Stonehaven and reflects that a number of works have been undertaken since the 2009 flood to manage flood risk. This option also reflects the ongoing maintenance of the watercourse undertaken by the Council.
- Option 1 - Direct defences to the Carron Water and Glaslaw Burn incorporating bridge raising of the Green Bridge, the White Bridge and the Red Bridge. Channel modification is also proposed to increase channel capacity. This scenario assumes an allowance for climate change within the analysis.
- Option 2 - Flood storage in the catchment upstream of Bridge of Fetteresso. Storage is capable of reducing flood flows in the Carron Water from the 0.5% AP (200 year) flood to the minimum channel capacity through Stonehaven (upstream of the Green Bridge); roughly equivalent to a flow in the region of the 50% - 20% AP (2-5 year) flood. Works on the Glaslaw Burn are also required.
- Option 3 - Flood storage in the catchment upstream of Bridge of Fetteresso. Storage is capable of reducing flood flows in the Carron Water from the 0.5% AP (200 year) flood to restrict the maximum wall height of defences through Stonehaven to 1.4m. Works on the Glaslaw Burn are also required.

6.2 Summary of flood damages

A summary of the original properties at risk and flood damages for Stonehaven are given in the table below. These are based on the following assumptions:

- Depth damage data provided by the FHRC MCM (2010 version) with values updated to July 2011;
- Estimated threshold levels defined by number of steps above ground levels (150mm per step plus LiDAR ground levels);
- Flood durations assumed to be less than 12 hours;
- Property areas defined by Mastermap areas;
- Property Present Value damages capped by market values;
- Flood damages for the option scenarios and above design events have been capped at the 200 year values.

Table 6-1: Summary of properties at risk

Option	Number of properties flooded by return period (years)							
	5	10	25	50	75	100	200	1000
'Do minimum'	0	5	163	280	307	340	372	427
Option 1: Direct defences	0	0	0	0	0	0	0	372
Option 2: Flood storage	0	0	0	0	0	0	0	372

Table 6-2: Summary of flood damages

Option	Flood damages by return period (years)							
	5	10	25	50	75	100	200	1000
'Do minimum'	£0	£76	£4,369	£7,516	£8,503	£9,324	£10,796	£13,357
Option 1: Direct defences	0	0	0	0	0	0	0	£10,796
Option 2: Flood storage	0	0	0	0	0	0	0	£10,796

Based on the above the total annual average damage (AAD) and Capped Present Value damages (PVd) are provided in the table below.

Table 6-3: Summary of AAD and PVd

Option	Total AAD (£k)	PVD (£k)	Capped PVd(£k)
'Do minimum'	451.4	13,458	12,517
Option 1: Direct defences	33.7	1,006	1,006
Option 2: Flood storage	33.7	1,006	1,006
Option 3: Flood storage and direct defences (1.4m)	33.7	1,006	1,006

In addition to the above direct flood damages, allowances for intangibles and indirect damages have also been estimated. These are summarised in the table below.

Table 6-4: Summary of indirect and intangible flood damages

Option	Indirect PV damages (£k)	Intangible PV damages (£k)	Total flood damages (£k)	Total damages avoided (£k)
'Do minimum'	1,293	1,384	15,195	-
Option 1: Direct defences	96	49	1,151	14,047
Option 2: Flood storage	96	46	1,148	14,047
Option 3: Flood storage and direct defences (1.4m)	96	46	1,148	14,047

6.2.1 Impact of climate change

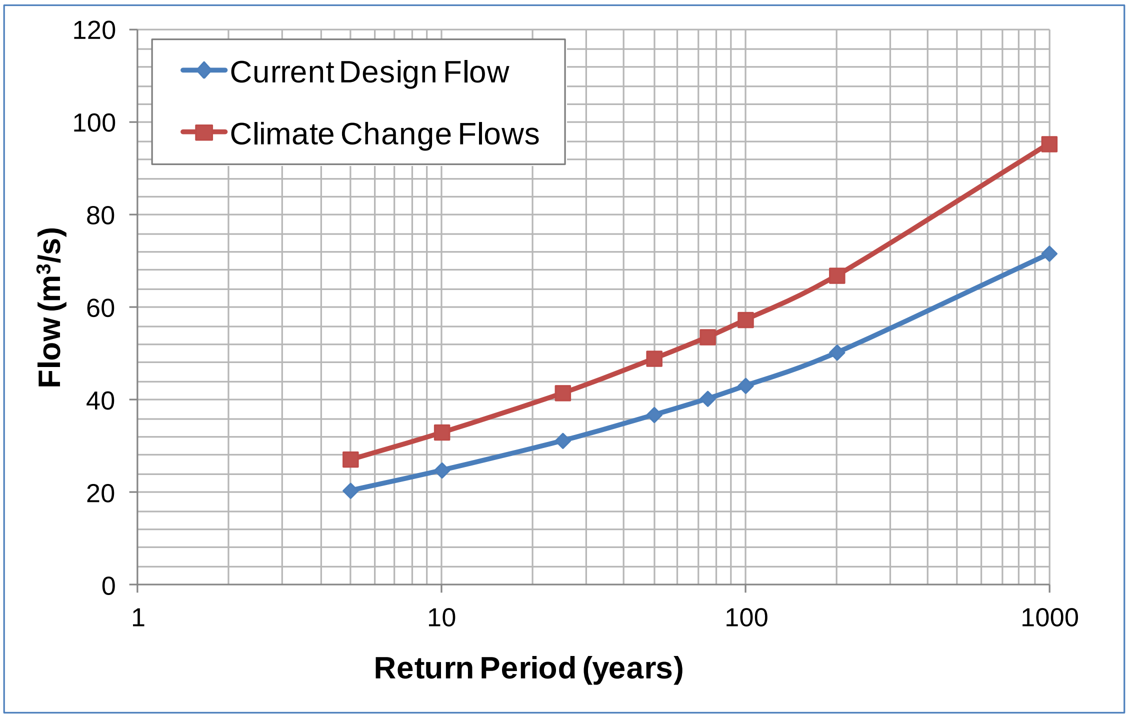
An allowance for climate change was considered as part of the earlier appraisal. This assumed that all options could be designed to incorporate the increased flood flows due to climate change. Further analysis and revisions to the climate change estimates using UKCP09 data suggest that whilst the direct defence option can accommodate an allowance for climate change, the storage option does not have the capacity to allow for increased flood flows and storage volumes needed to be stored.

For the purpose of assessing the direct defence option the damages avoided by the scheme will increase over time as flood flows increase. For example, a 100 year event will become a 30 year flood in 2080; thus the frequency of the flood and the damages will increase. The estimated future climate change frequencies are provided in the table below and shown graphically in Figure 6-1.

Table 6-5: Summary of the change in return period for future floods assuming a 33% increase in flood flows

Return Period (yr)	5	10	25	50	75	100	200	1000
AP (%)	**20**	**10**	**4**	**2**	**1.33**	**1**	**0.5**	**0.1**
Flow (m^3/s)	20.5	24.9	31.3	36.9	40.4	43.2	50.4	71.8
Climate change flows (m^3/s)	27.3	33.1	41.6	49.1	53.7	57.5	67.0	95.5
Estimated return periods (yr)	2	4	8	13	20	30	60	260

Figure 6-1: Impact of climate change on flood flows



These climate change estimates have been used to amend the probabilities for each return period flood assessed. AADs have been estimated on these revised probabilities at the end of the financial period. Discounting is then carried out assuming a linear increase in AADs to take into consideration this increase in flood damages as a result of climate change over time.

The total damages for the direct defence option are presented in Table 6-6. This has the impact of increasing the Present Value flood damages for the Do Minimum option from £15.2 to £22.5 million. The difference in terms of damages avoided between the two options is therefore enhanced due to the fact that Option 1 incorporates climate change, whereas Option 2 does not.

Table 6-6: Total flood damages assuming climate change

Option	Total flood damages (£k)	Total damages avoided (£k)
'Do minimum'	22,505	-
Option 1: Direct defences	2,195	16,351
Option 2: Flood storage	5,956	16,549

6.3 Economic appraisal

The benefit-cost analysis of the flood alleviation options has been carried out based on the methodology given in the 'Flood Prevention Schemes: Guidance for Local Authorities' report[11] by the Scottish Executive, April 2005. The principles are summarised as follows:

- Derive the damages associated with do-nothing;
- Derive the damages associated with each scheme option;
- Derive the benefits (damages avoided) associated with each option;
- Derive the costs for each option; and
- Derive the benefit-cost ratios for each option.

[11] Flood Prevention Schemes: Guidance for Local Authorities. April 2005. Scottish Executive.

In all cases, the benefits and costs are transformed into present values.

6.3.1 Assumptions

The following assumptions have been made:

- The life span of the scheme is assumed to be 100 years.
- Discounting of damages and scheme costs have been calculated using the revised Treasury discount rates as recommended by the 2003 revision to the Green Book[12]. This revision set a time varying discount rate of 3.5% for the first 30 years, 3% for years 31-75 and 2.5% for years 76-125. This equates to a Present Value factor of 29.81.

6.3.2 Benefit-cost results for Option 1 - Direct Defences

A summary of the benefit cost results for the direct defence option are provided in the table below. This option has a benefit cost ratio of 1.32 and a Net Present Value of £15.4 million. This shows that the scheme is cost effective and offers a long term benefit in terms of flood mitigation to Stonehaven.

Table 6-7: Summary of benefit-cost calculation for direct defence Option 1 (£k)

	'Do Nothing	Option 1: Direct defence
Total PV costs (£k)	-	9,998
Total PV costs + Optimism bias (£k)	-	16,531
PV damage (£k)	22,505	2,195
PV damage avoided (£k)	-	20,310
Net present value (£k)	-	3,779
Benefit-cost ratio	**-**	**1.23**

6.3.3 Benefit-cost results for Option 2

A summary of the benefit cost results for Option 2 are provided in the table 6-8. The option has a benefit cost ratios of 0.71 and a Net Present Value of £ -0.8 million. This shows that the scheme is not cost effective and does not provide a long term benefit in terms of flood mitigation to Stonehaven.

Table 6-8: Summary of benefit-cost calculation for Option 2 (£k)

	'Do Nothing	Option 2: Storage
Total PV costs (£k)	-	16,871
Total PV costs + Optimism bias (£k)	-	28,616
PV damage (£k)	22,505	2195
PV damage avoided (£k)	-	20,310
Net present value (£k)	-	-8,306
Benefit-cost ratio	**-**	**0.71**

6.3.4 Benefit-cost results for Option 3

A summary of the benefit cost results for Option 3 are provided in the table 6-9. The option has a benefit cost ratio of 0.76 a Net Present Value of £-0.6 million. This shows that the scheme is not cost effective and does not provide a long term benefit in terms of flood mitigation to Stonehaven.

[12] The Green Book: Appraisal and Evaluation in Central Government, January 2003. HM Treasury.

Table 6-9: Summary of benefit-cost calculation for Option 3 (£k)

	'Do Nothing	Option 2: Storage
Total PV costs (£k)	-	15,748
Total PV costs + Optimism bias (£k)	-	26,665
PV damage (£k)	22,505	2195
PV damage avoided (£k)	-	20,310
Net present value (£k)	-	-6,355
Benefit-cost ratio	**-**	**0.76**

6.3.5 Economic preferred option

Based on the economic appraisal carried out, the preferred option is the direct defence option. The storage option, following a more thorough analysis of the hydrology and the requirements for flood attenuation volumes, is not an economically viable option.

7 Conclusions

This report has assessed the potential options for mitigating flood risk to Stonehaven arising from the River Carron, Glaslaw Burn and Bervie Braes and proposed a preferred Flood Protection Scheme for Stonehaven. The options assessed where full flood defences (flood walls) in conjunction with channel modifications, raising of bridges that restrict the flow along the River Carron, and the attenuation of flows through the provision of upstream storage and a combination of upstream storage and direct defences.

Various options were tested for storage, however, the required storage volume resulted in this option being considered not cost beneficial and would also require flood walls within the town itself. When using storage to reduce the height of flood defences, the scheme was also not economically beneficial.

The final preferred scheme is for direct defences:

- Direct flood defences on the River Carron from immediately upstream of the Red Bridge NGR OS 386915 785636 to the coast at NGR OS 387522 785732.
- Occasional modifications to the River Carron channel between the Green Bridge and Bridgefield Bridge (a distance of 435m) removing approximately 1200 m^3 of material from the channel. This will also involve the reduction of the weir/sewer downstream of Green Bridge at NGR OS 387052 785636.
- Direct flood defences on the Glaslaw Burn from upstream of Carron Gardens to the upstream face of Low Wood Road
- Replacing the culvert under the Woodview Court Bridge with a box culvert with dimensions of approximately 4 m by 2 m with an invert level of 6.72 mAOD.
- Raising and relocating of the Green Bridge from its current soffit of 7.71 mAOD by 1.2 m to reduce its hydraulic impact and provide sufficient freeboard.
- Raising of the White Bridge by approximately 1 m to reduce its hydraulic impact.
- Raising of the Red Bridge by 1 m from is soffit of 9.01 mAOD to reduce its hydraulic impact and provide sufficient freeboard.

At present the most cost beneficial option is to install direct defences. Significant increases in flow estimates from previous studies have resulted in an increase in the required volume of potential storage areas. A combination of storage and direct defences may be able to reduce the aesthetical impact of the scheme however the associated costs would be prohibitive and this can be can also be achieved though more innovative solutions such as the inclusion of automatic flood defences as part of the scheme. The proposals offer a cost beneficial scheme which is environmentally sustainable.

Appendices

A Appendix - Climate Change Analysis

A.1 Reason for additional work

The impact of climate change on flood flows is a key risk in terms of the design and an area that would benefit from further assessments. Typically for flood studies, the potential effects of climate change are considered by up scaling by a factor of 20%, as recommended within SEPA's most recent guidance for flood risk assessments.

However, recent guidance for England and Wales has provided regionalised estimates of how climate change will impact upon river flows through the next century based on the UKCP09 projections. This information does not support Scottish catchments but is available for the Solway, Tweed river basins and Northumberland. These three regions are presented in the table below.

It is clear from the recommendations in use in England and Wales for adjacent regions with similar meteorological and hydrological processes, that the best estimate of climate change increase in flow by 2080 for Scottish catchments may be as much as 25-30% with a larger degree of uncertainty that should be tested further.

Region	Total potential change for 2020s	Total potential change for 2050s	Total potential change for 2080s
Tweed			
Upper range	25%	35%	35%
Best estimate	15%	20%	30%
Lower range	0%	5%	15%
Northumberland			
Upper range	25%	30%	50%
Best estimate	10%	15%	20%
Lower range	0%	0%	5%
Solway			
Upper range	25%	35%	65%
Best estimate	15%	20%	25%
Lower range	5%	15%	10%
Under the UKCP09 data, the 2020s classification covers the period 2010 to 2039, the 2050s the period 2040 to 2069, and the 2080s the period 2070 and 2099.			

SEPA has undertaken additional research into the impact of climate change on flood flows and commissioned a report from CEH (An assessment of the vulnerability of Scotland's river catchments and coasts to the impacts of climate change). The project reworked and re-presented the Defra/EA FCERM project FD2020's summary catchment-based results for those FD2020 catchments located in Scotland.

Unfortunately SEPA are not in a position to share the report more widely for a number of reasons. The main issue is that the outputs are potentially very difficult to interpret, and they keen to provide clear guidance to local authorities and others on how to incorporate climate change within projects.

Therefore in the mean time and to inform the design of the Stonehaven scheme a more thorough assessment of the UKCP09 data has been undertaken.

A.2 Methodology

Future increases in the peak flow for the years 2020, 2050 and 2080 have been calculated to inform the design, critical design constraints and any need for further flood risk intervention in the future.

This approach and additional assessment is deemed to be the preferred option for the assessment of climate change impacts on flood flows prior to further guidance issued by SEPA.

The assessment is a short term and simplified approach that avoids highly specialised and complex hydro-meteorological modelling.

UKCP09 provides scenarios for the upper and lower range of possible percentage increases in rainfall for 25km squares for the entire country. This information has been extracted from the online data source for the 'worst case' scenario; the high emission scenario. The percentage increase in monthly rainfall for winter months (December to February) has been used as a proxy for future increased flood flows, by assuming that a percentage increase in rainfall translates to the same increase in flood flows. The data extracted has been used to compare against the upper and lower range of flow increases presented by the Environment Agency for the Tweed and Northumberland catchments; deemed to be the most appropriate catchments for comparison.

A.3 Results and Discussion

Cumulative density functions (cdfs) of the UKCP09 percentage rainfall changes for the winter months (December to February) were downloaded from the UKCP09 website user interface:

http://ukclimateprojections-ui.defra.gov.uk/ui/admin/login.php.

UKCP09 Grid box 654 was selected as this grid box contained almost all of the Carron Water catchment area. For the purposes of analysis, it was assumed that the 80%, 50% (median) and 20% intervals on the cdfs could be used to represent the "Upper range", "Best estimate" and "Lower range" for the potential changes in rainfall (and flow) in the Stonehaven area. The purpose of this selection was to obtain a reasonable and practical range of percentage change values for analysis.

From the accompanying table, it can be seen that the resulting estimated changes for the Stonehaven area are similar to those previously estimated for the Tweed, Northumberland and Solway catchments. The "Best estimate" for the Stonehaven area ranges from 5% (for the 2020s) to 33% (for the 2080s). The "Upper range" and "Lower range" range from 19% (for the 2020s) to 67% (for the 2080s) and -7% (for the 2020s) to 10% (for the 2080s), respectively.

Region	Total potential change for 2020s	Total potential change for 2050s	Total potential change for 2080s
Tweed			
Upper range	25%	35%	35%
Best estimate	15%	20%	30%
Lower range	0%	5%	15%
Northumberland			
Upper range	25%	30%	50%
Best estimate	10%	15%	20%
Lower range	0%	0%	5%
Solway			
Upper range	25%	35%	65%
Best estimate	15%	20%	25%
Lower range	5%	15%	10%
Carron Water			
Upper range	14%	29%	53%
Best estimate	5%	17%	33%
Lower range	-3%	6%	17%

For reference, a list of potential changes to rainfall (and flow) for the Stonehaven area for each decade available under UKCP09 is listed in the table below. These values were estimated using an identical process to that described above.

Region	Total potential change for 2020s (2010 to 2039)	Total potential change for 2030s (2020 to 2049)	Total potential change for 2040s (2030 to 2059)	Total potential change for 2050s (2040 to 2069)	Total potential change for 2060s (2050 to 2079)	Total potential change for 2070s (2060 to 2089)	Total potential change for 2080s (2070 to 2099)
Upper range (80%ile)	14%	17%	23%	29%	37%	44%	53%
Best estimate	5%	7%	12%	17%	22%	27%	33%
Lower range (20%ile)	-3%	-1%	3%	6%	10%	13%	17%

B Hydraulic Model

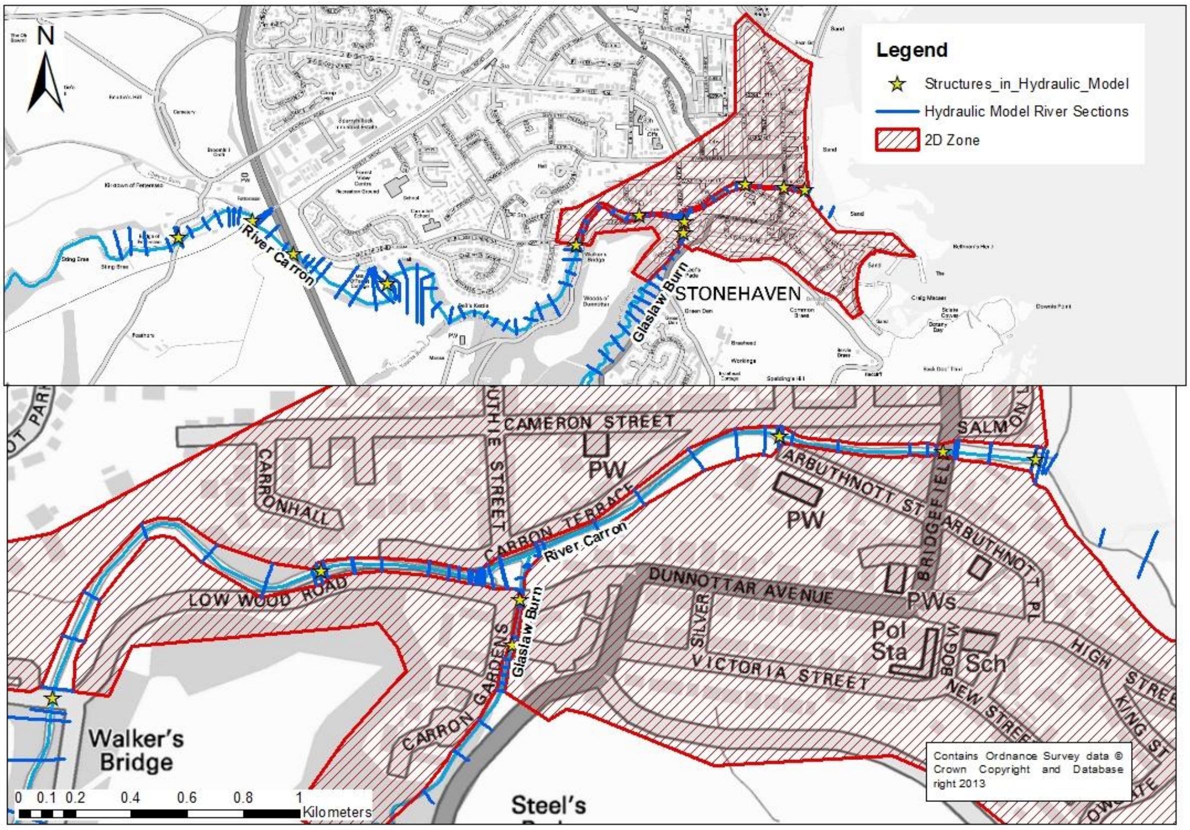
B.1 Model Geometry

The upper part of the River Carron to the Walker's Bridge is modelled as a 1D reach as is the upper section the Glaslaw Burn. 1D modelling is appropriate for these reaches as the river valleys are well defined and therefore flood routing is relatively simple. The 2D model domain covers the areas of Stonehaven downstream of the Walker's Bridge to the coastal outfall of the model and includes the lower section of the Glaslaw Burn from immediately upstream of Carron Gardens.

The 1D river model is linked to the 2D zone by hydraulic spills that represent the River Banks. When the water level between two sections exceed an adjacent bank level, water is permitted to flow into the 2D zone where it will follow overland flow paths. Flow is also allowed to re-enter the 1D river model if the depth in the 2D zone exceeds adjacent depths in the 1D reach..

The geometry of the hydraulic model is represented in Figure 3-1

Figure B-1: Hydraulic Model Geometry



B.2 Boundary Conditions

Boundary Conditions are required at the model limits; the upstream point of each reach and at the downstream limit of the watercourse. The boundary conditions used for hydraulic model are as follows:

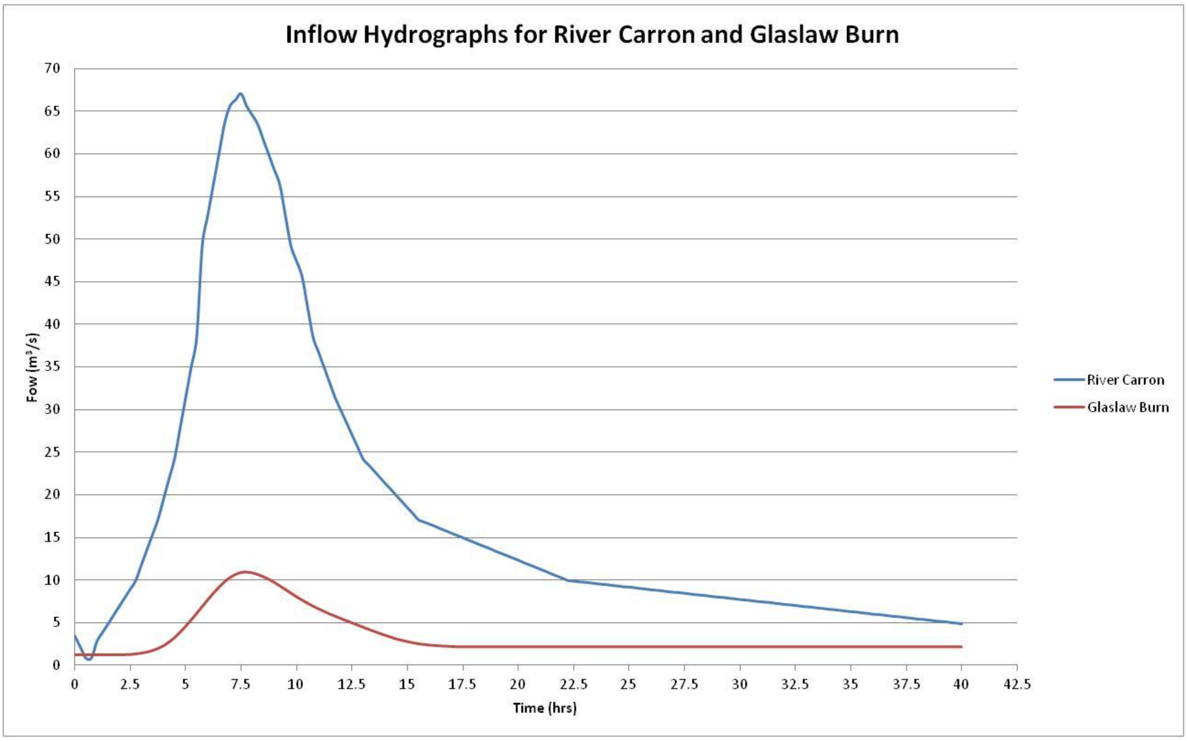
River Carron Upstream Boundary	Flow - Time Hydrograph
Glaslaw Burn Upstream Boundary	Flow - Time Hydrograph
River Carron Downstream Boundary	Time - Stage data representing tidal harmonic
Glaslaw Burn Downstream Boundary	Confluence with River Carron

Upstream Boundaries

The flow-time hydrographs that represent the upstream boundary conditions have been created using the peak flows estimated in Section 2.3.1. The hydrograph shape for the River Carron upstream boundary was generated using the methodology described in Archer et al[15] to generate a standardised hydrograph.

The design hydrographs for the River Carron and the Glaslaw Burn are illustrated in Figure B-2

Figure B-2: River Carron and Glaslaw Burn Hydrographs



Downstream Boundary

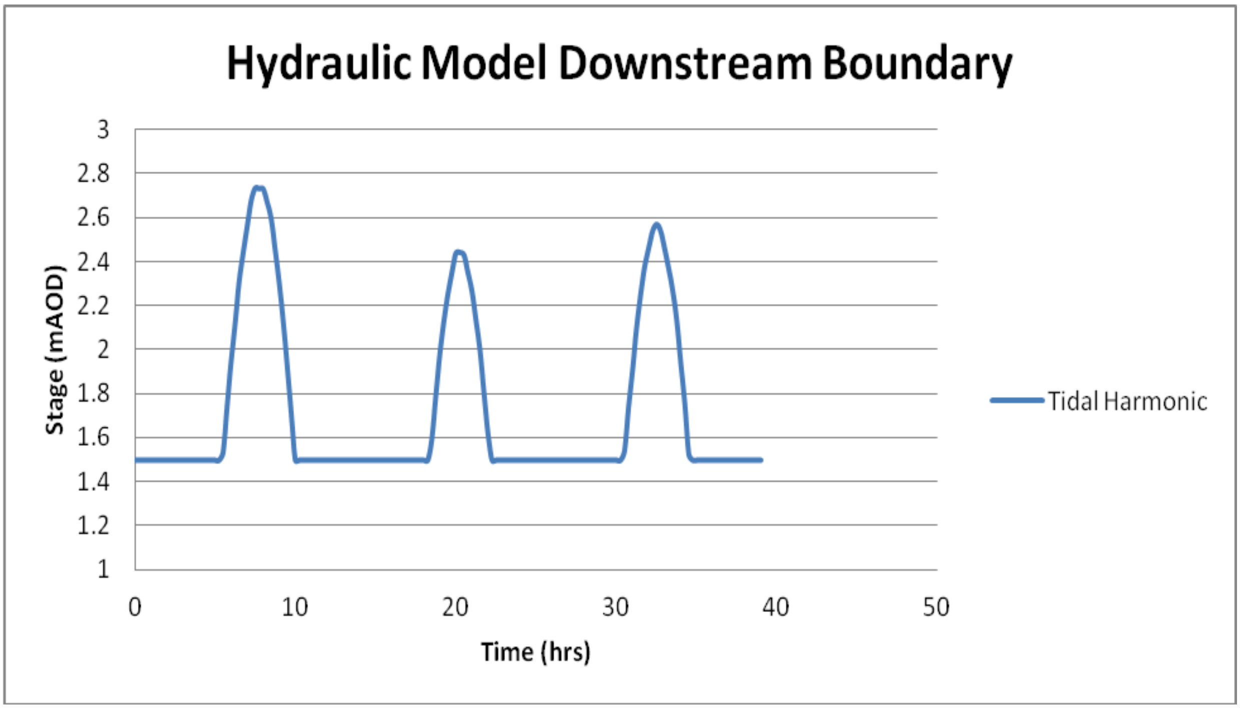
The downstream boundary of the model is the coastal outfall of the River Carron. The boundary has been modelled as a stage-time graph that represents the tidal harmonics of the sea. The tidal harmonic used was derived using extreme sea levels taken the Environment Agency's 2011 report on coastal flood boundary conditions[16] to represent peak tide levels consistent with a return period of 1 year and timed to coincide with the peak of the River Carron hydrograph to represent the worst case scenario. The model does not account for wave propagation up the channel.

The downstream boundary is illustrated in Figure B-3

[15] D. Archer, M Foster, D Faulkner and J. Mawdsley, 2000. The synthesis of design flood hydrographs. In: Flooding Risks and Reactions. Proceeding of the Water Environment 2000 conference, 5 October 2000. Institute of Civil Engineers, London.

[16] McMillan et al, 2011. Coastal flood boundary conditions for UK mainland and islands [project SC60064/TR2] Environment Agency report.

Figure B-3: Hydraulic Model Downstream Boundary



C Appendix - Flood storage analysis

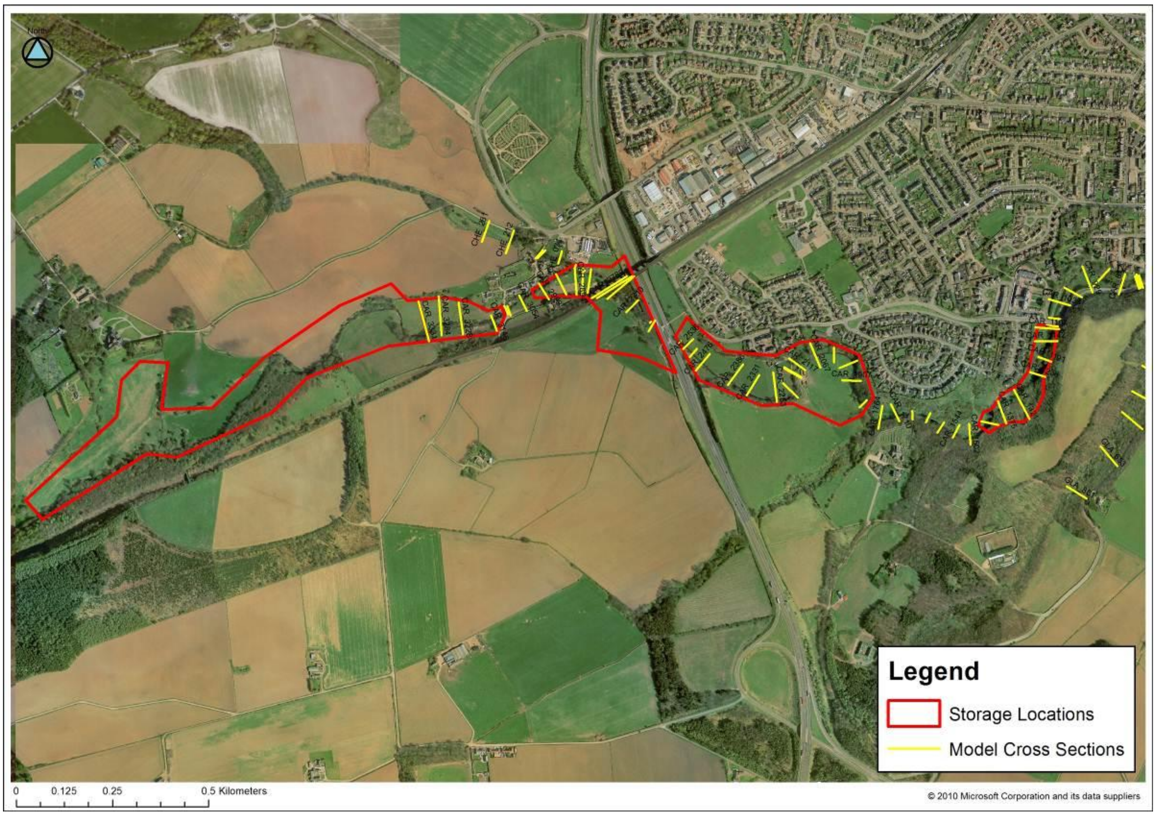
C.1 Background

The previous flood alleviation study for Stonehaven reviewed the options for flood mitigation via flood storage in the mid to lower catchment. This assessment assessed a number of locations based on available storage volumes derived by LiDAR topographic information. No specific modelling was undertaken to review and confirm that the available volumes were available and could attenuate flood peaks sufficiently. No geotechnical investigations were undertaken to assess site suitability

C.2 Review of previous storage locations

The previous storage assessment looked at a series of possible locations for the potential to provide flood storage sufficient to attenuate flood flows in Stonehaven. This assessment looked at the total volume required and the total volume available within each location. No modelling was undertaken and the volume required assumed a standard catchment critical design duration with no allocation for additional storage volumes for longer duration flood events. The following sites were investigated as shown in the figure below:

Figure C-1: Locations of storage sites investigates by previous assessment.



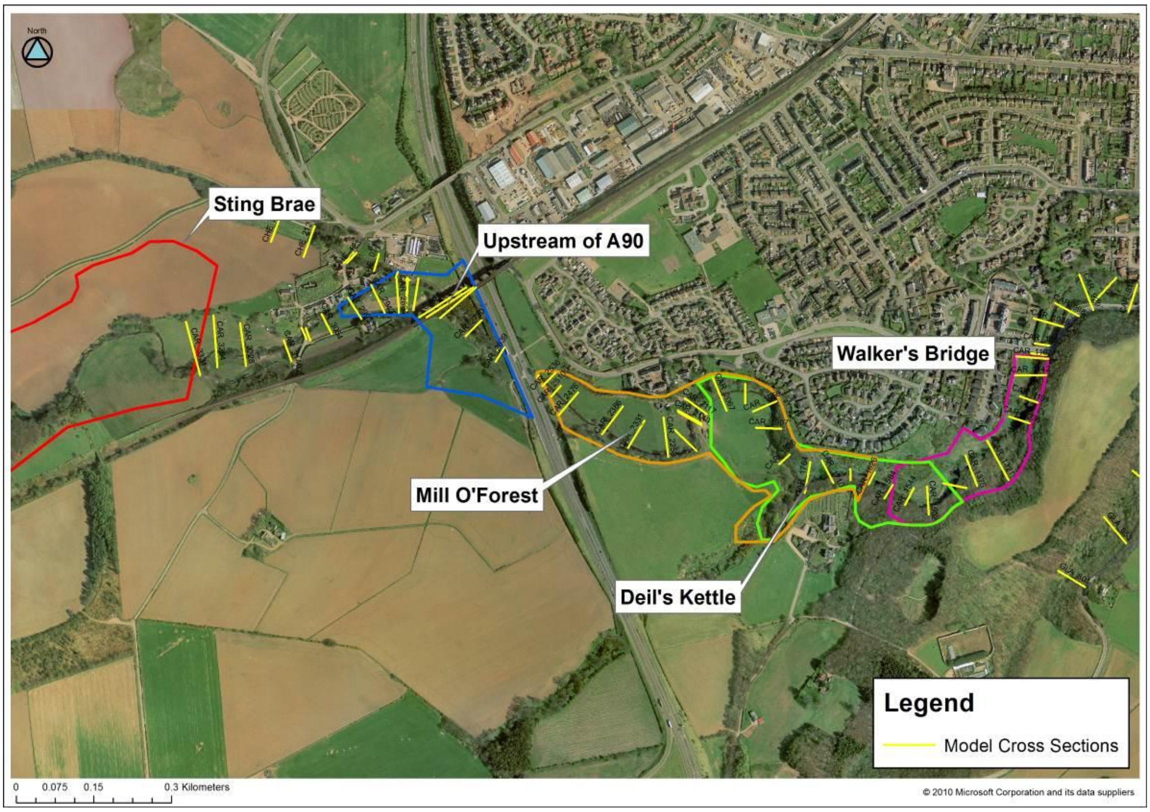
- **Walker's Bridge**. This is a very small storage area upstream of Walker's Bridge. The storage area maximum level was defined as the bridge deck (14.6mAOD). A review of the LiDAR levels suggests that the maximum water level is more likely to be in the region of 13.3mAOD on the right bank.
- **Mill O'Forest**. This is an area south of Mill of Forest Road, constrained by properties to the north and a number of out buildings on the floodplain itself. The storage area assumed that maximum levels would take into account these buildings with a maximum level of 22.3mAOD. Additional volumes could be retained in this area if this constraint was removed, through compulsory purchase of buildings and land.

- **Upstream of A90**. This is a small storage area upstream of the A90 road embankment. Levels are constrained by properties and riparian land to the north although the maximum level was previously based on the A90 deck level (31.1 mAOD).
- **Sting Brae**. This is a large area of agricultural land upstream of Bridge of Fetteresso. The storage area was split into 3 zones rather than a single storage area. Two locations were also assessed although the upper location is the only valid one due to the presence of properties immediately upstream of the Bridge of Fetteresso.

C.3 Definition of revised storage areas

The existing storage area proposals, digital mapping, aerial photographs and the LiDAR data were reviewed to check for additional potential storage areas and to revise the existing ones. Five areas were identified for further modelling analysis as illustrated in figure C -2.

Figure C -2: Revised storage sites investigated



Walker's Bridge. This has been retained although much of the area available already floods and there is not much addition storage above existing flood levels to retain additional storage volumes. The maximum flood level is constrained on the right bank by the levels along Low Wood Road; hence the maximum flood level at this point is 13.3 mAOD. A 1 m freeboard has also been assumed reducing this to 12.3 mAOD. Storage above this level would require a wall along the road which is not appropriate.

Deil's Kettle. A new storage area has been defined which is a downstream extension to the Milll O'Forest area. Between Walker's Bridge and Mill O'Forest, the Carron Water flows through a steep sided floodplain suitable for storage. Locating a dam between Riverside Drive and Murray Place would provide additional storage in the floodplain upstream. Access is constrained for construction and operational needs and would need to be reviewed further. Maximum flood levels would be constrained by the existing property levels in the Mill O'Forest area (23 mAOD). A 1m freeboard has also been assumed reducing this to 22 mAOD.

Mill O'Forest. This assumes the same areas as previous, but with an important difference and assumption; to make the most of the floodplain storage the current buildings and ownership of the land on the floodplain would be purchased to allow repeat flooding, together with removal of the buildings. Based on this assumption, the flood levels could be greater, thus providing a greater attenuation of flow from the previous assumption. Maximum flood levels would be constrained by the existing property levels in the Mill O'Forest area (28mAOD). A 1 m freeboard has also been assumed to bring this to 27 mAOD.

Upstream of A90. This has been retained although much of the area available already floods and there is not much additional storage above existing flood levels to attenuate flood flows. The maximum flood level is constrained by riparian properties upstream and the A90 road itself; hence the maximum flood level at this point is 31 mAOD. A 1 m freeboard has also been assumed reducing this to 30 mAOD.

Sting Brae. Rather than to split this area into multiple storage areas a single storage area was tested. This would maximise the available land and available storage within the floodplain. The land upstream includes land on the Fetteresso Castle estate and consultation with third parties would be essential. Other than the existing land-use there is relatively little infrastructure that constrains the flood storage; thus the maximum level is defined by the road to the north and the railway to the south. Maximum levels have been set at 47 mAOD when a 1 m freeboard is adopted.

C.3.1 GIS Analysis of storage areas

For the purposes of flood attenuation modelling an elevation/area relationship is required to consider the available storage in the area proposed. The LiDAR digital elevation model was used to determine this relationship at 0.5m intervals for each of the 5 areas assessed.

C.3.2 ISIS model set up

A series of simple ISIS models have been built to assess the storage and attenuation potential for each area. Each model consists of a series of units representing the inflow, the storage area (based on the area/elevation relationships), a weir and orifice unit to limit flows downstream and 2 cross sections downstream (taken from the original surveyed model at the appropriate location).

The inflows to the model are based on the 200 year flood (0.5% AP flood) for the total catchment. This is broadly appropriate for the lower 4 storage areas, but is a simplification for the upper catchment.

The models have been run for the catchment design storm duration (10.5 hour duration) and the orifice outlet varied to allow maximum attenuation up to the maximum flood level for each zone. This is an iterative process to obtain the necessary flow and level balance.

Providing storage on a catchment will consequently alter the concentration time due to the storage lag. It is important to find the storm duration that would result in the maximum flow downstream. This has been carried out by running the model for various storm durations and investigating the storage outflow and storage area flood levels.

C.4 Requirement for flood attenuation

Following revisions to the hydrology and revised modelling, the requirements for flood attenuation have changed. The minimum flow that is retained in bank is 16m^3/s, which occurs upstream of the Green Bridge. This is equivalent to a return period event of between a 2 year and 5 year flood.

Based on the above, all flood attenuation modelling has assumed that the maximum downstream flood flows (measured at the gauge) should be limited to the 2 year return period flood flow (14.5 m^3/s).

It should be noted that this capacity is based on the assumption of top of bank levels. Therefore it is assumed that no blockage of structures occurs and any informal embankments and walls present along the watercourse are of sufficient structural integrity to avoid failure under flood loading. This is deemed to be acceptable as previous flooding has not lead to significant failures.

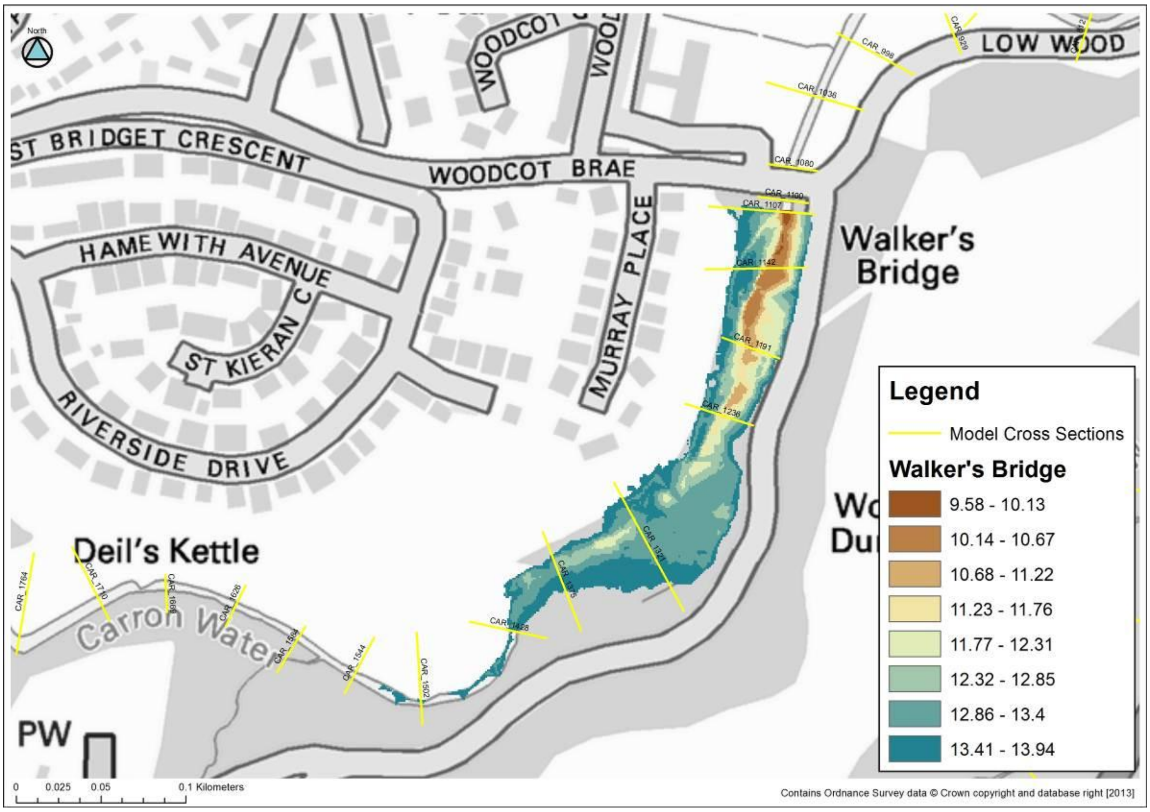
C.5 Model runs

The models for each of the proposed storage areas have been run and the results are presented in the sections below.

C.5.3 Walker's Bridge

The results below illustrate that this area does not provide any significant attenuation. This is probably due to the fact that much of the area upstream already floods and there is little additional area available to store flood water. Alternative flood attenuation durations have not been tested as this is not a viable option.

Figure C -3: Location and ground levels associated with Walker's Bridge zone



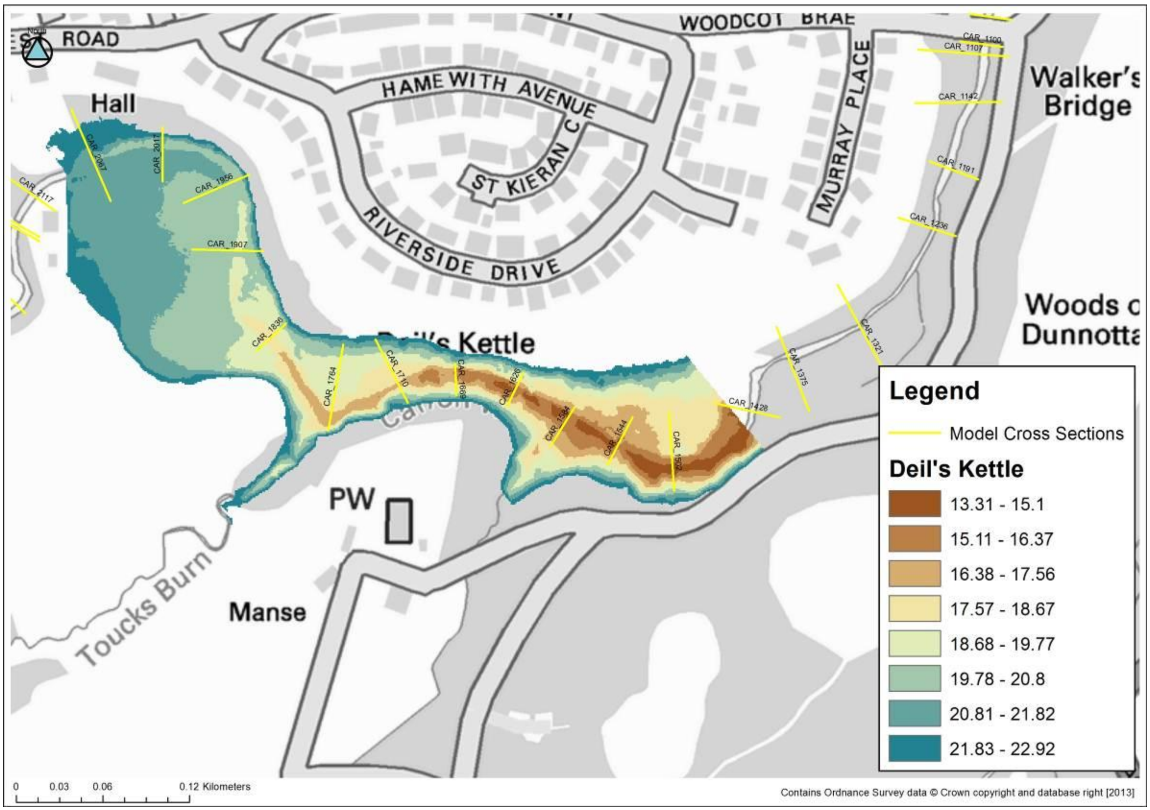
Parameter	Values for catchment critical storm duration	Values for storage optimal storm duration
Duration	10.5 hour	Not tested
Inflow	50.40 m^3/s	
Outflow	50.30 m^3/s	
Attenuation	0.10 m^3/s	
Minimum level	9.10 mAOD	
Maximum level	14.27 mAOD	
Embankment height	6.17 m	
Maximum safe level	12.30 mAOD	
Storage volume at max level	17,220 m^3	

C.5.4 Deil's Kettle

The results below illustrate that this area does provide some flow attenuation although this is not significant. This suggests that this area on its own would not be a viable option for flood attenuation. The height of the dam required is also very high for such a small volume available, suggesting that it is not a very efficient location for attenuation.

The critical design storm duration has been tested which suggests that the critical duration is 16.5 hours. The model was optimised for outflow and attenuation resulting in a maximum outflow of 48 m^3/s. This does not appear to provide sufficient attenuation to make this option viable.

Figure C -4: Location and ground levels associated with Deil's Kettle zone



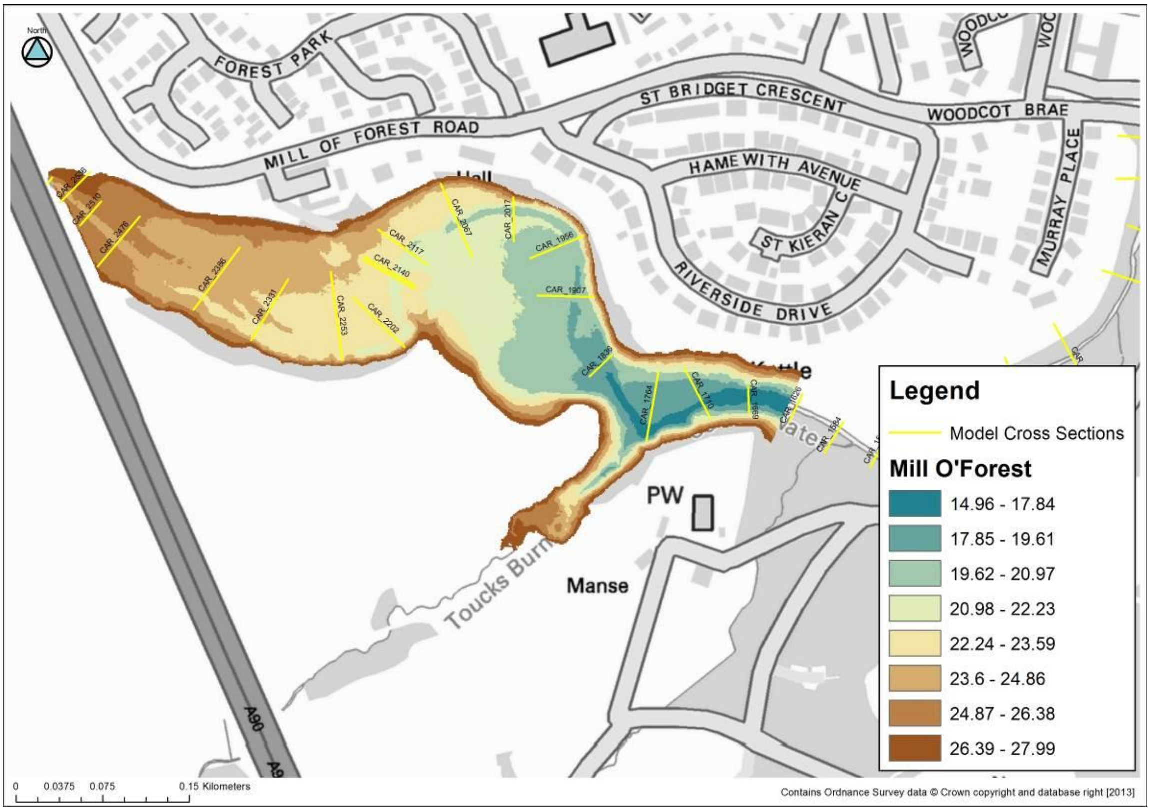
Parameter	Values for catchment critical storm duration	Values for storage optimal storm duration
Duration	10.5 hour	16.5 hours
Inflow	50.40 m^3/s	50.40 m^3/s
Outflow	46.10 m^3/s	48.14 m^3/s
Attenuation	4.30 m^3/s	2.26 m^3/s
Minimum level	12.97 mAOD	12.97 mAOD
Maximum level	21.55 mAOD	21.92 mAOD
Embankment height	9.58 m	9.95 mAOD
Maximum safe level	22.00 mAOD	22.00 mAOD
Storage volume at max level	81,800 m^3	95,920 m^3

C.5.5 Mill O'Forest

This area assumes that the land use of the existing floodplain and the buildings present would be acquired by the Council and the area used solely for the purposes of flood storage (amenity aspects could be incorporated with appropriate safety considerations). This increases the available storage. Initial results suggest that the 200 year flood flows can be attenuated from 50 m^3/s to approximately 37 m^3/s. This would require a 12 m high embankment to store up to approximately 270,000 m^3.

The critical design storm duration has been tested which suggests that the critical duration is 21.5 hours. The model was optimised for outflow and attenuation resulting in a maximum outflow of 39 m^3/s. This area therefore attenuates the 200 year flood flow by approximately 11m^3/s and reduces the 200 year flood flow to a flow equivalent to the 75 year flood.

Figure C -5: Location and ground levels associated with Mill O'Forest zone

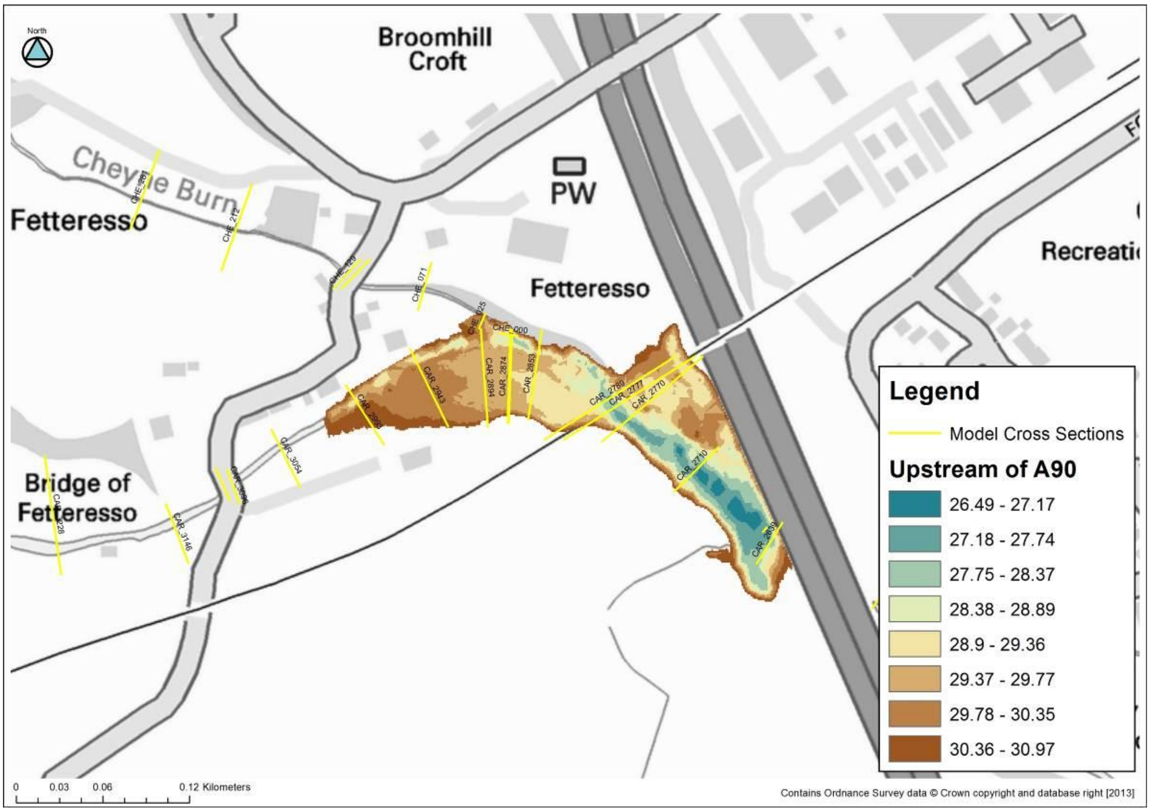


Parameter	Values for catchment critical storm duration	Values for storage optimal storm duration
Duration	10.5 hour	21.5 hours
Inflow	50.40 m^3/s	50.40 m^3/s
Outflow	36.50 m^3/s	39.13 m^3/s
Attenuation	13.90 m^3/s	11.27 m^3/s
Minimum level	14.62 mAOD	14.62 mAOD
Maximum level	26.42 mAOD	26.99 mAOD
Embankment height	12.80 m	13.37 mAOD
Maximum safe level	27.00 mAOD	27.00 mAOD
Storage volume at max level	272,390 m^3	313,520 m^3

C.5.6 Upstream of A90

The results below illustrate that this area does not provide any significant attenuation. This is probably due to the fact that much of the area upstream already floods and there is little additional area available to store flood water. Alternative flood attenuation durations have not been tested as this is not a viable option.

Figure C -6: Location and ground levels associated with the zone upstream of the A90



Parameter	Values for catchment critical storm duration	Values for storage optimal storm duration
Duration	10.5 hour	
Inflow	50.40m^3/s	
Outflow	50.30m^3/s	
Attenuation	0.10m^3/s	
Minimum level	25.61mAOD	Not tested
Maximum level	30.89mAOD	
Embankment height	6.28m	
Maximum safe level	30.00mAOD	
Storage volume at max level	30,990m^3	

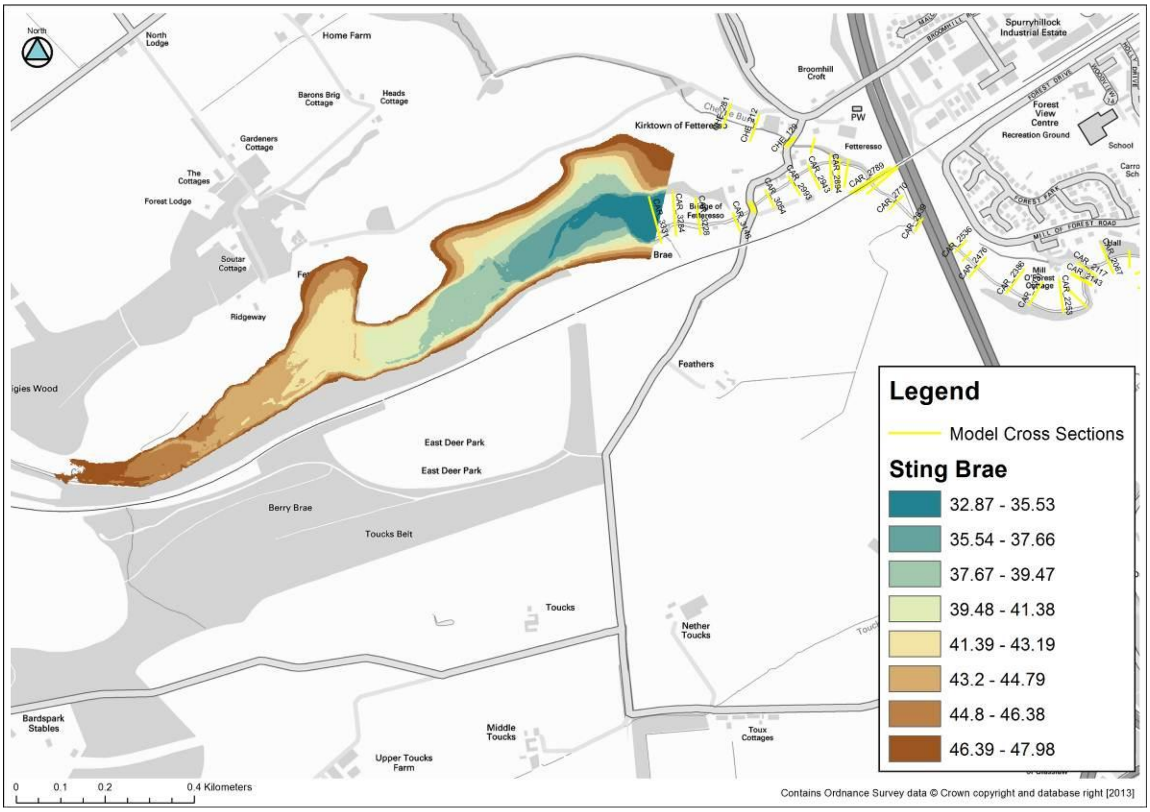
C.5.7 Sting Brae

The area upstream of the Bridge of Fetteresso has a relatively large floodplain/valley locally known as Sting Brae. Upstream of this is s portion of floodplain wider still next to the Fetteresso castle marked as Pond Haugh on OS mapping. The gradient of the channel is relatively steep which would require a large embankment to retain flood flows. Modelling the storage potential suggests that with a large dam, sufficient storage and attenuation could be achieved to reduce the 200 year flood flows significantly.

The critical design storm duration has been tested which suggests that the critical duration is 45.5 hours. The model was optimised for outflow and attenuation resulting in a maximum outflow of 15.1 m^3/s. This area has the potential to attenuate the 200 year flood flow to a flow equivalent to between the 2 - 5 year flood downstream. However, for very long duration flood events, the storage elevations required are above the limit defined by existing constraints.

This requires a very high embankment dam to retain the flood water that may not be environmentally or aesthetically acceptable. Modifications to review the dam height and acceptable downstream flows will be needed.

Figure C -7: Location and ground levels associated with the Sting Brae zone



Parameter	Values for catchment critical storm duration	Values for storage optimal storm duration
Duration	10.5 hour	45.50 hours
Inflow	50.40 m^3/s	43.23 m^3/s
Outflow	13.99 m^3/s	15.12 m^3/s
Attenuation	36.41 m^3/s	28.11 m^3/s
Minimum level	32.10 mAOD	32.10 mAOD
Maximum level	46.11 mAOD	48.33 mAOD
Embankment height	14.01 m	16.23 mAOD
Maximum safe level	47.00 mAOD	47.00 mAOD
Storage volume at max level	952,500 m^3	1,428,400 m^3

C.6 Summary of findings

Based on the analysis undertaken the 4 of the 5 potential areas identified do not provide the necessary flood storage to attenuate flood flows to the required levels. The only area that has the potential to attenuate flows sufficiently is the area upstream of Fetteresso in the Sting Brae area.

This location is upstream of the Cheyne Burn and therefore the assumption of total catchment flows at this storage area is not applicable. This means that additional storage is required at this location as the Cheyne Burn and the Glaslaw Burn are not attenuated.

In order to test the above a routing model of the catchment is required to incorporate flood flows from other tributaries within the catchment. LAG analysis is also required to ensure that the impact of flood storage on catchment durations is fully considered. Additional constraints that may also need to be reviewed with Council/third parties include:

- Embankment heights required.
- The railway line to the south.

- Access for construction.
- Fetteresso Estate land use.
- Environmental issues/fish passage etc.
- Costs and economic efficiency

D Appendix - Storage review considering wider catchment

D.1 Introduction

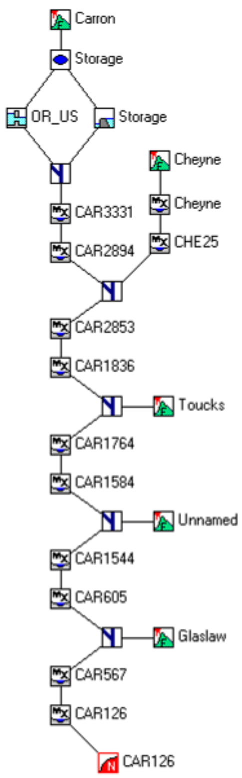
The above assessment is undertaken to review all of the potential areas of flood storage in order to refine the selection. Each of the above models assume total downstream catchment flow which is not appropriate for the upper catchment, particularly the Sting Brae location due to its location above the confluence with the Cheyne Burn.

Based on the above prioritisation process, the Sting Brae area (upstream of Fetteresso) looks to be the only technically viable solution that provides significant flood storage. To assess this further a more rigorous analysis has been undertaken to ensure that appropriate inflows to the storage location are valid and ensure that the attenuation of flows is sufficient to avoid flooding in Stonehaven.

D.2 Model build

A single ISIS model has been built to assess the storage and attenuation potential for the Sting Brae area. The model consists of a series of units representing all catchment and tributary inflows and hydraulic routing model sections (VPMC Cross Section) to model the transfer of flows between confluences. The storage area has been defined as before using an area/elevation relationship combined with an overflow weir and orifice unit to limit flows downstream.

Figure D-1: ISIS model schematic



The inflows to the model are based on the 200 year flood (0.5% AP flood) for the total catchment. Inflows have been defined for the following locations:

- Carron at Bridge of Fetteresso
- Cheyne at Fetteresso
- Toucks Burn at the confluence with the Carron
- Unnamed burn at the confluence with the Carron

- Glaslaw Burn at the confluence with the Carron

All catchment inflows were defined using the FEH Rainfall-Runoff approach in ISIS.

D.2.1 Model calibration

As this is a gauged catchment, existing hydrometric data has been used to calibrate and validate the model via: identification of the critical storm duration and global adjustment of SPR. The ultimate aim is to match the model outflow at both the location of the flood storage and at the gauged location upstream of the Glaslaw Burn, whilst retaining the relative difference in catchment descriptors.

D.2.2 Design Storm Duration

The model was set up and run without any upstream storage. The critical duration for the whole catchment was determined from FEH catchment characteristics and design rainfall from the Depth-Duration-Frequency (DDF) model in the FEH CD-ROM. The resultant design storm duration is 10.5 hours with a catchment rainfall depth of 94.3mm.

D.2.3 Model calibration

The design storm duration analysis resulted in a peak outflow of 46.6m^3/s which is lower than the target 200 year return period (0.5% AEP) statistical peak of 50.4m^3/s. In order to calibrate the catchment model to the statistical peak, the Standard Percentage Runoff (SPR) for each inflow boundary was adjusted to matching the statistical peak flow. This was undertook firstly to increase the SPR for the upper Carron catchment to match the statistical peak at the proposed storage location, and secondly by globally adjusting the tributary inflows.

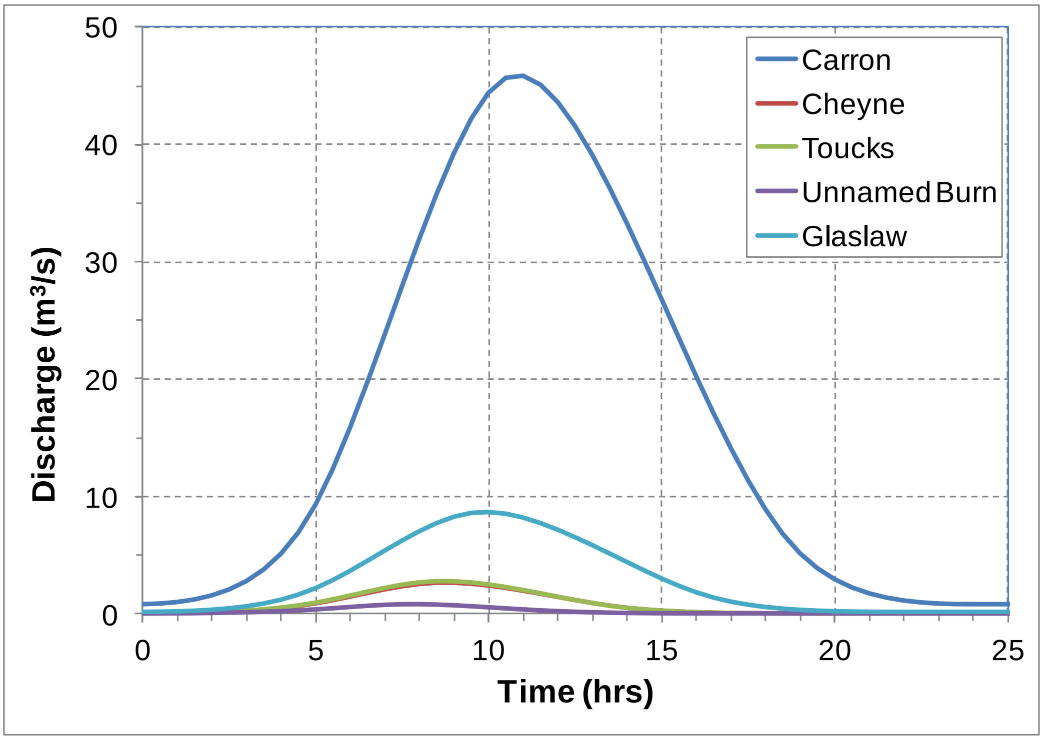
The upper Carron catchment was adjusted by increasing the SPR. The lower tributary catchments were calibrated by a global reduction in SPR of 25%. The Glaslaw Burn was not adjusted as this joins the Carron below the gauged location. The adjusted SPR values are shown in Table D-1 below.

Table D-1: Hydrological inputs into catchment model (flows assume 10.5 hour duration)

Catchment / Tributary	Original SPR	Calibrated SPR	Uncalibrated Flows (m^3/s)	0.5% AP Design Flow (m^3/s)
Carron at Bridge of Fetteresso	37.11	42.74	40.75	45.86
Cheyne	31.4	23.55	3.32	2.63
Toucks Burn	41.82	31.37	3.64	2.84
Unnamed burn	36.99	27.74	1.06	0.83
Glaslaw Burn	40.81	No change	8.69	8.69
Flow at gauge	N/A	N/A	46.60	50.40

Figure D-2 below shows the modelled outflow hydrographs for the catchment routing model of the Carron Water containing no formal storage areas.

Figure D-2: Calibrated flow inputs into catchment model (10.5 hour duration)



D.3 Storage testing

As already defined, the only viable storage location is the one upstream of Fetteresso Bridge. This was further tested to check if a storage solution on its own is feasible and the requirements for storage assuming a less stringent flow reduction in Stonehaven (a combined storage and direct defence option).

For each scenario, the model was run and the orifice area altered to achieve the desired attenuation in the upper catchment and to achieve the overall design flow in Stonehaven (as measured at the gauging station). This is an iterative process to obtain the necessary flow and level balance.

The outlets from the storage areas are modelled using orifice nodes. In terms of detailed design a more optimum and efficient outflow would be achieved with a HydroBrake or similar. At that staged a bespoke rating curve from manufacturers could be used to model outflow characteristics.

D.3.4 Reservoir lag

The introduction of formal storage areas into the catchment system causes the critical storm duration to change due to alterations in the concentration times due to the storage lag. The reservoir lag has been taken into account in estimating the design storm duration. The effect of a reservoir is to delay and attenuate the flood hydrograph. The more a reservoir attenuates flood inflows, the more sensitive it becomes to longer duration floods. In the case of a reservoired catchment, the design storm duration is extended by the reservoir lag using the following equation:

$$D = (Tp+RLAG)(1+SAAR/1000)$$

Where D = Design storm Duration

SAAR = Standard Average Annual Rainfall and

RLAG = Reservoir Lag

The critical duration in terms of downstream reservoir flows is dependent on the reservoir lag. However reservoir lag can only be determined once a flood hydrograph has been passed through the model and therefore an iterative modelling process is required to determine the critical duration. An iterative procedure has been undertaken to calculate the rainfall hyetograph, the net rainfall hyetograph and the subsequent inflow hydrograph which is then routed through the reservoir system until the design storm duration has stabilised.

Design rainfall data was obtained from the DDF modelling within the FEH CD-ROM to determine the design rainfall event inputs for a specific duration.

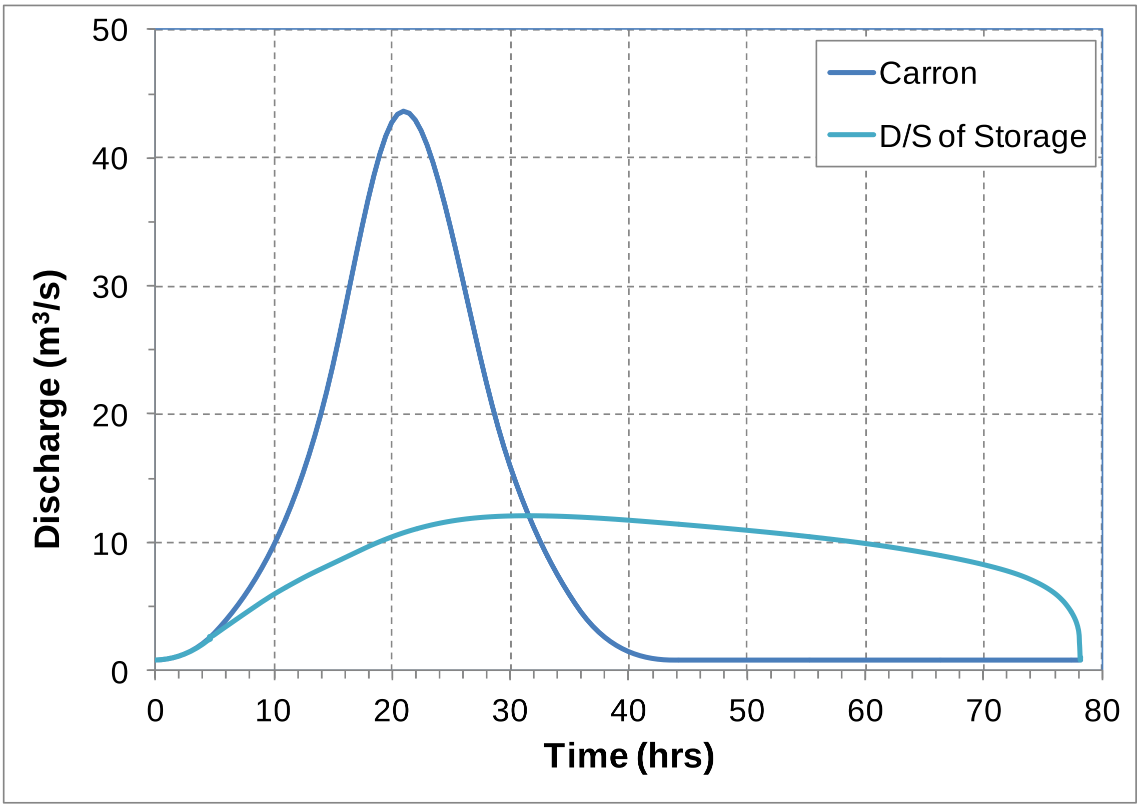
D.3.5 Design storm duration with flood storage at Fetteresso

The requirement of upstream flood attenuation is to reduce flood flows from the 200 year (0.5% AP) flood to the target flow of at least 15.6 m^3/s (between a 2 year and 5 year flood). The above LAG analysis was undertaken to find the critical storm duration for the catchment with the flood storage in place. The critical duration based on the above approach is 30.5 hours with a defined catchment rainfall depth of 141.8 mm. This results in a peak flow at the gauge location of 15.4 m^3/s. The results and key parameters for this scenario are given in Table D-2 below.

Table D-2: Model parameters for the 200 year, 30 hour duration model

Parameter	Value
Duration (hour)	30.5
Storage inflow (m^3/s)	43.6
Storage outflow (m^3/s)	14.4
Attenuation (m^3/s)	29.2
Flow at gauge (m^3/s)	17.4
Flow at outlet (includes Glaslaw) (m^3/s)	25.2
Maximum level in storage (mAOD)	47.0
Embankment height (mAOD) (based on ground level of 32.1)	14.9
Storage volume at max level (m^3)	1,132,800

Figure D-3: 0.5% AP (200 year) flood attenuation by storage at Fetteresso



The above analysis suggests that the storage in the upper catchment has the potential to attenuate flood flows, but this is constrained by a number of factors that limits this option. The constraints are as follows:

6. The maximum level in the storage area is 47.65 mAOD for the design storm duration. This will be higher for critical storm duration tests on maximum reservoir levels as longer duration floods will result in higher total volumes and higher storage levels (despite lower flood flows). The maximum level that storage could extend to is approximately 47.00 mAOD.
7. The ability of storage at this location to attenuate flood flows does not provide any significant freeboard to existing top of bank levels. To provide additional freeboard, additional storage would be required, or formalised flood defences in the lower reach.
8. The flood flow downstream of the gauge is 23.2 m^3/s. With the influence of the Glaslaw Burn, this would result in flooding to gardens in the reach downstream of the White Bridge. To prevent this would require additional attenuation upstream (shown not be possible) or storage on the Glaslaw as well.
9. The required storage would require an embankment over 16 m high (including freeboard). This is unlikely to be aesthetically acceptable to the local community. There may also be breach risks and concerns to the local community.
10. Inundation during flood events would flood large areas of the upstream catchment and grounds of the Fetteresso Castle. This may lead to objections.

D.3.6 Climate change assessment

As the existing scenario for flood storage in the upper catchment is not viable, the impact of climate change has not been tested. The hydrological analysis and modelling undertaken has shown that there is no spare capacity within the proposed storage area to attenuate additional flood flows (33% increase) associated with climate change.

Table D-3: Model parameters for the 200 year plus climate change model

Parameter	Value
Duration (hour)	30.5
Storage inflow (m^3/s)	43.6
Storage outflow (m^3/s)	26.6
Attenuation (m^3/s)	17.0
Flow at gauge (m^3/s)	28.8
Flow at outlet (includes Glaslaw) (m^3/s)	39.9
Maximum level in storage (mAOD)	47.0
Embankment height (mAOD) (based on ground level of 32.1)	14.9
Storage volume at max level (m^3)	1,132,800

D.4 Summary of flood storage option

The above tests represent a more rigorous assessment of the storage requirements for Stonehaven. Whilst an option representing storage on its own does significantly attenuate catchment flood flows, there are a number of technical aspects of such an option that limit this option as a suitable and sustainable option for flood mitigation in Stonehaven. Furthermore, this option is unlikely to be financially viable due to the large storage volumes and the height of the required embankment.

E Storage analysis of Glaslaw Burn

E.1 Locations assessed

The potential for flood storage on the Glaslaw Burn was not previously assessed. In order to assess the feasibility of storage on this burn a number of potential locations were selected for testing. Three sites have been selected and a simple model constructed to test the impact of flood storage at each. These locations are as follows:

- Downstream reach immediately upstream of Carron Gardens opposite Braehead Crescent
- Middle reach in the Woods of Dunnottar upstream of Braehead Crescent
- Upstream reach in the he Woods of Dunnottar upstream of the culvert and minor road

These locations are shown graphically in the figure below. All three locations are not ideal for flood storage due to the relatively steep catchment (and thus high embankments for the storage required), woodland location, and highly mobile bed and floodplain deposits.

Figure C -8: Location and ground levels associated with Walker's Bridge zone

E.2 Model runs

The models for each of the proposed storage areas have been run and the results are presented in the sections below.

E.2.1 Downstream location

The limit for storage at this location is constrained by the road levels to the east. A maximum elevation of 28 mAOD is assumed based on LiDAR elevations for the road and an assumed 1m freeboard between the road and the maximum water level.

The results below indicate that whilst some attenuation is possible at this location, it is not sufficient to reduce the 200 year flood with an allowance for climate change to the required 2 year flood flow. This is probably due to the fact that much of the area upstream already floods and there is little additional area available to store flood water.

Table D-4: Model parameters for the 200 year plus climate change

Parameter	Values for catchment critical storm duration
Duration (hr)	8.5 hour
Storage inflow (m^3/s)	10.9
Storage outflow (m^3/s)	8.3
Attenuation (m^3/s)	2.6
Maximum level in storage (mAOD)	18.8
Embankment height (m)	7.6 plus freeboard
Storage volume at maximum level (m^3)	43,900

E.2.2 Middle reach location

The limit for storage at this location is constrained by the road levels to the east. A maximum elevation of 33mAOD is assumed based on LiDAR elevations for the road and an assumed 1m freeboard between the road and the maximum water level.

The results below indicate that whilst some attenuation is possible at this location, it is not sufficient to reduce the 200 year flood with an allowance for climate change to the required 2 year flood flow. This is probably due to the fact that much of the area upstream already floods and there is little additional area available to store flood water.

Table D-5: Model parameters for the 200 year plus climate change

Parameter	Values for catchment critical storm duration
Duration (hr)	8.5 hour
Storage inflow (m^3/s)	10.9
Storage outflow (m^3/s)	6.5
Attenuation (m^3/s)	4.4
Maximum level in storage (mAOD)	32.9
Embankment height (m)	11.8 plus freeboard
Storage volume at maximum level (m^3)	74,200

E.2.3 Upstream location

The limit for storage at this location is constrained by the road levels to the east. A maximum elevation of 40mAOD is assumed based on LiDAR elevations for the road and an assumed 1m freeboard between the road and the maximum water level.

The results below indicate that whilst some attenuation is possible at this location, it is not sufficient to reduce the 200 year flood with an allowance for climate change to the required 2 year flood flow. This is probably due to the fact that much of the area upstream already floods and there is little additional area available to store flood water.

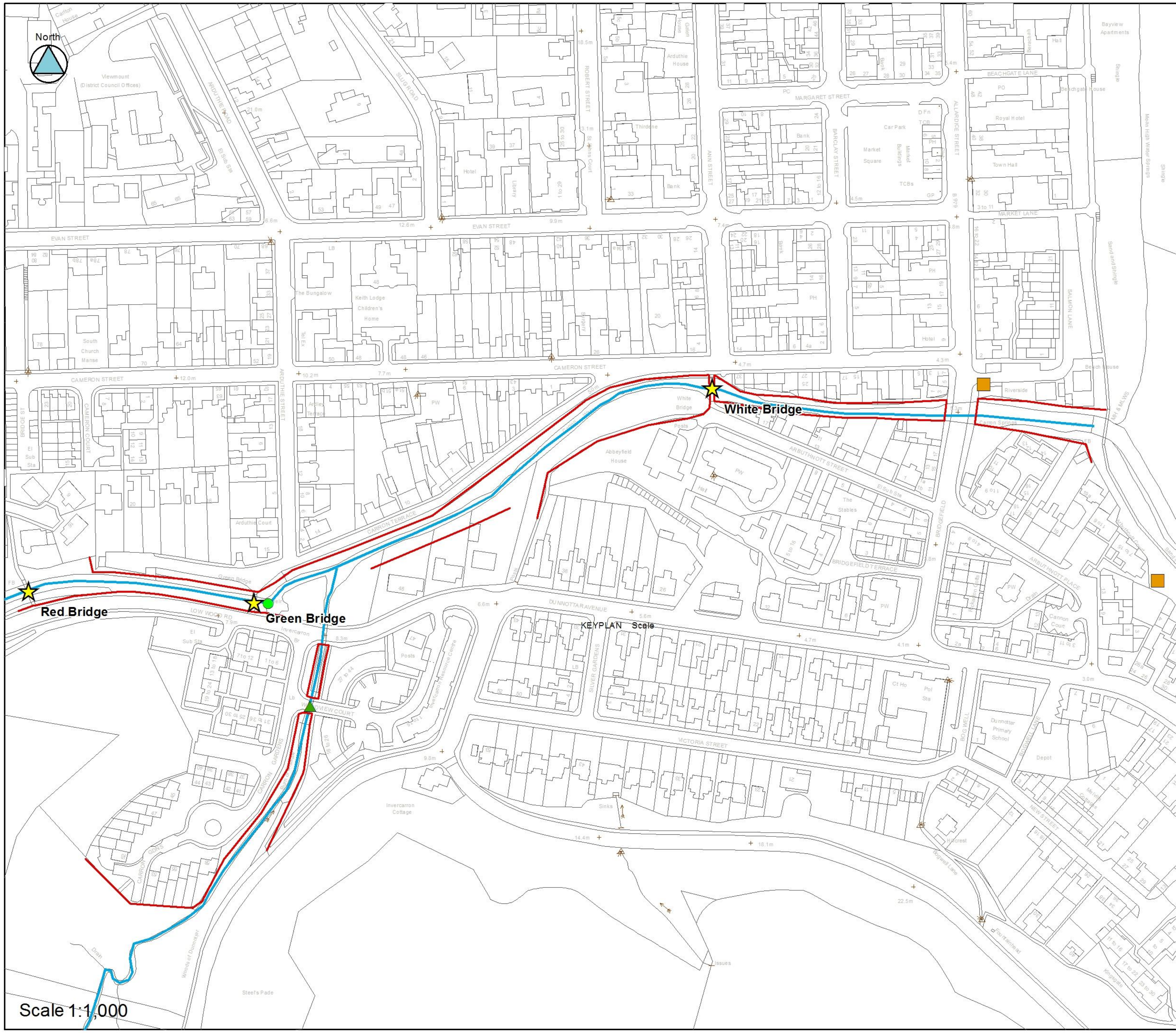
Table D-6: Model parameters for the 200 year plus climate change

Parameter	Values for catchment critical storm duration
Duration (hr)	8.5 hour
Storage inflow (m^3/s)	10.9
Storage outflow (m^3/s)	6.4
Attenuation (m^3/s)	4.5
Maximum level in storage (mAOD)	40.0
Embankment height (m)	10.0 plus freeboard
Storage volume at maximum level (m^3)	76,000

E.2.4 Summary of Glaslaw Burn storage

The above analysis suggests that although flows may be attenuated by up to 4.5 m^3/s this is insufficient to provide the required standard of protection for the scheme. No one single storage area provide sufficient storage to attenuate flood flows on the Glaslaw Burn from the 200 year flood with an allowance for climate change to the 2 year flood flow necessary to ameliorate flood risk in Stonehaven.

Appendix F Plan of Preferred Scheme



1:1,000

LEGEND

- Flood Walls
- Weir_to_be_removed
- Bridges to be Raised
- Culvert_to_be_replaced
- Potential_Locations_of_Pumping_Stations

FIGURE D1

PREFERRED SCHEME PLAN

Offices at

Atherstone

Doncaster

Edinburgh

Haywards Heath

Limerick

Newcastle upon Tyne

Newport

Saltaire

Skipton

Tadcaster

Thirsk

Wallingford

Warrington

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