Stonehaven Flood Protection Scheme

Hydrology and Hydraulic Modelling

June 2015

Aberdeenshire Council

Mott MacDonald

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Contents

Executive Summary

Mott MacDonald Ltd has been appointed by Aberdeenshire Council to carry out the detailed design of Stonehaven Flood Protection Scheme. This report presents a summary of the hydraulic modelling undertaken by Mott MacDonald to establish design flood water levels for the scheme.

The work builds on that undertaken by JBA Consulting in 2012 and 2013, who established design flow rates and the initial wall and embankment scheme concept. The assessment of design flows was reviewed by Mott MacDonald and confirmed appropriate by a Senior Hydrologist from SEPA. The application of these design flows has been updated from the JBA study due to a decrease in the reliance of the A90 culvert as a flow control.

The scheme is designed to protect Stonehaven from fluvial flooding for the 0.5% annual exceedance probability event from Carron Water and Glaslaw Burn. This report describes the 2D TUFLOW hydraulic model constructed to represent the watercourse, defining the flood levels and the improvements required to the conveyance of the channel to minimise required flood defence heights.

A varying height of freeboard has been added on top of the design flood levels to provide for the uncertainties related to the analyses. The height of the freeboard has been based on the statistical analysis of the sensitivity test of the key modelling parameters.

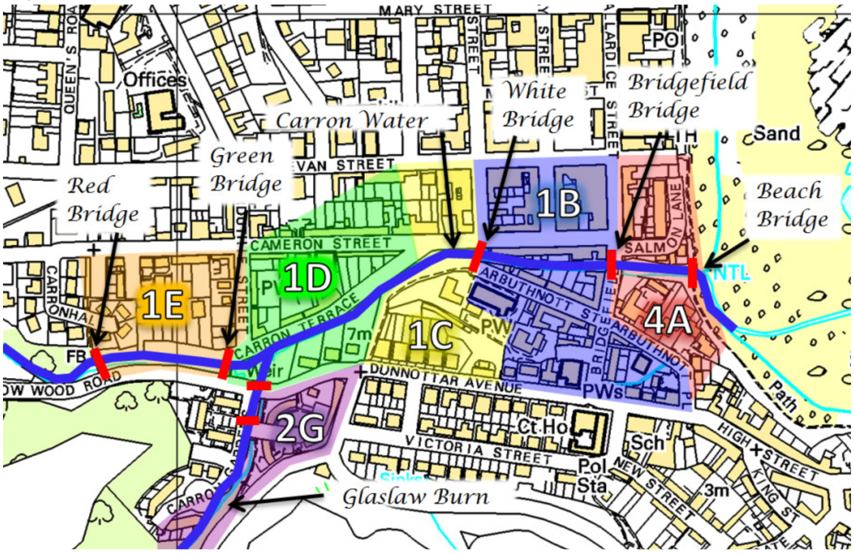
1. Introduction

1.1 Background

Mott MacDonald Limited (MML) has been instructed by Aberdeenshire Council (AC) to prepare the detailed design for the Stonehaven Flood Protection Scheme (SFPS).

The outline of the extent of the proposed scheme and two main watercourses (namely Carron Water and Glaslaw Burn) is presented in Figure 1.1.

Figure 1.1: Outline of the scheme



Source: Indicative zones within Stonehaven Flood Protection Scheme. Base map: © Crown copyright and database rights 2015. Ordnance Survey License 0100020767

The scheme is designed to protect Stonehaven from fluvial flooding for the 0.5% annual exceedance probability event from Carron Water and Glaslaw Burn. This report builds on the work undertaken by JBA Consulting in 2012 and 2013, who established design flow rates, the initial wall and embankment scheme concept and developed an InfoWorks-RS hydraulic model of the watercourse.

The JBA hydraulic modelling was reviewed by Mott MacDonald and documented in report reference 345087_001_A. This report identified that a 2D hydraulic model was required to be developed for the watercourse due to the nature of a number of specific flow constraints and options to be modelled.

1.2 Purpose of this report

This report presents a summary of the hydraulic modelling undertaken by Mott MacDonald to establish design flood water levels for the scheme.

2. TUFLOW hydraulic model

TUFLOW is a one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software. It simulates the complex hydrodynamics of floods and tides using the full 1D St Venant equations and the full 2D free-surface shallow water equations.

Two-dimensional modelling is specifically beneficial where the hydrodynamic behaviour in coastal waters, estuaries, rivers, floodplains and urban drainage environments have complex 2D flow patterns that would be awkward to represent using traditional 1D network models.

It also represents better solution for detailed modelling of the hydraulic structures and minor modifications inside the river channel. Storage volumes are implicitly included within the 2D modelling approach, based on the surface geometry and water flows.

The key inputs, detailed in the following sections, are required by TUFLOW are

- A Digital Terrain Model (DTM);
- Inflow and Tidal boundary;
- Roughness;
- Hydraulic Structures.

2.1 Model inputs

2.1.1 Digital Terrain Model (DTM)

The DTM has been based on the triangular interpolation of the point data from the 2013 topographic survey[1] (see Figure B.1 in Appendix B) and the use of zsh layers to refine the channel shape based on the survey. The hydraulic model is an in bank model only and flood walls have been included on both banks as zlines to contain the water.

A one metre grid was used for the hydraulic model, to represent the underlying topography and enables features such as weirs, bank slopes or minor river bed variations to be included in the hydraulic model.

The channel has been further adjusted for both Carron Water and Glaslaw Burn as discussed in Section 2.2.

[1] Drawing Number SH-RPS-00-00-DR-G-0001-P1.2, Revision P1.3, from 10 Oct 2013

2.1.2 Boundary Conditions

The upstream fluvial inflow boundaries at the Carron Water and Glaslaw Burn were specified as a discharge against time (QT) boundaries to represent the flood discharge at the watercourses.

The downstream tidal boundary was specified as level against time (HT) boundary to represent still water levels in the North Sea.

The details on the boundary conditions are presented in the hydrological analysis review in Appendix A.

2.1.3 Roughness values

Surface roughness was modelled using Manning's roughness coefficient 'n', as noted in Table 2.1. The roughness values were determined from standard tabulated values[2] based on the terrain types in the model. These were reviewed during the site walkover to confirm the values used.

Sensitivity analysis was undertaken on the value of Manning's roughness coefficient used, as discussed in Section 2.3.

Table 2.1: Manning's roughness coefficient values used

Surface features	Roughness Value
Normal Water Carron and Glaslaw Burn channel	0.040
Smooth sections of channel	0.030
Island after White Bridge	0.110

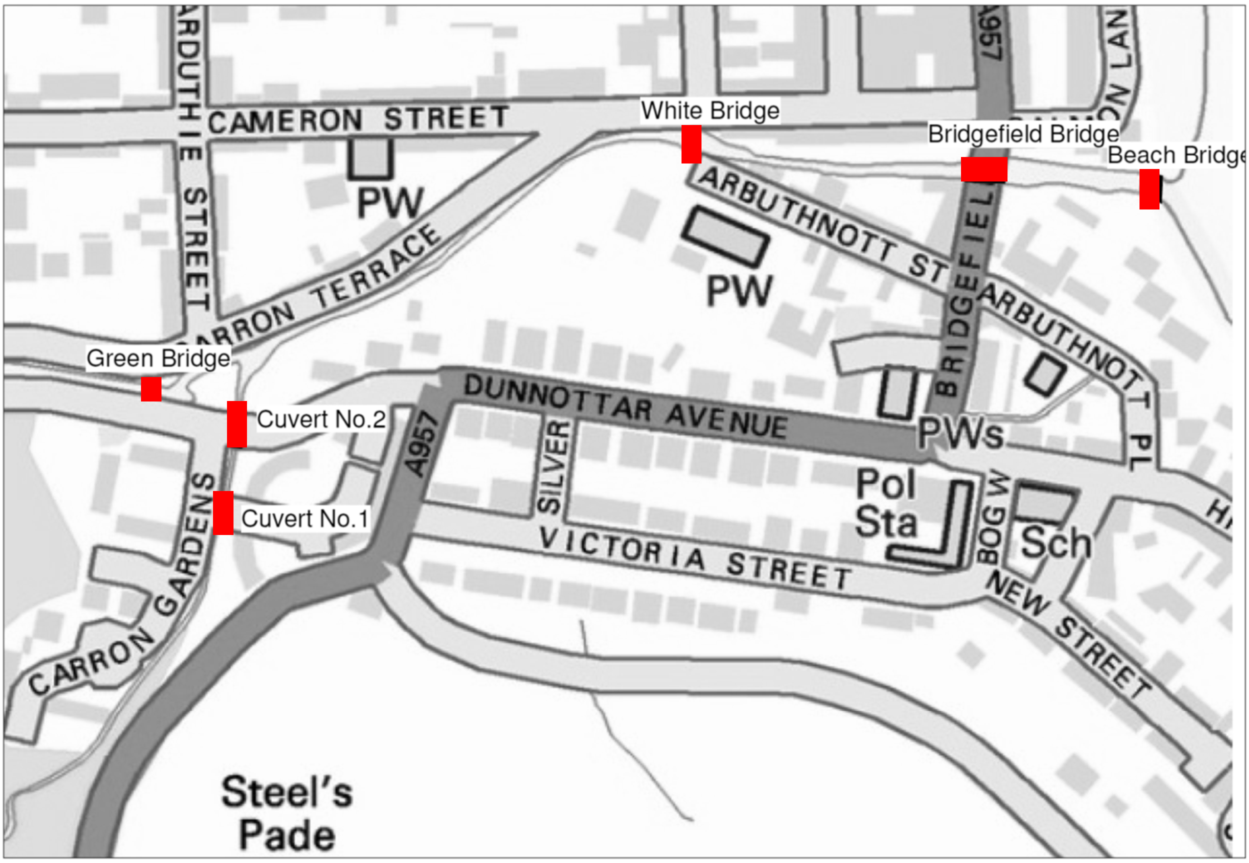
2.1.4 Hydraulic structures

A number of hydraulic structures (bridges and culverts) have been included in the 2D hydraulic model. Where possible, structures have been represented in the model as 2D elements with structure widths generally exceeding four 2D cells.

The locations of the key hydraulic structures are shown in Figure 2.1.

[2] Chow (1959) *Open Channel Hydraulics*

Figure 2.1: Location of the key hydraulic structures



Source: Indicative zones within Stonehaven Flood Protection Scheme. Base map: © Crown copyright and database rights 2015. Ordnance Survey License 0100020767

Carron Water

The Carron Water has five key hydraulic structures located within the modelled section.

- Red Bridge – Upstream of Green Bridge but excluded from the hydraulic model, as it is going to be rebuilt and raised.
- Green Bridge - excluded from the hydraulic model, as it is going to be rebuilt and raised.
- White Bridge –excluded from the hydraulic model, as it is going to be rebuilt and raised.

- Bridgefield Bridge – the existing road bridge has been represented in the model by a Layered Flow Constriction polygon (lfcsh). The applied hydraulic attributes are displayed in Table 2.2.
- Beach Bridge - the existing footpath bridge has been represented in the model by a Layered Flow Constriction polygon (lfcsh). The applied hydraulic attributes are displayed in Table 2.2.

Table 2.2: Carron Water Bridges – hydraulic modelling details

Parameter	Bridgefield Bridge	Beach Bridge
Invert	As river bed	As river bed
L1 Obvert (m AOD)	3.85	3.49
L1 Blockage	0	0
L1 FLC	0	0
L2 Obvert (thickness in m)	10	0.6
L2 Blockage	100	100
L2 FLC	0.5	0.5
L3 Obvert (thickness)	0.8	0.8
L3 Blockage	50	50
L3 FLC	0	0

Glaslaw Burn

Two hydraulic structures are at the Glaslaw Burn located within the modelled section. Table 2.3 presents the details of the applied hydraulic attributes.

- Culvert No.1 (upper) – The existing culvert has been represented in the hydraulic model by Flow Constriction (fcsh) polygons.
- Culvert No. 2 (lower) – The existing culvert has been represented in the hydraulic model by Flow Constriction (fcsh) polygons.

Table 2.3: Glaslaw culverts – hydraulic modelling details

Parameter	Glaslaw culvert No.1	Glaslaw culvert No.2
Obvert level	8.3m AOD	7.6m AOD
FC_Type	BD	BD
pBlockage	5*	5*
FLC below Obvert	0.1**	0.1**
FLC above Obvert	0.5	0.5
Mannings n	0.04	0.04

* - due to the size of the culverts, the blockage factor has been increased to 5%

** - due to the limited size of the culverts, the Form Loss Coefficient has been increased to 0.1

Sensitivity analysis on the hydraulic attributes at the key hydraulic structures was undertaken as described in the sensitivity analysis section (Section 2.3).

2.1.5 Model Parameters

The default TUFLOW model parameters have been used in the modelling. The sensitivity of the model to changes was investigated as part of the sensitivity analysis described below. A timestep of 0.5 seconds was used for the model.

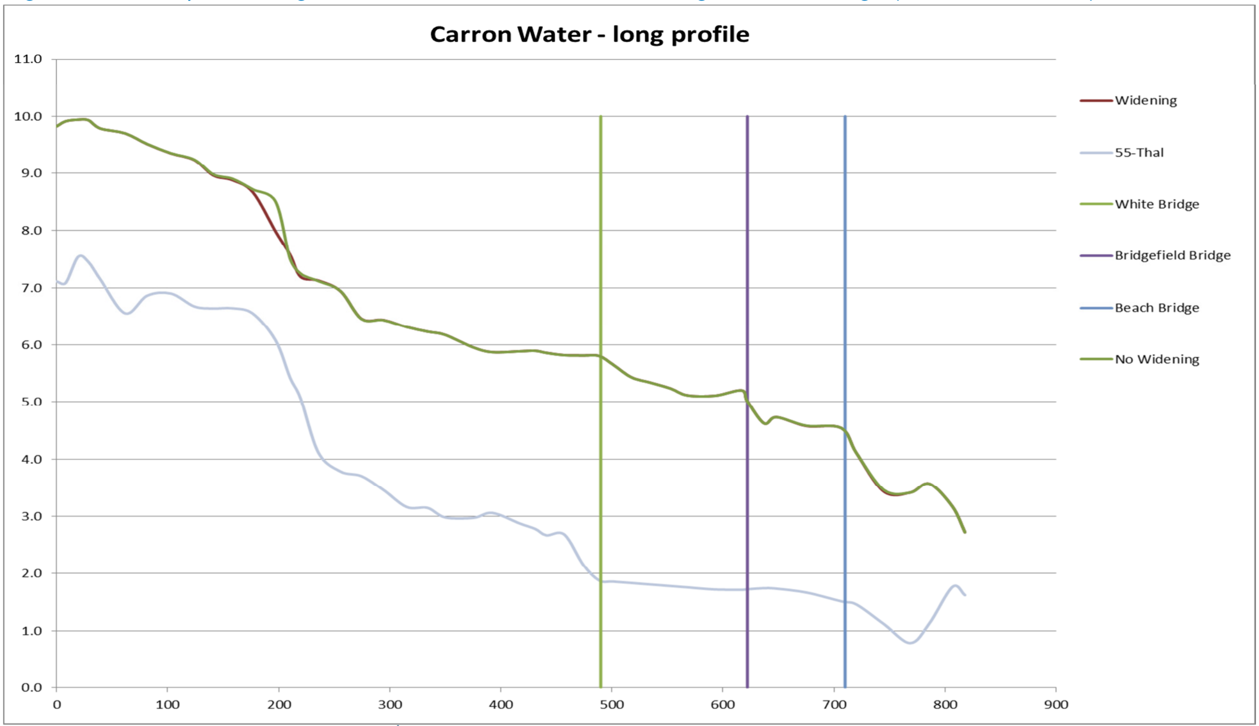
2.2 Optioneering and further modifications

Further changes have been made to the hydraulic model to represent the proposed modifications of the Carron Water channel.

2.2.1 River Bed adjustment at Green Bridge

The channel at the existing Green Bridge constriction has been adjusted to reflect the proposed new design. This included the relocation of the Green Bridge to the further downstream location and increasing the width of the channel. The channel widening took place at the left bank where approximately 3m of embankment was removed in the model.

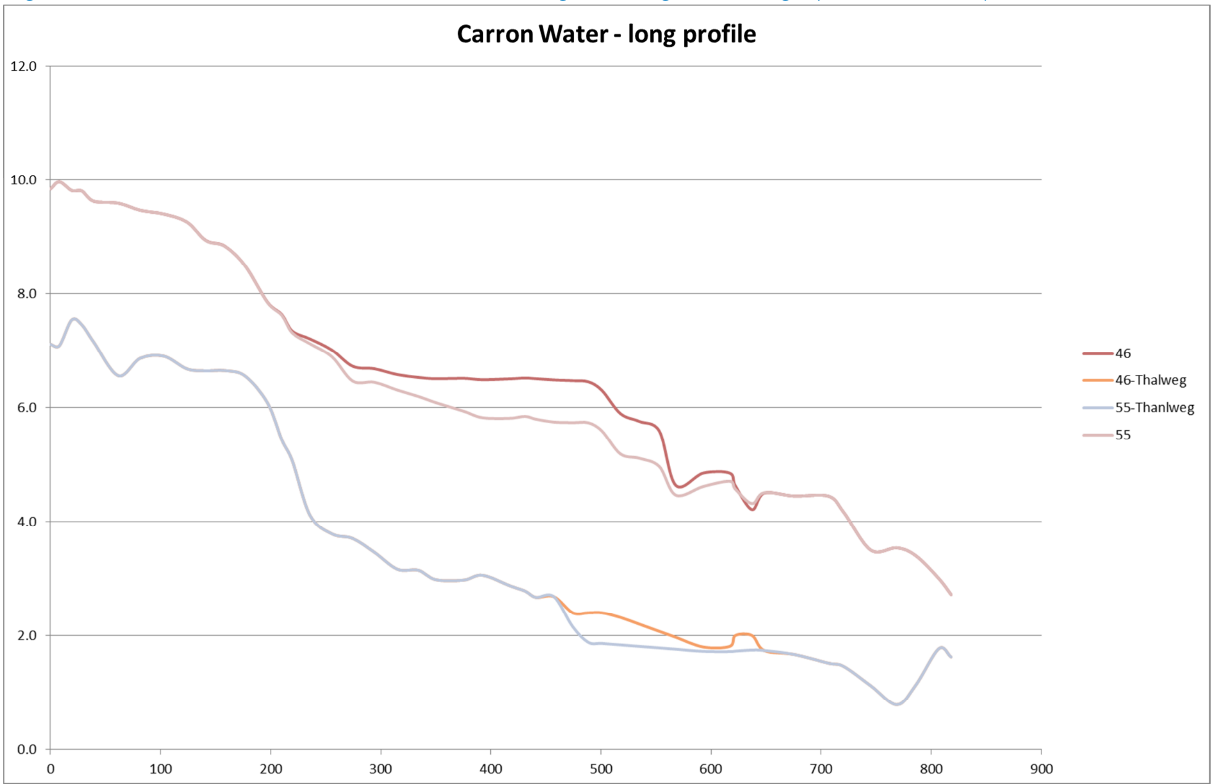
Figure 2.2: Graph showing the effect on water levels of widening at Green Bridge (Model 112 vs 113)



2.2.2 River Bed adjustment at White Bridge

The river bed levels have been adjusted at the location between the White Bridge and Bridgefield Bridge to improve the hydraulic properties of the channel. A zsh layer with the upstream elevation of 1.9m AOD at the White Bridge and 1.7mAOD at the Bridgefield Bridge was applied to reshape the river channel with a constant gradient and rectangular cross sections. Figure 2.2 shows there is a significant effect from this modification with an approximate 0.7m reduction in water levels at White Bridge.

Figure 2.3: River bed modification from White Bridge to Bridgefield Bridge (Model 46 vs 55)



2.2.3 Flood Relief Channel

A 1D element has been used to represent the proposed flood relief channel along the left side of the Carron Water. The channel starts just downstream of the White Bridge and discharges back to the Carron Water at the Bridgefield Bridge. Due to the existing buildings and infrastructure the size of the proposed channel has been restricted to the width of 2.4m and height of 1.2m. A Manning's roughness coefficient of 0.03 has been used for the culvert which allows for a natural bed to be used.

Several options of the flood relief channel arrangement have been examined in the hydraulic model, with the following proposed:

Inlet design

- The inlet is located approximately 10m downstream of the White Bridge
- The invert level of the inlet is 1.9m AOD
- The river bed levels around the inlet have been sweetened to improve the hydraulic conveyance towards the inlet.
- There is no trash screen modelled

Outlet design

- The outlet under the upstream face of the Bridgefield Bridge.
- The invert level of the outlet is 1.75m AOD

The hydraulic capacity of the culvert is estimated to be approximately 7m^3/s under ideal conditions. The applied scenario generated a peak discharge of approximately 4m^3/s during the 1 in 200 year fluvial flood event including an allowance for future climate change, representing approximately 5.1% of the flow.

The efficiency of the culvert could be further improved by suitable design changes but this would require additional work to investigate.

The details of the inlet and outlet arrangements are displayed in Figure C.1 in Appendix C).

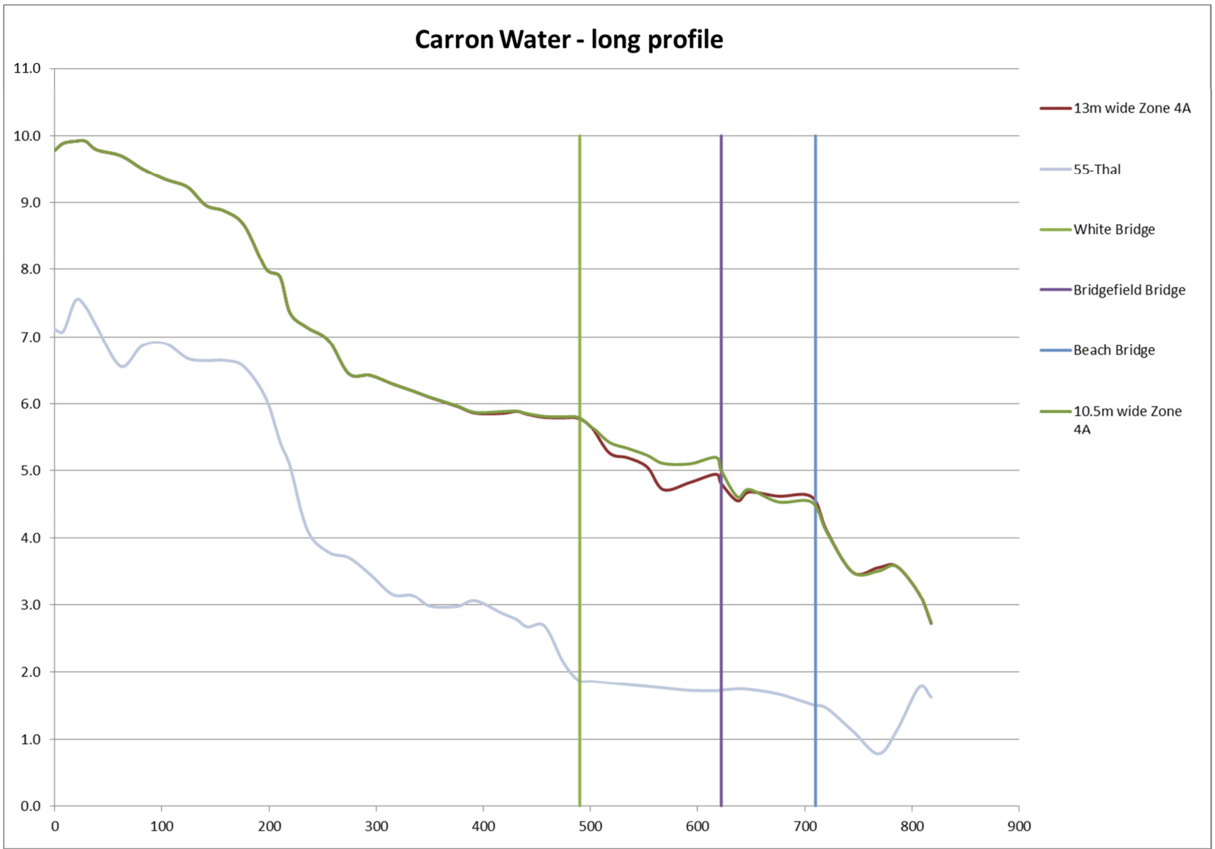
Extension to Beach Bridge

The flood relief channel in the hydraulic model was further extended to Beach Bridge in order to investigate the use of this option to facilitate construction. This option compared to the above scenario provided a minimal improvement in the flood water levels along the Carron Water channel (i.e. up to 0.12m for the 1 in 200 year fluvial flood event including an allowance for climate change) so could be considered, however, it was not considered necessary for the scheme.

2.2.4 Reducing the channel width between Bridgefield and Beach Bridge

The channel between Bridgefield Bridge and Beach Bridge was reduced to 10.5m to facilitate the construction of the scheme. This has the effect of increasing water levels upstream of Bridgefield Bridge.

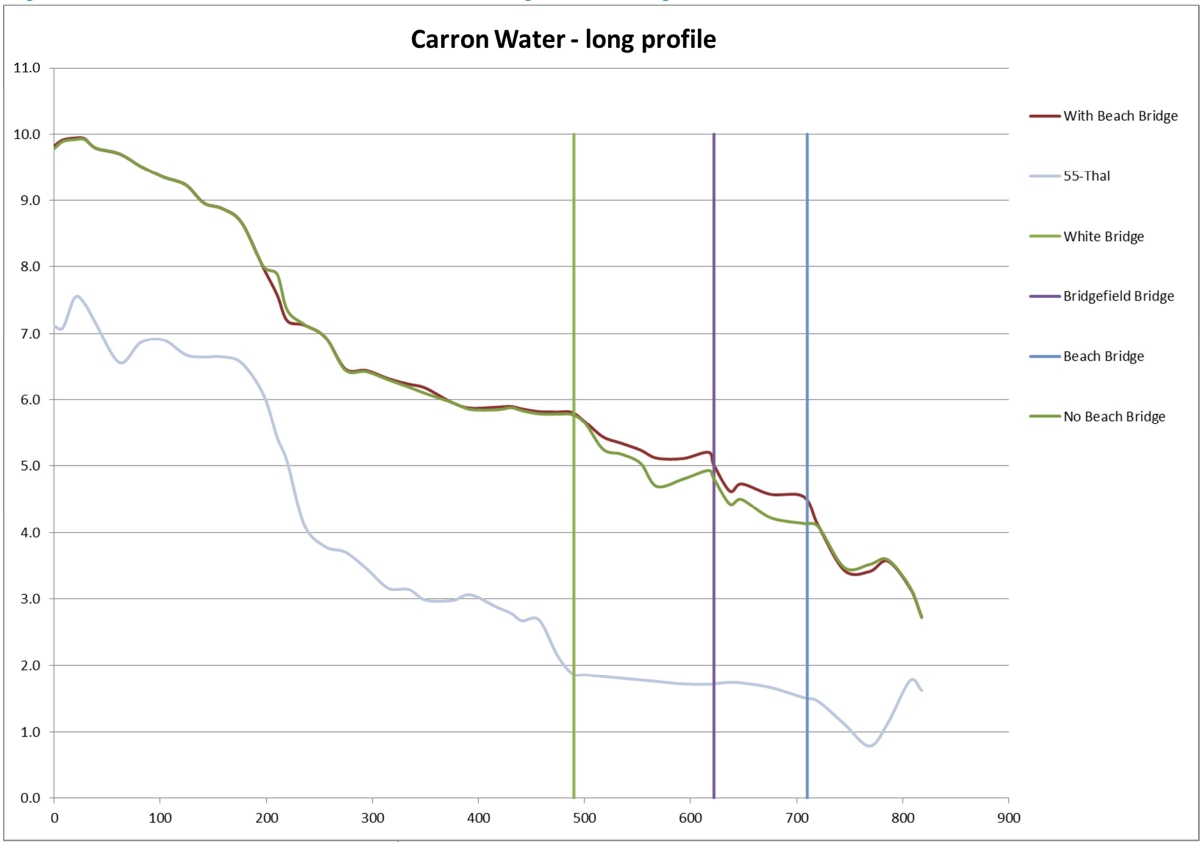
Figure 2.4: Effect on water levels from reducing channel width from Beach Bridge to Bridgefield Bridge (to 10.5m) – Model 110 vs 102



2.2.5 Removing Beach Bridge

The removal of Beach Bridge was investigated, due to the potential for this to be raised. This has the effect of reducing water levels from Beach Bridge up to White Bridge.

Figure 2.5: Effect on water levels from removing Beach Bridge – Model 112 vs 106



2.2.6 Widening of rock armour channel at outlet

The widening of the outlet channel was investigated, firstly by lowering a section of rock armour and secondly by widening the rock armour channel by 2m.

The results show that lowering the rock armour had little effect on water levels.

It is noted that widening the channel downstream of Beach Bridge does not reduce water levels below the soffit levels of the existing Bridgefield Bridge or Beach Beach, so these bridges remain at risk. It is also noted that widening the channel has a similar effect to removing Beach Bridge without widening, also that if the channel is widened, the removal of Beach Bridge has little effect on water levels. It is highlighted that there are otters present in the rock armour channel which would make channel modifications more difficult.

Figure 2.6: Effect on water levels from lowering rock armour – Model 100 vs 100BB

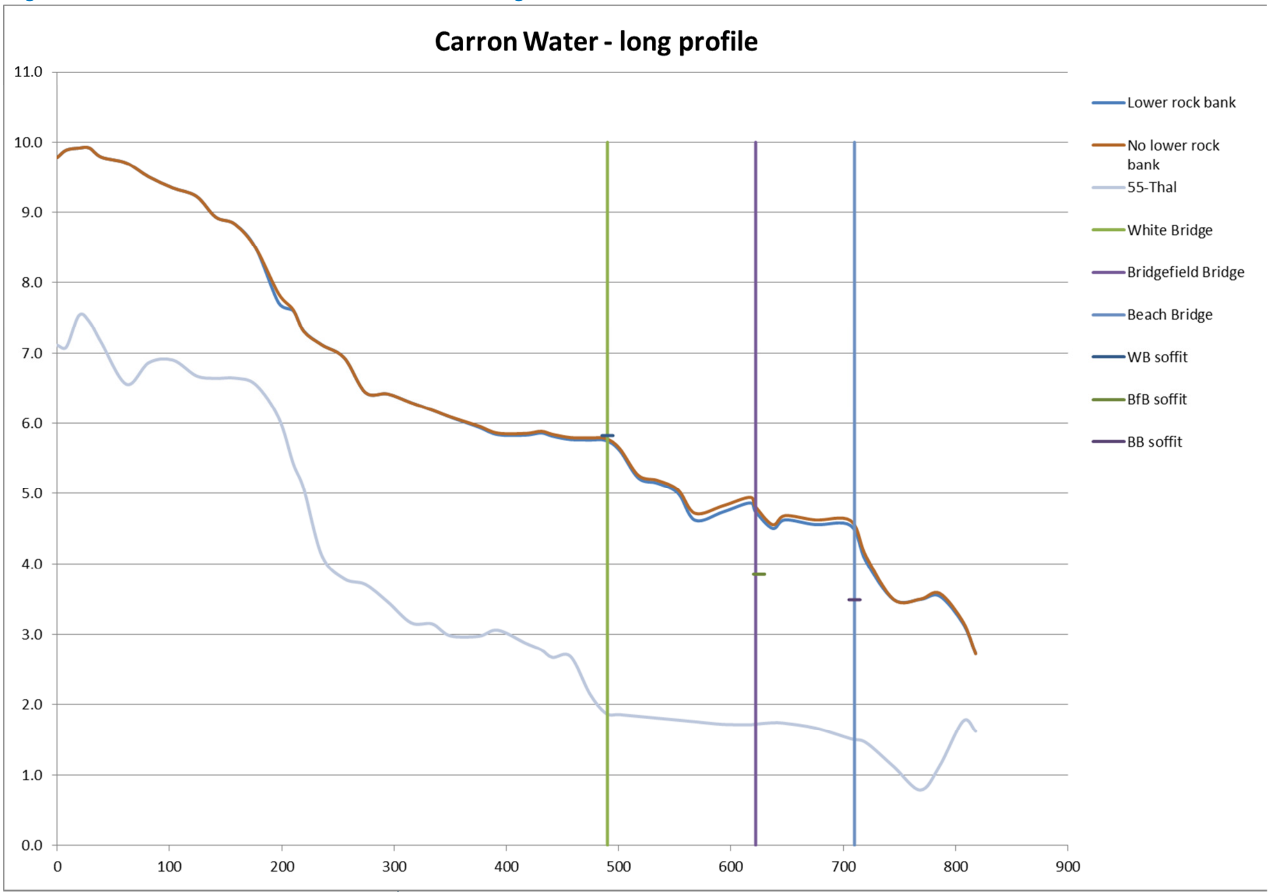
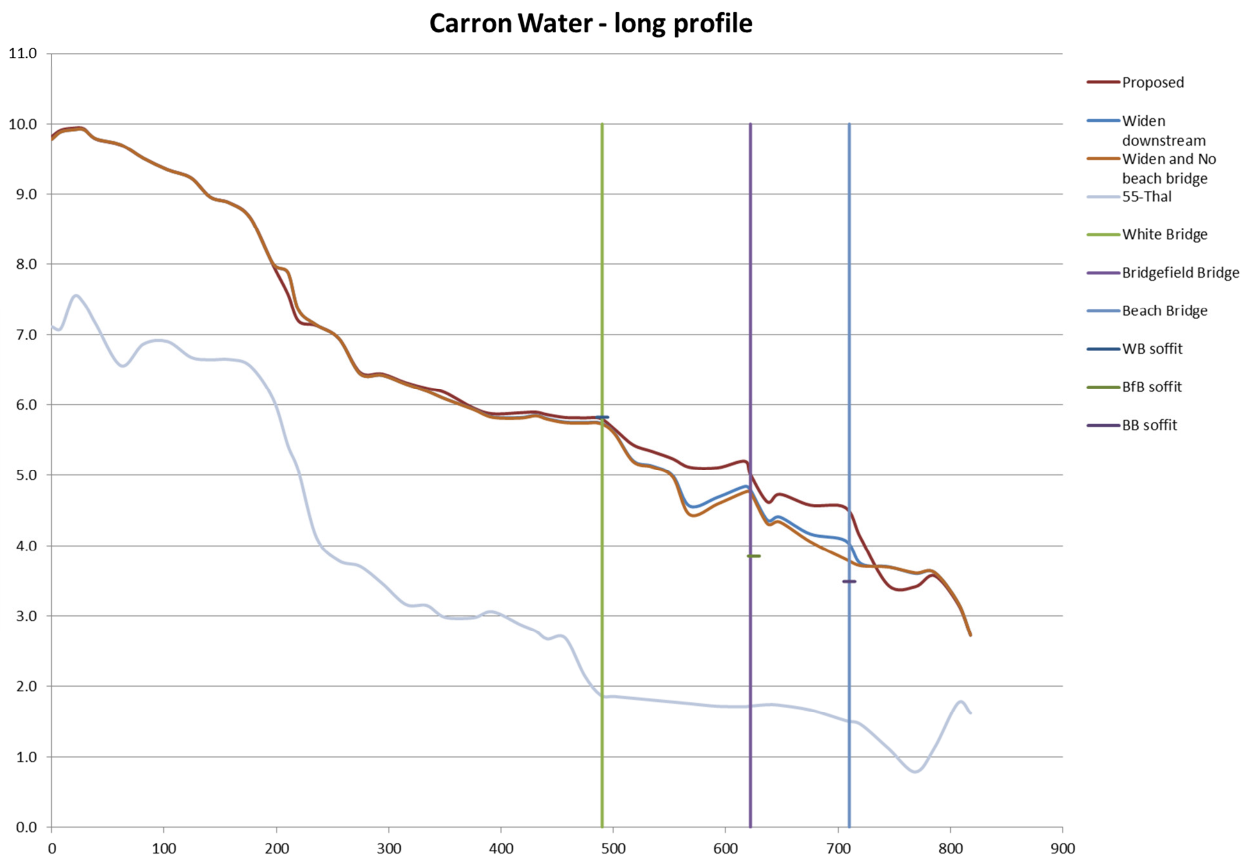


Figure 2.7: Effect on water levels from widening downstream of Beach Bridge – Model 112 vs 107 and 108



2.2.7 Adjustment on the Glaslaw Burn

The widening of the culverts and channel to 6m was modelled. The hydraulic model predicts that if the culverts and channel was widened then water levels upstream are reduced by approximately 2m, a significant effect. The effect of lowering the channel by approximately 1m was also investigated, which provides a constant bed gradient from the pipe crossing upstream of Culvert No 1 to the bed level downstream of the weirs. With this lowering, the width of Culvert No 2 was reduced to its existing footprint width, albeit a rectangular channel. Lowering the channel is predicted by the hydraulic model to reduce water levels by 1m. The results can be seen in Figure 2.8.

Figure 2.8: Effect on water levels modifications on Glaslaw Burn

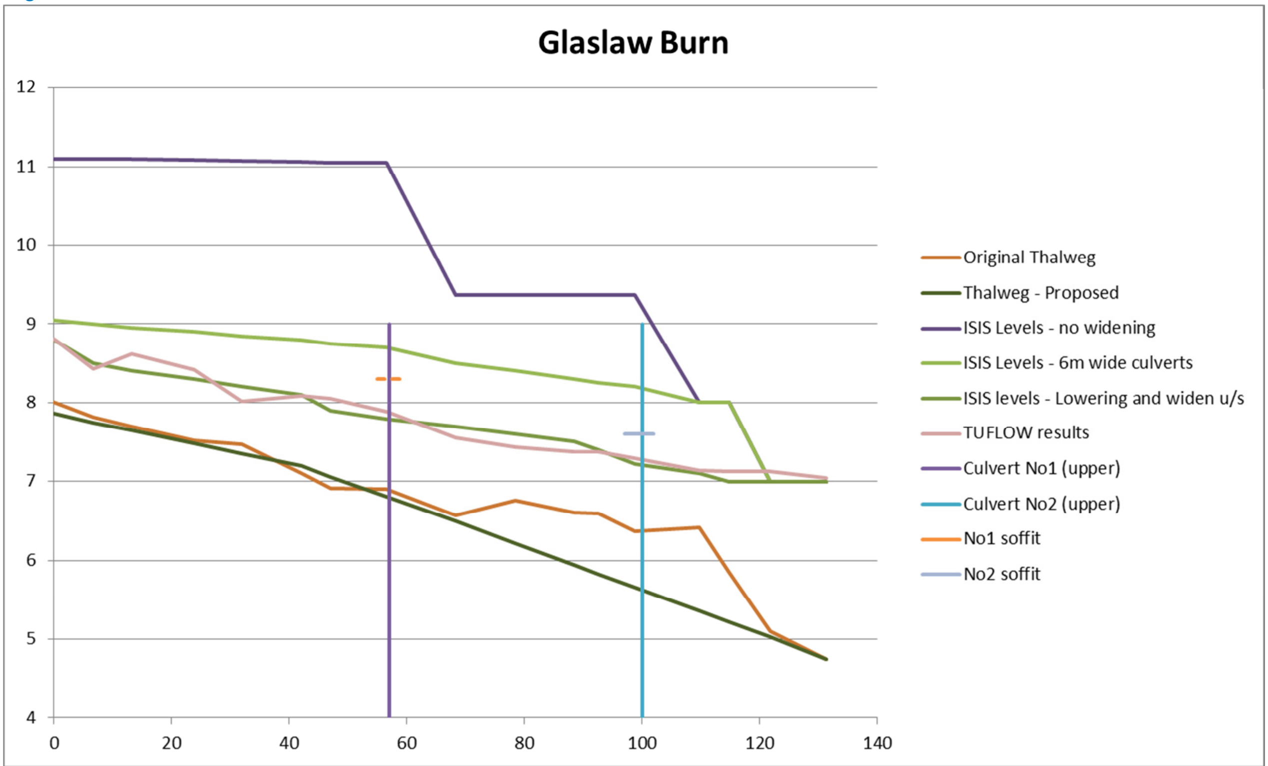
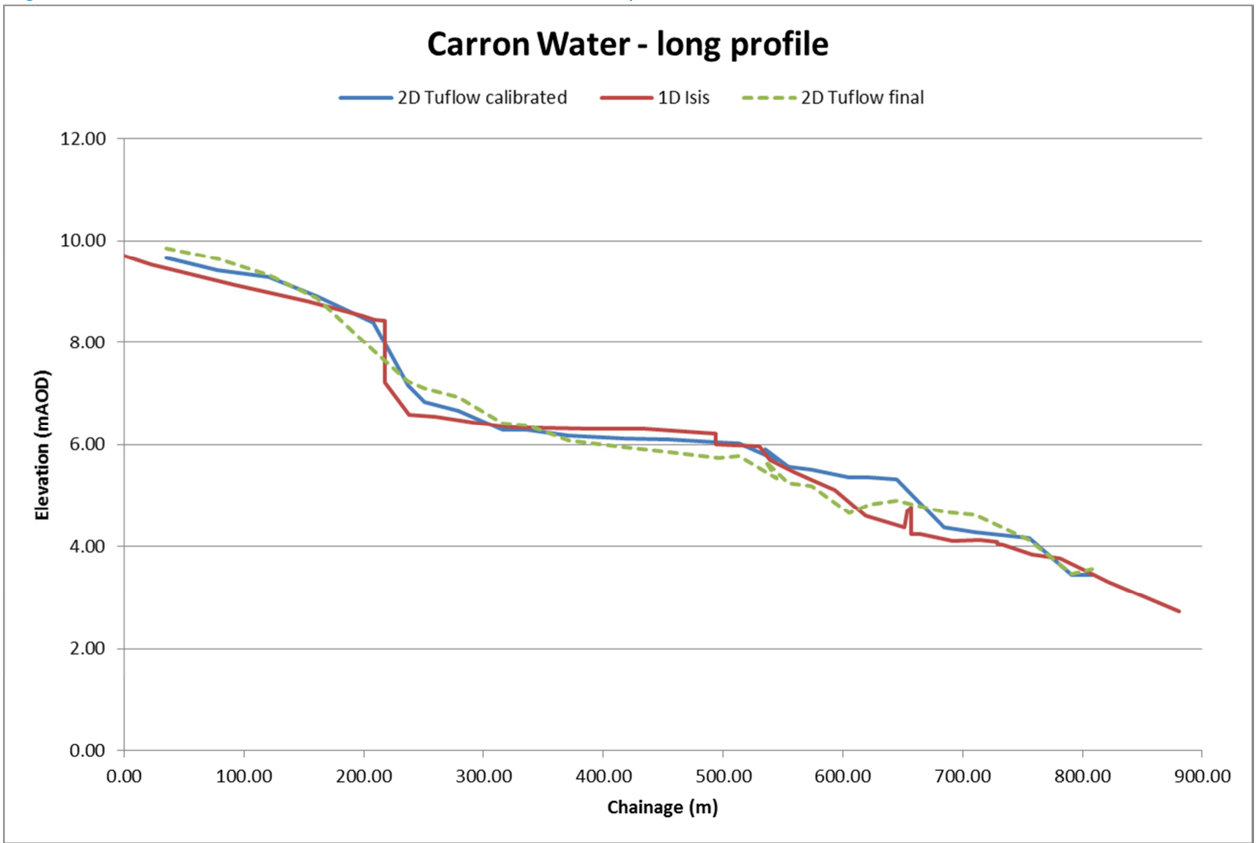


Figure 2.8 also shows the TUFLOW results in comparison to the ISIS results for the lowering and widening option. It can be seen that the water levels predicted by the two models are similar.

2.3 Calibration and sensitivity analysis

The model flood water levels were compared to the flood levels predicted by the 1D hydraulic model developed by JBA, which had been calibrated to the 2012 flood event. For the same flow conditions and channel shape the 2D model predicted a similar water level to the 1D hydraulic model, providing good calibration of the basic 2D model (see Figure 2.9).

Figure 2.9: Carron Water – 1D model and 2D model comparison



Due to the narrow width of Culvert No. 2 an ISIS hydraulic model was developed for the Glaslaw Burn in order to verify the flood levels upstream of the culverts, using boundary conditions derived from the TUFLOW model. The ISIS model produced similar water levels to the TUFLOW model, verifying the levels in the TUFLOW model as shown in the Figure 2.8.

All recorded flood events involve out of bank flow and do not have the magnitude of the design flood event. Therefore there is no calibration data for the in bank flood model. The main model uncertainties are where the modelled bridges are surcharged. No recorded flood event has surcharged the Bridgefield Bridge and Beach Bridge, and therefore, no calibration data is available for this.

Therefore, the following sensitivity analysis has been undertaken:

- The roughness coefficient was varied increased by ±10%, which had limited impact on the maximum flood water levels of ±0.1m.
- The upstream flow boundary at the Carron Water and Glaslaw Burn were varied by ±10%, which had a limited impact on the maximum flood water levels. The highest impact has been observed between White and Bridgefield Bridge, where the maximum water levels varied by ±0.3m.
- The downstream boundary increased by 0.5m which had an insignificant impact on the modelled water levels.
- The Form Loss Coefficient (Layer 2, i.e. above obvert level) at the Bridgefield Bridge was increased to 1, which increased maximum flood water levels upstream of the bridge by 0.5m.
- The Form Loss Coefficient (Layer 2, i.e. above obvert level) at the Beach Bridge was increased to 1, which increased maximum flood water levels upstream of the Beach and Bridgefield Bridge by 0.3m.

The values calculated for the sensitivity analysis have been used to determine the freeboard value to adopt as discussed below.

2.4 Freeboard

Freeboard is, in effect, a safety margin that allows for uncertainties. These include the uncertainties associated with the estimation of the design water level as well as wave effects, construction tolerances and long-term deterioration of the defences.

Allowances for waves are generally not large for typical fluvial defences. Therefore, this uncertainty has been assessed by the downstream boundary sensitivity and no further allowance for the waves has been added to the freeboard design.

The proposed flood defences consists of concrete flood walls which are classified as hard defences. A general requirement for the hard defences is a minimum freeboard of 0.3m. When considering embankment areas an additional allowance of 0.6m is needed to allow for deterioration of the flood defence standard due to:

- Settlement of defence due to consolidation of the foundation and, in the case of an embankment, consolidation of the earth fill;
- Degradation of the crest, such as wear to an embankment crest caused by cattle and agricultural machinery;

The uncertainties that impact directly on design water level include:

- Confidence limits for the hydrological and hydraulic data and calculations;
- Inaccuracies inherent in any physical or analytical models used;
- Variations in assumptions made about channel shape and form, hydraulic roughness, maintenance regime and sedimentation.

The magnitude of the uncertainties varies along the Stonehaven scheme and, therefore, the approach to these aspects of freeboard has been assessed for specific points along the scheme at an approximate spacing of 100m on the Carron Water and 20m at the Glaslaw Burn. These points are located to take into account the locations of the important hydraulic structures.

The uncertainty level in the hydrological and hydraulic analysis has been quantified by the series of the sensitivity analyses including the following parameters:

- Upstream boundary conditions (discharge)
- Downstream boundary condition (sea level)
- Roughness
- Hydraulic model accuracy
- Form Loss Coefficient at the bridges/culverts
- Blockage at the bridges/culverts
- Bed Levels

The sensitivity analyses have been run for each selected parameter with three scenarios, i.e. 5% Certainty, Best Estimate, 95% Certainty. Then, the variation of the water levels at each cross section has been statistically assessed using the standard deviation for each parameter. The sum of the standard deviations from all tested parameters has been assessed as the maximum possible variation in water levels at a given cross section and recommended as freeboard with regards the hydrological/hydraulic analysis.

Based on this assessment the freeboard at the Glaslaw Burn has been recommended to be 0.3m along the entire watercourse. The final recommendations on the freeboard level at the Carron Water vary along the scheme and are either 0.3m, 0.6m or 0.8m as summarised in Table 2.4.

Table 2.4: Freeboard summary for the Carron Water

Chainage (m)	Northing / Easting	Location description	Standard Deviation for the 95% Certainty (m)	Recommended freeboard (m)
Ch103	386955, 785655	Upstream end of the scheme	0.205	0.300
Ch197	387048, 785641	Upstream of the Green Bridge island	0.176	0.300
Ch294	387135, 785677	50m downstream of confluence with Glaslaw Burn	0.248	0.300
Ch431	387247, 785754	60m upstream of current White Bridge	0.527	0.600
Ch501	387315, 785758	Downstream of White Bridge, at the flood relief channel inlet location	0.551	0.600
Ch593	387406, 785747	30m upstream of Bridgefield bridge	0.784	0.800
Ch675	387488, 785744	35m upstream of Beach Bridge	0.359	0.600
Ch786	387564, 785675	Downstream end of the scheme	0.119	0.300

3. Modelling outcomes and discussion

3.1 Head losses at two downstream bridges

2D hydraulic modelling, using the TUFLOW hydraulic package, has been selected as the leading approach to determine the design flood water levels for the Stonehaven Flood Protection Scheme.

The model has been largely calibrated to the original 1D-2D hydraulic modelling constructed by JBA[3]. It is also noted that the original hydraulic model was calibrated to observed historic flood events.

The two downstream bridges, namely the Bridgefield Bridge and Beach Bridge, have been identified as the most critical constrictions in the channel with a significant potential to impact the flood water levels. As none of the historic flood events surcharged these two bridges, no detailed calibration of the hydraulic behaviour at the two locations during extreme discharges was possible. Therefore, a 1D ISIS model has been constructed by Mott MacDonald to further verify the results of the 2D modelling of the hydraulic head losses at the two critical downstream bridges.

The comparison of the determined head losses at the critical structures are presented in Table 3.1 for both, 1D and 2D hydraulic model.

Table 3.1: Hydraulic head losses at bridges

Location	1D head loss (m)	2D head loss
Bridgefield Bridge	1.03	0.40
Beach Bridge	0.08	0.24

It is considered that the 2D model provides a better representation of the flood channel due to the simplification of the 1D approach when the bridge becomes surcharged and that the bridge widens after the opening due to the culvert location, which makes the hydraulic assessment more complex.

It is noted that the freeboard added at the top of the design flood water levels includes for the uncertainty due to the hydraulic modelling (see Section 2.4). The largest freeboard of 0.8m has been recommended for the section just upstream of the Bridgefield Bridge. Therefore, the applied freeboard provides sufficient safety margin to accommodate the head losses of 1D model extent.

3

3.2 Smoothing the long profile of scheme

The maximum flood water levels have been extracted from the TUFLOW hydraulic model along the Carron Water and Glaslaw watercourse. The proposed heights of freeboard allowance have been added on the top of the flood water levels. The final minimum flood defence level has then been smooth out to provide a gradual decrease in flood defence level from the top to the bottom.

Figure 3.1 and Figure 3.2 provides the graphical summary. The detailed tabular summary of the design flood levels is presented in Table B.1 and Table B.2 in Appendix C.

The flood map with the proposed flood defences is presented in Figure B.2 in Appendix B.

Figure 3.1: Carron Water – Minimum flood defence levels (Model 112)

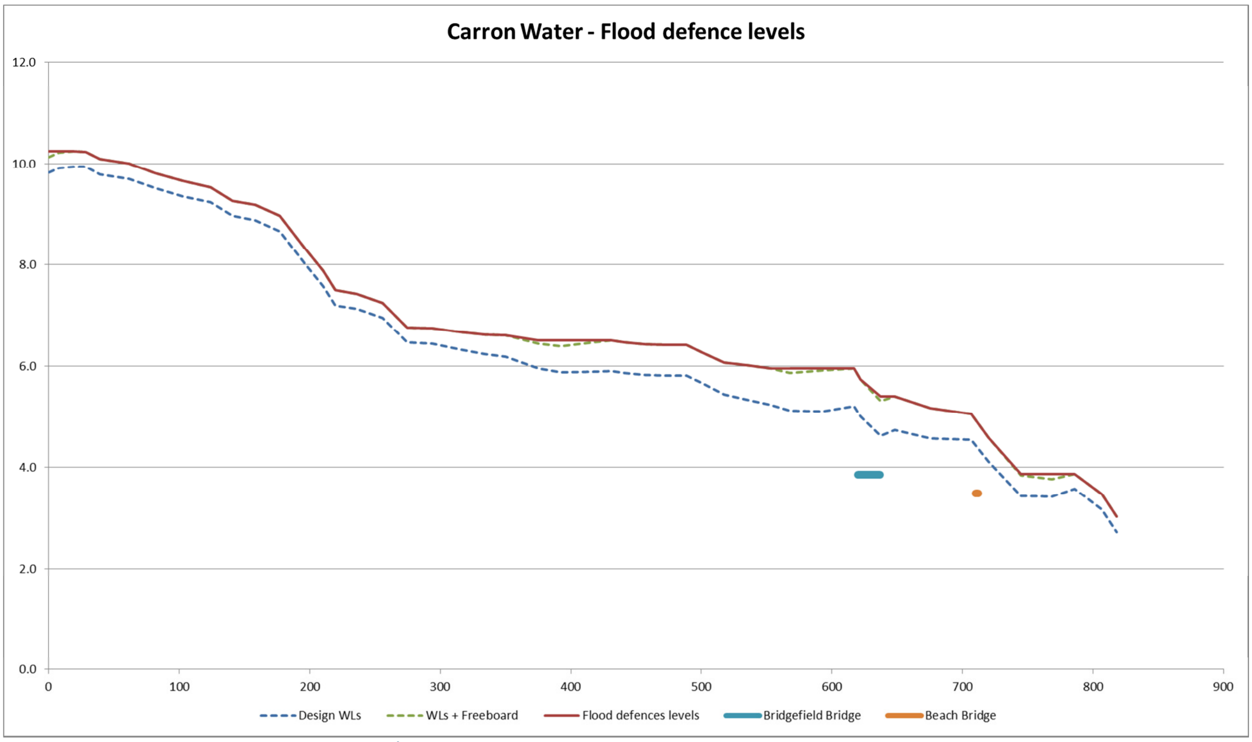
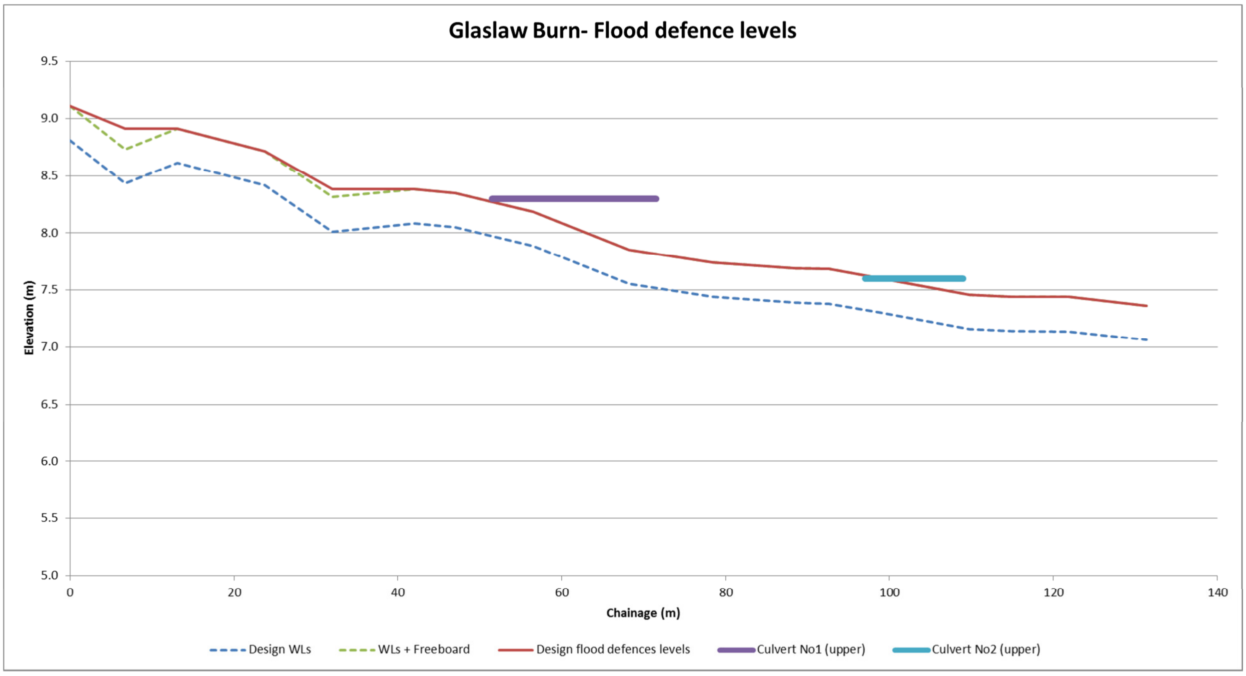


Figure 3.2: Glaslaw Burn – Minimum flood defence levels (Model 112)



Appendices

Appendix A. Hydrological analysis

The original hydrological analysis was presented by JBA in their Stonehaven River Carron Flood Alleviation Study report in July 2012. In November 2013, an update was published, namely Stonehaven River Carron and Glaslaw Burn Preferred Flood Protection Scheme Report, where the information gathered during and after flood event in December 2012 were included.

Both reports have been reviewed by SEPA and SEPA provided their comments as presented in Figure A.1. A brief summary of the undertaken analysis is provided in the following sections. The full account of the analysis is provided in the relevant JBA reports.

A.1 Carron Water

The hydrological analysis for the Carron Water has been based on the FEH Statistical Method and the observed data from the local SEPA river station on the Carron Water (OS NGR 8693 8565). In addition, JBA proposed to use a new model rating which reduced the flow estimates at high stages and which was subsequently agreed with SEPA.

QMED (Index Flood) estimation

Several methods for calculation QMED at the Carron Water at Stonehaven were applied in the JBA study. This included following:

- QMED from POT series;
- QMED from AMAX series extended with regression[4];
- QMED from catchment descriptors.

Consequently, the estimate of QMED derived from the AMAX series extended with regression was selected as the best option. The strong relationship between the two gauges provided a sufficient confidence in the method. This method also represented a conservative approach.

Growth curve

Several methods for generating the growth curve were applied. These included following:

- Ungauged pooling group analysis using the gauged Carron AMAX record only (Generalised Logistic (GL) distribution;

[4] The record from adjacent River Bervie at Inverbervie was used

- Single site analysis using an AMAX series derived from the gauged Carron record plus regression with the Bervie gauged record (GL distribution);
- Enhanced pooling group analysis an AMAX series derived from the gauged Carron record plus regression with the Bervie gauged record (GL distribution).

A review of historical flood events in Stonehaven was undertaken to establish the most appropriate growth curve. Based on this the enhanced pooling group approach was selected as the best representation of the flood growth curve of the Carron Water catchment.

Update after December 2012 flood event

JBA carried out an update of the hydrological analysis in 2013 following the flood event occurred in December 2012. Consequently, the statistical analysis was updated with the additional gauged data. The final design peak flow estimates for the Carron Water are presented in Table A.1.

A90 flow adjustment

The 1D hydraulic model undertaken by JBA starts upstream of the A90 culvert. The inflows in Table A.1 were run through the 1D hydraulic model and the flows just upstream of the Red Bridge were used as the input to the 2D TUFLOW model. During the review of the 1D hydraulic model it was identified that the A90 culvert had a significant impact on water levels and flows in watercourse, not evident on the ground. This was adjusted in the 1D hydraulic model and the revised flows just upstream of Red Bridge were used in the TUFLOW model.

A.2 Glaslaw Burn

The FEH Rainfall Runoff method was used to estimate the peak flows at the Glaslaw Burn. The method estimates peak flows based on the FSR method and catchment characteristics extracted from the FEH CD-ROM v3. The final design peak flow estimates for the Glaslaw Burn are presented in Table A.1.

A.3 Climate change allowance

Data from UKCP09 was analysed and the impact on flood flows was estimated for a number of intervals until 2080. Based on this, the 33% increase due to the climate change for 2080 was adopted for design. This is higher that a general recommendation of SEPA (20%) and it is in line with the conservative assumptions.

A.4 Peak Flow Estimates

A summary of the peak flow estimates used in the model can be seen in Table A.1 (For Carron Water this is the flow upstream of the A90 culvert).

Table A.1: Peak Flow Estimates

Annual probability	Return period (years)	Carron Water Model Rating with regression (m3/s)	Glaslaw Burn FEH R-R (m3/s)
50%	2	14.5	2.5
20%	5	20.5	3.4
10%	10	24.9	4.2
4%	25	31.3	4.6
2%	50	36.9	6.2
1%	100	43.2	7.1
0.5%	200	50.4	8.2
0.5% + cc	**200 + cc**	**67.0**	**10.9**
0.1%	1000	71.8	11.7

Source: JBA report, April 2013

A.5 Downstream (tidal) boundary

The tidal boundary curve was generated in the previous JBA study in 2012. A stage-time hydrograph representing tidal harmonic was used. The tidal harmonic used for the downstream boundary was derived using extreme sea levels taken from the Environment Agency's 2011 report on coastal flood boundary conditions and also takes into account tidal surge.

The applied tidal curve in the hydraulic modelling has a peak level of 2.732m AOD. This approximately equals to the 1 year coastal flood event.

Table A.2 below shows that peak sea levels for selected coastal flood return periods.

Climate change

Based on the estimates from the UKCP website for the Stonehaven location[5], climate change effects to 2080 have been considered by increasing sea level by 0.266m (© UK Climate Projections, 2009).

Table A.2: Coastal flood levels

Annual probability	Return period (years)	Level (m AOD)
	1	2.73
50%	2	2.80
20%	5	2.89
10%	10	2.97
2%	50	3.12
1%	100	3.19
0.5%	200	3.25
0.5% + cc	**200 + cc**	3.52
0.1%	1000	3.39

Source: Environment Agency

[5] http://ukclimateprojections-ui.defra.gov.uk. © Crown Copyright 2009. The UK Climate Projections (UKCP09) have been made available by the Department for Environment, Food and Rural Affairs (Defra) and the Department of Climate Change (DECC) under licence from the Met Office, UK Climate Impacts Programme, British Atmospheric Data Centre, Newcastle University, University of East Anglia, Environment Agency, Tyndall Centre and Proudman Oceanographic Laboratory. These organisations give no warranties, express or implied, as to the accuracy of the UKCP09 and do not accept any liability for loss or damage, which may arise from reliance upon the UKCP09 and any use of the UKCP09 is undertaken entirely at the users risk.

Figure A.1: SEPA response on the hydrological analysis

FLOOD RISK HYDROLOGY RESPONSE TO REQUEST FOR INFORMATION RELATING TO FLOOD RISK

NORTH AREA (DINGWALL OFFICE)

Site:	Stonehaven				
SEPA Ref:	**No PCS - direct from JBA**		**Planning Ref:**		**n/a**
Hydrology Contact:	Claire Wheeler	**Ext:**		**Date:**	22 July 2013
Documents Reviewed:	Stonehaven December 2012 Flood Event Review - Final Draft April 2013				

Summary

We have been asked to review the report by JBA with particular reference to agreeing the design flows. Work is progressing with the design of flood alleviation works and design flows are critical to ensuring the adequate and appropriate design of such a scheme.

We (Malcolm MacConnachie) previously reviewed a study which was published in July 2012 and agreed that the methodology used to assess design flows was appropriate. This report essentially reviews and updates that study in light of the information gathered during and after the flood event which affected Stonehaven on 23rd December 2012.

Technical Report

1. We welcome the revision of design flood flows for Stonehaven as a result of the latest serious flood event to affect the town. We agree that the SEPA rating for the Carron gauge is not likely to be a reliable way of calculating high flows as the rating is not calibrated for those events. We previously agreed the approach used to develop a modelled rating instead.

2. The flows have been updated resulting in an increase in the design flows. We welcome the updated estimates and take reassurance from the upward revision of those figures. It is apparent that flooding in Stonehaven results from a complex interaction of flood sources and mechanisms which introduces significant uncertainties when trying to estimate design flows (where all the variables could differ in any given event).

3. The flow estimates for the Glaslaw Burn have greater uncertainty than for the Carron due to the absence of any gauged or recorded data for the catchment. We agree with the decision to adopt higher flows estimated from the Rainfall Runoff method instead of those previously used, as they seem more in line with the observations of flooding from the most recent event.

4. Early in the report there is some analysis of rainfall data from the event. We have previously highlighted that the SEPA raingauge at Cheyne is likely to have underestimated rainfall in the days leading up to the event. The gauge was thought to be under-recording and was fixed on the morning of 22nd December. We are confident in the data for the 24 hours leading up to the peak of the event, but the days and possibly weeks prior to that are likely to have been underestimated. This was supported by the

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monthly total which was recorded in the check gauge for the month of December. Our estimation is that the rainfall from early December to the morning of 22nd may have been 50% more than recorded by the gauge for the same period. No reference to the issue is made in the report and rainfall return periods may have been underestimated as a result, particularly for durations longer than 24 hours.

5. Our analysis of the rainfall indicated that the antecedent conditions may have been a significant factor in the severity of the flood event. The rarity of the rainfall over the preceding 10 days may have been relatively much more severe. Neither this, nor the point raised above may have a bearing on the peak flow experienced on 23rd December and given that the purpose of the report is to inform the design of a scheme, may not be a significant issue. It may be that the antecedent conditions contributed more to the severity of overland flow experienced and how it contributed overall to the flood event.

6. Although the report focuses on the peak flows in the watercourses, it is evident that any scheme of works will have to consider surface water drainage and overland flow contribution in addition to the flow from the burns.

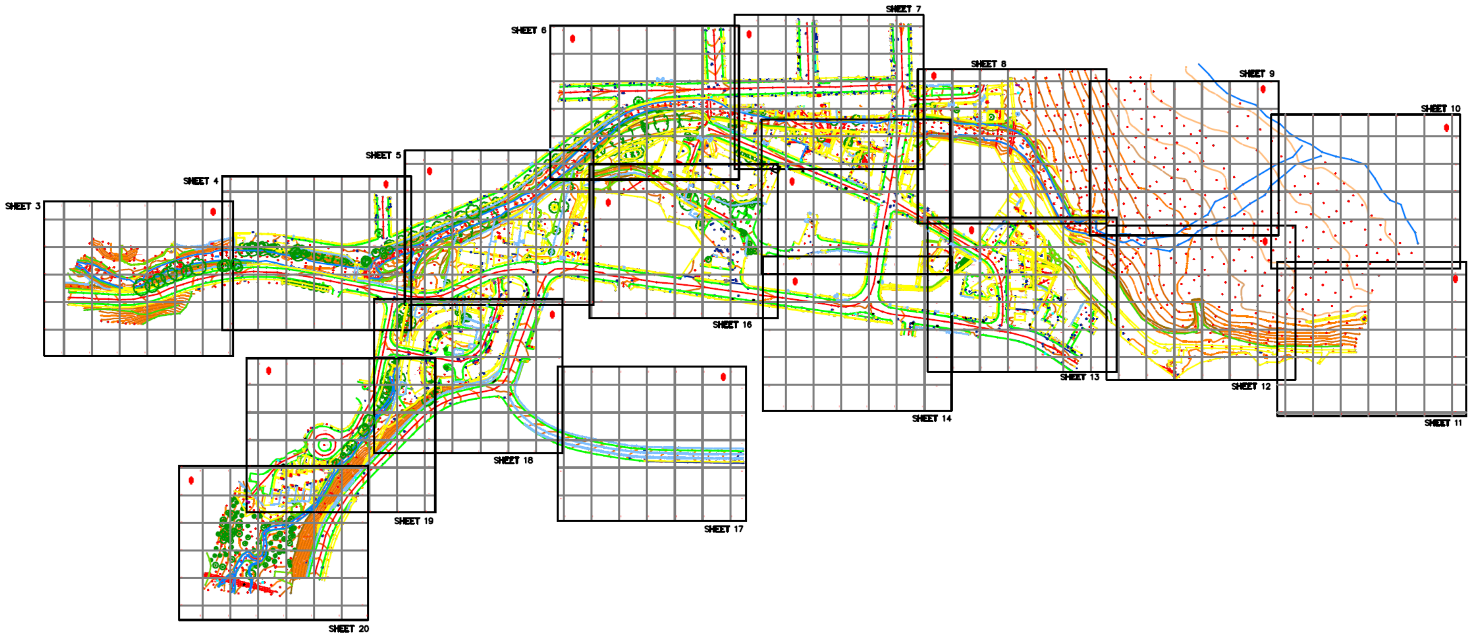
7. There is a recent example of flooding and a flood alleviation scheme in Rothes in The Moray Council area which may have some similarities and may be a helpful case study when considering options for Stonehaven. Moray Council may be able to provide further information on the scheme if requested to.

8. We noted a number of typographical errors in sections 1 and 2 of the report which you may wish to correct.
    - Paragraph 1 of 1.1 (line 3) “in tense” (remove space)
    - First line of 2.1 “December 2013” (2012)
    - Line 3 of para 2 of 2.1 “The flood water waters” (remove water)
    - Line 1 of para 3 of 2.1 “upstream of the Woodview Court” (remove the)

9. The recommendations made in section 6 of the report seem appropriate and proportionate.

Appendix B. TUFLOW model

Figure B.1: Topographical survey extent from 2013



Source: Aberdeenshire Council

Figure B.2: 1 in 200 year + cc fluvial flood event (flood defences in place)

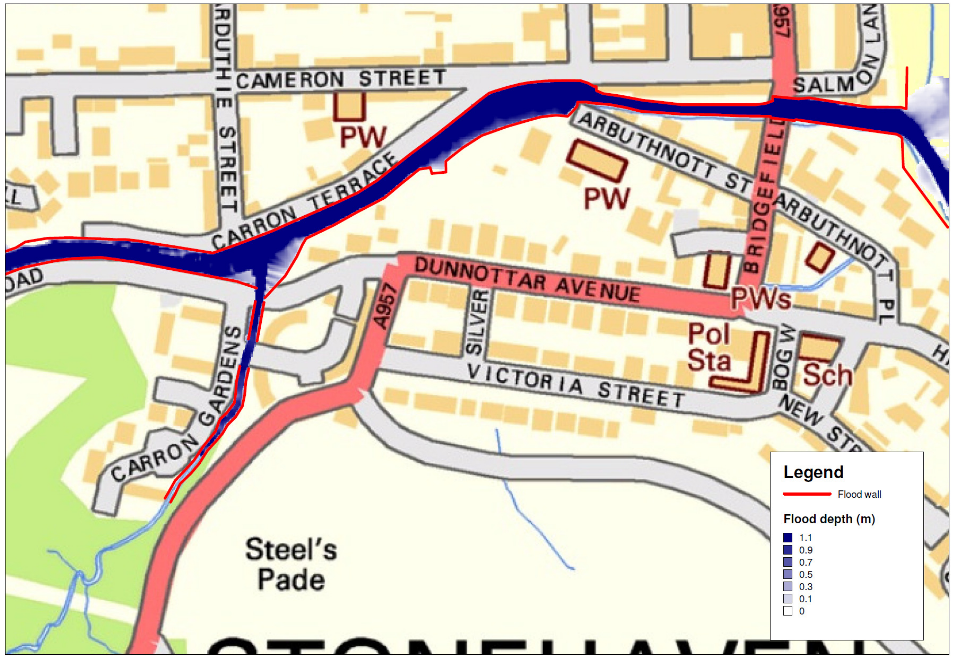


Table B.1: Carron Water – design water levels and minimum flood defence levels (Model 112)

Chainage	Design WLs	Proposed Freeboard	WLs + Freeboard	Flood defences levels
0	9.824	0.300	10.124	10.242
8	9.913	0.300	10.213	10.242
19	9.942	0.300	10.242	10.242
28	9.933	0.300	10.233	10.233
39	9.787	0.300	10.087	10.087
62	9.702	0.300	10.002	10.002
82	9.513	0.300	9.813	9.813
103	9.351	0.300	9.651	9.651
124	9.235	0.300	9.535	9.535
141	8.961	0.300	9.261	9.261
158	8.879	0.300	9.179	9.179
177	8.666	0.300	8.966	8.966
197	7.986	0.300	8.286	8.286
210	7.580	0.300	7.880	7.880
220	7.193	0.300	7.493	7.493
236	7.127	0.300	7.427	7.427
256	6.942	0.300	7.242	7.242
274	6.463	0.300	6.763	6.763
294	6.441	0.300	6.741	6.741
315	6.316	0.346	6.662	6.662
334	6.235	0.387	6.622	6.622
350	6.177	0.422	6.599	6.599
375	5.957	0.478	6.435	6.497
392	5.874	0.515	6.389	6.497
417	5.889	0.570	6.458	6.497
431	5.897	0.600	6.497	6.497
441	5.862	0.600	6.462	6.462
457	5.821	0.600	6.421	6.421
474	5.815	0.600	6.415	6.415
488	5.808	0.600	6.408	6.408
501	5.659	0.600	6.259	6.259
518	5.435	0.636	6.071	6.071
535	5.340	0.674	6.014	6.014
553	5.238	0.713	5.951	5.951
568	5.118	0.746	5.863	5.946
593	5.112	0.800	5.912	5.946
617	5.204	0.742	5.946	5.946
622	5.011	0.729	5.740	5.740
637	4.624	0.693	5.316	5.399
648	4.733	0.666	5.399	5.399
675	4.577	0.600	5.177	5.177
707	4.542	0.515	5.056	5.056
720	4.105	0.478	4.583	4.583
745	3.432	0.411	3.843	3.872
769	3.415	0.347	3.762	3.872
786	3.572	0.300	3.872	3.872
807	3.168	0.300	3.468	3.468
818	2.724	0.300	3.024	3.024

Source: TUFLOW

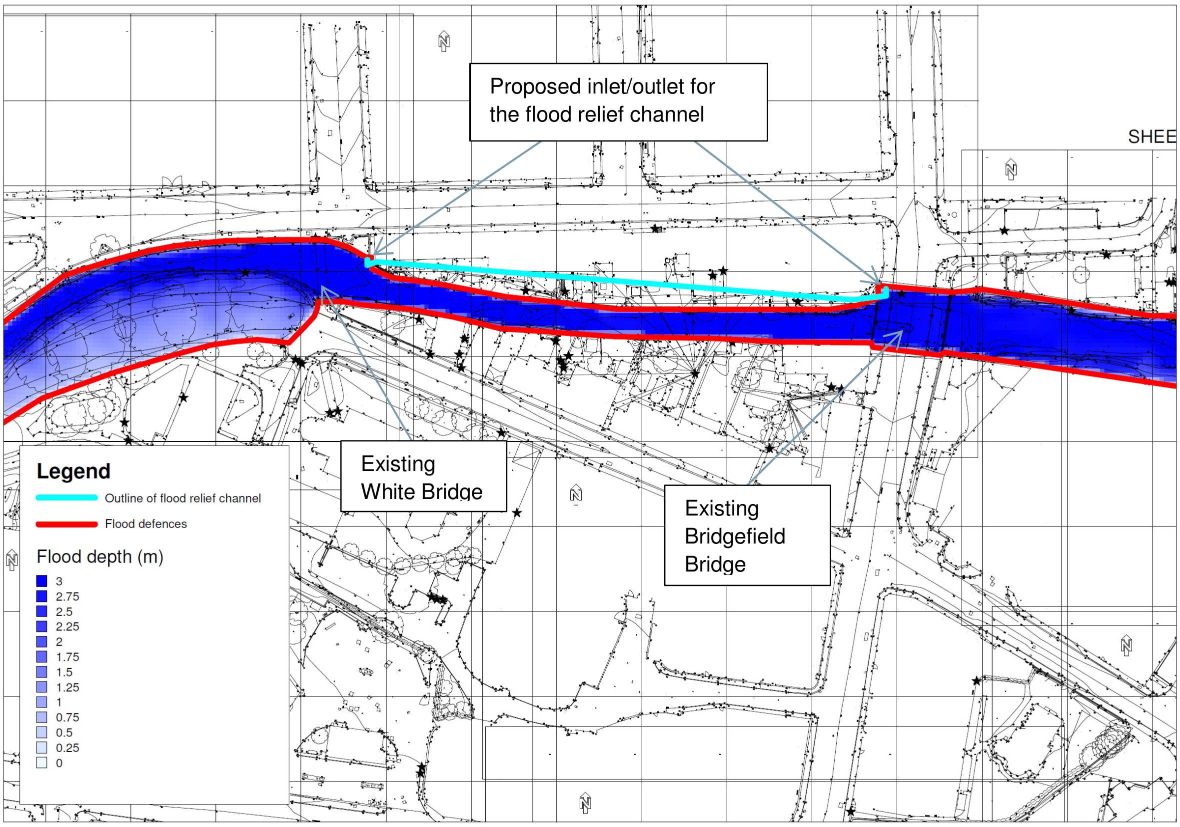
Table B.2: Glaslaw Burn – design water levels and minimum flood defence levels (Model 112)

Chainage	Design WLs	Proposed Freeboard	WLs + Freeboard	Flood defences levels
0	8.809	0.300	9.109	9.109
7	8.433	0.300	8.733	8.915
13	8.615	0.300	8.915	8.915
24	8.415	0.300	8.715	8.715
32	8.013	0.300	8.313	8.383
42	8.083	0.300	8.383	8.383
47	8.050	0.300	8.350	8.350
57	7.886	0.300	8.186	8.186
68	7.556	0.300	7.856	7.856
78	7.440	0.300	7.740	7.740
88	7.389	0.300	7.689	7.689
93	7.383	0.300	7.683	7.683
99	7.307	0.300	7.607	7.607
110	7.160	0.300	7.460	7.460
115	7.144	0.300	7.444	7.444
122	7.139	0.300	7.439	7.439
131	7.063	0.300	7.363	7.363

Source: TUFLOW

Appendix C. Optioneering

Figure C.1: Flood relief channel design



Source: The topographical survey from Aberdeenshire Council