

Technical Abstract

The Bin Brook is a tributary of the River Cam which runs through Cambridge from the south-west, before joining the Cam. In 1978, the Bin Brook flooded several properties in the Gough Way (GW) estate, due to insufficient capacity in the culvert which runs under the estate. After this event, a relief channel was constructed around GW, to divert water at high flows. Despite this, flooding occurred again in 2001, affecting 38 properties in the GW area. Since then, the Environment Agency (EA) has commissioned several studies to assess the flood risk in the area and develop potential flood mitigation measures. However, none of these options have been deemed economically viable and no new flood defence system has been implemented.

The main aim of this project was to develop a one-dimensional hydraulic model of the Bin Brook network and to use this to assess the efficiency of potential flood mitigation measures for the GW area. The flood mitigation measures considered were: a) to increase the GW relief channel capacity by increasing its depth by 0.5 m, and b) to construct a 150,000 m³ or 210,000 m³ flood storage area (FSA) upstream of the GW area. Flood Modeller, a programme for building models of open-channel networks and simulating the behaviour of flows through them, was used throughout the project.

The hydraulic model developed was based on an existing model of the Bin Brook, built for the EA by Halcrow Group Limited. In this project, the EA model was simplified by removing some components, such as hydraulic structures far upstream and downstream of the GW area, and inflows whose peak flowrates were small enough to be neglected. The simplified model was shown to match the results of the full model to a good accuracy (+/- 10 cm), within the GW area.

The flood mitigation measures were then incorporated into the simplified model and tested by running unsteady simulations for flood return periods ranging from 5 to 200 years. The peak water levels for each return period were then compared to those of the current Bin Brook network configuration.

The deeper relief channel was found to have a large impact for small events, reducing the peak water levels in the GW area by 10 to 35 cm for 1 in 5 and 1 in 10 year events. However, for larger events, the peak levels were only reduced by up to 10 cm. It was found that the standard of protection for the most vulnerable areas would be increased from 1 in 5 years to 1 in 10 years.

In contrast, the impact of the FSA schemes increased with the size of the flood event. Peak levels were reduced by up to 69 cm for a 1 in 50 year event and 78 cm for a 1 in 100 year event. It was found that the standard of protection for the most vulnerable areas would be increased from 1 in 5 years to 1 in 50 years by the larger FSA.

The project concludes that while both schemes could offer a higher standard of protection to the GW area, the FSA scheme (both the 150,000 m³ and the 210,000 m³ reservoirs) is more suitable and offers greater benefits.

Contents

1	Introduction	3
1.1	The Bin Brook: Overview and Background	3
1.2	Objectives	6
1.3	Previous Work	6
1.3.1	Bin Brook Flood Alleviation Scheme: Pre-Feasibility Report, Hal- crow Group Limited, 2002	6
1.3.2	Bin Brook Hydraulic Model QA Report, JBA Consulting, 2010 . . .	7
1.3.3	Case Study of the Bin Brook Flood and Its Mitigation Measures, Andreas Papadogiannakis, 2015	9
2	Open Channel Modelling	9
2.1	Steady Uniform Flow	9
2.2	Unsteady Flow	10
3	Flood Modeller Software	11
3.1	Computational Method	11
3.2	Simulation Types	11
3.3	Hydraulic Units	12
3.4	Flood Estimation Methods	13
4	Bin Brook Model Development	14
4.1	EA Model	14
4.2	Simplified EA Model	16
4.3	Bin Brook Boundary Conditions	16
4.4	Optimal Storm Duration	18
4.5	Verification of the Simplified Model	21
5	Bin Brook Model Applications	23
5.1	Deepening the Relief Channel	23
5.1.1	Initial Model	23
5.1.2	Simplified EA Model	26
5.1.2.1	Test Design	26
5.1.2.2	Results	28
5.1.2.3	Practical Issues	31
5.2	Flood Storage Area	32
5.2.1	Test Design	33
5.2.2	Results	37
5.2.3	Practical Issues	41

6	Future Work	45
7	Conclusions	45
8	Acknowledgements	46
A	Risk Assessment Retrospective	47
B	Photographs of Key Features of the Bin Brook Network	48
C	Flood Storage Area - Impact On Peak Levels in Main Channel	49

1 Introduction



Figure 1: Flooded paddock adjacent to Barton Road on 21 Oct, 2001. Photograph courtesy of Keith Glover.

1.1 The Bin Brook: Overview and Background

The Bin Brook is a tributary of the River Cam, which runs through Cambridge from the south-west, passing through many residential areas and the grounds of several colleges, before joining the River Cam. A map of the area is provided in Figure 2.

In 1978, the Bin Brook flooded the Gough Way (GW) estate, located in the south-west of Cambridge, damaging many properties. This occurred due to the insufficient flow capacity of the GW culvert, through which the Bin Brook flows under GW. This event led to the construction of a relief channel which encircles the estate, diverting flood flows away from the culvert. Despite this, in October 2001, flooding occurred again, affecting 38 residences in the GW and Barton Road areas, as well as the grounds of Robinson,

Clare Hall and St John's Colleges. It is estimated that the return period of this flood event¹ was somewhere between 50 years and 100 years [1].

Since then, the Environment Agency (EA) has commissioned several studies to assess the flood risk in the area and to develop potential flood mitigation schemes. In 2001, a study considered increasing the standard of protection² in the area by enlarging the GW relief channel, or constructing a flood storage area (FSA) upstream of the M11. However, both of these options were rejected as they did not meet the Department for Environment, Food & Rural Affairs' economic criteria for grant aid [1]. In 2007, interest in an FSA scheme was renewed, but it was once again rejected for financial reasons. Since then, little has progressed and no new flood defences have been provided. Despite this, the Gough Way Residents Association (GWRA), a group of GW residents, continues to communicate with the EA to promote the development of a flood mitigation scheme which would better protect their homes.

¹Return period is a probabilistic measure, defined as the average time interval between flood events of larger than a certain magnitude. It may be easier to think of a 100 year flood event as a 1 in 100 year flood event, or as a 1% annual probability of flooding

²Standard of protection describes the level of protection from flooding provided to a certain area

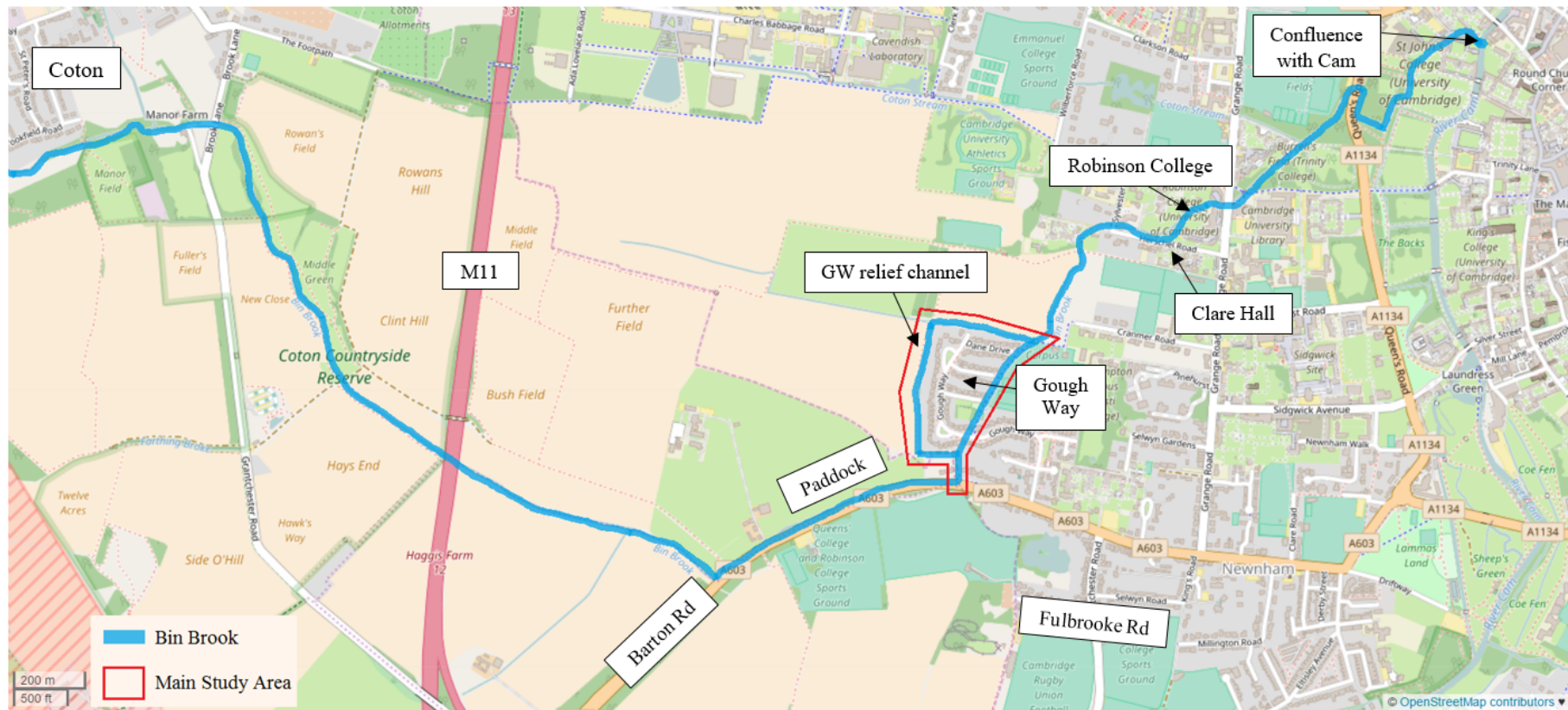


Figure 2: Map of the Bin Brook with the main study area shown in red, courtesy of www.openstreetmap.org/

1.2 Objectives

The main objective of this project was to assess the efficiency of several potential flood mitigation schemes, with the main focus being on the GW estate, henceforth referred to as the main study area. The tasks were as follows:

- To get in touch with the GWRA to arrange a visit to the main study area, and to obtain any information and suggestions they may have concerning the Bin Brook.
- To develop a 1D hydraulic model of the Bin Brook network.
- To assess the level of accuracy to which this model is able to reproduce the results of the existing EA model.
- To develop proposals for potential flood mitigation schemes, and to assess their efficiency using the hydraulic model.
- To make a recommendation on the most suitable flood mitigation scheme.

1.3 Previous Work

1.3.1 Bin Brook Flood Alleviation Scheme: Pre-Feasibility Report, Halcrow Group Limited, 2002

Following the October 2001 flood, the EA commissioned a report from the Halcrow Group to investigate the current flood risk and potential mitigation measures in the affected areas. The options considered were:

1. Do Nothing - Cease maintenance of existing infrastructure and allow the channels and culverts to deteriorate.
2. Maintain Existing - Do the minimum required to up-keep the existing infrastructure.
3. Channel Enlargement - Increase the capacity of the relief channel or of the Gough Way culvert.
4. Flood Storage - Construct an FSA upstream of the M11 motorway.

Halcrow developed a 1D hydraulic model of the Bin Brook, henceforth referred to as the EA model, extending from the village of Coton to its confluence with the river Cam to study the behaviour of flows through the network. Details about the hydraulic structures and topographic data for channel profiles and floodplain ground levels was gathered for the model.

The main conclusions drawn concerning the flood mechanism were as follows:

- Upstream of the M11, there is adequate flow capacity and the standard of protection is high.
- The capacity of the Barton Road culverts is low, therefore high flows bypass them by overtopping across the paddock towards the south-west corner of GW (around a 1 in 10 year event).
- The GW culvert can only pass up to approximately 2.3 m³/s, representing a design capacity of less than 5 years, beyond which additional flow will start to utilise the relief channel.
- Levels downstream of GW are largely governed by the Robinson College pond sluice gate.
- The standard of protection in the GW area ranges from 1 in 10 to 1 in 40 years, which is below the standards for dense urban areas of 1 in 50 to 1 in 200 years. In addition, predicted climate change would result in this dropping to 1 in 5 to 1 in 20 years within 50 years time.

The preferred option was the FSA, however it did not satisfy the economic criteria for grant aid eligibility. As a result, no action was taken other than to continue maintaining the existing network.

1.3.2 Bin Brook Hydraulic Model QA Report, JBA Consulting, 2010

The purpose of this study was to create a flood hazard map of the Bin Brook study area for a 1 in 100 year flood event. A 1D-2D linked model was created, based on the 1D model built by Halcrow, and was used to generate the flood outline. The study also looked at the model's sensitivity to Manning's roughness and the downstream boundary condition.

The report looked at the effect of a +/-20% change in Manning's roughness. The difference in water levels, in metres Above Ordnance Datum (mAOD), due to varying Manning's roughness can be seen in Figure 3. The report found that the influence of the assigned Manning's roughness is small, with the mean change in peak water levels being less than 5 cm for either a 20% increase or decrease in Manning's.

At the confluence with the Cam, the boundary condition has been set as a constant water level as the stage in the Cam is controlled by weirs and sluice gates at Jesus Green Lock [1]. The report looked at the effect of a +/-0.5 m change in this downstream level. The resulting difference in water level along the modelled reach can be seen in Figure 4. The report found that the influence of the downstream level is small, with the mean change in peak water levels being less than 5 cm for a +/-0.5 m change in stage. Water levels more than 800 m upstream of the boundary are unaffected.

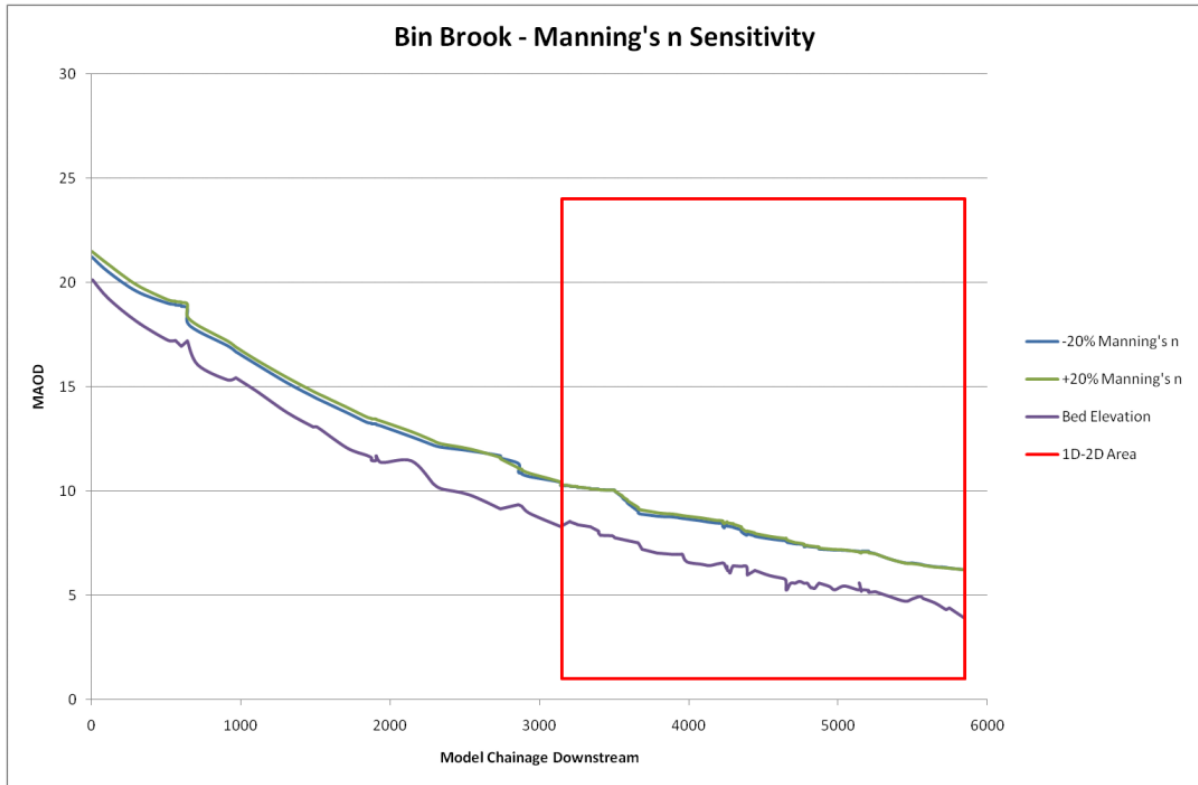


Figure 3: Effect of variation in Manning's roughness on water levels (mAOD) along the length of the model (m) [2]

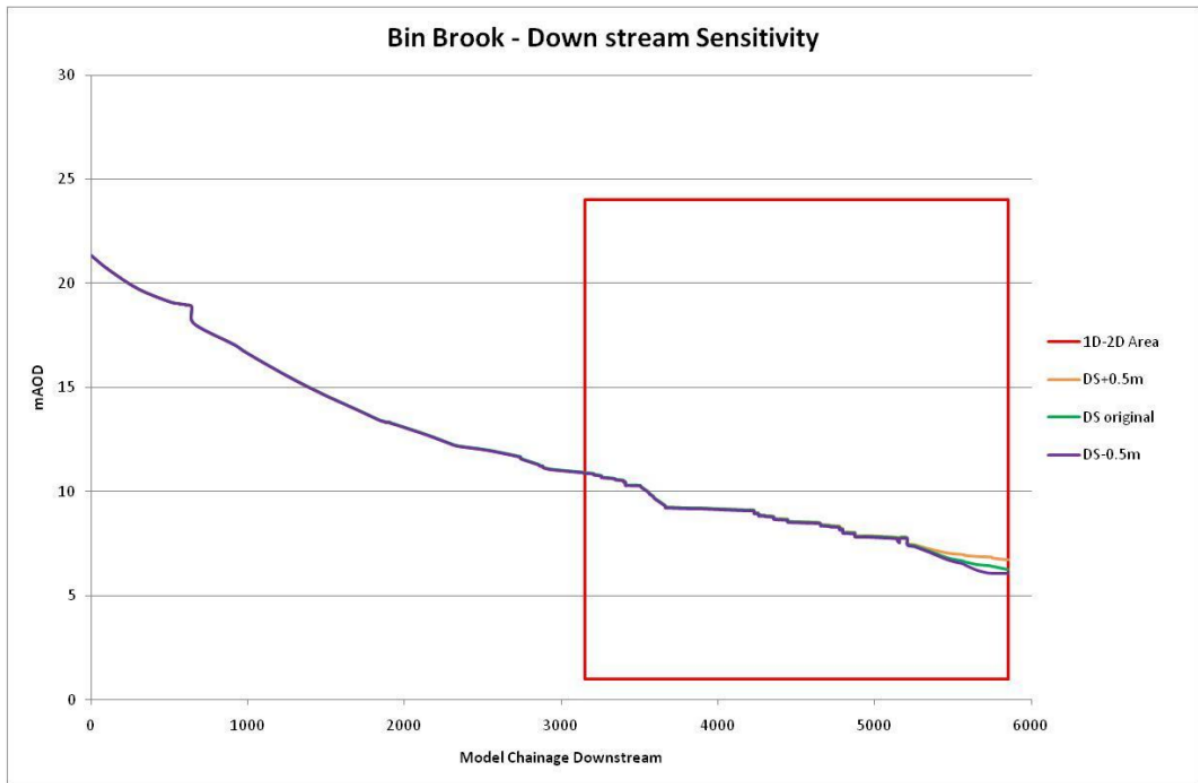


Figure 4: Effect of variation in downstream stage boundary on water levels (mAOD) along the length of the model (m) [2]

1.3.3 Case Study of the Bin Brook Flood and Its Mitigation Measures, Andreas Papadogiannakis, 2015

A previous 4th year MEng student worked on the study of the Bin Brook flood. The study used a simplified version of the EA model. The mitigation measures assessed were:

1. The addition of a new relief channel
2. The increase of the Gough Way culvert capacity

The first proposal involved constructing a new relief channel to divert flow from a section upstream of the first Barton Road culvert, across the paddock, and to the south-west corner of the existing relief channel. This was a logical route for the new relief channel, as it is the direction in which flows naturally go once the left bank at Barton Road is breached [1]. However, the potential capacity of the new channel was limited due to the presence of a Cambridge Water pipe running underneath the paddock at an unknown depth. It was shown that the addition of another relief channel would only provide protection to the Gough Way estate in small events. For 1 in 50 and 1 in 100 year events, the decrease in peak water level was found to be only 10 cm or 5 cm, respectively.

The second proposal looked at increasing the capacity of the GW culvert by: a) increasing its diameter or b) choosing a lining material with low Manning's roughness. For a 1 in 100 year event, it was found that increasing the culvert's diameter from 1.55 m to 2m would decrease the peak water level at the culvert inlet by 10 cm. Replacing the current culvert material with PVC (which would reduce Manning's roughness from 0.03 to 0.01) made practically no change to the water levels, and it was concluded that roughness does not have a big influence on the flow.

The report concluded that, of the options considered, only the increase in culvert diameter would have a meaningful impact on the standard of protection. Increasing the culvert capacity would reduce the flood levels for 1 in 100 year event to those of a 1 in 50 year event in the current situation.

2 Open Channel Modelling

Open channel flow is defined as the flow of water in a channel with a free surface. The types of flow considered in this study are steady uniform flow and unsteady flow.

2.1 Steady Uniform Flow

In steady uniform flow, water depth is constant with time and distance. The gravitational forces are balanced by the frictional resistance forces. The flow is governed by the Chézy equation (1) and the Manning equation (2) :

$$U = C\sqrt{R_h S_b} \quad (1)$$

$$C = \frac{1}{n} R_h^{1/6} \quad (2)$$

where U is the mean velocity, C is the Chézy coefficient, R_h is the hydraulic radius (the flow area, A , divided by the wetted perimeter, P), S_b is the bed slope and n is Manning's roughness coefficient, which is a measure of the channel's frictional resistance.

Knowing that discharge, Q , is equal to velocity times flow area, these equations can be combined to give an equation for flowrate:

$$Q = \frac{1}{n} \frac{A^{5/3}}{P^{2/3}} S_b^{1/2} \quad (3)$$

This allows Q to be calculated, given the depth, or vice versa. It is only valid for channels of constant cross-section, so generally does not apply to natural channels. For more detailed information on this topic, the reader should refer to “Hydraulics in Civil and Environmental Engineering”, Chapter 5 [6].

2.2 Unsteady Flow

In unsteady flow, water depth varies with time and distance. Unsteady flows are governed by the equations for conservation of mass and conservation of momentum, known as the continuity equation (4) and the dynamic equation (5). In 1D, these are:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (4)$$

$$\frac{1}{g} \frac{\partial U}{\partial t} + \frac{U}{g} \frac{\partial U}{\partial x} + \frac{\partial h}{\partial x} = S_b - S_f \quad (5)$$

where Q is the discharge at section x , A is the flow area at time t , q is the net volumetric inflow, U is the mean velocity, h is the depth at section x at time t , S_b is the bed slope and S_f is the energy gradient.

These equations, known as the Saint-Venant equations, cannot be solved analytically, so various mathematical models are employed to obtain a numerical solution. For more detailed information on this topic, the reader should refer to “River Mechanics”, Chapter 5 [9].

3 Flood Modeller Software

Flood Modeller is a software package which allows the user to build models of open channel systems. The free version of this software was used to analyse the behaviour of the Bin Brook network throughout the project. Flood Modeller allows the user to simulate the flow of water through a channel network using a 1D solver, which calculates water levels and discharge at each cross-section in the model, and a 2D solver, which simulates how flood water flows across the surrounding terrain. For the purposes of this project, only the 1D solver was used.

3.1 Computational Method

As discussed, in Section 2.2, the Saint-Venant equations, which describe the motion of water in open channels, cannot be solved analytically. Flood Modeller uses the Preissmann scheme to solve for discharge and water depth. This is an implicit finite difference scheme, which is widely used for 1D modelling. The scheme gives an exact solution of the linearised form of the Saint-Venant equations. For more information about the Preissmann scheme, the reader should refer to “River Mechanics”, Chapter 11 [10].

3.2 Simulation Types

Flood Modeller can be used for two main types of simulation: steady and unsteady.

Steady simulations are used for steady, open channel flow, and require minimal inputs. An estimate of flow split at junctions is usually sufficient to generate a steady flow water profile.

Unsteady simulations are used for unsteady, open channel flow. This allows the user to study how the water profiles will be affected when the inflow is varied over time (due to a rainfall event, for example). To run an unsteady simulation, the user must provide the following:

1. Initial conditions for discharge and stage (water level referenced to some datum). These can be obtained by performing a steady simulation.
2. Simulation start and end times - the period of time over which the user wishes to simulate the flow.
3. Timestep - the time interval at which calculations will be performed. Careful consideration must be given to the selection of this parameter. The Courant Number, Cr , is a dimensionless parameter which is used to define the stability condition of a model:

$$Cr = \frac{c\Delta t}{\Delta x} \quad (6)$$

where Δt is the timestep, Δx is the distance between cross-sections and c is the characteristic wave propagation velocity.

For explicit schemes, the value of Cr is limited to 1, but for an implicit scheme such as that used by Flood Modeller, this can be increased to 5-10. Generally, if a simulation fails due to instability, this can be resolved by reducing the timestep. Flood Modeller has an option for using an adaptive timestep, which automatically adjusts the timestep to the largest it can be without affecting model stability as the simulation progresses. This option was used for most of the unsteady runs carried out in this study.

3.3 Hydraulic Units

Flood Modeller offers a wide range of components for building models. Each ‘building block’ used in a model is referred to as a node. This section will give an overview of the components available, the principles that govern the flow through these nodes, and the inputs required from the user. When using a hydraulic model, it is important to remember that the accuracy of the representation of the real flows and structures will only be as good as the validity of the data provided by the user. For further details of the methodology used to model each type of unit, the reader should refer to the Flood Modeller Manual [4].

River Unit: This is the most basic unit, used to represent channel cross-sections. Discharge is computed using Eqs. (4) and (5). Inputs: cross-section geometry, distance to next cross-section, Manning’s roughness at each point on the cross-section.

Junctions: Used when the channel splits into multiple branches. The assumption is that the stage at all connected nodes is equal. Inputs: node labels of all connected nodes.

Bridges: There are two types of bridges: the US BPR bridge, which uses the method developed by the US Bureau of Public Roads to calculate the discharge at bridges, and the Arch bridge (which allows multiple arches), which uses the method developed at HR Wallingford to calculate the discharge. Inputs: bridge geometry, width, Manning’s roughness and the angle between the direction of flow in the channel and the normal to the main axis of the bridge.

Conduits: Used to represent circular, rectangular or irregular culverts. A culvert is formed of a culvert inlet, two or more conduit units, and a culvert outlet. In the conduit, the flow is described by Eqs. (4) and (5). The culvert inlet models the losses at the entrance of the culvert while the culvert outlet models the losses at the exit. Inputs: conduit geometry, Manning’s roughness, distance to next conduit unit, inlet and outlet

loss coefficients and trash screen configuration (if applicable).

Vertical Sluice Gate: Used to model a bank of vertical sluice gates. The sluice gate unit has 11 different modes of operation, each of which has a corresponding equation for discharge. Inputs: weir geometry, gate geometry, gate operation, coefficient of approach velocity for weir flow, under-gate flow and over-gate flow.

Reservoirs: Used to model areas into which water may spill. The assumption is that the stage at all connected nodes is equal. The rate of water level rise, $\frac{\partial h}{\partial t}$, is related to the net discharge, Q by the following relationship:

$$Q - A(h)\frac{\partial h}{\partial t} = 0 \quad (7)$$

where $A(h)$ is the surface area of the reservoir at elevation h [4].

Inputs: set of surface area-elevation co-ordinates describing the reservoir geometry.

Spill Units: Used to represent flow spilling out of the channel, or past an overtopped structure. The overtopping flow is modelled as in-line or lateral flow over an irregular weir. It operates in three principle modes: zero flow (where the stage in the connected nodes is lower than the bank level defined), free flow (where the bank has been breached, but the stage in the spill is still lower than that in the connected nodes) and drowned flow (where the weir is submerged). For free flow, the discharge, Q , is defined as:

$$Q = C_d b h^{1.5} \quad (8)$$

where C_d is the weir coefficient, b is the width of the spill section and h is the height of water above the crest [4].

Inputs: weir coefficient and the profile of weir crest elevation.

Hydrological Boundaries: Used to set the boundary conditions at inflow points and downstream boundaries. They are defined as curves of either discharge against time, or stage against time. Inputs: there are many different types of hydrological boundaries, requiring different inputs. The most basic allow the user to define a custom boundary condition. More complicated boundary units can generate the inflow curve according to some flood estimation method, as will be further discussed in Section 3.4.

3.4 Flood Estimation Methods

To study the effects of flood mitigation schemes, we need a method for flood prediction.

The FEH (Flood Estimation Handbook) boundary unit in Flood Modeller uses the unit hydrograph rainfall-runoff method to generate a curve of discharge against time (referred to as a hydrograph) based on a specified return period.

A brief overview of this method for an *ungauged* catchment, such as the Bin Brook, is given below [5]:

1. The unit hydrograph is estimated based on certain catchment descriptors. A unit hydrograph describes the relationship between rainfall and runoff for a certain catchment.
2. The rainfall profile for the specified return period is generated using the FEH's depth-duration-frequency (DDF) model. This model allows the design rainfall at any location in the UK to be estimated.
3. The runoff hydrograph is obtained by convolving the design rainfall profile with the unit hydrograph.

An example of the outputs generated by the boundary unit can be seen in Figure 7.

When using this method to generate inflow hydrographs for a model, it is important to remember that flood prediction is uncertain. The estimates are based on historical data collected from roughly similar catchments, so uncertainty is unavoidable [11].

4 Bin Brook Model Development

The Bin Brook model extends from the village of Coton (around 2 km upstream of the main study area) to the brook's confluence with the River Cam. It is classified as 'main river' from Barton Road to the Cam, which makes the EA responsible for promoting flood mitigation measures along this reach [1]. The brook runs through several culverts along Barton Road, then turns northwards into the GW estate. At this point, it is joined by a tributary flowing from the Fulbrooke Road area, to the south. At the entrance of GW, the channel branches into the main channel, which flows under GW through a culvert, and the relief channel, which circles GW. The two re-join at the north-east corner of GW.

The brook then flows through the grounds of Clare Hall and Robinson College, which has a pond fed by the Bin Brook in which the water levels are controlled by a sluice gate. The brook then passes underneath Grange Road, through the grounds of St John's College and under Queen's Road, before finally joining the River Cam.

In October, a trip to the main study area was organised to better visualise the hydraulic structures located along the water course, and relate them to their nodal representations. Photographs of key structures taken during this trip can be found in Appendix B.

4.1 EA Model

The 1D model built by Halcrow for the EA was obtained by Andreas Papadogiannakis for his study of the Bin Brook flood. It has also been used as the basis for the hydraulic model developed in this project. The EA model is made up of 447 nodes, and includes all the hydraulic structures along the water course, eight inflow boundary conditions and

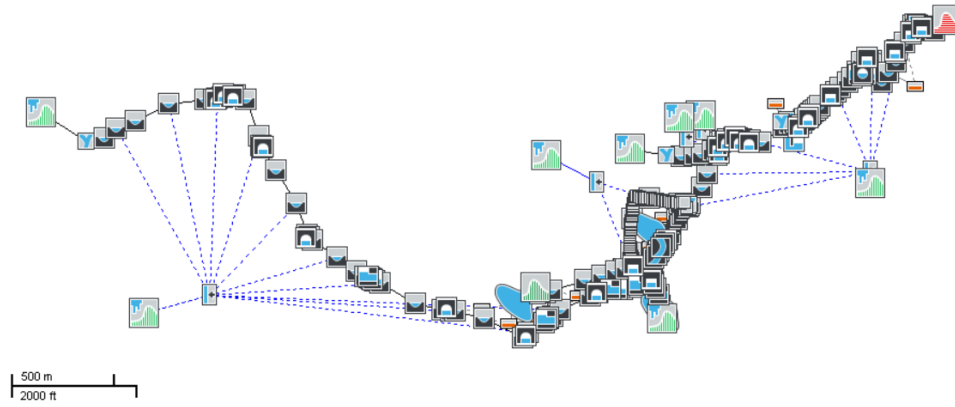


Figure 5: Environmental Agency Model

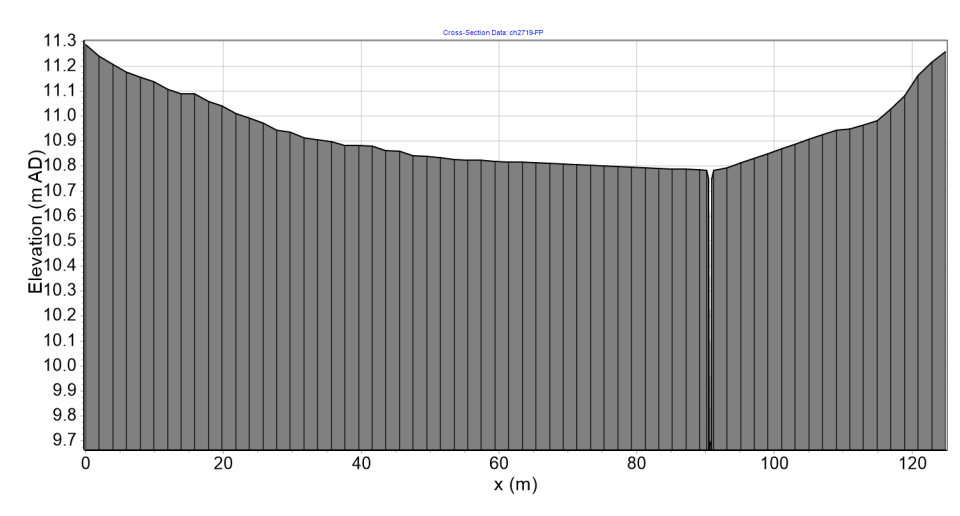


Figure 6: River section modelling drain, extended to include the paddock ground elevation

one downstream boundary condition. A screenshot of the model can be seen in Figure 5. Several important features should be noted:

- The drain which runs across the paddock to the south-west corner of the relief channel has been modelled by six river units. These are connected to the Barton Road channel units by spill sections, to allow the flow of overtopping water across the natural floodplain to be modelled. Though the drain itself is only 50 cm wide, the sections have been extended to a width of 100 m, such that they include the ground elevation profile of the paddock, as shown in Figure 6. There is one additional inflow boundary, “dummy_in”, at the upstream end of this drain. This is a constant flow-time boundary of $0.1 \text{ m}^3/\text{s}$, which does not represent a real flow, but which is necessary to assist model stability as Flood Modeller is very poor at dealing with dry channels.
- Due to the sparsity of surveyed cross-sections in certain reaches, many “interpolate” sections have been used. These are river sections containing no cross-sectional data,

but at which the programme will still perform calculations. It does this by calculating a weighted average of the properties of the river sections either side of it, producing a hybrid section based on its location [4]. This helps to reduce model sensitivity due to sudden changes in cross-section. In this model, many interpolated sections are used in the main area of interest (at 10 m intervals in the Gough Way area), in order to ensure water levels are computed at all critical locations.

- The Robinson sluice gate is modelled as being permanently open as the Robinson College porters have instructions to raise the gate in winter and drop it during summer [2]. It has been assumed that they would open it in case of a large rainfall event, regardless of the season.

4.2 Simplified EA Model

Flood Modeller Free was used for this project and while this version of the software allows models of any size to be viewed, it can only run simulations for models containing a maximum of 250 nodes. Therefore, to manipulate and run simulations with the EA model, it first had to be simplified by removing many nodes. The following steps were followed for the selection of nodes to be deleted:

1. Two of the inflows (CH_ditch_us and CH_ditch_lat) were found to make a negligible contribution to the total peak discharge ($<0.17\%$), as can be seen in Table 1, and thus were removed, along with the river units connecting them to the main part of the network .
2. River and interpolate sections were removed such that the distance between cross-sections did not exceed 50 m in the main study area (Gough Way) or 400 m upstream and downstream of this area.
3. Hydraulic structures upstream and downstream of the main study area were removed, starting with the least obstructive.

In total, 217 nodes were deleted, leaving a few spares for modelling potential mitigation measures in the later stages of the project. Although deleting nodes will have reduced how accurately the model represents the real life network, it also has some advantages: the reduced complexity can improve the stability of simulations and make it easier to monitor the model and troubleshoot errors. Section 4.5 will look at the effect of this simplification on the model's accuracy.

4.3 Bin Brook Boundary Conditions

In order to run, a model needs at least one upstream and one downstream boundary condition. Generally, upstream boundaries are flow-time boundaries, and downstream

Inflow Label	Peak discharge (m ³ /s)
SC1	4.610
SC2	4.526
SC3	0.788
SC4	1.132
SC5	2.894
CH_BB_lat	0.092
CH_ditch_us	0.016
CH_ditch_lat	0.007

Table 1: Peak discharge of inflow boundaries for a 14.5 hour, 1 in 100 year event

Inflow Label	Inflow Description
SC1	Upper catchment
SC2	Lateral inflows from upstream limit to Gough Way
SC3	Fulbrooke Road tributary
SC4	Drains on left bank of Gough Way relief channel
SC5	St John's tributary and remaining inflows
CH_BB_lat	Lateral inflows downstream of Gough Way

Table 2: Description of inflows represented by model boundary units

boundaries are stage-time boundaries.

For the Bin Brook model, the downstream condition is a constant water level boundary, as the stage at this location in the River Cam is controlled by weirs and sluice gates at Jesus Green Lock, as was discussed in section 1.3.2.

The model has six upstream flow-time boundaries, each representing the inflow from a different catchment, as described in Table 2. The boundary unit at the very upstream end of the model, SC1, is the only one which contributes to the baseflow in the network. The inflows are defined as FEH boundary units, which generate a hydrograph based on the following inputs provided by the user:

- Catchment details: The catchment area, the urban extent of the land cover and the standard annual average rainfall
- Event data: The storm duration, the storm area (usually equal to the catchment area), the standard percentage runoff and the data interval, t , which is the time interval used to generate points to plot the rainfall profile and unit hydrograph
- Event rainfall data: The rainfall return period, the storm return period and the six DDF model parameters (catchment descriptors), which can be obtained from the FEH CD-ROM database

- Unit hydrograph data: The mean drainage path length, mean drainage path slope, the proportion of time the catchment soil moisture deficit was below 6 mm in the period 1961-1990 and the calibration factor for the time to peak, T_p , of the unit hydrograph
- Options: The magnitude of the baseflow (which can be zero) and the time delay (which shifts the start time of the hydrograph)

The method used to produce this inflow hydrograph was discussed in Section 3.4.

4.4 Optimal Storm Duration

To assess the performance of potential flood defences, it is desirable to test them against the worst case scenario. For a given return period, the critical flood event, which gives the largest flow/stage, depends on the storm duration. This is because the rainfall profile obtained from the DDF curve, which can be obtained from the FEH CD-ROM database, depends on the storm duration as well as the return period [5]. As a result, different storm durations produce different hydrographs for the same return period, as is illustrated by Figure 7.

There are several methods for choosing the critical storm duration. For critical peak discharge, the FEH recommends a storm duration, D given by:

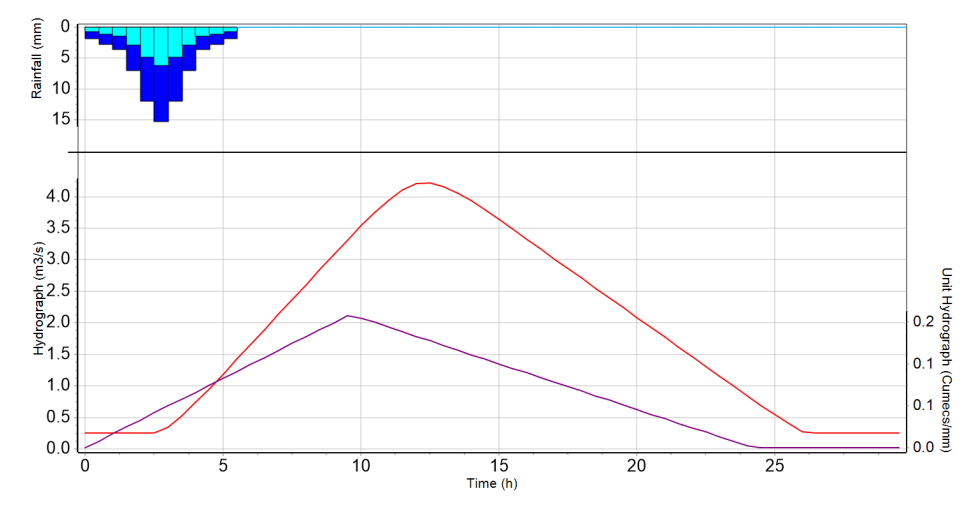
$$D = \left(1 + \frac{SAAR}{1000}\right) T_p \quad (9)$$

where $SAAR$ is the standard annual average rainfall, and T_p is the time to peak of a t -hour hydrograph given by:

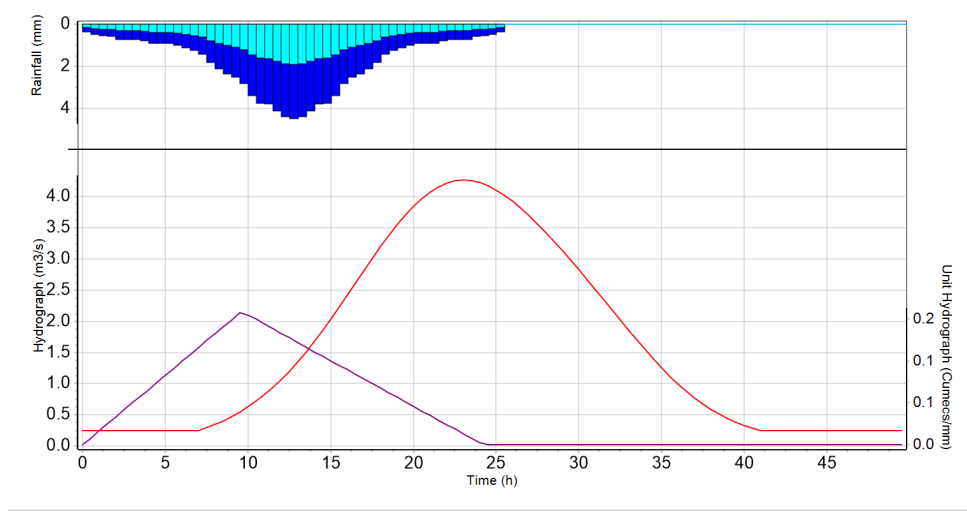
$$T_p = 4.27DPSBAR^{-0.35}PROPWET^{-0.80}DPLBAR^{0.54}(1 + URBEXT)^{-5.77} + \frac{t}{2} \quad (10)$$

where $DPSBAR$ is the mean drainage path slope (m/km), $PROPWET$ is the proportion of time catchment soil moisture deficit was below 6mm during the period 1961-1990, $DPLBAR$ is the mean drainage path length (km), $URBEXT$ is the urban extent and t is the time interval at which points of the hydrograph curve are calculated [5]. The catchment data for all the inflows was extracted from the EA model, and equation (9) was used to compute the recommended storm duration for each inflow. The results can be seen in Table 3.

It can be seen that there is a wide range of D for this set of catchments. There is no obvious way to average these values to obtain a single storm duration for the whole model. With multiple inflows, the hydrographs interact with each other as they travel through the network, resulting in a different magnitude and time of peak flow at each point in the network. This is related to the location of the inflows relative to that point,



(a) 1 in 100 year hydrograph for a storm duration of 5.5 hours at boundary unit SC1



(b) 1 in 100 year hydrograph for a storm duration of 25.5 hours at boundary unit SC1

Figure 7

Inflow Label	D (hours)
SC1	9.98
SC2	8.25
SC3	10.32
SC4	8.55
SC5	4.69
CH_BB_lat	2.17

Table 3: Critical storm duration, D, for model inflows

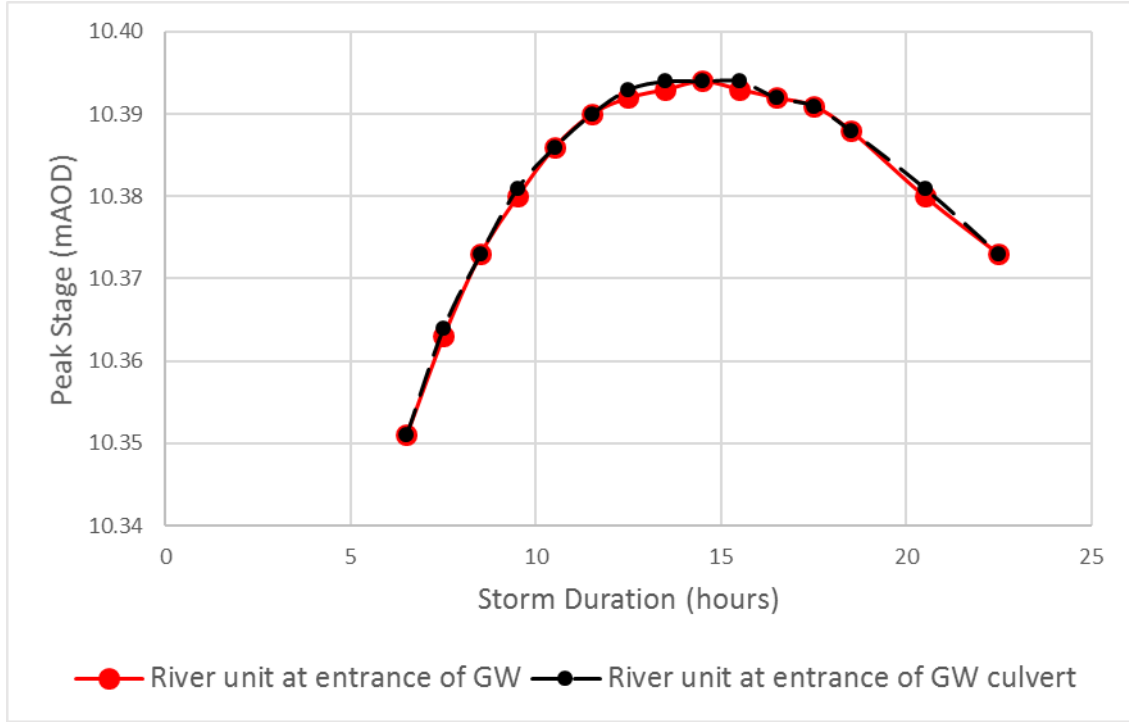


Figure 8: Effect of storm duration on peak stage

and the resulting hydrograph at each point in the network will depend on the complex interaction between the upstream inflow hydrographs.

As an alternative approach, it was decided to find the storm duration for all inflow boundaries which gave the highest peak water level at the critical cross-sections in the model. These are the river section at the entrance of Gough Way, just upstream of junction between the main channel and the relief channel, and the river section at the entrance of the Gough Way culvert. Simulations were ran for 1 in 100 year events, with storm duration varying from 6.5 to 22.5 hours. The resulting peak water levels these sections were extracted and plotted against storm duration (see Figure 8). As the sections are close, the curves are very similar and both peak at 14.5 hours. The variation in peak water level is quite small; the difference between the largest stage achieved and that from a 6.5 hour storm is less than 5 cm, for example.

The Optimal Storm Duration tool in Flood Modeller is able to run a large batch of simulations, varying the storm duration at each boundary unit individually to find the combination of storm durations which gives the largest stage at the section of interest. Unfortunately, this tool is not available in Flood Modeller Free. Therefore, for the purposes of assessing the performance of potential mitigation measures, it was deemed that a storm duration of 14.5 hours was a suitable critical duration. This duration was employed for all subsequent tests discussed in this report.

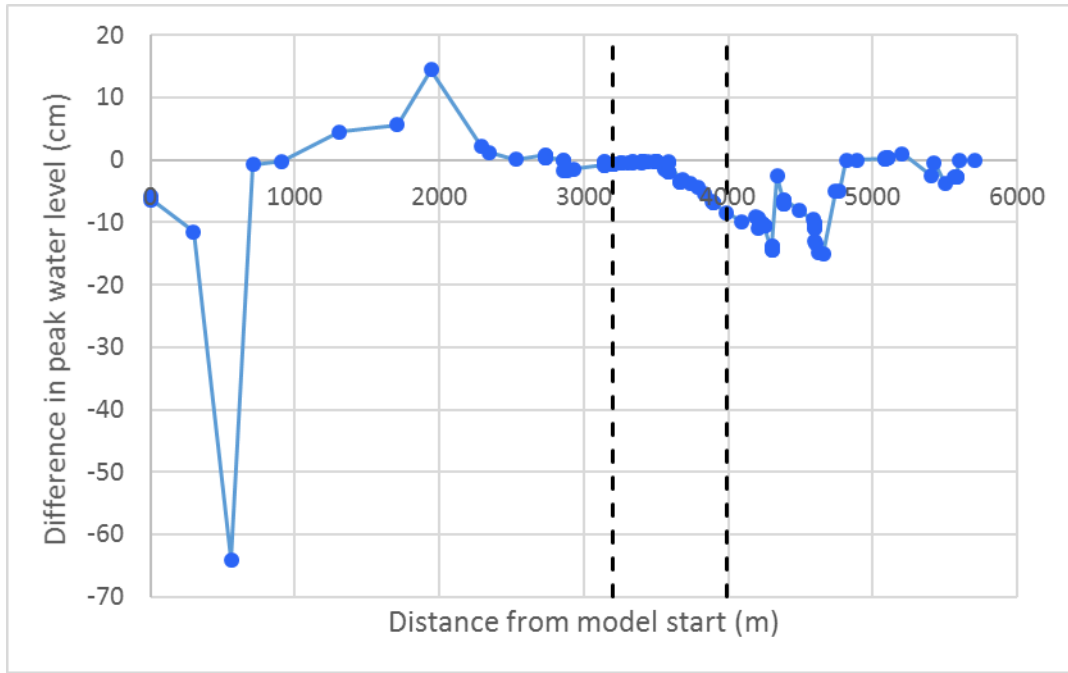


Figure 9: Difference in peak water levels between the simplified model and the EA model in the main channel

4.5 Verification of the Simplified Model

Before proceeding further with the study, it was necessary to verify that the simplification of the EA model had not resulted in an unacceptable loss of accuracy in the main study area. As the full model could not be run in Flood Modeller Free, it was sent to a colleague with access to the full version of the software, who kindly ran it and sent back the results. A 100 year return period was used for the model verification.

When calibrating their model, Halcrow stated that peak flood levels were largely matched within 0.05 m compared to the reference event, and that this was well within survey and model tolerances [1]. Based on this statement, the accuracy of the simplified model was considered to be very good where peak levels fell within 0.05 m of the EA model output, and good where the peak levels fell within 0.1 m. The change in peak stage at each node was plotted along the length of the channel, such that a negative difference represents a reduction in stage compared to the EA model. Figure 9 shows the change along the main channel (from Coton to the confluence with the Cam), and Figure 10 shows the change along the Gough Way relief channel. The black dashed lines represent the delimitation of the main study area.

As can be seen from Figures 9 and 10, the differences in peak water levels in the main study area all fall within 10 cm of the EA model results. They correlate very well at the upstream end of the study area, and only begin to deviate slightly at the downstream end, due to the removal of structures in the downstream reach. It is important to note that, in the main channel, the peak levels are always lower, so more conservative flood levels may

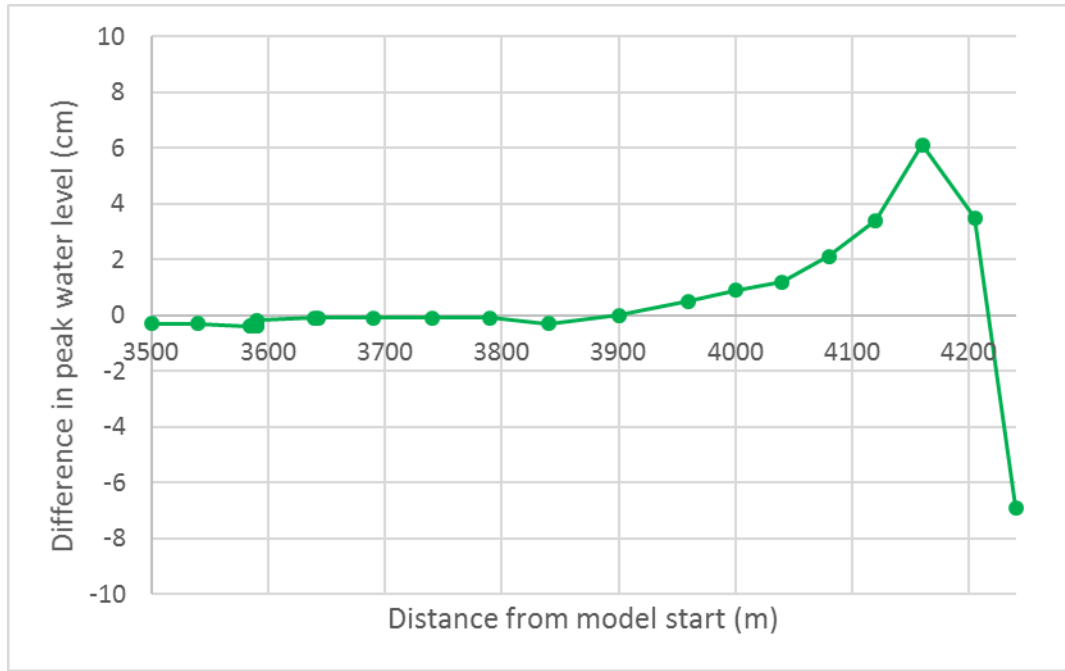


Figure 10: Difference in peak water levels between the simplified model and the EA model in the relief channel

need to be employed to determine if flooding has occurred at those cross-sections. On the other hand, at the downstream end of the relief channel, the peak levels tend to be slightly higher, which is likely due to the reduction in the number of interpolated sections between river sections with fairly different profiles in this zone.

Upstream and downstream of the main study area, the peak levels are not so well matched, but nonetheless largely remain within 15 cm of the EA model levels. This is as expected, as many structures were removed from these reaches. There is one at the upstream end of the model, where there is a very large discrepancy of 0.64 m. Upon investigation, it was observed that this section is located directly downstream of two bridge units which were deleted in the model simplification. These represented a significant constriction to the flow, so it makes sense for the water level to be significantly lowered by their removal. This was considered acceptable as it does not seem to have had a significant effect on the accuracy of the levels further downstream.

It was concluded that a good level of accuracy was achieved in the main study area, and these results were deemed to match the EA model sufficiently well to proceed with the study using the simplified model.

5 Bin Brook Model Applications

5.1 Deepening the Relief Channel

5.1.1 Initial Model

In Michaelmas term, a highly simplified 1D model of the GW area was built in order to become familiar with the software, verify the model results with hand calculations and carry out a preliminary investigation of the relief channel efficiency.

In the current configuration, the channel splits at the entrance of the GW estate; the main channel passes underneath GW through a 157 m long culvert, and then continues for 231 m overground, while the relief channel circles the estate over a distance of 740 m before re-joining the main channel at the exit of GW. However, the relief channel only starts to divert a portion of the flow once the water depth in the main channel has already reached approximately 0.5 m, due to the bed elevation of the relief channel at the upstream junction being 0.5 m higher than that of the main channel. The bed levels at the downstream junction are approximately equal.

For the initial model, the channel geometry was simplified to uniform rectangular cross-sections with constant bed slope so that the equations for steady, uniform, open-channel flow could be used. Note that the constant bed slope simplification meant that there was a step change in bed level at both ends of the relief channel, unlike in the actual network. Figure 11a shows the schematic representation of the model in Flood Modeller, and Figures 11b and 11c show the simplified geometries of the main channel and relief channel cross-sections, respectively. The relief channel was assigned a lower Manning's roughness coefficient as it is concrete-lined along a large proportion of its length.

In steady, uniform, open-channel flow, the discharge is given by equation (3). By expressing A and P in terms of width, W , and water depth, h , we can derive an equation for Q in terms of h :

$$Q = \frac{W^{5/3}}{n} \left(\frac{h^{5/2}}{W + 2h} \right)^{2/3} S_b^{1/2} \quad (11)$$

Equation (11) was used to calculate the flow in the main and relief channels for water depths of 0 to 1.5 m, which is the maximum depth allowed before the main channel bank is over-topped. Note that, for the relief channel, the water depth, h , is 0.5 m less than that in the main channel (referred to as the inflow water depth) due to the step change in bed elevation. These results were compared to those obtained by running steady simulations in Flood Modeller and were found to be fully consistent. This is as expected, and was not so much interpreted as a verification that the software works as it should (Flood Modeller is, after all, used for flood modelling by professionals across the water engineering industry), but rather as a verification that no mistakes were made by the user

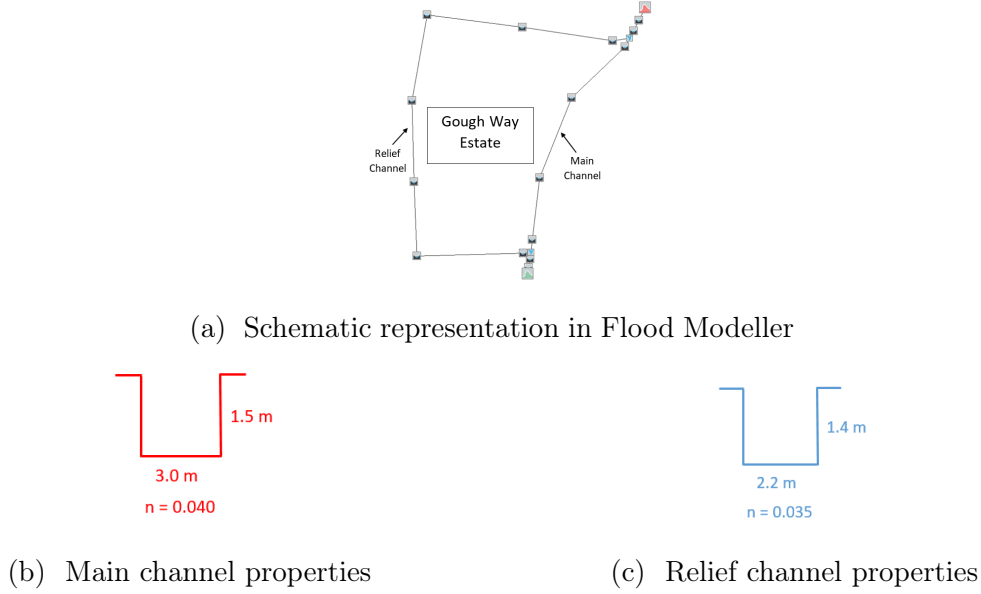


Figure 11

in setting up the models units, boundary conditions and initial conditions.

The relief channel is long and has a shallow slope, therefore there were some concerns about how much it was able to reduce the flowrate in the main channel for large flow events. To investigate the efficiency of the relief channel, the results were used to compute the ratio of flow in the relief channel to total incoming flow for different inflow water depths (Figure 12, blue curve), and the flow velocities in each branch (Figure 13).

It was found that the proportion of flow carried by the relief channel increases with water depth, but only reaches about 22% before the main channel becomes full. In addition, the velocity in the relief channel is quite slow compared to that in the main channel, despite it having a lower roughness coefficient. This is due to its shallow slope, and the fact that the wetted perimeter term in Eq. (1) will be consistently lower in the relief channel due to its smaller width and water depth.

Clearly, as the relief channel has already been constructed, it would be desirable to improve the level of flood protection it provides to the GW estate. One of the options considered by Halcrow (see Section 1.3.1), was to increase channel capacity in the GW area by increasing the width of the relief channel by 50%. However, this would require the acquisition of land to extend the channel boundary. This adds cost and complexity to the scheme, so instead, it was decided to explore the possibility of increasing the relief channel capacity by deepening it. This would involve deepening the relief channel by 0.5 m, effectively removing the step change in bed level at the upstream junction.

This change was made to the network, and the discharge for inflow water depths of 0 to 1.5 m was computed, as before. Note that, the water depth in the relief channel is now equal to the water depth in the main channel. The curve of the proportion of total

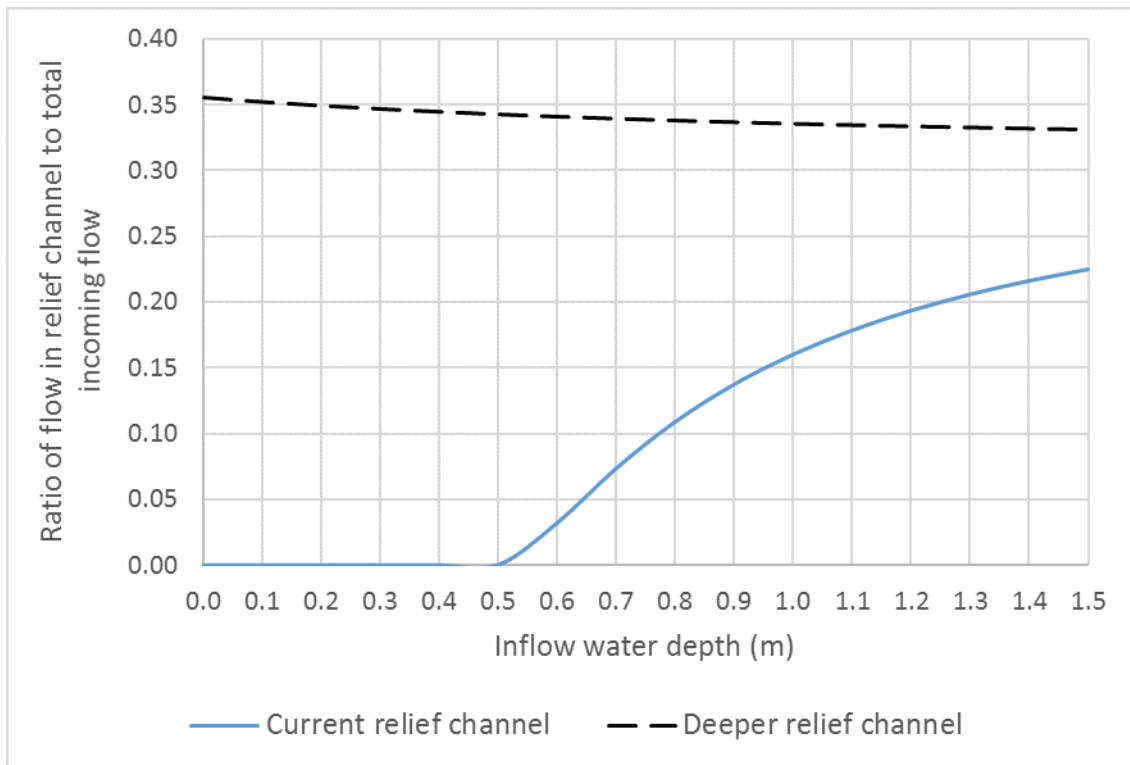


Figure 12: Variation of proportion of total flow in the relief channel with inflow water depth

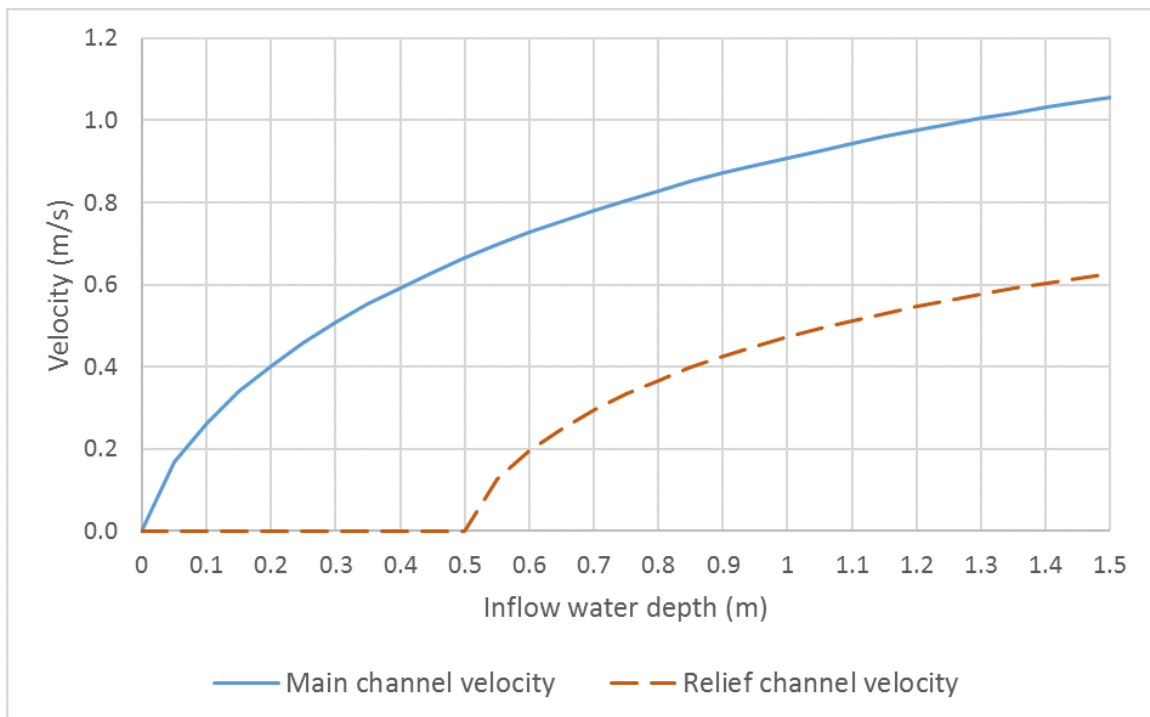


Figure 13: Variation of flow velocities in the main and relief channels

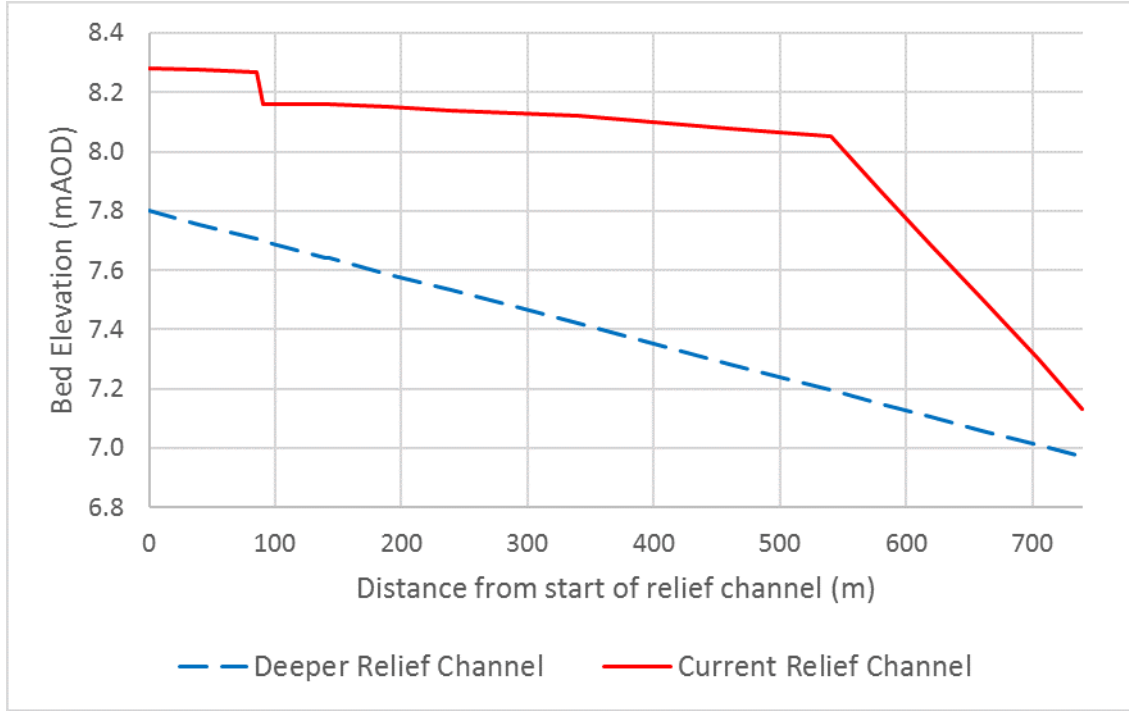


Figure 14: Relief channel bed profile

flow being carried by the deepened relief channel was plotted on the same graph as that of the current relief channel, as can be seen in Figure 12. The proportion of flow carried by the relief channel now decreases with water depth, but is consistently higher than for the current channel, reaching about a third at the maximum water depth of 1.5 m. The added flow area increases the maximum total flow capacity of the system from $6.1 \text{ m}^3/\text{s}$ to $7.1 \text{ m}^3/\text{s}$, a 16% increase. The effect on the relief channel velocity was also positive, but small, which was expected as although the water depth was increased, the slope remained the same.

The results of this preliminary investigation suggested that deepening the relief channel could significantly improve the standard of protection to the GW area. It was therefore decided that the impact of this option should be investigated in the simplified EA model, to get a much more accurate picture of the effects on the real network.

5.1.2 Simplified EA Model

5.1.2.1 Test Design

The channel network around the GW area is in reality far more complicated than the initial model. Not only is the channel geometry non-rectangular, it also varies along the channel, and there are several hydraulic structures limiting the flow capacity. In large flood events, the channel along Barton Road is over-topped and water flows across the paddock to the south-west corner of GW, adding a significant inflow to the relief channel.

Unlike in the initial model, the slope of the current relief channel is not constant, as

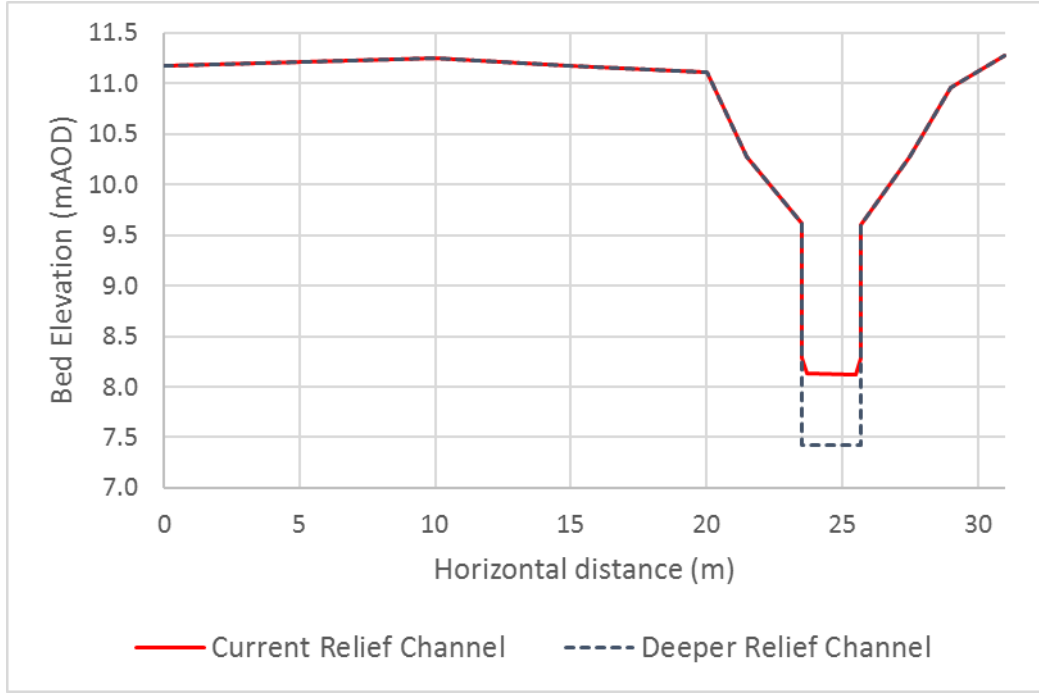


Figure 15: Example of a relief channel cross-section

can be seen from its bed profile (Figure 14). It is quite flat for the first 550 m, before becoming steeper after the final bend, along the northern boundary of GW. The relief channel nodes were edited such that the bed level at the upstream and downstream cross junctions would match those in the main channel, and such that the slope along the relief channel would be uniform, to avoid stagnant water at low levels. The blue curve, Figure 14, shows the new bed profile. An example of the change in cross-sectional profile can be seen in Figure 15, which shows the cross-section of a river unit in the relief channel.

Unsteady simulations were ran for a 1 in 5, 1 in 10, 1 in 20, 1 in 40, 1 in 50, 1 in 100, 1 in 150 and 1 in 200 year event. The simulation length was 40 hours, in order to allow the peak flow to pass through the system, and the water levels to recede back to their baseflow levels.

At first, the altered model had some difficulties running, producing unexpectedly noisy results, or crashing before completion due to a water level calculated at one of the time steps being unphysically large. This was solved by taking the following steps:

- The baseflow of the inflow boundary unit SC4, which feeds two relief channel river nodes, was increased from 0 to $0.2 \text{ m}^3/\text{s}$ to support the stability of that branch as the flood waters recede.
- The maximum number of iterations performed at each timestep was increased from 11 to 16. This helps to increase the likelihood of convergence [4].
- The Preissmann box weighting factor was increased from 0.7 to 0.9. This has to do with the numerical scheme used.

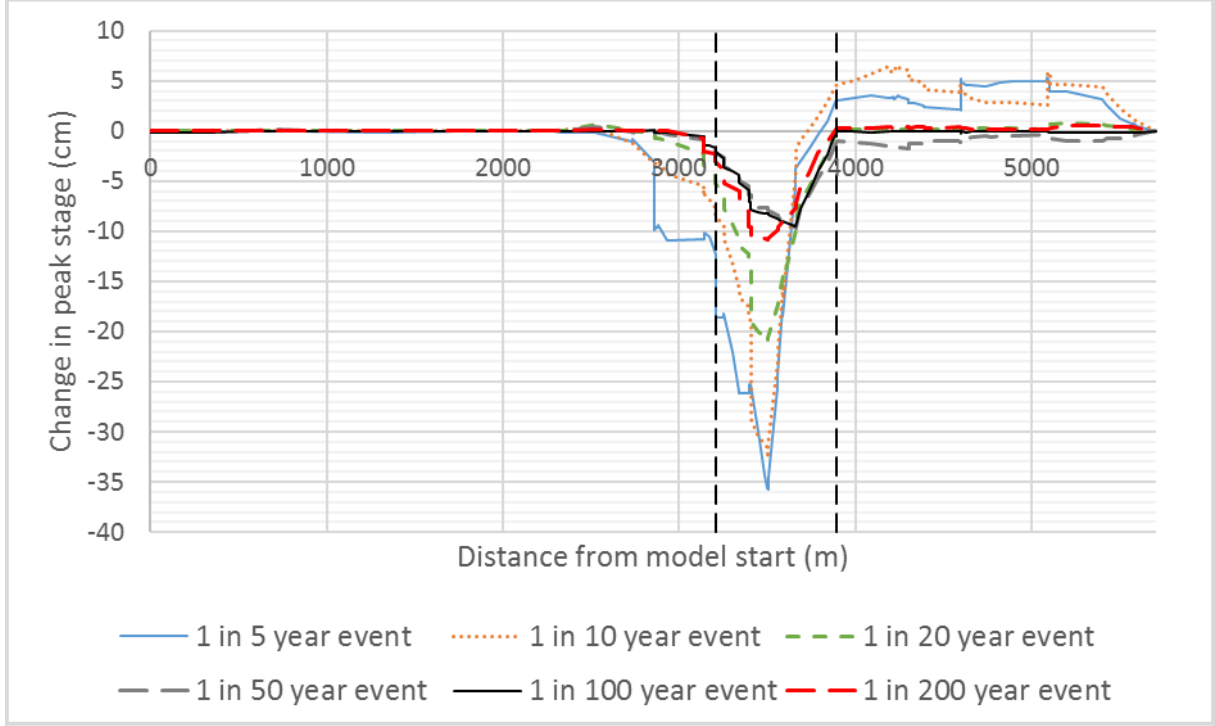


Figure 16: Change in peak stage in the main channel due to deepening the relief channel

- The option to use an automated Preissmann slot for river sections was turned on. This adds a small triangular slot at the base of all river units in the model to improve stability by avoiding very small water depths (near dry sections) when the discharge is small [4].

Having made these changes, all the simulations were successfully ran from 0 to 40 hours. To assess the improvement due to the deepening of the relief channel, these results were compared to those of the simplified EA model, with the baseflow at boundary unit SC4 also increased to $0.2 \text{ m}^3/\text{s}$ (the reference case), so that any changes in water levels could only be attributed to the change in relief channel geometry.

5.1.2.2 Results

Figure 16 shows the change in peak stage along the main channel, from the upstream limit near Coton, to the confluence with the Cam. The black, dashed, vertical lines represent the delimitation of the main study area. Figure 17 shows the change in peak stage along the relief channel. The results for the 1 in 40 and 1 in 150 year events have been omitted to improve the clarity of the graphs. In both figures, the x-axis plots the distance, in m, along the channel from the upstream limit of the model. Note that, as the relief channel is 347 m longer than the main channel between the upstream and downstream GW junctions, the downstream junction is at a “distance from model start” of 3893 m in Figure 16, but at a “distance from model start” of 4240 m in Figure 17. The change in peak stage at each node was defined as the peak stage in the deeper relief channel model

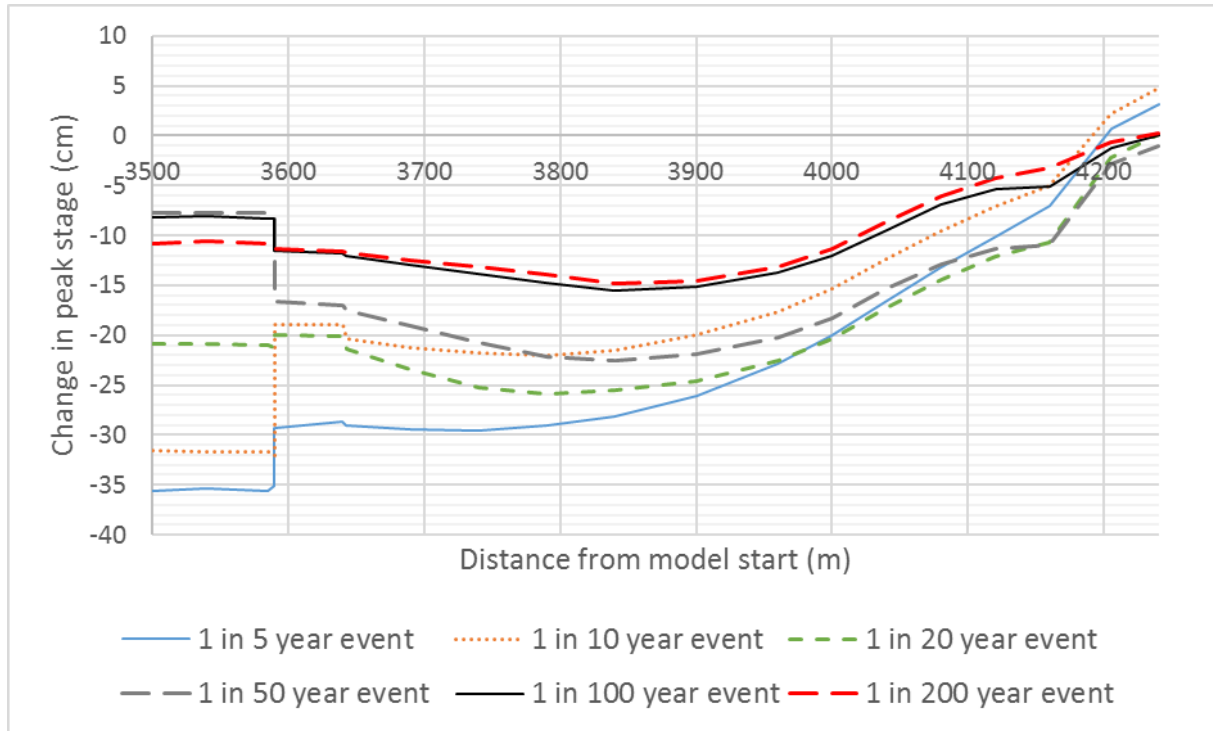


Figure 17: Change in peak stage in the relief channel due to deepening the relief channel

minus the peak stage in the reference model, such that a positive change represents an increase in peak stage, whilst a negative change represents a decrease, compared to the reference case.

Looking at Figure 16:

For all return periods tested, it can be seen that significant improvements are achieved in the main study area, with reduction in peak levels as high as 36 cm for a 1 in 5 year event. The reductions in flood levels are most significant near the upstream end of GW (“distance from model start”: 3500 m), which is especially beneficial as this is the area with the lowest level of protection at present. At the GW culvert inlet, one of the most critical locations, there is a stage reduction of around 10 cm for all large events.

The flows slightly upstream of GW are also reduced by up to 10 cm, providing a higher level of protection to Barton Road. Despite being upstream of the relief channel, these levels are affected by the change in relief channel geometry as the flow is subcritical, therefore the conditions depend on both the upstream and downstream flows [6]. The decrease in downstream levels due to the deepening of the relief channel causes a decrease in levels some distance upstream. The levels far upstream are unaffected, as can be seen from 0 to 2500 m. For higher return periods, this effect becomes less noticeable.

Downstream of the main study area, for the 1 in 5 and 1 in 10 year events, the peak water levels are slightly increased by the deepening of the relief channel. This is due to the peak discharge in the relief channel being larger, and travelling along the relief channel faster as a result of the change in channel geometry. The resulting hydrograph

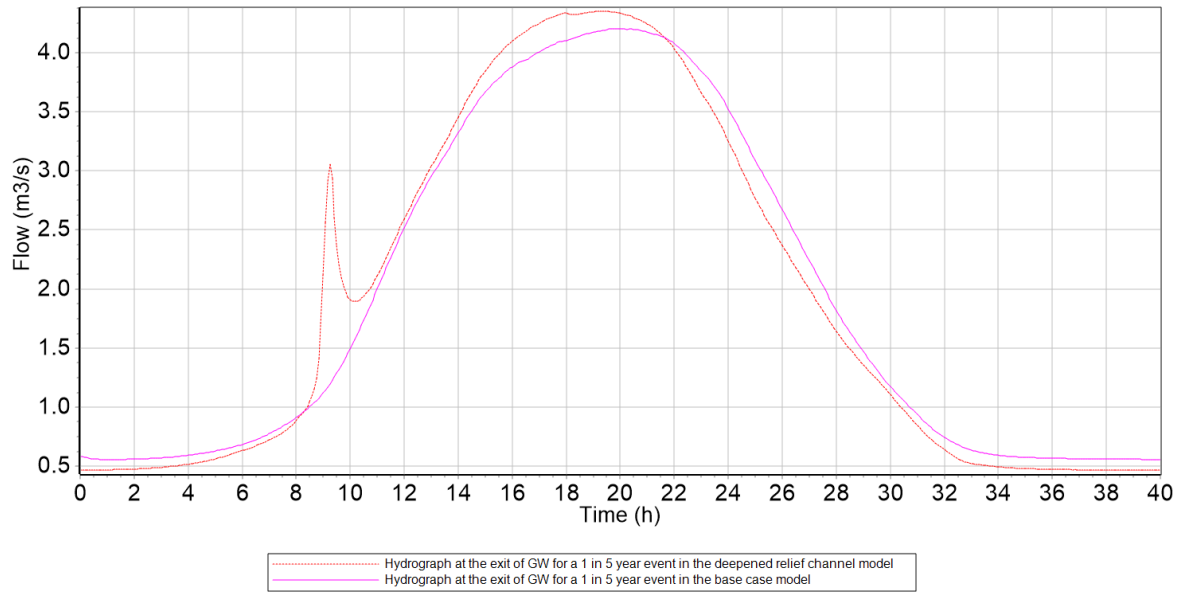


Figure 18: 1 in 5 year hydrograph at outflow of GW for the deeper relief channel model and the base case model

at the downstream end of GW is therefore shifted slightly earlier has a larger peak than in the reference case. This is illustrated in Figure 18 which shows this hydrograph for the 1 in 5 year simulation. It can be seen that the peak for the base case occurs at 20 hours, while the peak for the deeper relief channel case occurs at 19 hours and is slightly larger. The resulting increases in downstream peak levels are all less than 6.5 cm. While this should not be problematic for most downstream areas, which currently have a standard of protection of 1 in 20 years or better [1], some low-lying areas at Clare Hall could experience more frequent flooding as a result of this increase. Precise surveying of flood levels at those locations would be required to fully assess the impact of this increase in peak stage.

For larger events, this effect becomes negligible as the additional flow diverted to the relief channel is only a small fraction of the total flow in the system (<10%). The effect on the peak stage of downstream sections is then negligible (less than 1 cm).

Looking at Figure 17, it can be seen that the peak water levels in the relief channel are reduced by up to 36 cm. For large events, there is a 10 cm, or greater, reduction in water levels in the upstream end of the channel, which is most vulnerable to flooding. At 3590 m, there is a footbridge which forms a constriction in the channel, and the larger the flow, the larger the increase in stage across the bridge. For small return periods, this results in a step up in change in peak stage, as can be seen from the 1 in 5, 1 in 10 and 1 in 20 year curves. For larger return periods, this explanation is no longer valid as the water level is so high that the footbridge is overtopped, and part of the flow bypasses it through a spill unit. The reduction in stage on the upstream side of the bridge results in a reduction of

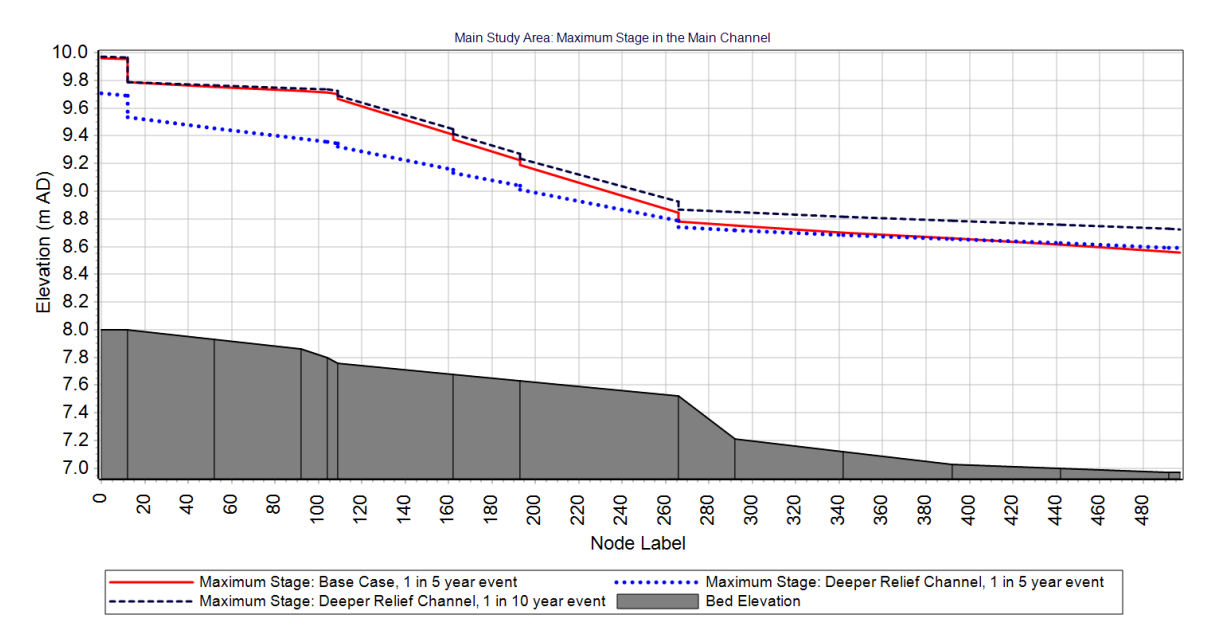


Figure 19: Peak water levels in the main channel between the upstream and downstream ends of the main study area

flow through the spill unit, hence the reduction in peak stage on the downstream side is even larger, as can be seen from the 1 in 50 and 1 in 100 year curves. For a 1 in 200 year event, this effect is barely noticeable as the bridge becomes fully submerged.

In order to estimate the improvement in the standard of protection to the GW area, the new peak levels were compared to the peak levels for a 1 in 5 year event in the reference case, which are considered to be safe even for the most vulnerable points near the GW culvert inlet. Figures 19 and 20 show the maximum water levels in the main study area, along the main channel and the relief channel respectively. The red and blue curves show the peak levels for a 1 in 5 year event for the reference case and the deeper relief channel, respectively. It can be seen that there is a big improvement. The black curve shows the peak levels for a 1 in 10 year event for the deeper relief channel. It can be seen that these have largely been lowered to the same level as the peaks for the 1 in 5 year reference case on the upstream side of the main study area. We can therefore conclude that deepening the relief channel improves the standard of protection to the most vulnerable points from a 1 in 5 year standard to a 1 in 10 year standard.

5.1.2.3 Practical Issues

There are several practical issues which would need to be considered in a project appraisal report if this mitigation scheme were to be carried forward:

- Increasing the depth of the channel from 1.7 m to up to 2.3 m close to a residential area may pose a significant health and safety risk. This may require the installation of lighting, so that the ditch can be seen when it is dark, and the erection of a barrier

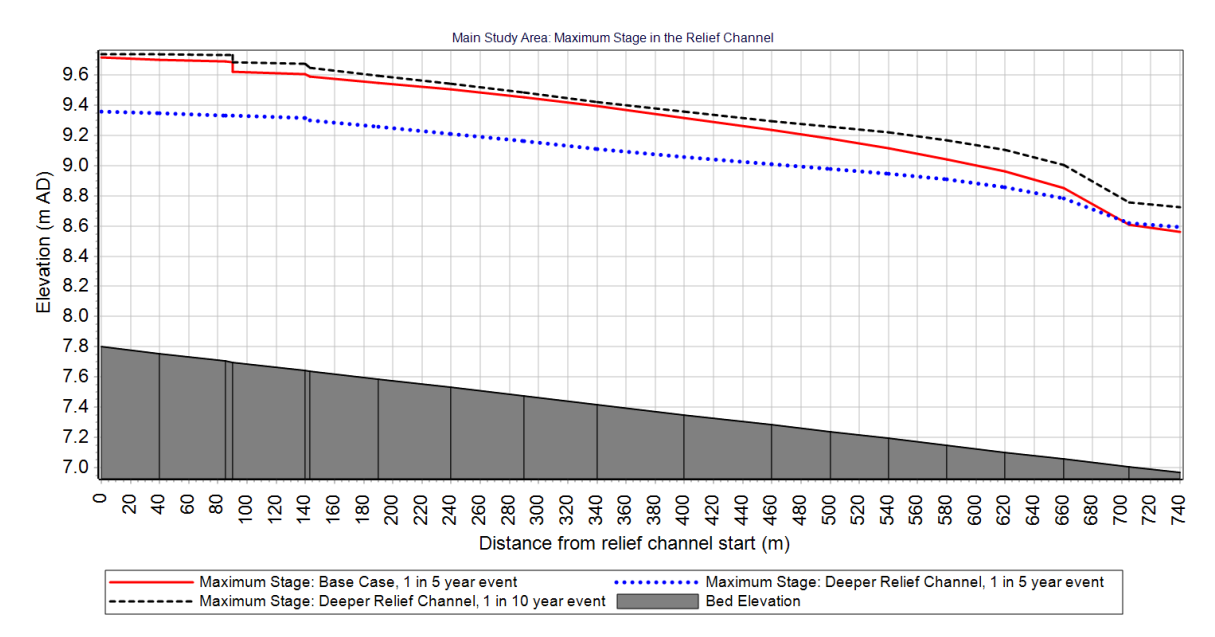


Figure 20: Peak water levels in the relief channel

to prevent people from falling in. The severity of this hazard should be assessed in the risk assessment for the scheme.

- When the water in the channel is at baseflow and the discharge and velocity are very low, the water in the relief channel may be stagnant, which can be unpleasant and pose a health risk as it is an ideal breeding environment for many bacteria.
- As the downstream flood risk in small events is increased by the deepening of the relief channel, some localised protection measures may be needed in vulnerable, low lying areas, such as Clare Hall, to ensure that their standard of protection is not reduced as a result of the additional protection to GW.
- Information about existing services would need to be gathered to ensure that the deepening of the relief channel would not interfere with any existing pipework. Re-routing existing infrastructure would significantly increase the cost of the scheme.
- This schemes requires approximately 400 m³ of earth to be excavated, and the demolition and reconstruction of the lining of the concrete sections. Such heavy construction works in a residential area would be very disruptive to residents.

5.2 Flood Storage Area

In 2002, when Halcrow produced the Bin Brook Pre-Feasibility report, they explored the option of providing an upstream FSA near the M11 motorway to reduce the downstream levels. The EA favoured this option, and it was carried through to project appraisal phase in 2007. The pre-feasibility report had estimated the storage volume requirement

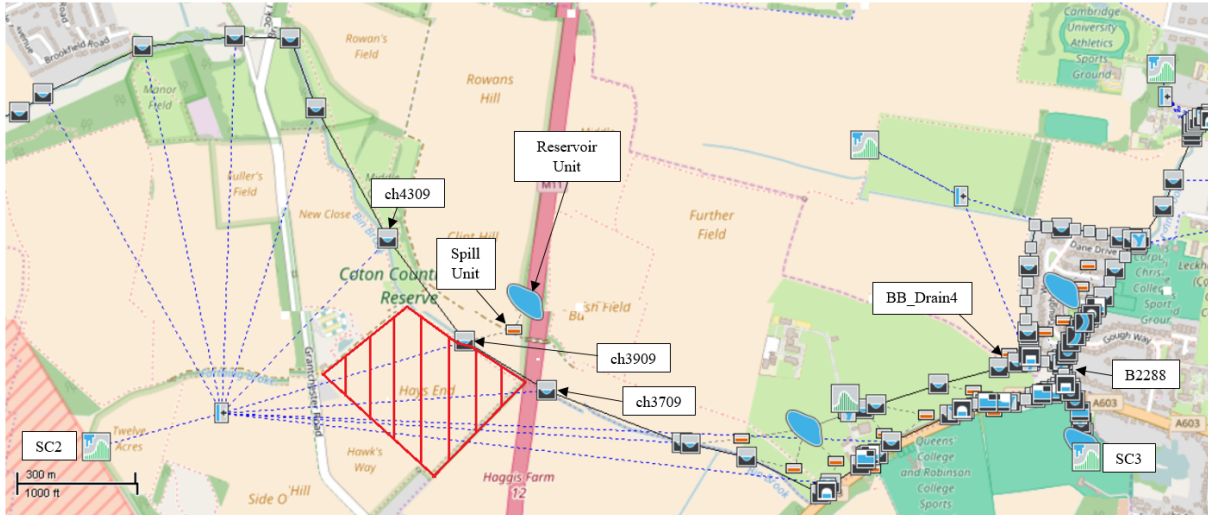


Figure 21: Proposed location of Bin Brook flood storage area, shown in red, and schematic representation of FSA model in Flood Modeller (*Map: www.openstreetmap.org/*)

to be of the order of $150,000 \text{ m}^3$ to provide a 50 year standard of protection. This value was revised to $215,000 \text{ m}^3$ in the project appraisal report, shortly before the scheme was rejected on the basis that the benefits offered were not sufficiently high compared to the “maintain existing” option [7].

As the FSA option was favoured by the EA, and neither of the Halcrow reports actually tested it in the hydraulic model, it was decided that this study would carry out a more thorough investigation of the benefits available from such a scheme.

5.2.1 Test Design

When the reservoir option was being considered by the EA, the proposed location was a plot of land just upstream of the M11 motorway. The land is owned by the Cambridgeshire Preservation Society, which offered it free of charge due to the ecological benefits which would arise from the creation of a wetland area. Since this is likely to be the only area available for the reservoir, it was also used as the location for the reservoir in this study. The area of land available is approximately $112,000 \text{ m}^2$, and the length of the edge of the plot adjacent to the brook is approximately 350 m. The location can be seen on the map in Figure 21. The figure also shows the schematic representation of the model produced for testing the FSA, with key nodes labelled for clarity of the subsequent discussion.

There are several types of reservoir used for flood storage. The main two are online reservoirs and offline reservoirs. For the online type, the baseflow passes through the FSA, whereas for the offline type, the baseflow passes outside of the FSA [8]. For this scenario, the offline type was the natural choice as the land available for the reservoir is adjacent to the channel, meaning that the brook would need to be re-routed to pass through the FSA to form an online reservoir. A schematic representation of an offline reservoir can be

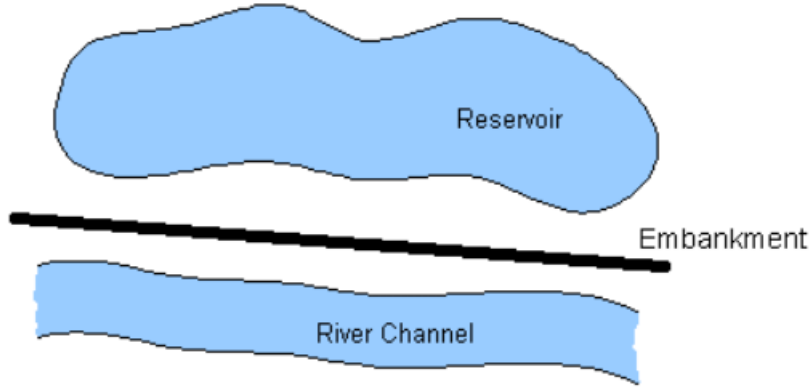


Figure 22: Overhead schematic of an offline reservoir [4]

seen in Figure 22.

Next, the storage requirement had to be determined. As mentioned above, Halcrow gave an initial estimate for a 50 year standard of protection of $150,000 \text{ m}^3$, but later revised this up to $215,000 \text{ m}^3$. It was hypothesised that this error arose because Halcrow must have based the initial volume on the hydrograph at river unit ch3909, which is adjacent to the reservoir. However, between this location and GW, the inflow boundaries SC2 and SC3 further contribute to the total flow, resulting in the total volume reaching GW being higher than at ch3909. To provide a 50 year standard of protection to the properties at GW, the reservoir volume must be based on the hydrographs at the sections feeding the GW main and relief channels, which are B2288 and BB_Drain4, as shown in Figure 21. To verify this theory, and to size the reservoir for this study, the required volume was calculated based on the ch3909 hydrograph and on the B2288 and BB_Drain4 hydrographs. To obtain the storage volume required to provide an x year standard of protection, the volume under the 1 in 5 year hydrograph was subtracted from that under the 1 in x year hydrograph, as we know that a 1 in 5 year event does not cause damage to any areas in the network, so only flows in excess of this must be diverted to the reservoir. This is illustrated by Figure 23, where the 1 in 5 year and 1 in 50 year hydrographs at river unit ch3909 have been plotted, and the estimated reservoir volume is represented by the blue area. The trapezium rule was used to find the volumes underneath the flow-time curves. A summary of the estimated required volumes can be seen in Table 4.

As can be seen, the volumes calculated closely match those estimated by Halcrow, further reinforcing the above theory about the cause of the underestimate in Halcrow's pre-feasibility report. To assess the flood mitigating effects different size FSAs, it was decided that reservoir volumes of $150,000$, $210,000$ and $285,000 \text{ m}^3$ would be tested. Considering the surface area of land available, these volumes correspond to reservoir depths of 1.3 m, 1.9 m and 2.5 m, respectively.

The reservoir had to be incorporated into the Bin Brook model to run the tests. Starting from the simplified EA model, a reservoir unit was inserted and connected between

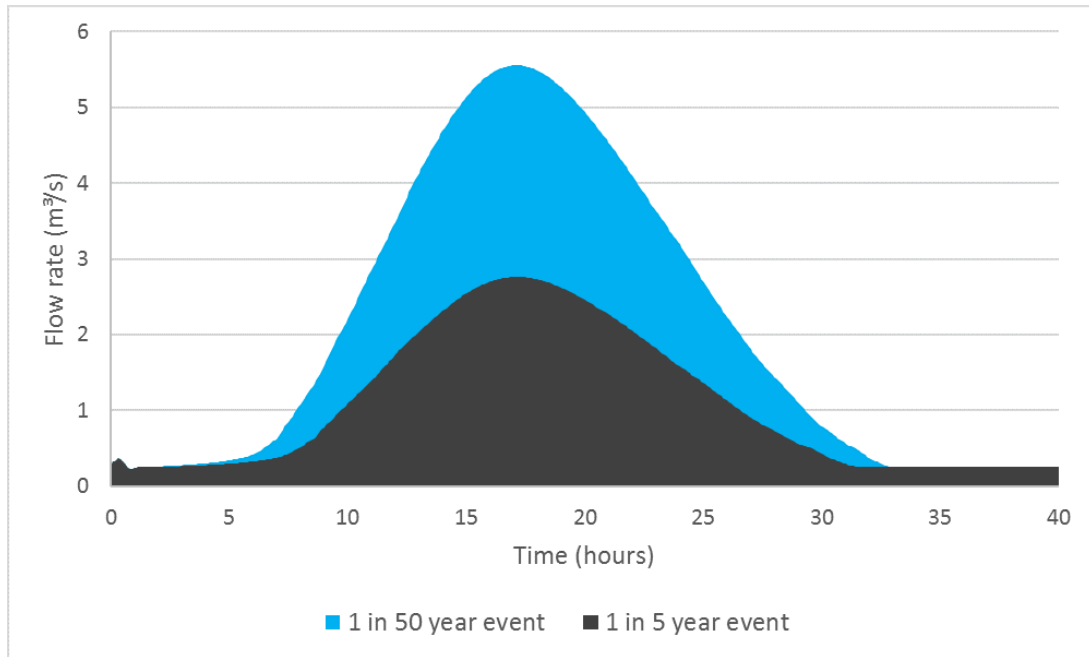


Figure 23: Hydrographs at river section ch3909 for 5 year and 50 year storm return periods, with the calculated storage volume represented by the blue area

	Calculated Volume (m ³)		Halcrow Estimate (m ³)	
Design for a 1 in 50 year event	Based on ch3909	145,000	Pre-feasibility report	150,000
	Based on B2288 & BB Drain4	210,000	Project appraisal report	215,000
Design for a 1 in 100 year event	Based on ch3909	190,000	Not Available	
	Based on B2288 & BB Drain4	285,000	Not Available	

Table 4: Flood storage requirements based on hydrograph volume calculations and Halcrow Group Limited estimates

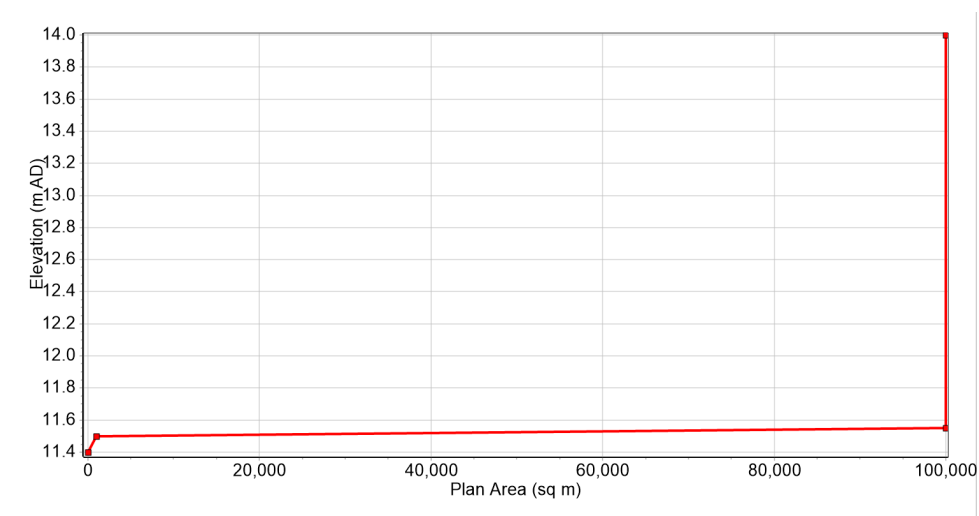


Figure 24: Reservoir geometry for the 150,000 m³ FSA

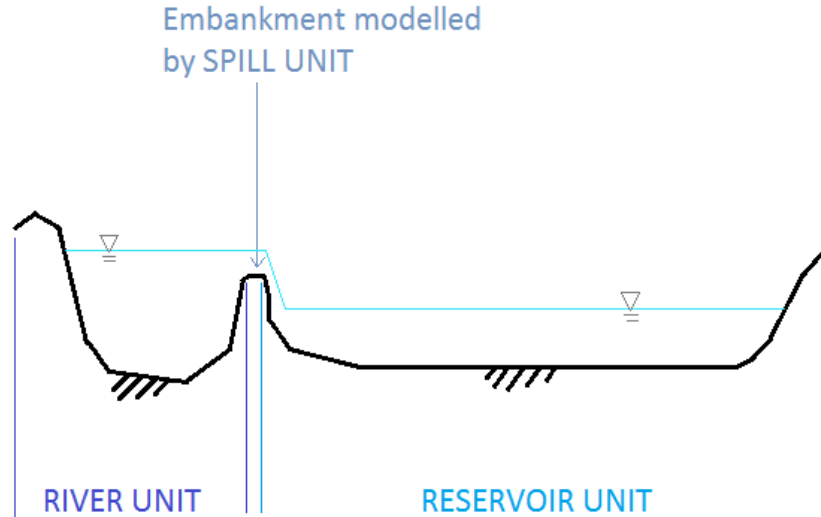


Figure 25: Cross-sectional view of reservoir modelling

the river sections ch3909 and ch3709, using a spill unit, as can be seen in Figure 21. The reservoir unit allows the user to input the desired profile of elevation against surface area. As an example, the profile for the 150,000 m³ FSA is included (see Figure 24). Note that a small slot has been added in the bottom of the reservoir, so that the initial water depth could be set to 0.05 m to avoid the instability issues that tend to arise with dry sections. The spill unit is used to define the embankment elevation profile between the two river units, which are 232 m apart. A schematic representation of this arrangement can be seen in Figure 25 to aid visualisation.

Consideration was given to the height of the embankment, the required flowrate over the embankment and the elevation of the reservoir:

- The height of the embankment was set such that the volume of water spilling over it in the flood event on which the storage volume was based would be equal to the volume of the reservoir.
- For the reservoir to be efficient, the flowrate into the reservoir must be large enough for the reservoir to fill within the time taken for the flow in the channel to peak and recede back to baseflow. The rate of flow from the river section to the reservoir is controlled by the spill unit. For free flow, the discharge into the reservoir is given by Eq. (8). For flood embankments, weir coefficients, C_d , in the range 0.8 to 1.2 are recommended [4]. The lowest value, 0.8, was used in the model as a conservative estimate, and was found to give sufficiently high flows in all cases.
- The elevation of the bottom of the reservoir was chosen such that the design volume would be enclosed between the bottom of the reservoir and the embankment level. For larger events, the excess will cause the embankment to become submerged as the water level in the reservoir and river channel continues to rise.

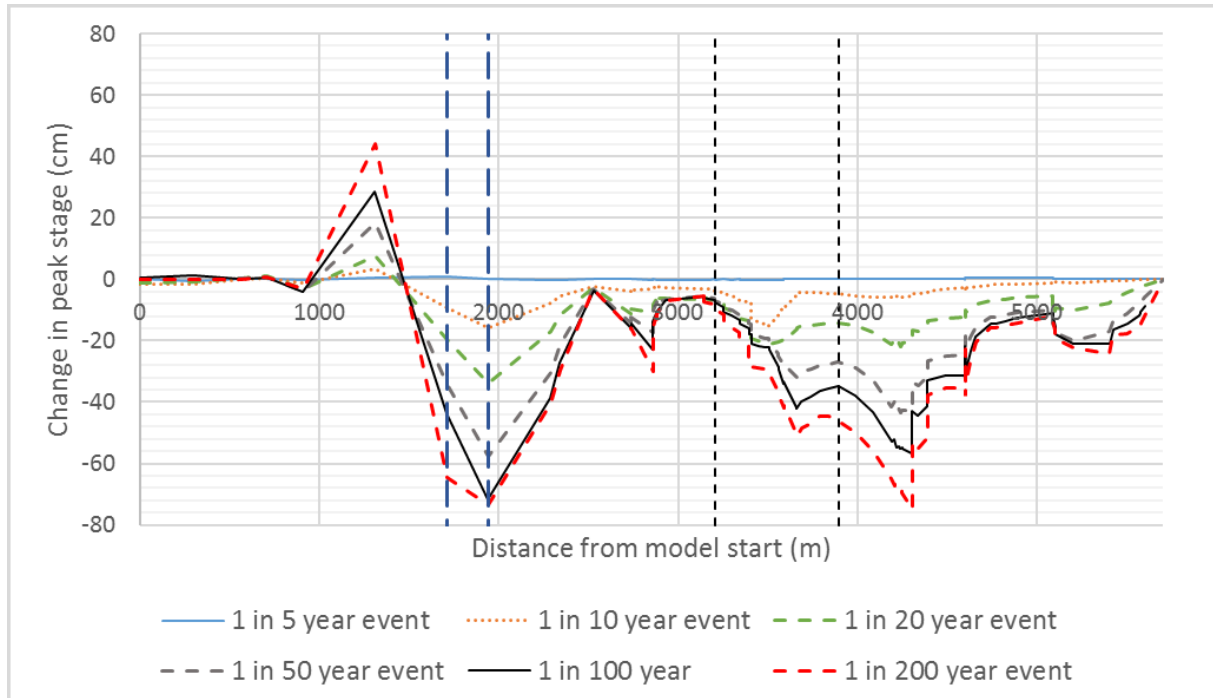


Figure 26: Change in peak stage in the main channel due to 150,000m³ FSA

As the only change made to the model was the addition of the reservoir, the results from the unaltered simplified EA model were used as the base case for comparing with the FSA results. The initial condition for the stage in the reservoir at time zero was entered manually, and set such the reservoir would be initially empty.

Unsteady simulations of duration 40 hours were ran for a 1 in 5, 1 in 10, 1 in 20, 1 in 40, 1 in 50, 1 in 100, 1 in 150 and 1 in 200 year event. The model encountered no stability issues for these runs, except with the 285,000 m³ FSA, which will be discussed in Section 5.2.2.

5.2.2 Results

After all the simulations for the base model and the FSA model were ran, the peak stages at all nodes were extracted.

Figures 26 and 28 show the change in peak stage along the main channel from the upstream limit near Coton, to the confluence with the Cam, for the 150,000 m³ FSA and the 210,000 m³ FSA, respectively. The black, dashed, vertical lines represent the delimitation of the main study area, and the blue, large-dashed, vertical lines represent the nodes between which the reservoir is located. Figures 27 and 29 show the change in peak stage along the relief channel for the 150,000 m³ FSA and the 210,000 m³ FSA, respectively. Change in peak stage was defined as described in Section 5.1.2.2. Note the difference in scale of the y-axis.

Looking at the general shape of the curves, it can be seen that the inclusion of an FSA reduces the maximum water levels at all sections downstream of it. For the main channel,

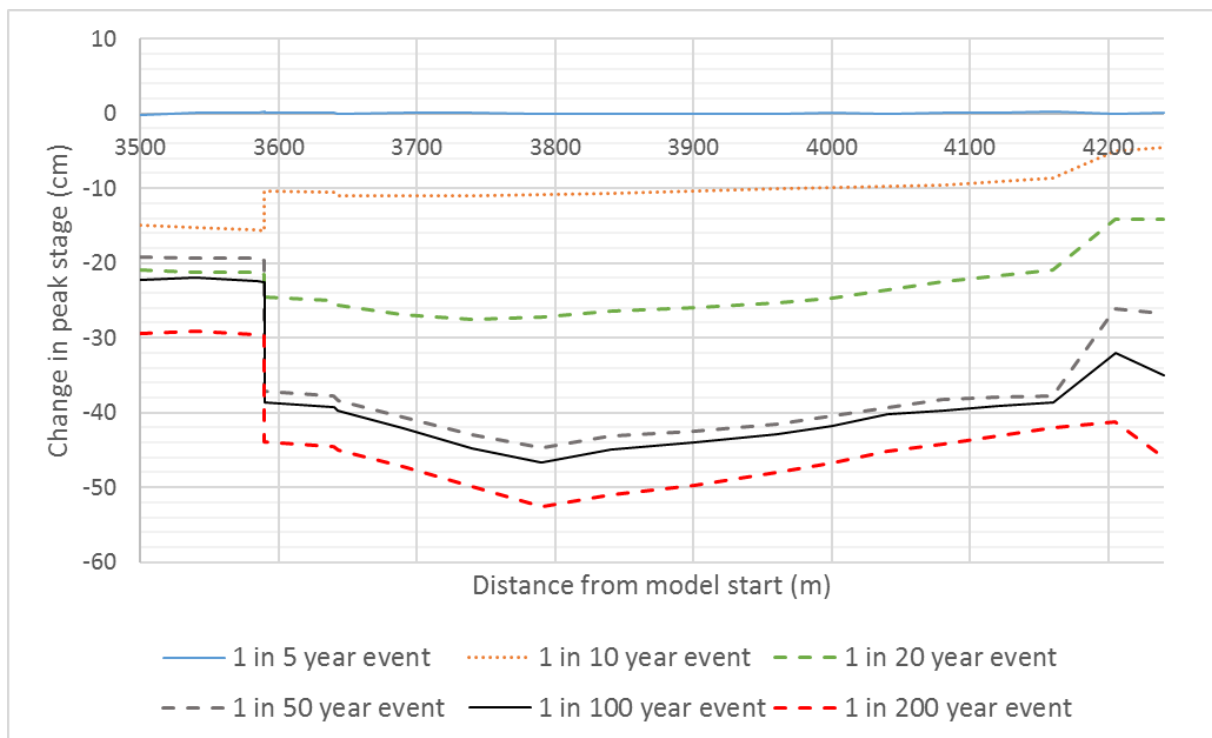


Figure 27: Change in peak stage in the relief channel due to 150,000m³ FSA

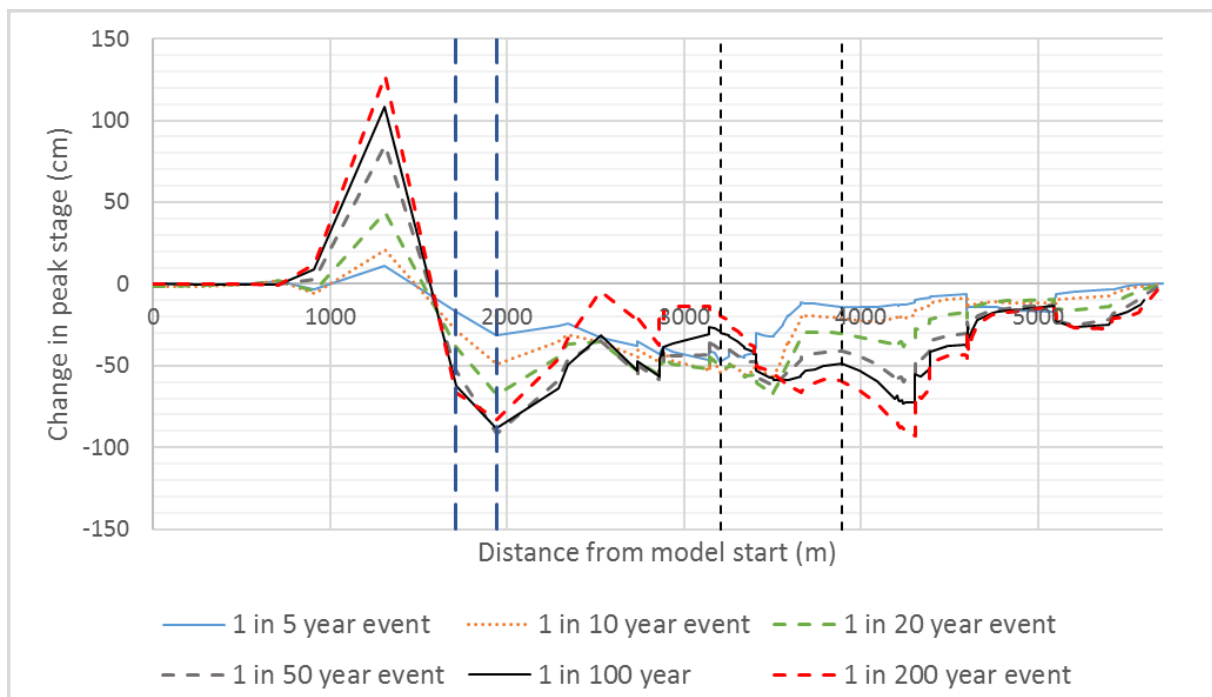


Figure 28: Change in peak stage in the main channel due to 210,000m³ FSA

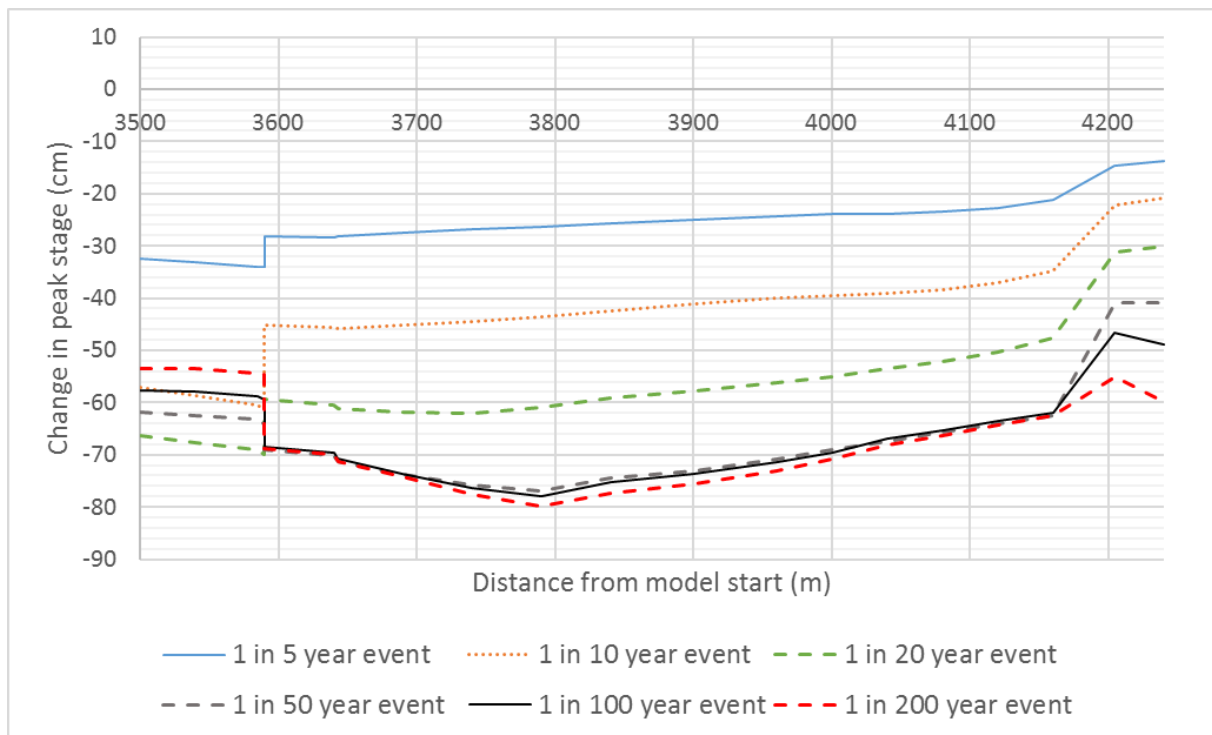
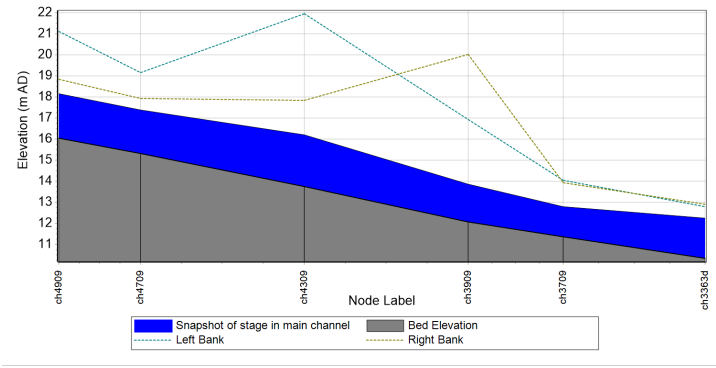


Figure 29: Change in peak stage in the relief channel due to 210,000m³ FSA

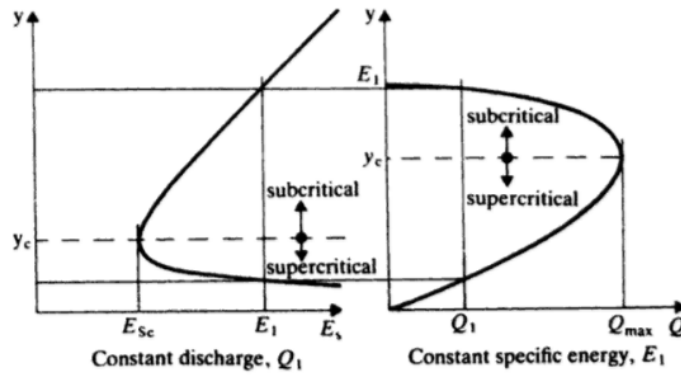
the greatest reduction in peak stage is achieved at section ch3709 (1942 m), which is the river unit at the downstream end of the embankment between the channel and the reservoir. At 2527 m the effect of the reservoir on the peak level is relatively small. This is especially noticeable on the graph for the smaller reservoir, where the curves for all storm return periods are pinched at around -3 cm. This is due to the fact that, at this point, which is on the Barton Road reach, the left bank is very low, and the water starts spilling out onto the paddock at relatively small events. This means that the water depth is relatively fixed by the elevation of the bank between the river section to the floodplain. It cannot be much improved upon unless the flow is so reduced that the stage does not reach the spill level at all.

Further downstream, there are several points with a sudden jump in change in peak stage. These are due to the presence of hydraulic structures such as the Barton Road culvert (2859 m), various bridges (4303 m, 4388 m), the Robinson sluice gate (4598 m) and the Queen's Road culvert (5102 m). Finally, at the downstream end of the main channel, the change in peak levels tends to zero as the downstream level is fixed by the boundary condition. A key point to note is that, unlike for the deepened relief channel, the reduction in peak water levels increases with the severity of the storm event. For a 1 in 50 year event, peak levels in the main study area are reduced by up to 28 cm and 69 cm by the 150,000 m³ and 210,000 m³ FSAs, respectively. For a 1 in 100 year event this is increased to 45 cm and 78 cm.

In the relief channel, the peak water levels are also significantly reduced. The change



(a) Example of flow transition from subcritical to supercritical between ch4309 and ch3709 for a 200 year event in the 150,000 ³ model



(b) Variation of specific energy, E , and flowrate, Q , with water depth, y [6]

Figure 30

in peak stage at the downstream end of the relief channel matches that at the downstream end of the main study area for the main channel, as the two branches re-join, thus the stages must be equal. The jump at 3590 m is due to the presence of a footbridge, as discussed in Section 5.1.2.2.

Directly upstream of the FSA, at river section ch4309 (1310 m), there is an increase in maximum stage. For large events, especially for the bigger reservoir, this is very large, reaching as high as 130 cm for a 1 in 200 year event. This is due to the flow regime transitioning from subcritical at ch4309 to supercritical at ch3709. Specific energy, E , is a measure of the energy of a flow with the channel bed as datum. It is a function of the water depth and flowrate, which gives a specific energy curve, as illustrated in Figure 30b. For a certain depth, known as the critical depth, y_c , there is a minimum in the magnitude of specific energy. It can be shown that flow is subcritical when the depth is greater than y_c , and supercritical when it is smaller than this value [6].

The stage from ch3909 to ch3709 is constrained to the height of the embankment (until the reservoir is full, at least), therefore the water depth is reduced and passes through the critical depth at some point between these sections. This transition in flow regime

results in a much larger water depth at the upstream section, as can be seen in Figure 30a. Further downstream from the reservoir, the flow regime returns to subcritical.

When attempts were made to run the model with the 285,000 m³ reservoir, which was designed to provide a 1 in 100 year standard of protection, it was found that the increase in depth at ch4309 was unacceptably large. This is because, in order to capture this volume of water from the upstream part of the network, the embankment height had to be set very low, so that essentially any flow above baseflow could be captured. Not only is this quite impractical, it also caused the section downstream of the reservoir to dry out in large events, as the flowrate into the reservoir was large compared to that in the channel, leading the simulation to crash. This proposal was therefore rejected.

To check if the two reservoirs were able to meet their design return periods, the maximum stage profiles of the 1 in 50 year event were plotted along with the maximum stage profile for a 1 in 5 year event in the base case. This was to check if the addition of the reservoir successfully reduced the levels to within the safe limit. Figure 31 shows the maximum water levels in the main channel, for the main study area only, for clarity. For completeness, the full profile is included in Appendix C. Figure 32 shows the maximum water levels in the relief channel.

Looking at these curves, we note that the smaller reservoir does not have sufficient capacity to reduce the peak levels in the GW area to the 1 in 5 year base case peak levels. The larger reservoir matches the 1 in 5 year base levels very well, indicating that the reservoir volume estimation method based on the hydrographs at B2288 and BB_Drain4 was highly suitable and accurate.

In addition to increasing the standard of protection to the most vulnerable areas of the Bin Brook network from 1 in 5 years to 1 in 50 years, the larger reservoir decreases the peak levels of a 1 in 100 year event to those of a between 1 in 5 and 1 in 10 year event, while the smaller reservoir decreases them approximately to those of a 1 in 20 year event in the current network, meaning that the extent of flooding would be much more limited than at present for larger events.

5.2.3 Practical Issues

If the provision of an FSA were to be carried forward as a flood defence scheme for the Bin Brook, there are several issues which would require further consideration:

- As well as constructing the reservoir and embankment, a pumping system would also need to be installed to return the water from the reservoir to the channel after the storm event has passed, adding additional costs.
- The size of the reservoir will affect the cost, with larger volumes generally resulting in higher costs [8]. The ratio of cut to fill also has an impact as it usually determines

the quantity of soil which must be carried to or from site.

- Although the banks at section ch4309 are high, and the increase in peak level at this section did not cause flooding for any of the return periods tested, it may be necessary to provide additional local flood defences to this area. This should be considered especially carefully due to the proximity of the M11.
- If possible, using the reservoir construction as an opportunity to create a wetland could provide many ecological benefits.

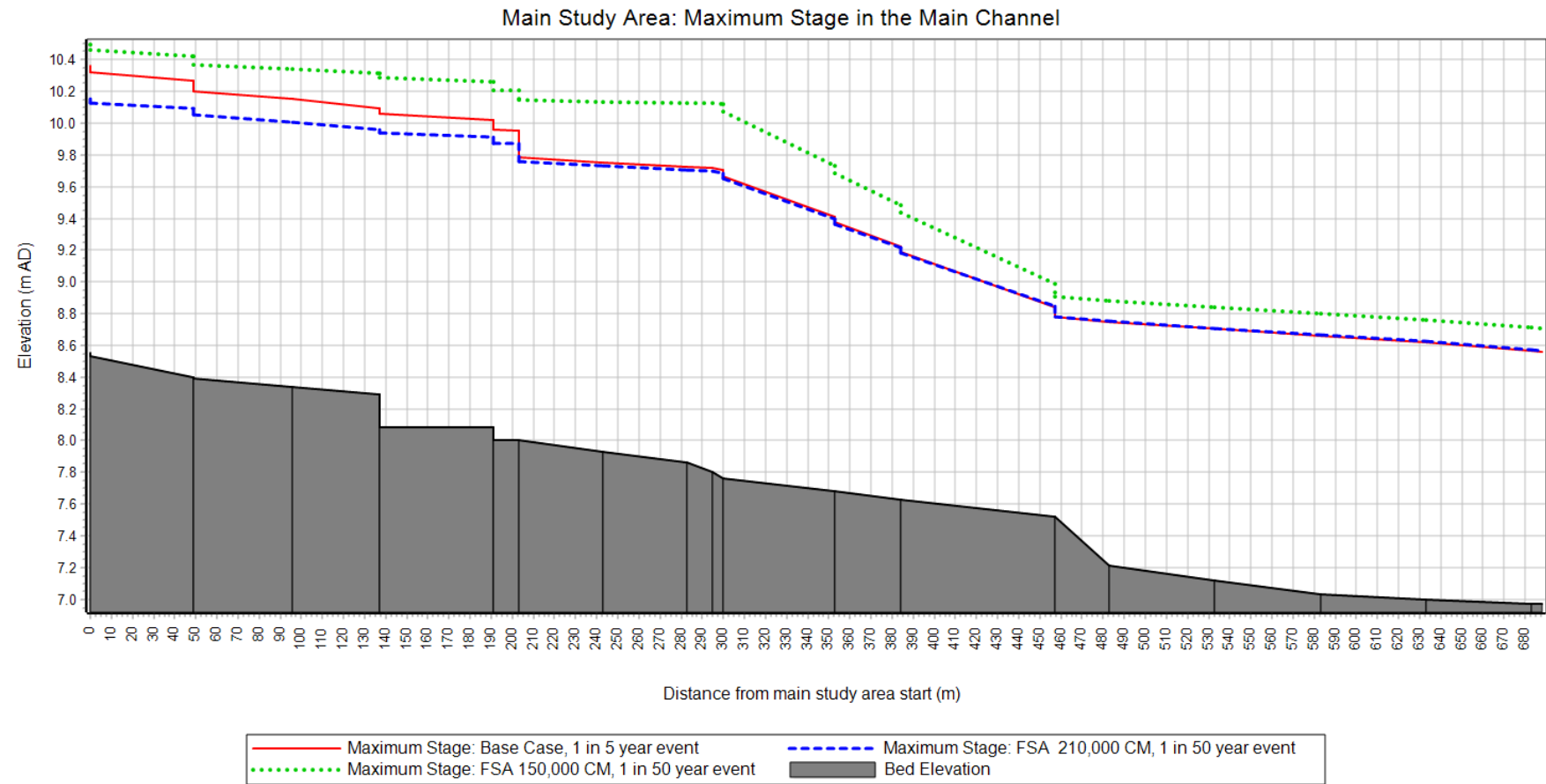


Figure 31: Peak water levels in the main channel from the upstream to the downstream end of the main study area, Blue: 1 in 50 year peaks, FSA 210,000 m³, Green: 1 in 50 year peaks, FSA 150,000 m³, Red: 1 in 5 year peaks, Base case

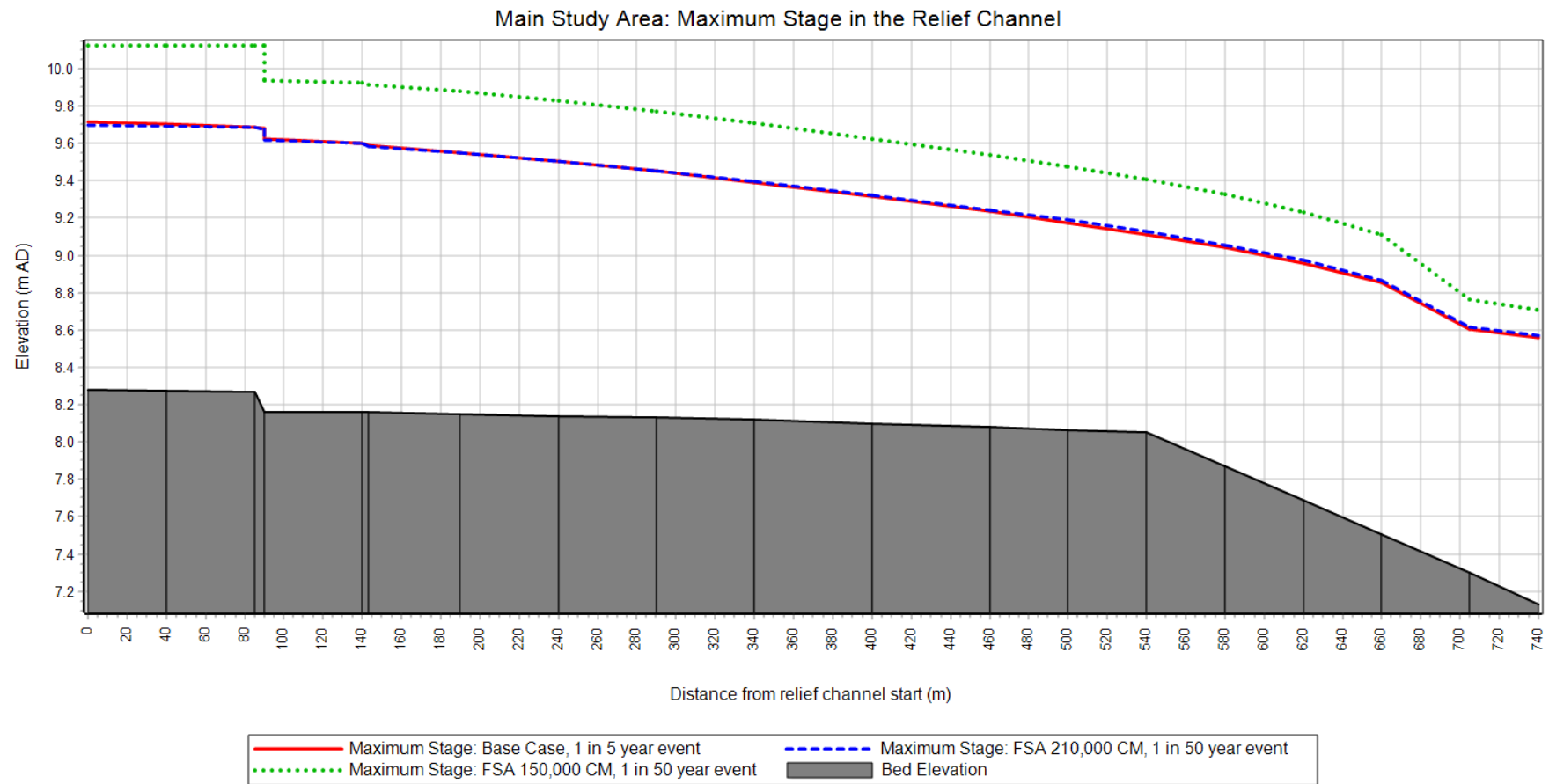


Figure 32: Peak water levels in the relief channel, Blue: 1 in 50 year peaks, FSA 210,000 m³, Green: 1 in 50 year peaks, FSA 150,000 m³, Red: 1 in 5 year peaks, Base case

6 Future Work

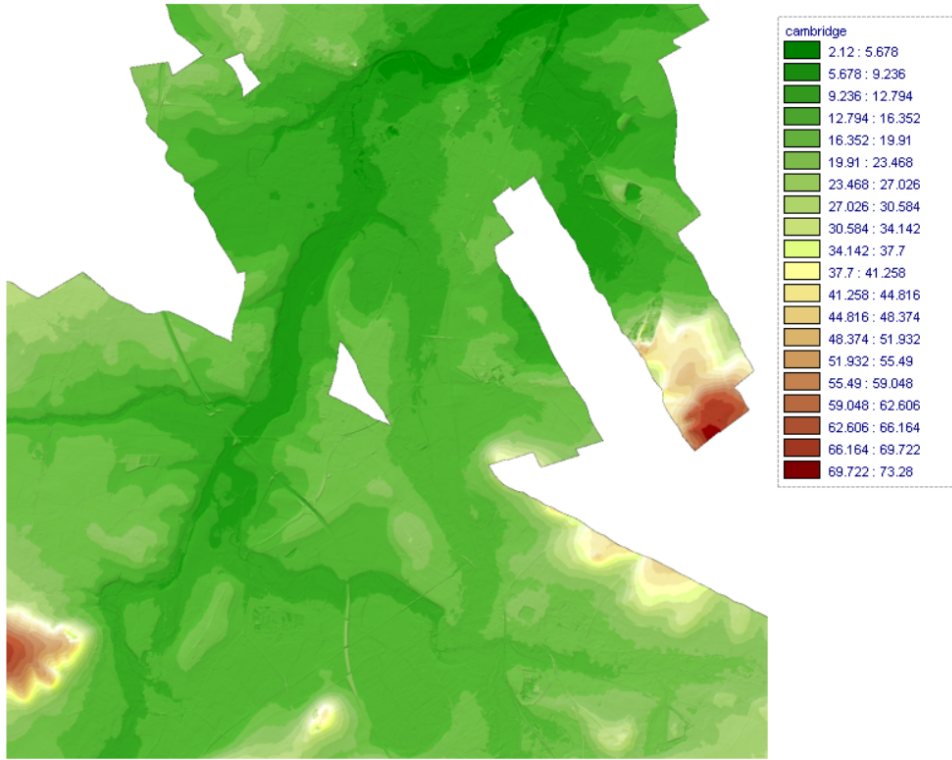


Figure 33: Ground elevations (mAOD) in the Bin Brook area, Light Detection and Ranging (LIDAR) data courtesy of Survey Open Data: <http://environment.data.gov.uk/ds/survey/index.jsp#/survey?grid=TL45>

This study has made use of Flood Modeller’s 1D modelling tools to simulate flood levels and flowrates in the river channel. This is a good way of checking if flood thresholds at specific locations are exceeded and of generating water level profiles. To see how flood water will spread across the surrounding land once it escapes the channel, a 1D-2D linked model can be used to generate a flood map. By providing ground elevation data, such as that shown in Figure 33, this tool can be used to simulate where the water will spread once it spills out of the 1D river cross-sections. This would be particularly useful to compare the flood extent for the different mitigation schemes considered in this project. It would also be a very useful tool for pitching a scheme to the EA.

7 Conclusions

Over the course of the project, all the tasks outlined in Section 1.2 were successfully carried out, allowing the following conclusions to be made:

- Flood Modeller software is a very useful tool for designing flood defences, which produces realistic flow profiles for complex channel networks. The FEH boundary

units make testing flood defences against different return periods very straightforward. Having said this, it is important to bear in mind that the accuracy of the software can only be as good as the data it is provided with, and it is very difficult to confirm the validity of all the inputs required by the model nodes.

- It was found that the simplified model derived from the EA model had a good level of accuracy in the main study area, validating its use for the analysis of the flood mitigation schemes.
- Increasing the capacity of the relief channel would somewhat increase the protection to the main study area, reducing the peak levels of the current network by around 10 cm for large events (50 to 200 years). The standard of protection to the most vulnerable areas would be improved from 1 in 5 years to 1 in 10 years.
- Providing an upstream FSA could significantly improve the standard of protection in the main study area. It was found that constructing a 210,000 m³ FSA could reduce the peak levels of a 1 in 50 year event to those of a 1 in 5 year event, which would avoid flooding at any point in the network in a 50 year event. The extent of flooding in larger events would also be significantly reduced. Constructing an even larger FSA was found to be not feasible due to the increase in maximum peak stage at the section upstream of the reservoir being too large.

Having analysed the effects of increasing the channel capacity and providing a FSA, this study recommends that the latter is the most appropriate and beneficial flood mitigation scheme for the Bin Brook flood. It is recommended that a more detailed study be carried out in the full Bin Brook model for a range of reservoir volumes. Although the smaller FSA was shown to provide smaller improvements than the large one, the lower cost of a smaller volume may make it the most optimal solution for a government organisation such as the EA, which has to carefully consider and justify its spendings.

8 Acknowledgements

I would like to thank Keith Glover, for sharing all his knowledge of the Bin Brook flood with me; Dr Xiaolin Wang, for sending me the EA model results, and for providing her Flood Modeller expertise throughout the project; and the members of the Gough Way Residents Association, for sharing their observations and suggestions and inviting me to their annual flood meeting.

Finally, I would like to thank my supervisor, Dr Dongfang Liang, for his support and encouragement throughout this challenging project, and for his ability to advise me while allowing me the freedom to steer the project in the direction that interested me most.

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A Risk Assessment Retrospective

In Michaelmas, a risk assessment was carried out, detailing the expected hazards for the project. Lone working in proximity to a river channel in an isolated area, potentially with heavy surveying equipment, was identified as the greatest risk. However, throughout the year, I was always accompanied on my trip to the brook, and no surveying equipment was actually required, so the risk was very minimal. However, it is always best to plan for all likely outcomes in a risk assessment, so I would not remove these hazards from the risk assessment entirely, if I were to write it again. It is better to be a bit too cautious than to risk an accident.

B Photographs of Key Features of the Bin Brook Network



(a) Diversion channel, concrete section



(b) Diversion channel, earth section



(c) Gough Way culvert inlet



(d) Robinson pond sluice gate

Figure 34

C Flood Storage Area - Impact On Peak Levels in Main Channel

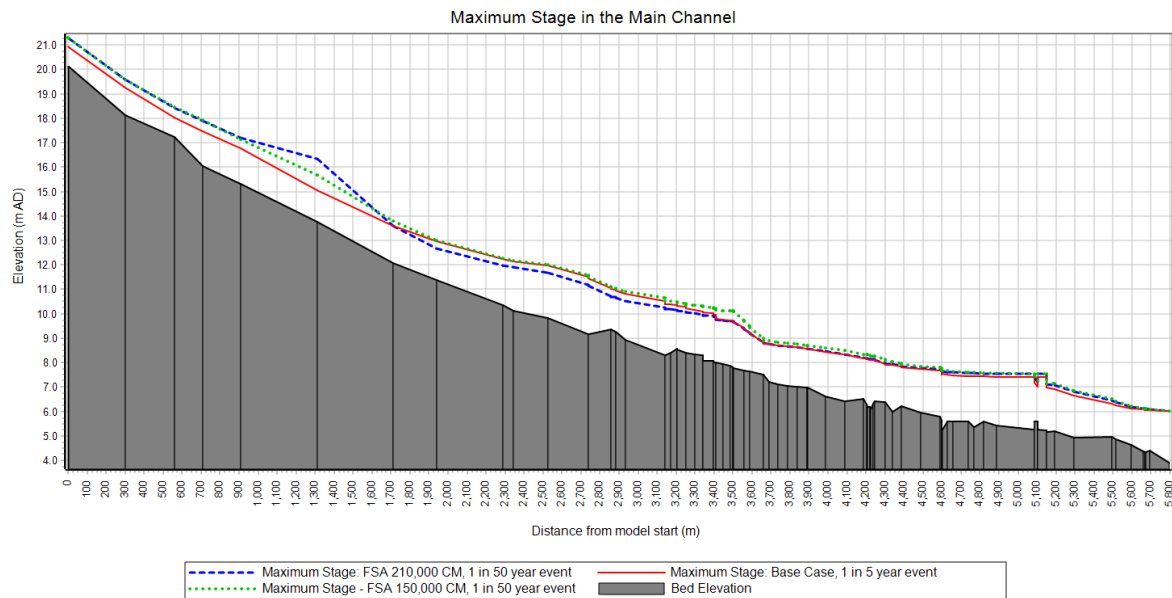


Figure 35: Peak water levels in the main channel from Coton to the confluence with the Cam, Blue: 1 in 50 year peaks, FSA 210,000 m³, Green: 1 in 50 year peaks, FSA 150,000 m³, Red: 1 in 5 year peaks, Base case