Appendix E

Hydraulic Modelling Report

P3Eco (Bicester) Ltd and A2Dominion Group

NW Bicester Eco Development

Exemplar Site - Hydraulic Modelling Report

CONTENTS

Appendices

1 INTRODUCTION

1.1 Project Appointment

Hyder Consulting (UK) Limited (HCL) were appointed by A2Dominion Group and P3Eco (Bicester) Ltd to undertake a Flood Risk Assessment (FRA) for the proposed NW Bicester eco development. This FRA required hydraulic modelling of the River Bure and tributaries to determine the baseline flood risk on the property, and model any necessary mitigation works.

1.2 Background to the Study

P3Eco (Bicester) Ltd and A2Dominion Group intend to construct a residential development to the northwest of Bicester. The NW Bicester eco development will comprise approximately 5,000 homes, secondary school, a number of primary schools, retail and commercial space along with health care and other community facilities. Approximately 40% of the overall site will be green open space, including playing fields, semi private and public open space. The first phase of the eco development will be an Exemplar for future development, which will comprise residential homes, land for a primary school, a nursery, and areas of commercial and retail property.

The River Bure and two of its tributaries run through the centre of the proposed development. This area is upstream of the extent of the Environment Agency's River Bure hydraulic model, and therefore detailed flood extents have not been produced for the site. The tributaries of the River Bure in this area have not been included in the EA's Flood Zone Maps. Therefore the flood risk to the site has not been fully quantified. Hydraulic modelling of the watercourse was necessary to fully determine the flood risk to the site.

This report outlines a Level 3 Flood Risk Assessment (FRA) for the Exemplar Site development only. The remainder of the NW Bicester eco development site will be covered in a separate FRA.

1.3 Aims and Objectives

This study will:

- Estimate design flows for the River Bure and its tributaries for the 20-year, 100-year, 100-year (plus climate change) and 1000-year events;
- Construct a hydraulic model of the River Bure and tributaries and use it to define the baseline flood risk to the site for the design events; and
- Identify and undertake any mitigation modelling necessary to ensure that the development site is protected from flood risk while maintaining the existing hydraulic regime downstream of the development site.

2 STUDY AREA CHARACTERISTICS

2.1 Location

The study area is located on the north-western edge of Bicester, near Caversfield House off the B4100. Figure 1 below shows the location of the proposed development site (marked in red) within the River Bure catchment (outlined in green).

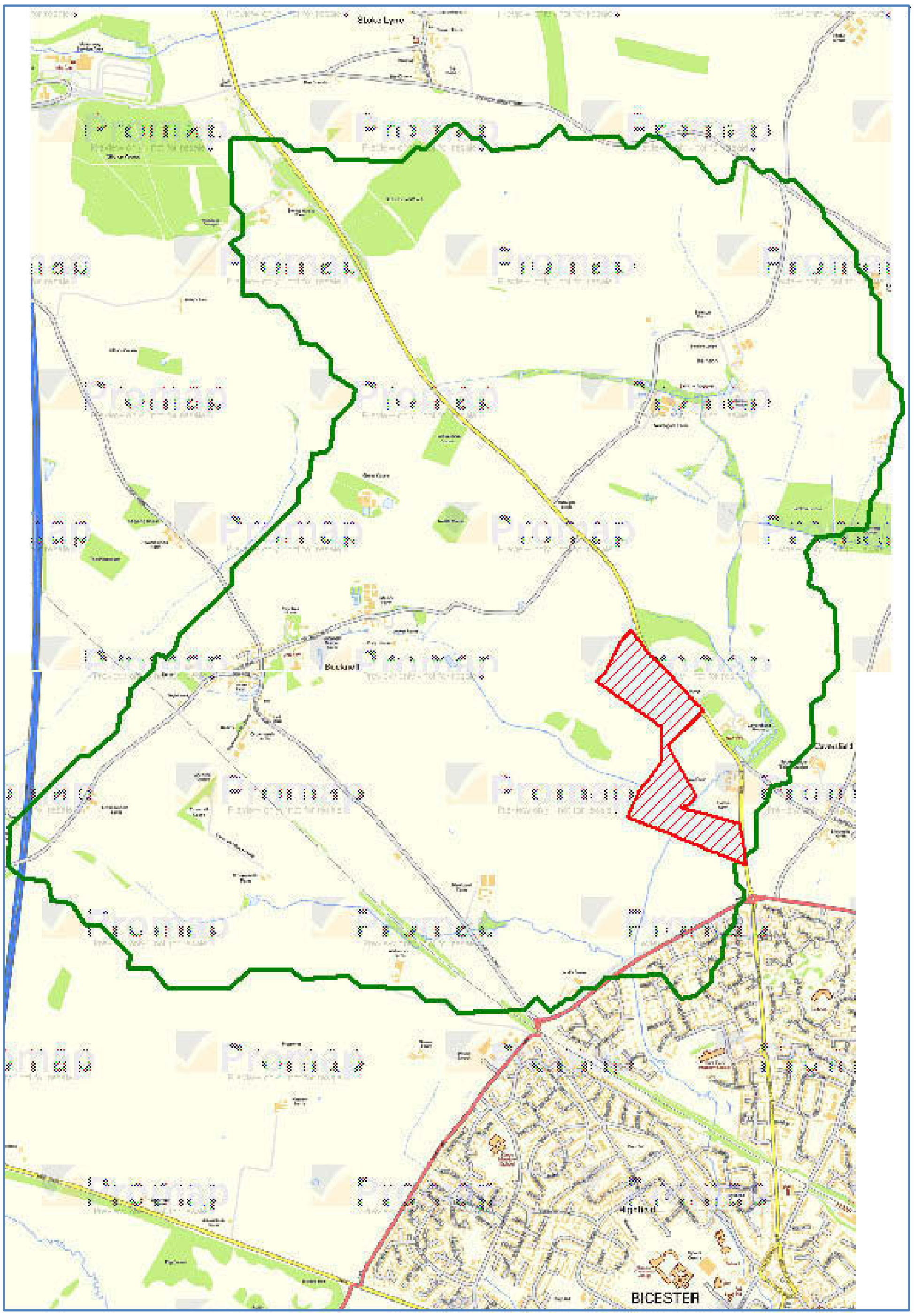

Figure 1 – Site Location Plan

2.2 Catchment

The River Bure and tributaries drain an area of approximately 10.5 km² (calculated to the A4095) which is predominantly rural. The development site is located towards the downstream end of the catchment. FEH catchment descriptors show that the catchment is permeable. Further detail is provided within the hydrology discussion in Section 3.

2.3 Watercourse

The Bure flows in a southerly direction from Caversfield House to a culvert beneath the A4095. Downstream from this it flows in an open channel between Lucerine Avenue and Purslane Drive. There is a tributary flowing in an easterly direction from Bucknell which converges with the Bure downstream of Home Farm, and another tributary which flows in an easterly direction from Crowmarsh Farm and converges with the Bure at the A4095 culvert.

2.4 Historical Flooding

There are no historical records of flooding within or around the site from either the EA or the SFRA.

2.5 Modelling History

Hydraulic modelling of the River Bure has been undertaken for the Environment Agency. However, the River Bure model does not extent far enough upstream to cover the proposed development site.

3 HYDROLOGICAL ASSESSMENT

Design flood flows for the River Bure and its tributaries were estimated using Flood Estimation Handbook Statistical method, including the permeable catchment adjustment procedure, due to the nature of the catchment. The Revitalised Rainfall Runoff Method (ReFH) and the Institute of Hydrology 124 (IoH 124) methods were considered for use but were deemed unsuitable. The ReFH method was deemed inappropriate as its application in permeable catchments is not recommended and the EA Flood Estimation Guidelines “advise users to avoid IoH124 for flood estimation on most small catchments.”

Prior to undertaking the flow estimation, FEH catchment descriptors were checked against available information. The catchments of the tributaries are too small to enable separate flow estimates to be undertaken. Therefore, for consistency the flow estimates were undertaken for the River Bure catchment at the downstream point of interest and these flows were proportioned by catchment area to obtain the estimates at the other required locations.

3.1 The FEH Statistical Method

The FEH Statistical method bases the estimation of future flood events on trends in historical flood flow data (AMAX) from a single gauged site or a group of gauged catchments (a pooling group analysis). The generation of peak flow estimates is a two-stage process.

3.1.1 Estimation of the Index Flood (QMED)

QMED, the median annual flood flow (the index flood event) is estimated where possible using gauged AMAX data recorded on the subject watercourse at the location of interest. In ungauged catchments an empirical equation that includes a number of ‘catchment descriptors’, such as area and soil type, is used and ideally, an adjustment is made based on flow data from a local, hydrologically similar ‘donor’ catchment.

Catchment descriptors for the River Bure catchment to the downstream extents of the model, the A4095, were exported from the FEH CD-ROM v3 and are presented in Table 3-1.

Table 3-1 River Bure catchment descriptors

Descriptor		Value
AREA	Catchment Area (km^2)	10.48
FARL	Index of the influence of reservoirs and lakes	0.974
PROPWET	Index of the proportion of time that soils are wet	0.32
BFIHOST	Base flow index	0.857
SPRHOST	Soil index of the percentage runoff	13.1
DPLBAR	Index describing catchment size and drainage path configuration (km)	2.8
DPSBAR	Index of catchment steepness (m/km)	16.8
SAAR	Standard Average Annual Rainfall (mm)	647
URBEXT2000	Index of catchment urbanisation	0.0078

Local gauges were assessed for their suitability for use in the adjustment of QMED, however none were deemed suitable donors due to the low SPRHOST values observed at the site. In the absence of suitable recorded data on the River Bure and neighbouring catchments QMED was estimated to equal **0.33 m^3/s**, using the catchment descriptor equation with an adjustment for urbanisation.

3.1.2 Determination of Flood Growth Curve

The second stage of the method involves the determination of a flood growth curve, a statistical relationship between the relative magnitudes of high return period flood events and QMED.

The WINFAP-FEH v3.0.003 software package with HIFLOWS v3.02 data was used to determine the flood growth curve. The software enables the 'pooling' and analysis of data from hydrologically similar catchments to produce a flood growth curve based on a weighted average of the individual growth curves from the AMAX records at each of the pooled gauging stations.

A pooling group was compiled at the site, with a target return period of 100 years. The pooled growth curve was fitted using a Generalised Logistic distribution, and was considered statistically "strongly heterogeneous". A review of the pooling group was undertaken and sites 203046, 32029, 25011, 22003, 27010 were removed from the group due to hydrological dissimilarities between the catchments draining to these gauges and the subject site. Stations 50009 and 36009 were also investigated as they are outliers on the L-moment graphs but no reason was established to justify their removal. In order to retain the required number of station years within the pooling group two stations 27073 and 48004, were added, The resultant growth factors and peak flow estimates are presented in Table 3-2 below.

Table 3-2 Peak flow estimates for design flood events at site

Return Period (Annual Occurrence Probability)	Growth Factors	Peak flows for design events (m^3/s)
1 in 2 year (50%)	1	0.33
1 in 20 year (5%)	1.96	0.65
1 in 50 year (2%)	2.58	0.85
1 in 100 year (1%)	3.22	1.06
1 in 100 year plus climate change	-	1.27
1 in 1000 year (0.1%)	7.04	2.32

A 20% allowance for climate change was added to the 1 in 100 year flow estimate, in accordance with the PPS25 and the standard design life estimates for residential property.

As hydraulic modelling required full flow hydrographs, rather than peak flow estimates, hydrographs were developed using the ReFH modelling software and the peaks of the hydrographs were scaled to the FEH Statistical flows presented in Table 3-2.

3.2 Adjustments to Hydrology

As part of the hydraulic model build, information was requested from the Environment Agency's River Bure model to inform the downstream boundary condition for the Bicester eco development model. The Environment Agency supplied stage and flow hydrographs for nodes at the A4095 road bridge, and peak stage and flow values for the node located closest to the chosen downstream boundary point of the Bicester eco development model.

When this information was received, it was noted that the peak flows in the EA's River Bure model were significantly higher than those calculated in the hydrological assessment in Section 3.2.1 above. A comparison of the EA's River Bure flows at the downstream boundary with the hydrology calculated in Section 3.2.1 is shown in Table 3-3 below, for a range of events.

Table 3-3 Comparison of calculated flows with EA River Bure model flows

Return Period	Hydrological Assessment	EA River Bure model Node BU.3056	Percentage increase
20-year	0.65	2.45	377%
50-year	0.85	2.75	324%
100-year	1.06	3.03	286%
100-year plus climate change	1.27	3.43	271%
1000-year	2.32	4.41	190%

It was determined that the EA had conducted significant temporary gauging in the catchment and used this data in calculating the hydrology for the River Bure model. The gauging data and River Bure hydrology report could not be provided in the timescale available for the NW Bicester eco development modelling, and therefore it was not possible to use the gauged information to inform the hydrological assessment.

For this reason, it was decided to use the peak flows supplied from the River Bure model in the Bicester eco development model, as the additional gauging undertaken means that the River Bure model flows are likely to be more accurate. This is particularly important given that the assessed hydrology was significantly lower than the River Bure model flows, which could lead to underestimation of the flood risk to the site.

The shapes of the hydrographs used in the modelling were calculated by using the same ReFH hydrograph used for the hydrological assessment and scaling it to the new peak flows. The final hydrographs are shown in Figure 2 below.

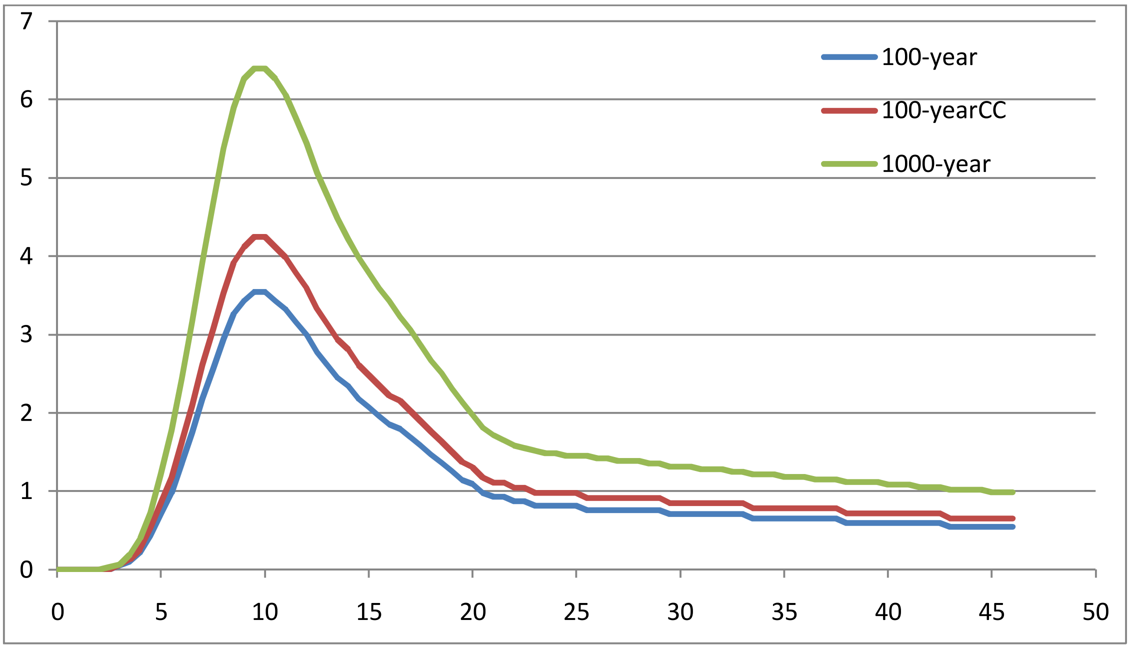

Figure 2 – Final total hydrographs

The hydrographs were then scaled by the same factors used in the hydrological assessment to divide the single flow for the River Bure into three flows for the Bure and its two tributaries. Baseflows of 0.15 cumecs were used for each tributary. The final 100-year flows for each tributary are shown in Figure 3 overleaf.

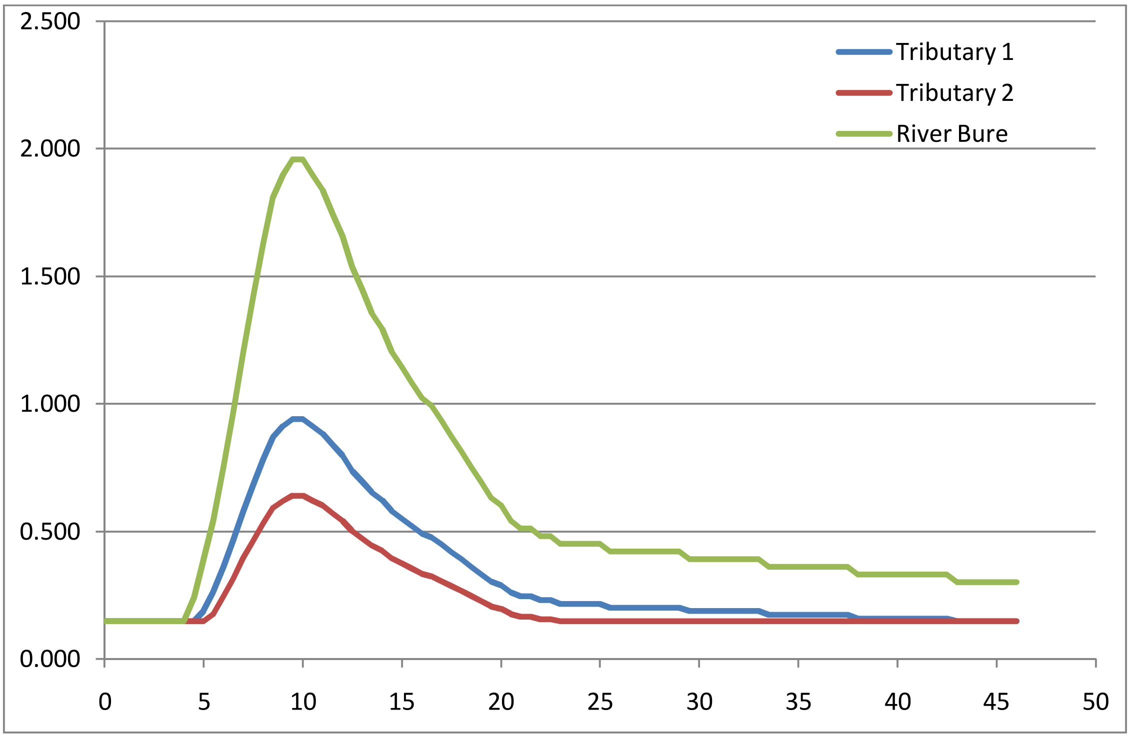

Figure 3 – 100-year flows for each watercourse

4 MODEL BUILD

4.1 Approach

An unsteady state 1D ISIS model of the River Bure and associated tributaries and floodplains was constructed.

4.1.1 Software

ISIS river modelling software (version 3.3.0.88) was used to construct the model.

4.2 Model Conceptualisation

4.2.1 Model Extents

The model contains three watercourses and a lake outflow as detailed in Table 4-1 and shown in Figure 4 overleaf.

Table 4-1 Watercourses contained in model

Watercourse	Name in model	Length of reach (m)	Upstream extent (NGR)	Downstream extent (NGR)
River Bure	Tributary 3 (T3) down to confluence with Tributary 2 (T2) down to confluence with Tributary 1 (T1) to downstream extent of model	1952	458174, 225414	457695, 223804
Tributary 1	Tributary 1 (T1)	2588 (to confluence with T2)	455409, 224548	457606, 224230
Tributary 2	Tributary 2 (T2)	1510 (to confluence with T3)	456707, 225662	457979, 224508
Lake outflow	Tributary 4 (T4)	260 (to culverted confluence with T3)	458207, 225342	458100, 225070

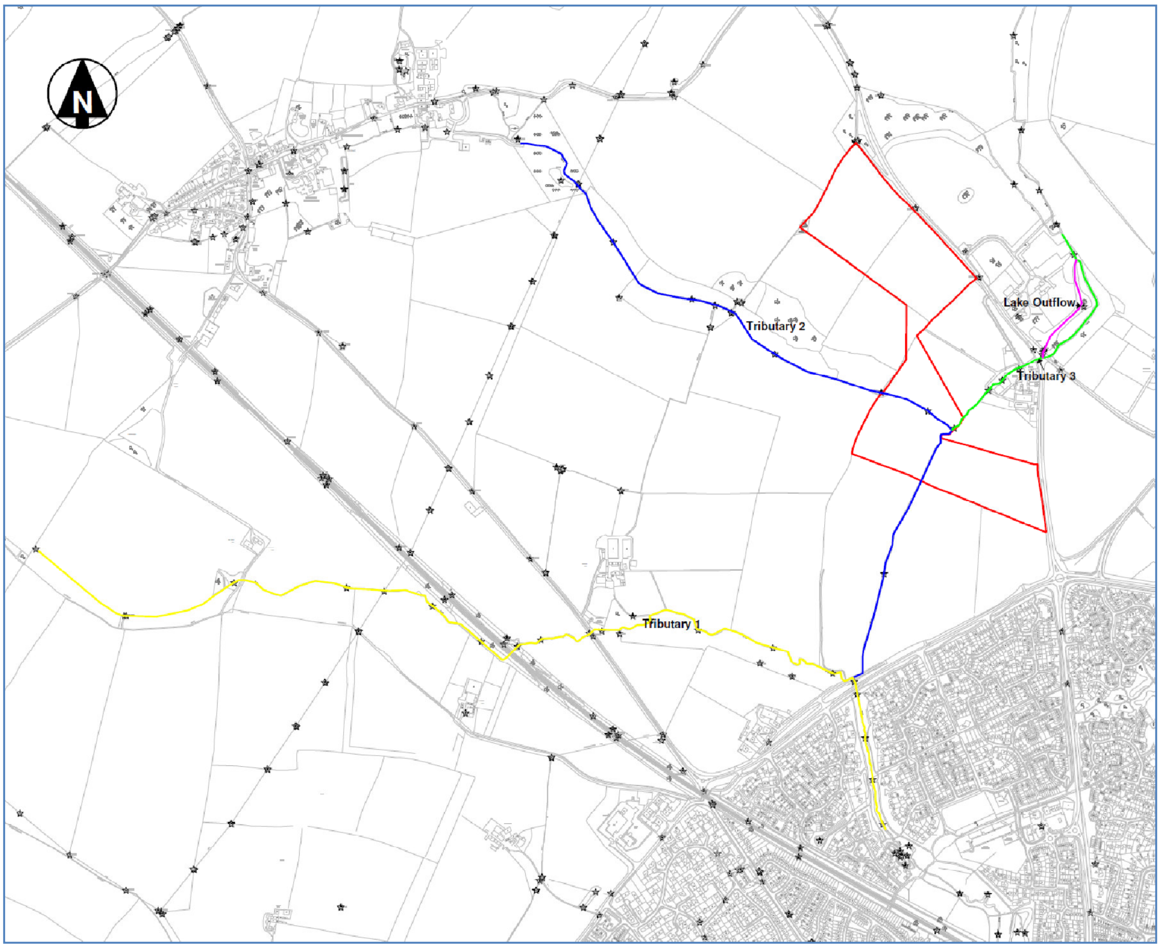

Figure 4 - Modelled Watercourses

4.2.2 Model Boundaries

The upstream boundaries for the ISIS model of the River Bure and tributaries were defined as QT boundaries. Inflow hydrographs for each design event were input at the upstream extents of the River Bure and its two tributaries. Table 4-2 below provides a summary of the different scenarios that were simulated in order to assess the existing and future flood risk on the proposed development site. Figure 5 overleaf shows the locations of the model boundaries.

Table 4-2 Modelled Scenarios

Scenario	Return period	Upstream boundary conditions	Downstream boundary condition
1:20	1 in 20 year	T1 – 0.652 m^3/s T2 – 0.444 m^3/s T3 – 1.355 m^3/s	77.12m
1:100	1 in 100 year	T1 – 0.942 m^3/s T2 – 0.641 m^3/s T3 – 1.958 m^3/s	77.21m
1:100+20%	1 in 100 year with climate change (20%)	T1 – 1.130 m^3/s T2 – 0.769 m^3/s T3 – 2.350 m^3/s	77.25m

1:1000	1 in 1000 year	T1 – 1.701 m³/s T2 – 1.157 m³/s T3 – 3.535 m³/s	77.37m

The downstream boundary condition was taken from node BU.3056 of the Environment Agency hydraulic model developed for the River Bure through Bicester.

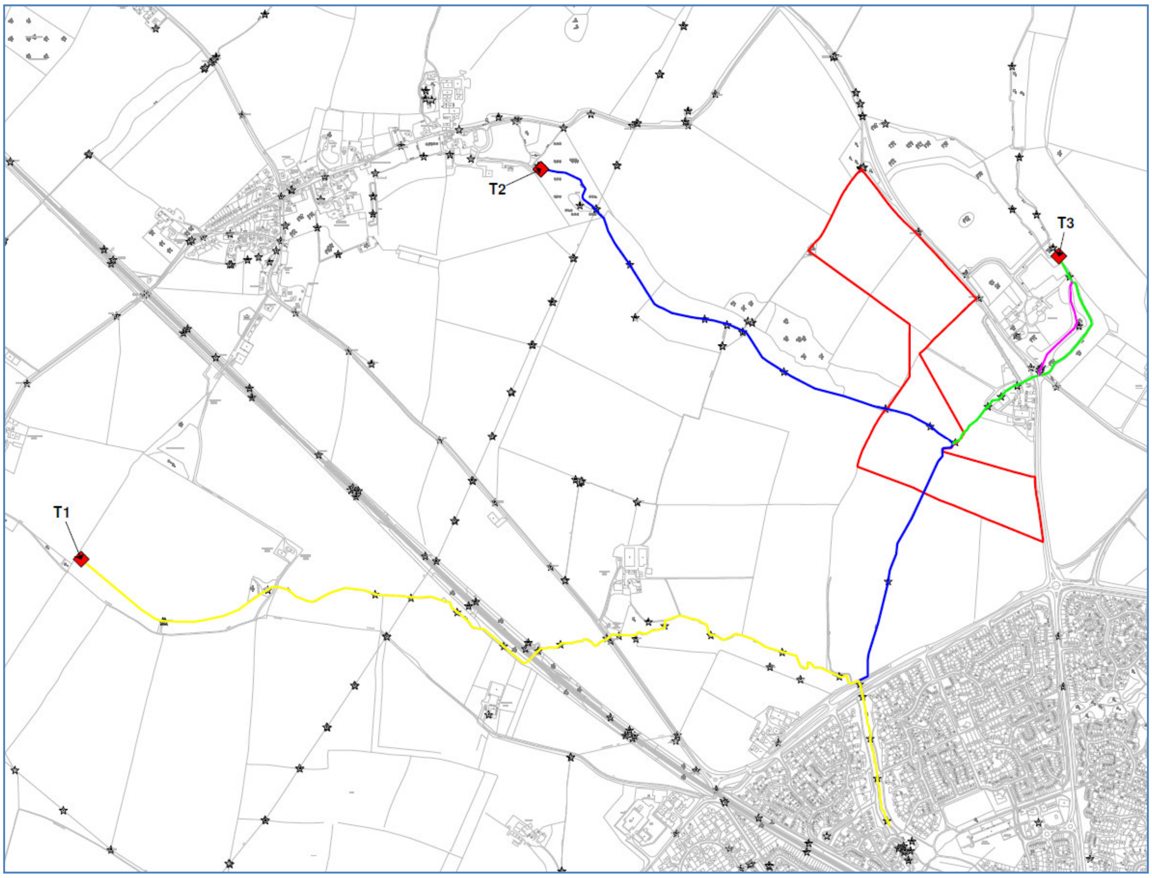

Figure 5 – Model Boundary Locations

4.2.3 Data Input

The majority of the cross-section data in the model was generated from two cross-section surveys. The majority of the model was informed by Hyder's in-house surveyors, who also conducted a topographical survey of the Exemplar site and survey information necessary to model the connection between the River Bure and the lake at Caversfield House (discussed in Section 3.3.3 below). Additional survey was collected by Maltbys Land Surveyors to supplement the existing survey information. In particular extra information was gathered at the confluences of the watercourses and at the pond outflow (named as T4).

4.2.4 Model Construction

Baseline Geometry

Open channel sections were represented using ISIS River Nodes; sections were truncated if necessary to ensure that floodplain was predominantly represented in the 2D domain.

Interpolated cross sections were added as required to improve definition of the river channel profile and to improve stability in areas where significant backwater effects were observed.

When the model was run, a number of sections were shown to be ‘glass-walling’, with modelled water levels higher than the highest ground level in the section. In some cases, the existing Hyder topographical survey was used to extend these sections across the floodplain. However, this information did not cover the full extent of the modelled watercourses. To extend the remaining sections, additional topographical information was required.

It was also identified after the initial runs that the lake at Caversfield House was connected to the River Bure at its upstream end, allowing flow along the lake to its culverted outlet. To model this flow path and the interaction between the lake and the River Bure, it was decided to model the lake using cross-sections. As survey data was not available for the lake, one LiDAR tile was purchased to aid in creating the lake sections and extending the River Bure sections in the area. The base level of the lake sections was taken from points on the survey of the River Bure that showed the left bank of the lake.

For any remaining glass-walling sections that weren’t covered by either the site topographical survey or the LiDAR, a 5 m DTM was used to extend sections. As this was the least accurate of all the topographical information available, it was only used where other more accurate information could not be obtained.

4.2.5 Structures

The baseline model incorporates a number of structures that have been modelled using the survey data provided. The structures are listed in Table 4-3.

Table 4-3 Structures in model

ISIS node	Structure	ISIS unit
T1-2723	Small field ditch culvert	Symmetrical conduit
T1-2391	Inline pond outflow	Symmetrical conduit
T1-2064	Small field ditch culvert	Symmetrical conduit
T1-1564	Railway culvert	Symmetrical conduit
T1-1300	Road culvert	Arch bridge
T1-1051	Small field ditch culvert	Symmetrical conduit
T1-0452	Small footbridge	Orifice
T1-0427	Bridge under track	Arch bridge
T1-0416	Bridge under road	Symmetrical conduit
T2-1461	Small field ditch culvert	Symmetrical conduit
T2-0779	Small field ditch culvert	Orifice
T3-0741	Small footbridge	Orifice
T3-0637	Small culvert	Orifice
T3-0356	Permanent sluice board (see below)	Spill
T3-0355	Penstock (see below)	Vertical sluice
T3-0354	Road culvert	Symmetrical conduit

T3-0301	Road culvert	Symmetrical conduit
T3-0256	Small footbridge	USBPR bridge
T3-0176	Small bridge	USBPR bridge
T3-0157	Small bridge	USBPR bridge
T4-pondweir	Lake inflow from T3-0687	Orifice
T4-0025	Lake outflow culvert	Orifice
T4-0019	Small arched gap in wall over outflow channel	Arch bridge
T4-0015	Road culvert	Symmetrical conduit

In general the ISIS node used has matched the type of structure found in the study area. However, orifice units were used where model stability was compromised by using a bridge or culvert. This usually occurs when these units are surcharged as they do not handle the transition between normal and orifice flow very well and can cause model instability.

A number of culverts in the model use the symmetrical conduit unit with a thin 'hat' on the unit. This is because ISIS does not solve pressurised flow very well and therefore by using a small 'hat' the open channel equations are still used without a significant loss of accuracy in calculating water levels.

Flows into and out of these culverts are modelled using spill units rather than culvert inlet/outlets as these units are coming under increasing critique as was highlighted at the ISIS user group in November 2009. In Bicester most of the culverted sections are small and are part of the field ditch system. The culvert inlet/outlet units have been designed with larger culverts in mind and therefore using spill units is the methodology that has been followed here.

A full model schematic has been supplied in electronic format with the ISIS model.

T3-356

The series of structures just upstream of the face of the B4100 road culvert is particularly complex. The model was informed by the survey information and from talking to the surveyors who undertook the work. Photographs from the surveys are shown in Figure 6.

Figure 6 – Photographs of the structures at T3-356

The first structure is a wooden board which has been placed in the channel and allows no water to pass under it, causing water to weir over the top of it to flow “through” the structure. After speaking with the surveyors they suggest the channel is likely to be ephemeral at this location. This structure was modelled as a spill with a coefficient of 1.2 in the ISIS model.

Water that flows over this structure enters a sump 1.37 m below the crest of the board. The water then flows through a penstock gate, which is open and then into another sump. Water then enters the road culvert, which is set 0.41 m above the base of the sump. This culvert extends for approximately 35 m before issuing at T3-314.

The first sump was modelled by repeating the section at the upstream face of the penstock, which was then linked to a vertical sluice unit with the weir information taken from the long section. The breadth of the weir was altered to ensure that the bore area was correct. The second sump was modelled using two river units with the geometry provided by the surveyors for the immediate downstream face of the penstock. However, the second section was raised to the bed level at the face of the road culvert as informed by the long section. The culvert was modelled using the spill and symmetrical conduit schematisation as described above.

During model runs, adjustments to this schematisation were required to deal with poor convergence in the model. These adjustments consisted of changing the bed levels of culvert sections T3-0335ca, T3-0335cb and T4-0000 to flatten the slopes of these culverts, and the addition of a spill at T3-0335ca to deal with the drop in bed level. The ‘hats’ on these culverts were also removed. These changes stabilised the model and removed the poor convergence.

A comparison of the long section at this location in ISIS and from the survey is shown in Figure 7.

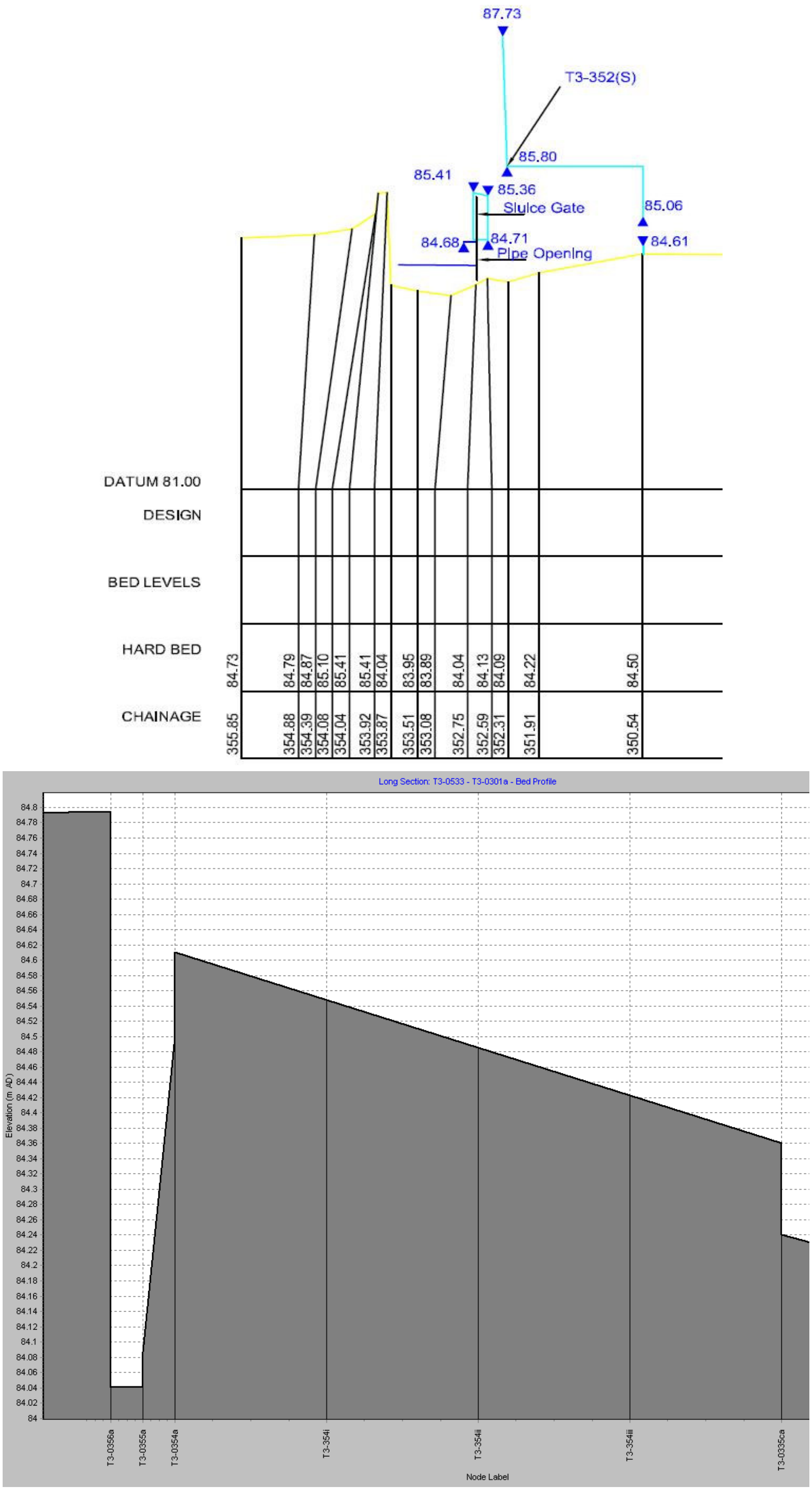

Figure 7 – The surveyed (top) and ISIS long sections at this location. Please note that the spill and vertical sluice gate information is not shown in the ISIS output.

T1/T2 Confluence

The confluence between T1 and T2 is another unusual configuration. The two watercourses flow towards each other and then at the confluence, the combined watercourse flows at ninety degrees through a culvert (see Figure 8). The watercourse then reverts to open channel before entering a second culvert under the road.

At the actual confluence the bridge unit (T1-0427a) was informed from the survey. Then 1 m back from this the same channel profile was used but with only the bed (25.714 LHB to 36.601 RHB) and not any of the banks. For each watercourse, the upstream section (T1-0450b and T2-0055) was copied to become the most downstream section before the confluence but the bed levels were dropped to the same level as at the points mentioned above (25.714 for T1 and 36.601 for T2) on the long section through the confluence provided by the surveyors. The sections were altered to include the wall on the relative bank. These channel profiles were copied into spill units and linked to the confluence section with the spill unit using a junction. This was because without a spill unrealistic water level profiles were obtained upstream of the confluence. The spill unit had a value of 1.5 to help model some of the energy losses that will occur at this location.

This schematisation was seen as the best that could be achieved given the information obtained. However, it is possible that the energy losses are not fully accounted for but there was no information available for an additional general loss unit to be used.

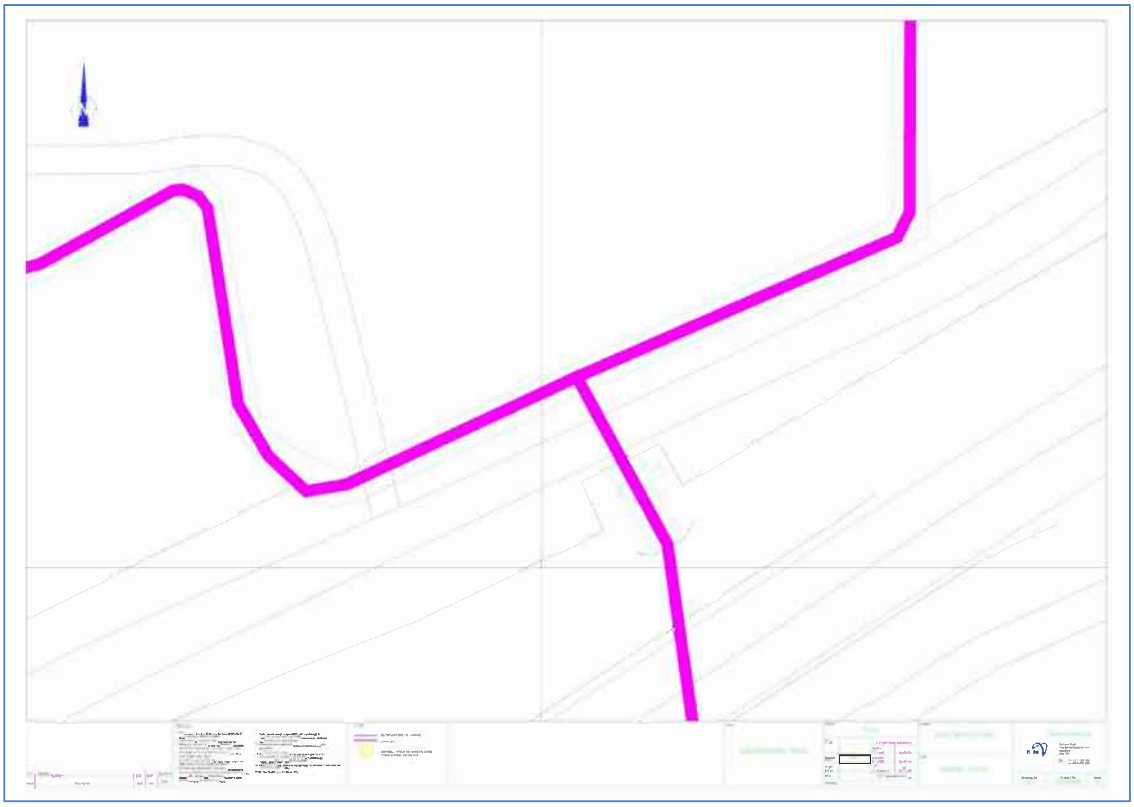

Figure 8 – The confluence between T1 and T2

4.2.6 Manning's Roughness Coefficients

The resistance to flow in a channel or over a floodplain is defined in a hydraulic model by the use of a roughness coefficient, Manning's number, otherwise known as Manning's '*n*'. The Manning's '*n*' range of values used in the model, as outlined in Table 4-4, were based on site visit observations and published values (Chow, 1959).

Table 4-4 Adopted Range of Manning's '*n*' coefficients

Location	Manning's '*n*'		Type of Channel / Floodplain and Description
Channel	Min	0.04	Clean, winding, some pools and shoals with some weeds and stones.
	Max	0.05	Winding, some pools and shoals, lower stages, more ineffective slopes and sections with weeds and stones.
Floodplain	Min	0.02	Concrete or tarmac.
	Max	0.07	Medium to dense brush in winter.

4.3 Model Runs

For all model runs, a time step of 1 second was used. Design hydrograph simulations were run for 50 hours. Most run parameters have not been altered from the default values. Parameter dflood has been raised as several sections in the model around the confluence with T1 and T2 have low bed levels and therefore can end up with very high water levels. Raising dflood prevents this from causing a problem during the model runs. The parameter maxitr has also been raised to aid model stability.

The model runs carried out as part of this study are listed below.

- 20-year;
- 100-year;
- 100-year with 20% climate change; and
- 1000-year.

4.4 Model Calibration

Unfortunately no recorded water level or flow data was available at the site and therefore model calibration was not possible. To gain further confidence in the model, sensitivity analysis was undertaken as detailed in Section 4.5.

4.5 Sensitivity Testing

Model sensitivity tests are undertaken to determine the level of uncertainty in the predicted water levels associated with key model parameters. For consistency, all sensitivity tests have been carried out using the 1 in 100 year flow. The following tests were undertaken:

- Manning's 'n' values increased by 20%
- Manning's 'n' values decreased by 20%
- Downstream boundary increased by 0.5m

- Spill coefficients increased by 20%
- Spill coefficients decreased by 20%

4.6 Post-Development Model

The proposed development for the Exemplar Site includes the removal of an existing bridge structure at T2-0779a, the addition of two large bridge structures where new roads cross the watercourse, and significant reshaping of the watercourse floodplain. Figure 9 below shows the proposed development with these changes highlighted. Post-development modelling was undertaken to determine the impact of the proposed development on flooding in the area.

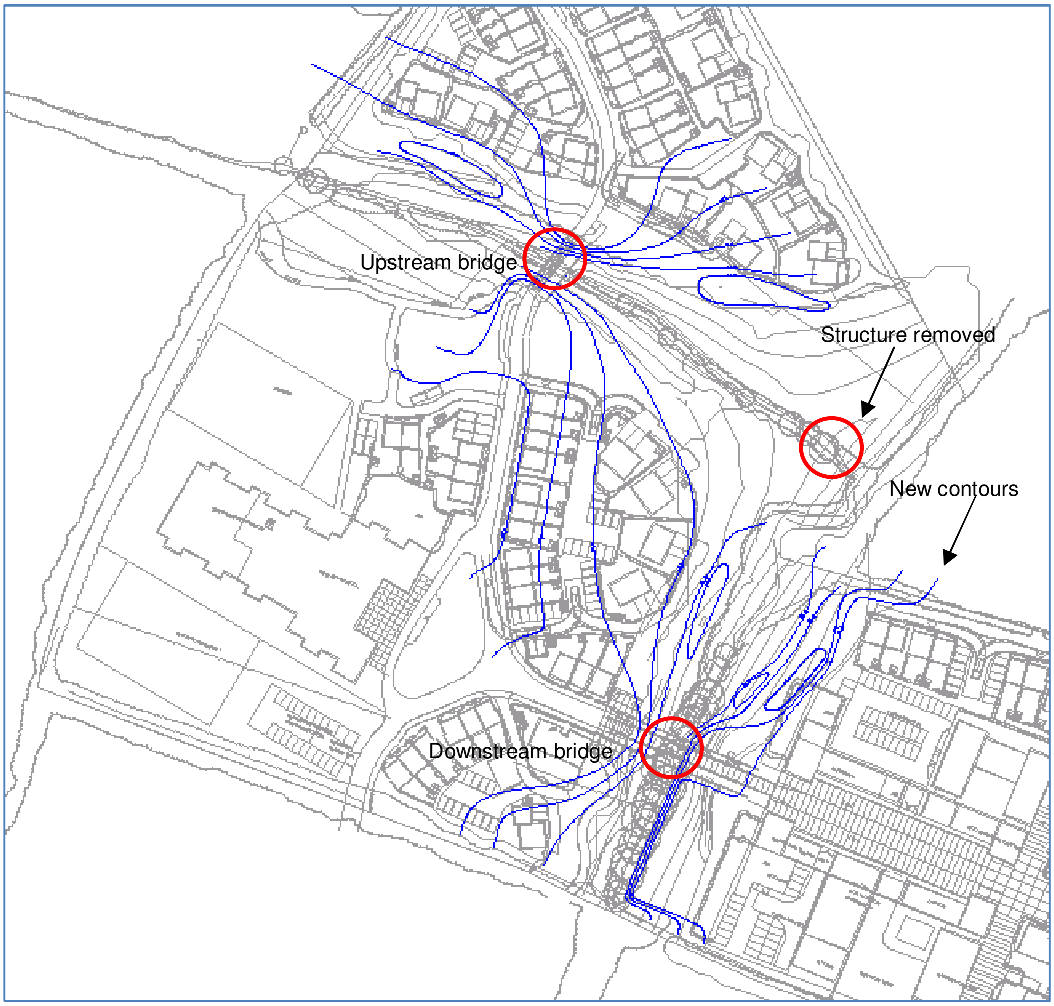

Figure 9 – Proposed development changes

To model the changes in floodplain topography and the two bridge structures, additional cross-sections were required as circled in red on Figure 10 overleaf.

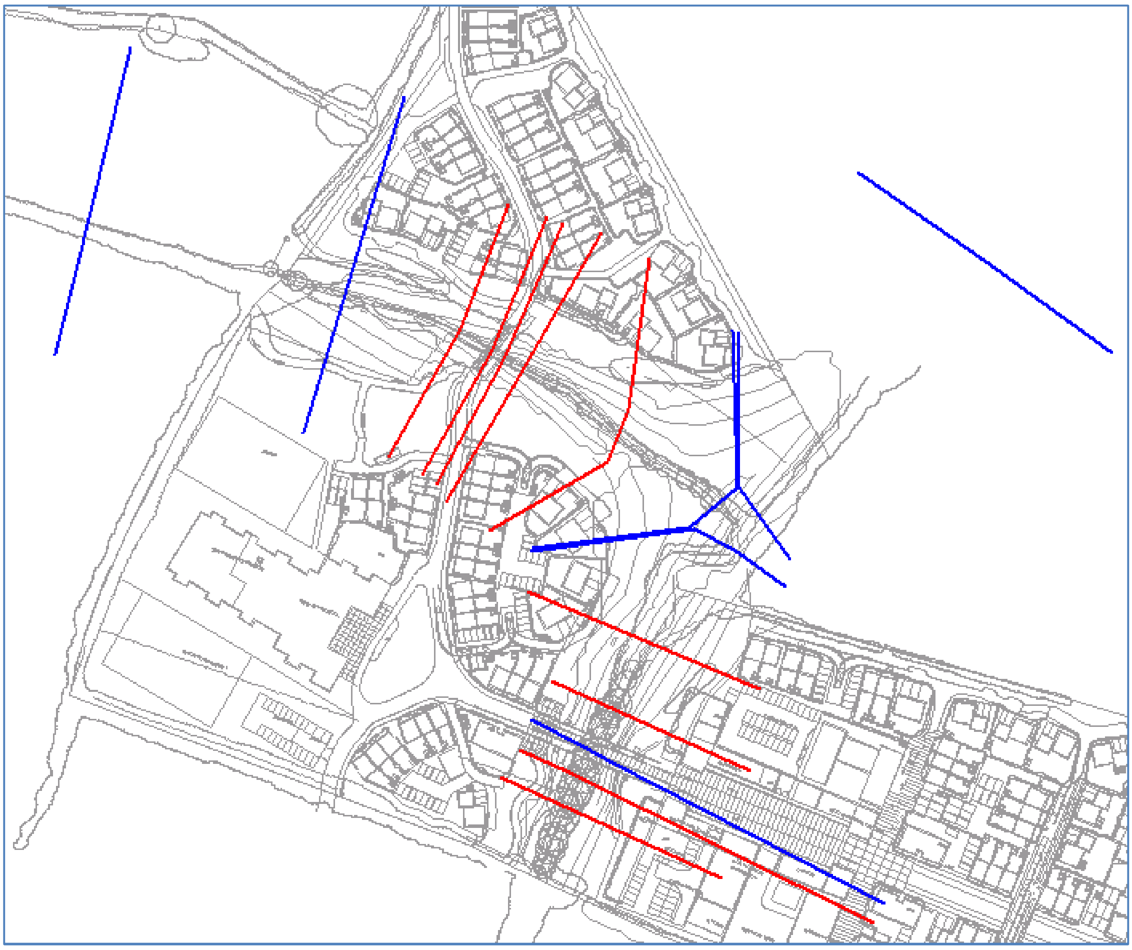

Figure 10 – New cross-sections for model

In order to allow comparison of levels and velocities with the baseline model, the baseline model was re-run for the 100-year with climate change and the 1000-year flood events to include new sections at the locations shown in Figure 10 above. For the baseline model, these sections were interpolated from topographical survey taken of the area.

Figure 11 overleaf shows the proposed design for the two bridge culverts. These culverts were represented in the model using symmetrical conduit units. The existing shape of the channel bed was used as the base of the culvert.

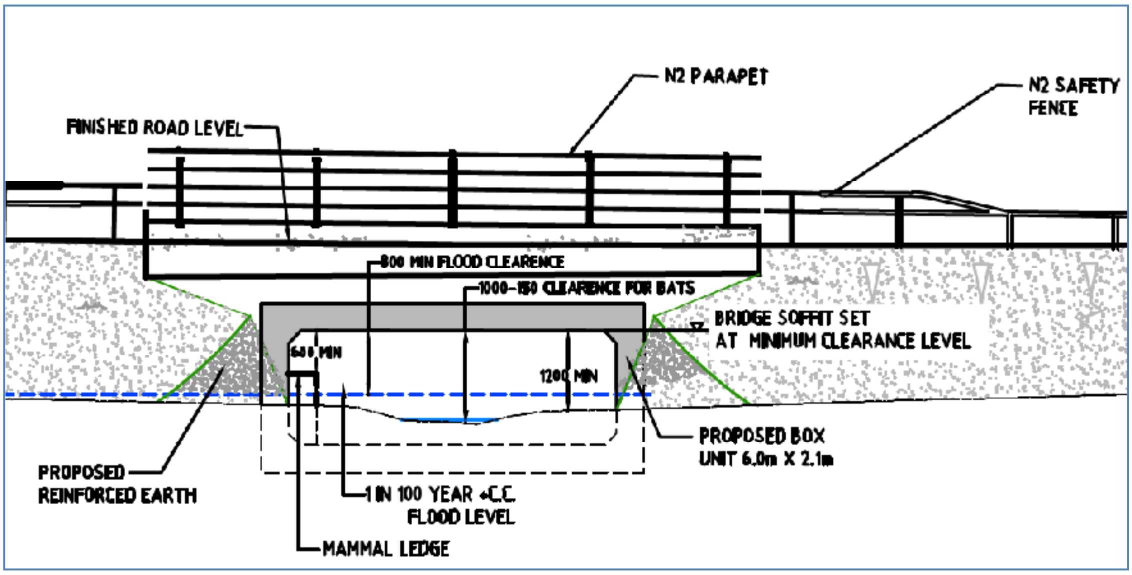

Figure 11 – Proposed bridge culvert

5 MODEL RESULTS

5.1 Results

Modelled water levels and flows for the ISIS model nodes are summarised in Appendix A.

Appendix B contains figures which illustrate the flood extents and depths for each of the four design events.

Figure 12 below shows the development site with the modelled 100-year and 1000-year extents (i.e. Flood Zones 2 and 3), shown in dark blue and light blue respectively.

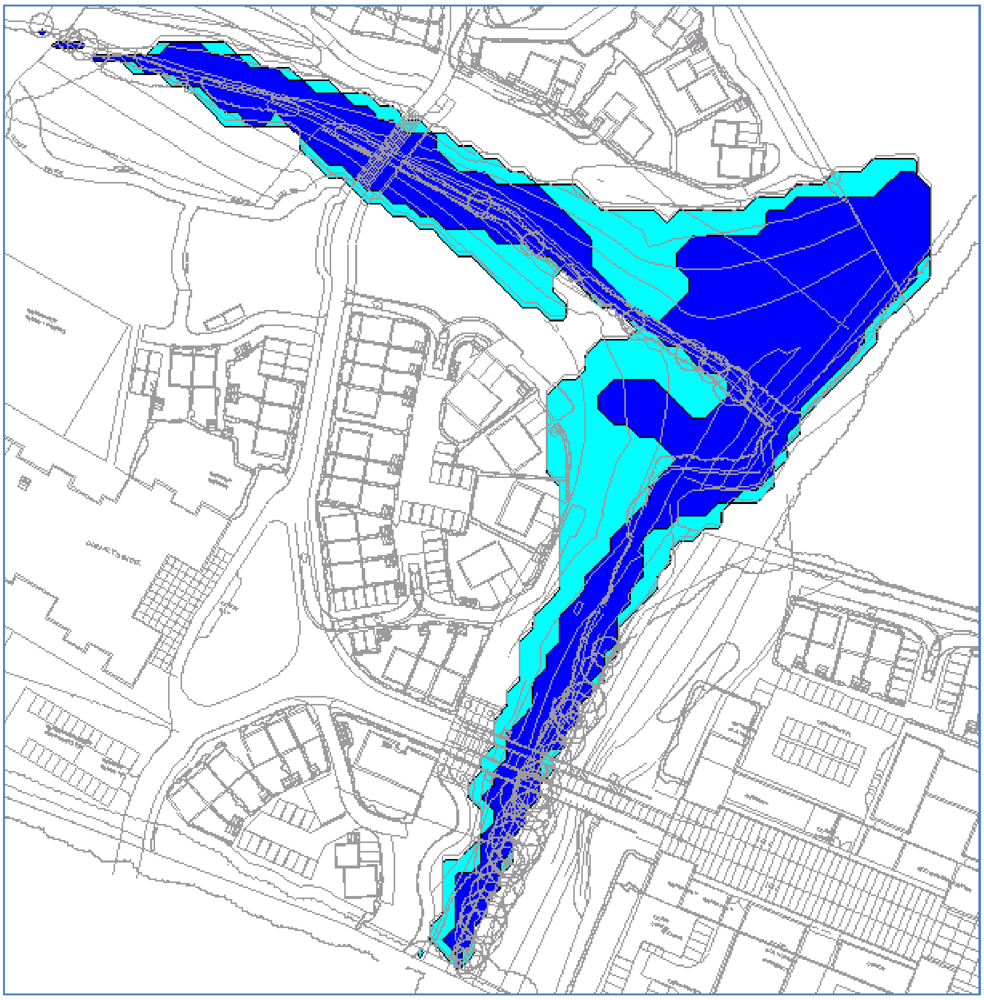

Figure 12 - Modelled Flood Risk to Site

5.2 Discussion of Results

The flood extents shown Appendix B and in Figure 12 above show the baseline flood risk to the development site from the River Bure and its tributaries. The extents show that the majority of the proposed development site is in Flood Zone 1 with a low risk of flooding (greater than 1 in 1000 years). There are small areas of Flood Zones 2 and 3 around the watercourses

The model predicts that floodwater is generally confined to the valleys in which the watercourses flow, with ponding occurring at confluences and upstream of constricting structures. The model does not predict any overland flow occurring. On the Exemplar site, flooding occurs predominantly on the flatter land around the confluence between the River Bure and the northernmost of the two tributaries (T3). Away from the confluence, flooding is confined to the relatively narrow valley of the watercourse.

Table 5-1 below shows the modelled peak water levels through the development site for each return period. Cross-section locations are shown on Figure D2 in Appendix B.

Table 5-1 Development Site Modelled Peak Water Levels

Node Label	20-year	100-year	100-year with climate change	1000-year
T2-0952	84.64	84.67	84.68	84.70
T2-0779a	83.27	83.34	83.38	83.49
T2-0777b	83.27	83.34	83.38	83.49
T2-0756a	83.26	83.34	83.38	83.49
T2-0756b	83.26	83.34	83.38	83.49
T2-0636	82.67	82.77	82.81	82.91
T3-0157a	83.45	83.54	83.59	83.71
T3-0152b	83.45	83.54	83.59	83.71
T3-0011	83.26	83.34	83.38	83.49

5.3 Sensitivity Test Results

As discussed in Section 4.5, the following sensitivity tests have been run using the 100-year design event.

- Manning’s ‘n’ values increased by 20%
- Manning’s ‘n’ values decreased by 20%
- Downstream boundary increased by 0.5m
- Spill coefficients increased by 20%
- Spill coefficients decreased by 20%

Table 5-2 overleaf summarises the ISIS results for the sensitivity tests, showing the average, minimum and maximum changes in modelled water level for the whole model, while Table 5-3 shows the values for the reach through the development site.

Table 5-2 Sensitivity Test Results Summary – Whole model

Sensitivity Test	Maximum	Minimum	Average
Manning’s n values increased by 20%	0.08	0.00	0.03
Manning’s n values decreased by 20%	0.00	-0.10	-0.03
Downstream boundary raised 0.5m	0.50	0.00	0.01
Spill coefficients increased by 20%	0.01	-0.07	-0.01
Spill coefficients decreased by 20%	0.12	-0.02	0.02

Table 5-3 Sensitivity Test Results Summary – Development Reach

Sensitivity Test	Maximum	Minimum	Average
Manning's n values increased by 20%	0.06	0.00	0.03
Manning's n values decreased by 20%	-0.02	-0.07	-0.06
Downstream boundary raised 0.5m	0.00	0.00	0.00
Spill coefficients increased by 20%	0.00	-0.01	0.00
Spill coefficients decreased by 20%	0.00	-0.01	0.00

5.3.1 Model Parameters

Changing the spill coefficients in the model made virtually no difference to modelled water levels across the development site. Across the whole model, changing the spill coefficients did cause changes in modelled water level, particularly in the area around the confluence of T1 and T2, with changes of +/- 70mm to 120mm in this area. This indicates that in general the model is not particularly sensitive to spill coefficients, with the exception of the T1/T2 confluence.

Increasing Manning's 'n' values also made little difference to modelled water levels, with a maximum increase of approximately 80mm over the whole model, and 60mm across the development site. Decreasing Manning's 'n' values gives slightly greater changes in modelled water levels, with a maximum increase of 100mm at one section only. Based on these results, the model is not considered to be overly sensitive to changes in Manning's 'n' values.

The results of the sensitivity tests to Manning's 'n' and spill coefficients show that the model is not very sensitive to changes in these parameters.

5.3.2 Downstream Boundary

Increasing the downstream boundary level by 0.5m caused an increase in water level at the downstream-most section of the model, but no significant changes in modelled water level elsewhere in the model. The small reach of the model affected by the change indicates that the model is not overly sensitive to changes in downstream boundary.

5.4 Post-Development Modelling

Adding section to the baseline model caused some minor changes to the modelled flood extents for the 100-year with climate change and the 1000-year events, particularly in the upstream reach of the tributary. This is shown in Figure 13 for the 1000-year event, where the previous extent is shown in black and the revised extent in blue.

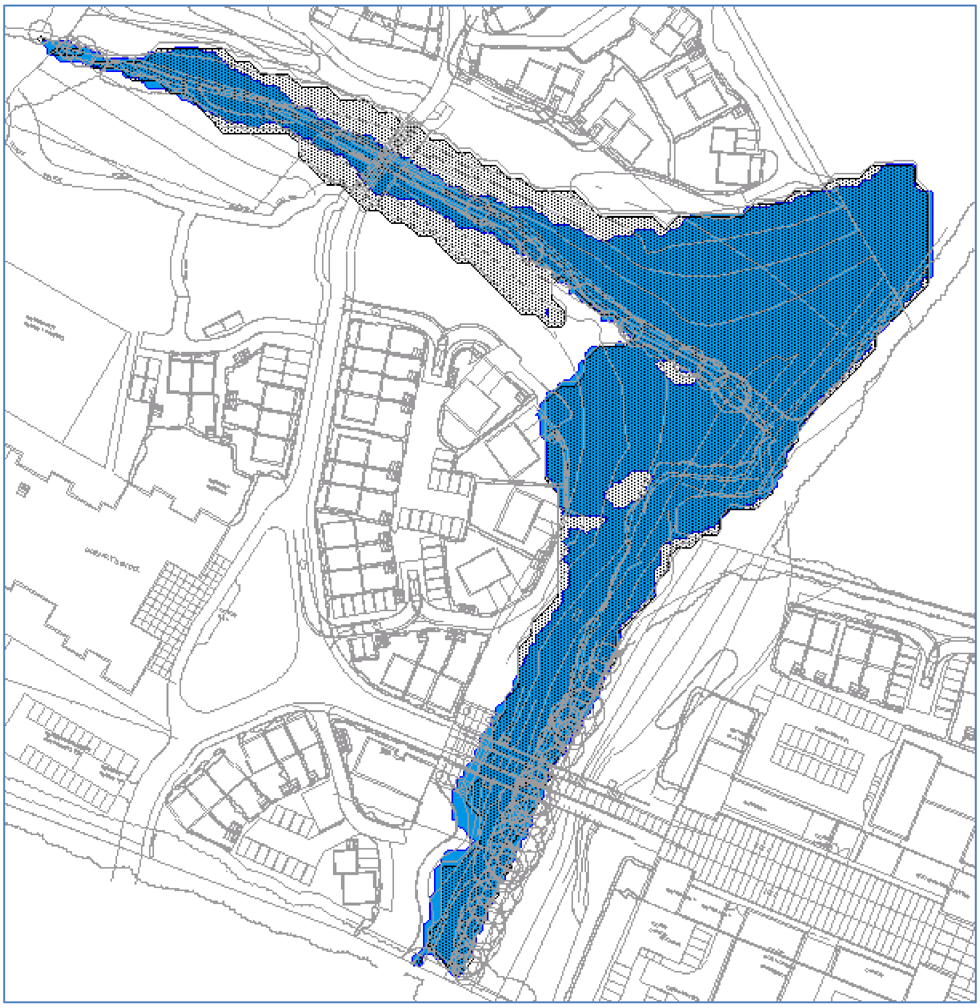

Figure 13 – Differences in baseline extent

Figure 14 overleaf shows the change in flood extent caused by the proposed development for the 1000-year flood event, where the revised baseline is shown in blue and the post-development extent in black.

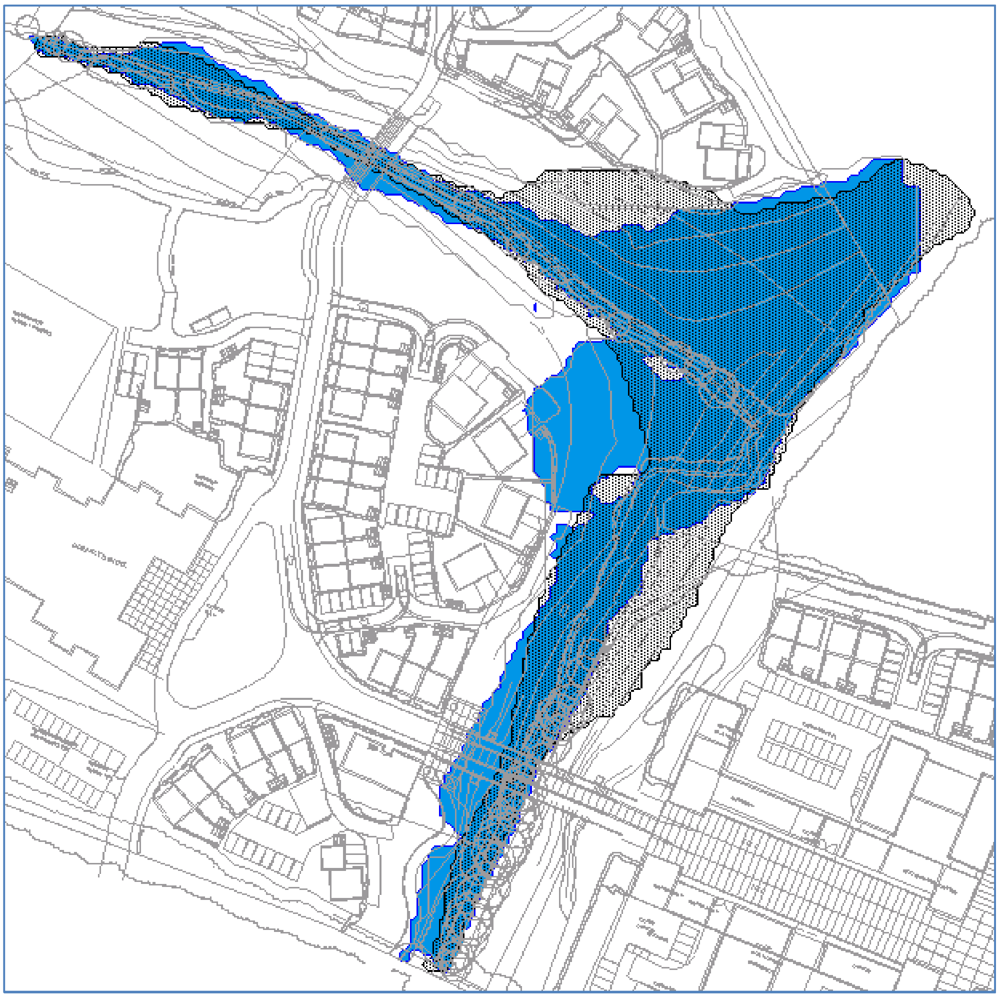

Figure 14 – Post-development extent comparison

This shows that around the proposed bridge on the tributary, the contouring and bridge structure have little impact on the modelled flood extents. Downstream of this area, the re-contouring causes additional flooding on the open space on the left bank, but this does not threaten the proposed development. At the confluence with the River Bure, the re-contouring has significantly reduced the flood extent on the western side of the confluence, removing the area of flooding that had impacted on gardens and roads in the proposed development. Downstream of this area, the landscaping associated with the second bridge has decreased the flood extent at the bridge location and downstream, but increased the flood extent upstream of the bridge on the left bank of the River Bure. This also does not threaten the proposed development.

A comparison of modelled flood levels through the development site is shown overleaf in Table 5-4, with the upstream bridge (T2-0887) and downstream bridge (T2-0636) sections highlighted. These results show that modelled water levels through the reach are generally lower, with moderate increases of 50mm to 110mm immediately upstream of the downstream bridge (T2-0636). This indicates that this bridge in conjunction with the narrowing of the channel at this point is causing a slight obstruction to flow in this area. Decreases of approximately 90mm to 110mm around T2-0756a are likely to be related to the removal of the existing structure in this area.

Table 5-4 Post Development Modelled Water Levels Summary – Development Reach

Label	100-year with Climate Change			1000-year		
	Baseline	Post Development	Difference	Baseline	Post Development	Difference
T2-0952	84.66	84.68	0.02	84.69	84.73	0.03
T2-0902	83.97	83.97	0.00	84.03	84.02	-0.01
T2-0887	83.88	83.83	-0.05	83.94	83.90	-0.04
T2-0887d	83.78	83.78	0.01	83.84	83.84	0.00
T2-0872	83.67	83.69	0.02	83.74	83.74	0.00
T2-0827	83.42	83.36	-0.05	83.49	83.43	-0.05
T2-0777b	83.34	83.23	-0.11	83.43	83.34	-0.09
T2-0756a	83.33	83.22	-0.11	83.42	83.34	-0.09
T2-0756b	83.33	83.22	-0.11	83.42	83.34	-0.09
T2-0686	83.05	83.08	0.03	83.16	83.23	0.06
T2-0656	82.94	83.00	0.05	83.05	83.17	0.11
T2-0636	82.88	82.89	0.01	83.00	83.05	0.05
T2-0636d	82.81	82.77	-0.04	82.92	82.90	-0.03
T2-0611	82.65	82.61	-0.04	82.75	82.72	-0.03
T2-0462	81.60	81.60	0.00	81.69	81.69	0.00

Table 5-5 overleaf shows a comparison of velocities through the development reach. This shows increases in velocities in the reaches around both proposed bridges (T2-0902 to T2-0827, and T2-0636 to T2-0611). Figure 15 shows the areas of increased velocity on the proposed development plan.

Figure 15 – Reaches showing increased velocity

Table 5-5 Post Development Modelled Velocities Summary – Development Reach

Label	100-year with Climate Change			1000-year		
	Baseline	Post Development	Difference	Baseline	Post Development	Difference
T2-0952	0.708	0.635	-0.073	0.792	0.639	-0.153
T2-0902	0.915	0.981	0.066	0.955	1.058	0.103
T2-0887	0.505	0.669	0.164	0.523	0.707	0.184
T2-0887d	0.992	1.033	0.041	0.992	1.033	0.041
T2-0872	0.676	0.741	0.065	0.719	0.741	0.022
T2-0827	0.513	0.649	0.136	0.516	0.662	0.146
T2-0777b	0.651	0.65	-0.001	0.651	0.65	-0.001
T2-0756a	0.296	0.368	0.072	0.296	0.368	0.072
T2-0756b	0.726	0.671	-0.055	0.729	0.671	-0.058
T2-0686	0.759	0.45	-0.309	0.802	0.455	-0.347
T2-0656	0.673	0.604	-0.069	0.685	0.606	-0.079
T2-0636	0.674	0.994	0.32	0.676	1.082	0.406

T2-0636d	1	1.337	0.337	1.002	1.458	0.456
T2-0611	0.901	1.003	0.102	1.027	1.151	0.124
T2-0462	1.283	1.283	0	1.284	1.283	-0.001

Some of the increases in velocity shown are potentially significant, with velocities at the downstream bridge (T2-0636) increasing by approximately 40-60%. This has the potential to cause scour in the areas around the bridges, which may be a particular problem at the downstream bridge due to larger velocities and the presence of a narrow channel with steep banks. Detailed design will need to assess the scour potential in this area and provide appropriate protection to prevent any scour.

The proposed development causes no significant change in flood extents, levels or velocities downstream of the development site.

5.5 Stability and Convergence

A check of the convergence plots for each design run shows that the 20-year, 100-year and 100-year plus climate change runs all show poor convergence at the same points in the flow hydrograph, as shown in Figure 16, Figure 17, and Figure 18 below and overleaf.

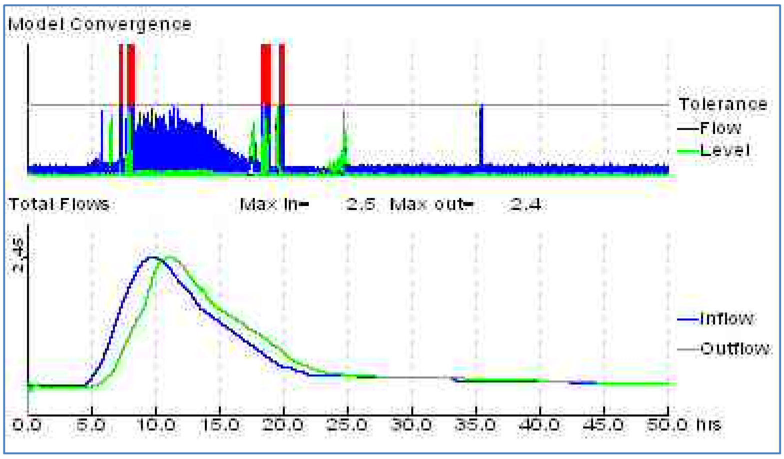

Figure 16 – 20-year convergence plot

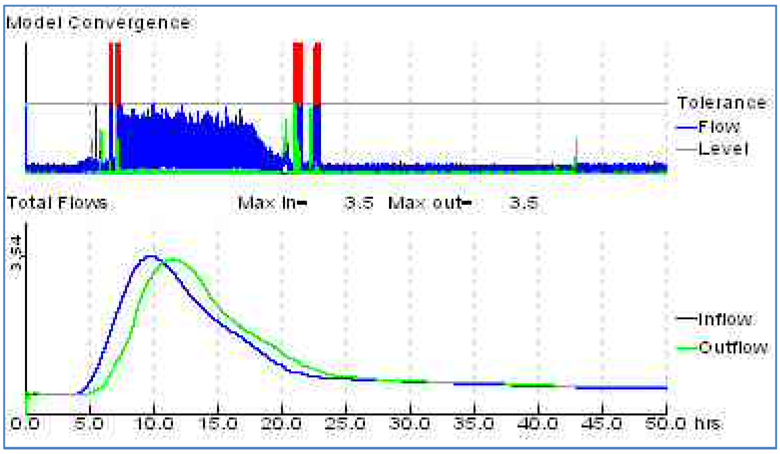

Figure 17 – 100-year convergence plot

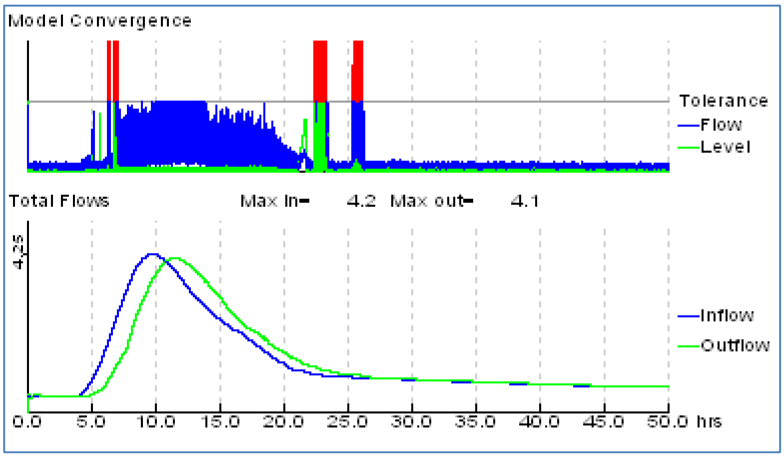

Figure 18 – 100-year with climate change convergence plot

This poor convergence is occurring around the structure at T4-0025, and appears to occur when the water level downstream of the structure reaches the upstream sill level of the orifice representing the structure. The model seems to be having difficulty with the transition from free flow to orifice flow, as shown by the instability in the unit mode plot in Figure 19. Mode 2 is free flow, while Model 4 is orifice flow.

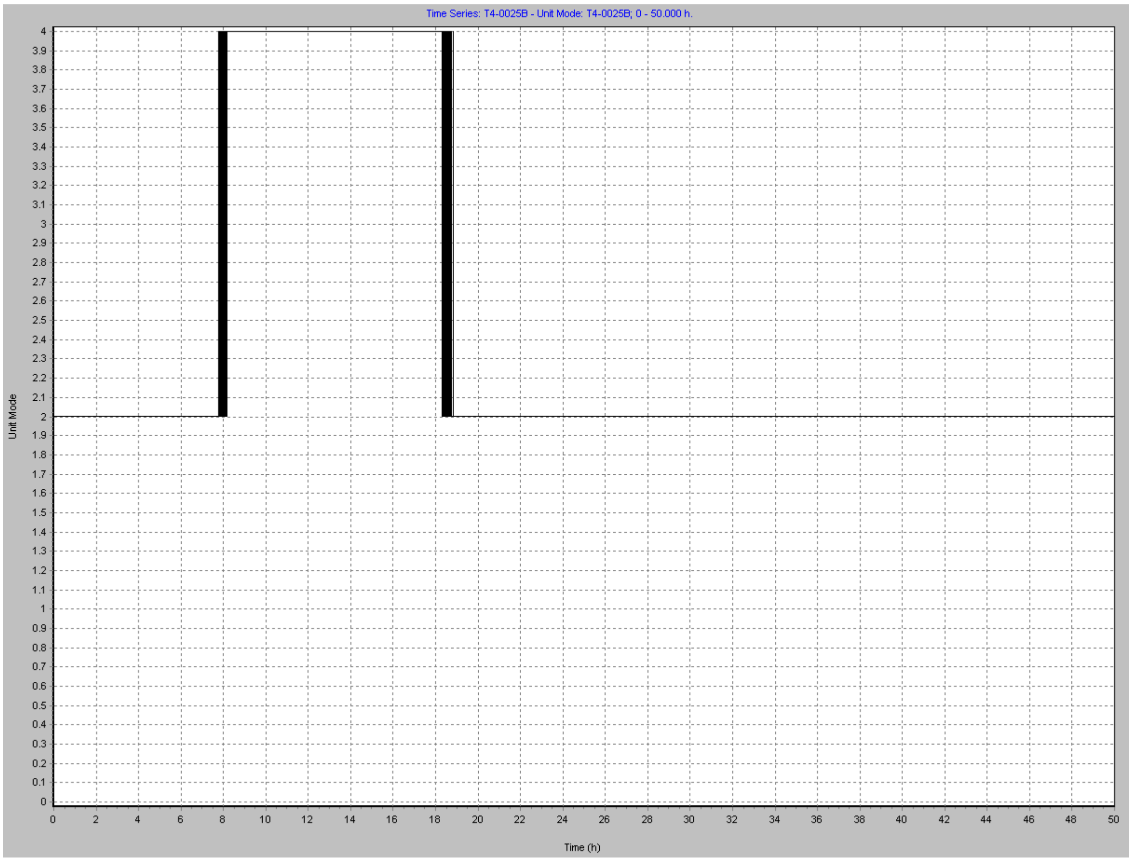

Figure 19 – 20-year unit mode plot for T4-0025

A check of the stage and flow results downstream of this structure (see Figure 20) show that there is instability in the flow hydrograph (red) through this structure, but it has a much smaller impact on the stage hydrograph (blue). As the instability generally occurs before and after the peak of the flow hydrograph, and is located upstream of the Exemplar site and outside of the total eco development site, it is not considered to have a significant impact on the results of the hydraulic modelling and the resultant FRA conclusions.

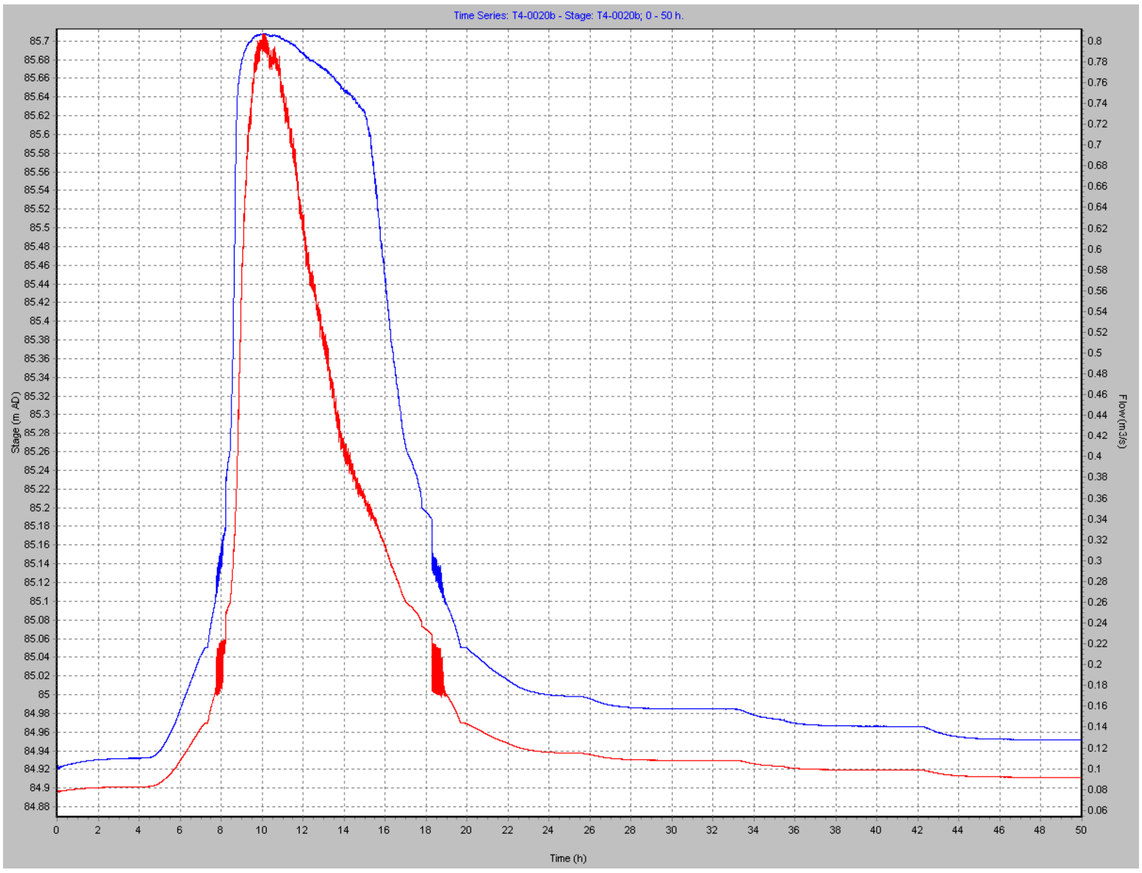

Figure 20 – Modelled stage and flow at section T4-0020

The 1000-year convergence plot is shown in Figure 21. This run has the same poor convergence as described above, and also shows some poor convergence based around the structure at T3-0265. This appears to occur at the point at which the downstream water level nears the soffit of the structure.

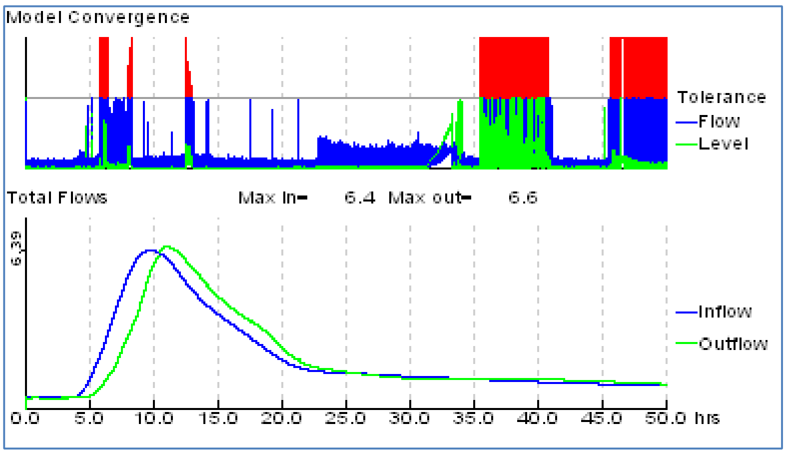

Figure 21 - 1000-year convergence plot

This instability impacts on both the stage and flow hydrographs, but does not appear to impact on peak levels and flows significantly. It is not considered to have a significant impact on the results of the hydraulic modelling and the resultant FRA conclusions.

6 MODEL LIMITATIONS

6.1 General

The hydraulic model has been constructed using the best available data, and from a range of sources. Whilst some checks have been made to confirm the suitability of the data, Hyder Consulting cannot be held responsible for errors in third party works.

The model is considered to be a best representation of reality within the current constraints of modelling; accuracy is inherently related to the quality and extent of data available.

6.2 Hydrology

There is insufficient hydrometric data available to enable validation or calibration of the model. Therefore, there is a degree of uncertainty associated with the fluvial flow estimates used in this modelling study.

7 CONCLUSIONS

- A 1D hydraulic model of the River Bure and its tributaries was constructed using ISIS;
- Design events for the 1000-year, 100-year, 100-year (climate change), and 20-year were run and flood depths mapped to quantify baseline flood risk from the River Bure and tributaries;
- The results of these runs show that the development site is predominantly in Flood Zone 1, with small areas of Flood Zones 2 and 3 around confluences of watercourses and upstream of restrictive structures;
- Sensitivity testing has shown that the model is not overly sensitive to changes in model coefficients or downstream boundary levels;
- Post-development modelling has shown that the proposed development changes flood extents through the development site to reduce the impact on the development, without causing impacting flood extents off-site;
- The new bridge structures cause increases in velocities which have the potential to cause scour;
- The findings of the study are subject to the limitations discussed in section 5.4.