

Date:	23 rd December 2020	Version:	01
To:	Nick Hale, SERT		
From:	Eric Gillies, cbec eco-engineering UK Ltd		
Project:	UK20-1057 Emm Brook, hydraulic modelling update 2020		
Subject:	Model results, setup details, and sensitivity tests		

1. INTRODUCTION

This short report describes an update to the cbec 2019 2D Emm Brook hydraulic model. Users should also consult the original report: cbec 19-1001 Emm Brook, Woosehill Fish Passage Improvements, Final Report (cbec, April 2019).

The model is updated to include all bridges, pipe crossings, improved frictions and boundary condition, but the basic topography, hydrology and design remains unchanged. Updates to model setup, inclusion of hydraulic structures, and sensitivity tests are contained in the Appendix. The appendix also contains a short note on how the model solver is equivalent to TUFLOW FV and how the model may be converted to run on that platform.

1.1 FLOOD MODELLING DESCRIBED IN THE REPORT

The following model runs were undertaken as part of the modelling update.

Table 1 Model update run matrix

RP	RUN TYPE	TYPE OF OUTPUT IN REPORT	REPORT SECTION
5	design	DEPTH MAP	RESULTS
20	design	DEPTH MAP	RESULTS
100	design	DEPTH MAP, OUTLINE AND TABULATED VELOCITIES	RESULTS
100+25%	design	DEPTH MAP	RESULTS
100+35%	design	DEPTH MAP	RESULTS
100	existing	DEPTH MAP, LEVEL DIFFERENCE AND OUTLINE	RESULTS
100+25%	existing	LEVEL DIFFERENCE MAP	RESULTS
100+35%	existing	LEVEL DIFFERENCE MAP	RESULTS
100	Unsteady	LEVEL DIFFERENCE MAP, PASS FORWARD FLOW	SENSITIVITY
100	Friction n+40%	LEVEL DIFFERENCE MAP	SENSITIVITY
100	No structures	TEXT	SENSITIVITY
100	Boundary + 0.25 m	LEVEL DIFFERENCE MAP	SENSITIVITY
100	Mesh dependency	TEXT	SENSITIVITY

Hydrological inflows are as contained in the original report, but are summarised here in Table 2. Flows were computed using WinFAP FEH pooling group methods. Please see the original report for the hydrological assessments and comparison with EA flood model flows.

Table 2 Peak model inflows (WinFAP FEH pooling group)

Return period [years]	Peak discharge at upstream end of site [m ³ /s]
5	5.36
20	7.69
100	10.84
100 CC (25%)	13.55
100 CC (35%)	14.63

2. RESULTS

2.1 INUNDATION AND FLOOD OUTLINES

Figure 1 to Figure 3 show the inundation depths for the 5, 20 and 100 year return period floods. Figure 4 and Figure 5 compare the design inundation to the existing inundation at the 100 year return period, and Figure 6 to Figure 8 shows the effect of 25% and 35% climate uplifts. Note that the inundation extents have increased as a result of adding in a more conservative downstream boundary condition (extracted from the larger EA Lower Loddon flood model).

No properties are flooded at the 100 year return period, including climate uplifts, as a result of the design, and there is no significant difference in flood outline as a result of the design. Design flood levels are lower at the upper end of the site, and so upstream flood risk is not increased.

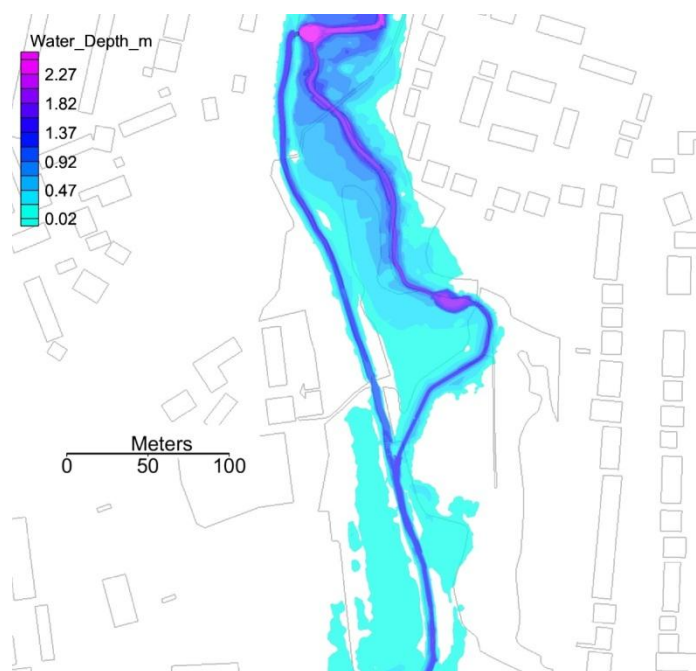


Figure 1 Design channel water depth [m], for the 5 year return period flow. Contains OS Data, Crown copyright.

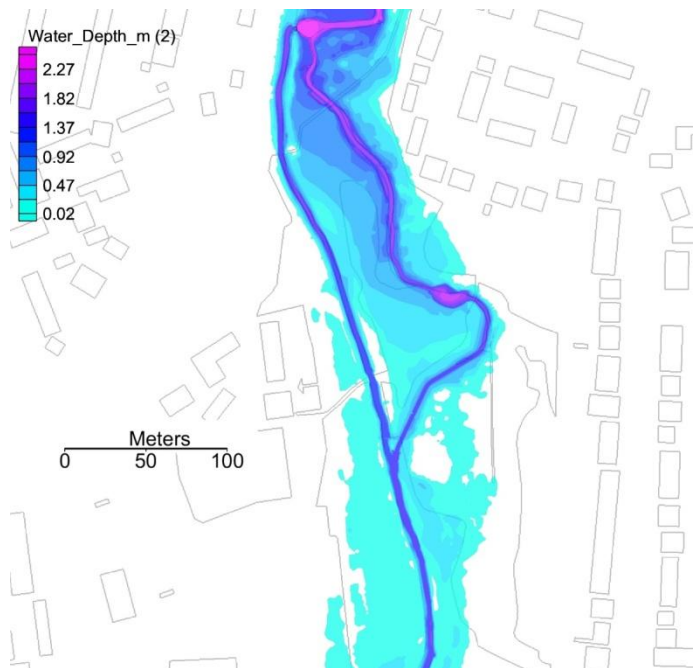


Figure 2 Design channel water depth [m] for the 20 year return period flood. Contains OS Data, Crown copyright.

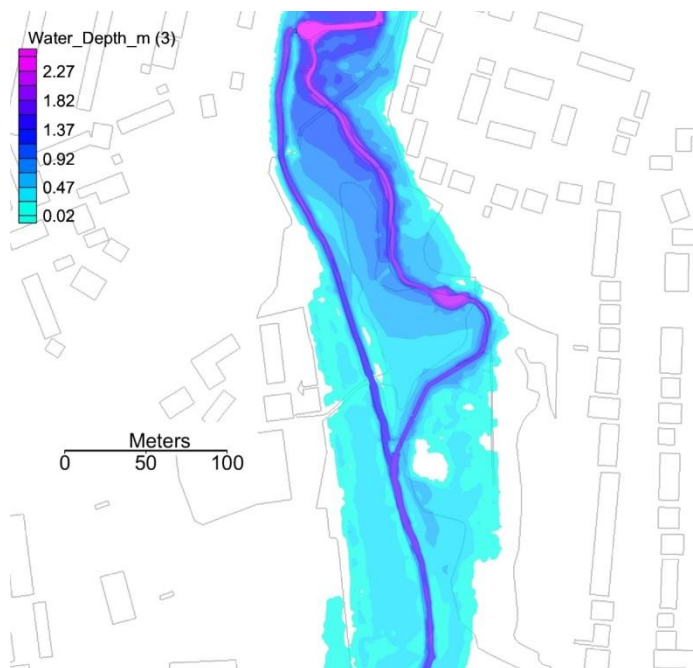


Figure 3 Design channel water depth [m] for the 100 year return period flood. No properties are inundated within the design area. Contains OS Data, Crown copyright.

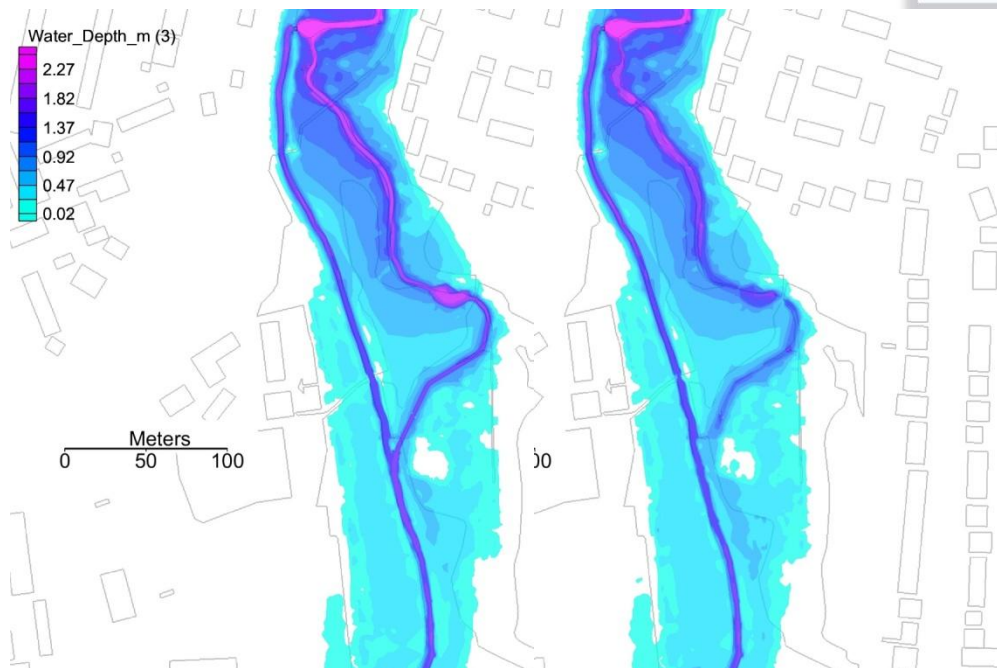


Figure 4 Comparison between inundation pattern, 100 year return period flood. Left frame design condition; right frame existing condition. Contains OS Data, Crown copyright.



Figure 5 Comparative 100 year return period flood outline. Black line, Lower Loddon EA flood model (2009). Red line, existing condition 2D model. Green line, design condition 2D model. There is no difference to property between the existing and design model results within the design area. Contains OS Data, Crown copyright.

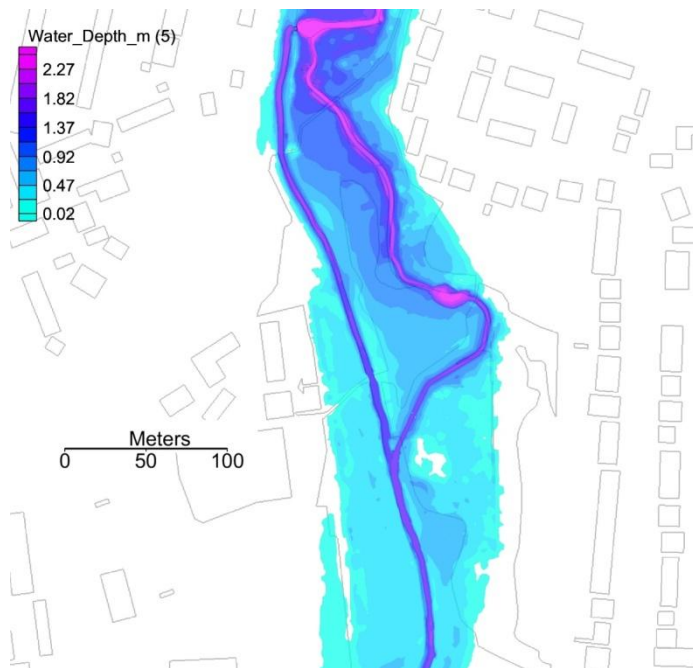


Figure 6 Design inundation for the 100 year return period flood including an 25% uplift for climate change. No properties are inundated within the design area. Contains OS Data, Crown copyright.

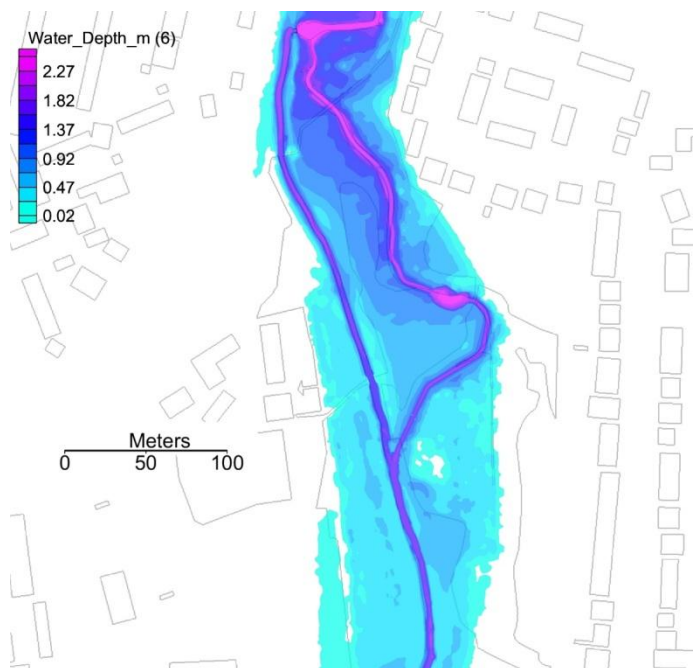


Figure 7 Design inundation for the 100 year return period flood including an 35% uplift for climate change. Contains OS Data, Crown copyright.

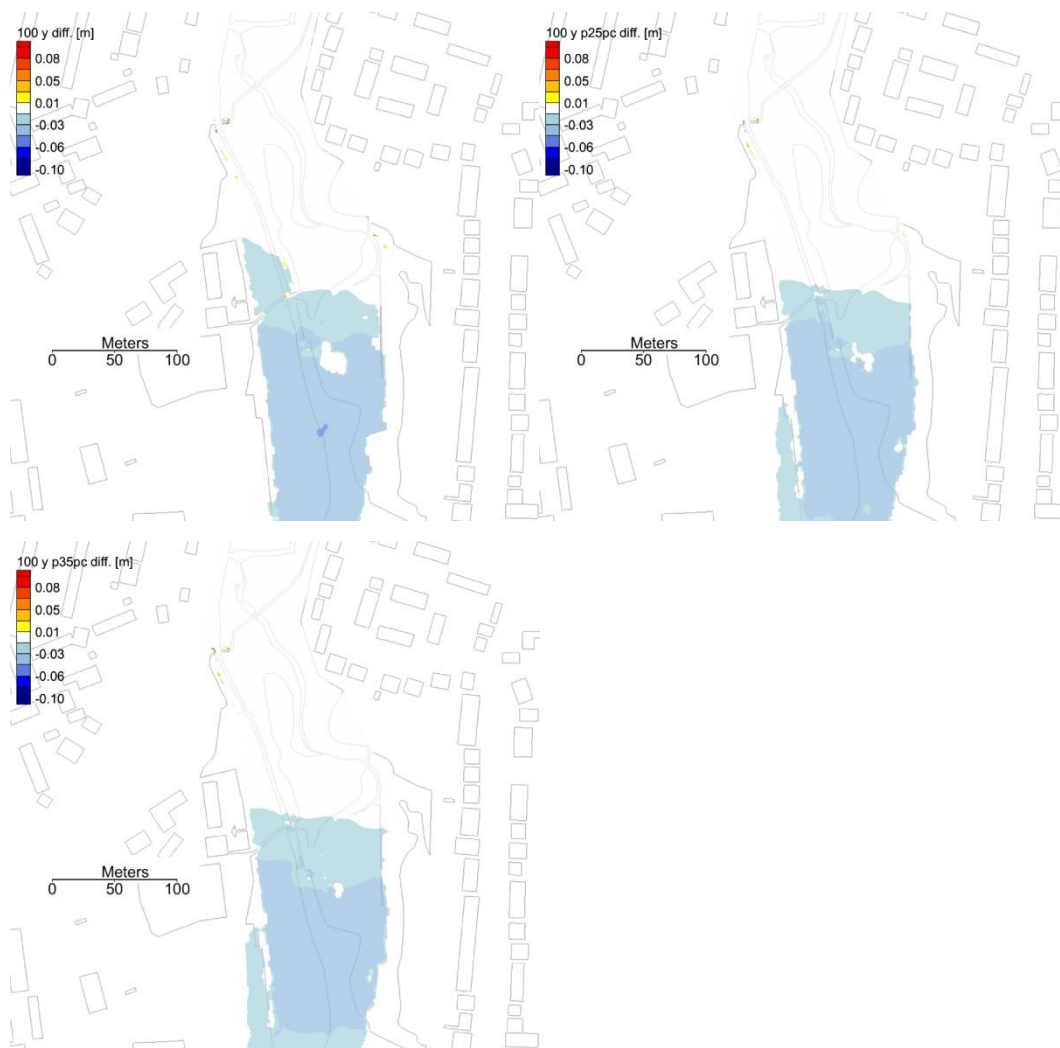


Figure 8 Water level difference maps for 100 year (top left), 100 year plus 25% climate uplift (top right) and 100 year plus 35% uplift (bottom left). Shown is design level minus existing level: the design has a marginally lower level at the upper end of the site. Contains OS Data, Crown copyright.

2.2 PASS FORWARD FLOW/DOWNSTREAM FLOOD RISK

Downstream flood risk is assessed by computing the hydrograph at the downstream end of the site, and comparing existing and design conditions. Figure 9 compares the hydrograph at the model exit boundary computed using unsteady, time accurate modelling of the 100 year flood. There is no increase to pass forward flow during the hydrograph and so no increase to downstream flood risk.

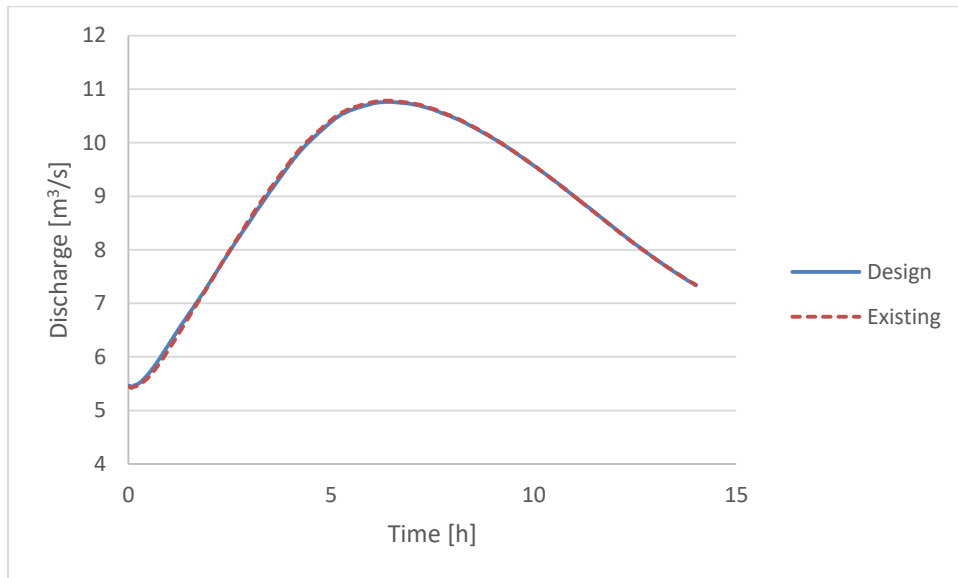


Figure 9 Pass forward hydrographs for design and existing conditions, 100 year flood.

2.3 PEAK VELOCITIES AT BRIDGE STRUCTURES

Figure 10 maps the peak velocities during the 100 year hydrograph. Velocities at the bridges are tabulated in Table 3.

Velocities at each of the bridge crossings are reasonable and all less than 1.5 m/s.

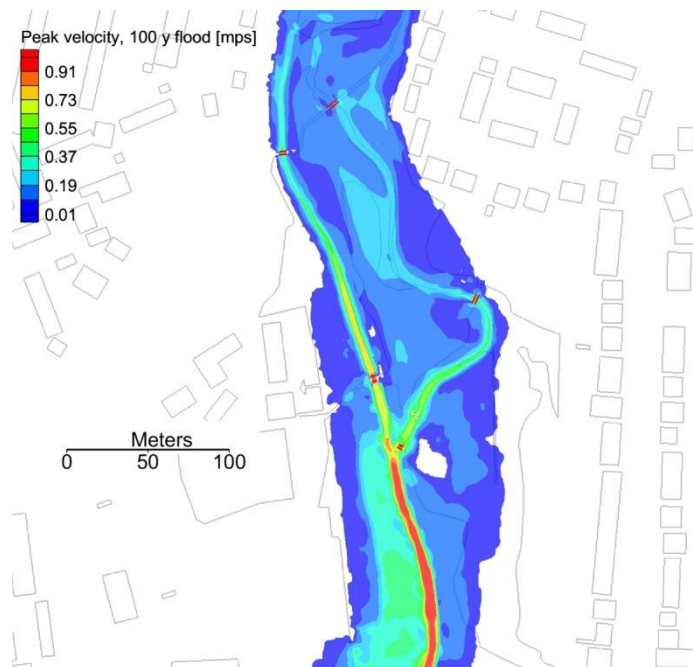


Figure 10 Peak velocity during the 100 year flood hydrograph (unsteady results). Contains OS Data, Crown copyright.

Table 3 Velocity peaks at key structures during the 100 year return period hydrograph.

Bridge Name	Peak 100 y RP velocity [m/s]
Kingfisher	1.06
Un-named, existing channel north	0.40
South Bridge/control	1.08
Upstream of pond	0.46
Northern bridge, design channel	0.33
Between Alder tree trunks	0.86

APPENDIX A

HYDRAULIC MODELLING SET UP

A.1 Hydraulic model solver

The basic model set-up and solver (SRH-2D v 3.2) for the 2020 model update is the same as described in cbec report 19-1001 Emm Brook Woosehill Fish Passage Improvements, (cbec, April 2019). However, since that report, cbec, and independent consultants, have undertaken additional benchmarking of the SRH-2D code against the TUFLOW FV code more familiar to the environment agency. TUFLOW FV is one of the fully 2D codes that have been benchmarked by the Environment Agency¹, and it is applicable across the full range of Environment Agency flood risk modelling requirements.

Table 4 Comparison of solver capabilities, SRH-2D, Mike 21 and TUFLOW FV (source US FHWA)

	Basic Capabilities					Hydraulic Structures ¹					Other Capabilities			
	Unstructured Mesh	Conservation of Mass	Conserves Momentum	Supercritical Flow	Unsteady Flow	Bridge Piers ²	Bridge Pressure Flow	Weirs ³	Culverts ⁴	Gates	Wind Shear	Rainfall	Infiltration & Exfiltration	Mobile Bed
Mike21C	X	X	X	X	X									X
HEC-RAS-2D	X	X	X ⁷	X	X			X	X	X		X		
RiverFlow2D	X	X	X	X	X	X	X		X		X	X	X	X
SRH-2D	X	X	X	X	X	X	X	X	X	X		X	X	X
TELEMAC	X	X	X	X	X	X					X			X
TUFLOW- (FV & Classic)	X	X	X	X	X	X	X	X	X	X	X	X	X	X

SRH-2D has all of the capabilities of TUFLOW, save for wind shear modelling. Not only that, but the model meshes for SRH-2D and TUFLOW FV are both generic '.2dm' format (so the meshes are interchangeable, and both set up using Aquaveo SMS v13) and there are only minor differences in script file set up to run these meshes in either SRH-2D or TUFLOW.

Figure 11 shows a comparison between TUFLOW's computation of meandering river levels compared to SRH-2D (a benchmark test undertaken by cbec). The results have no significant differences.

¹ Benchmarking the latest generation of 2D hydraulic modelling packages. Report SC120002, Environment Agency 2013.

Eden Vale Young (EVY) were commissioned in 2020 to compare the performance of SRH-2D against TUFLOW FV for a proposed re-alignment of the channel and floodplain of the River Rom in Essex². Their independent analysis stated that the water levels in the location of the proposed restoration measures computed by SRH-2D and TUFLOW FV were in very good agreement.

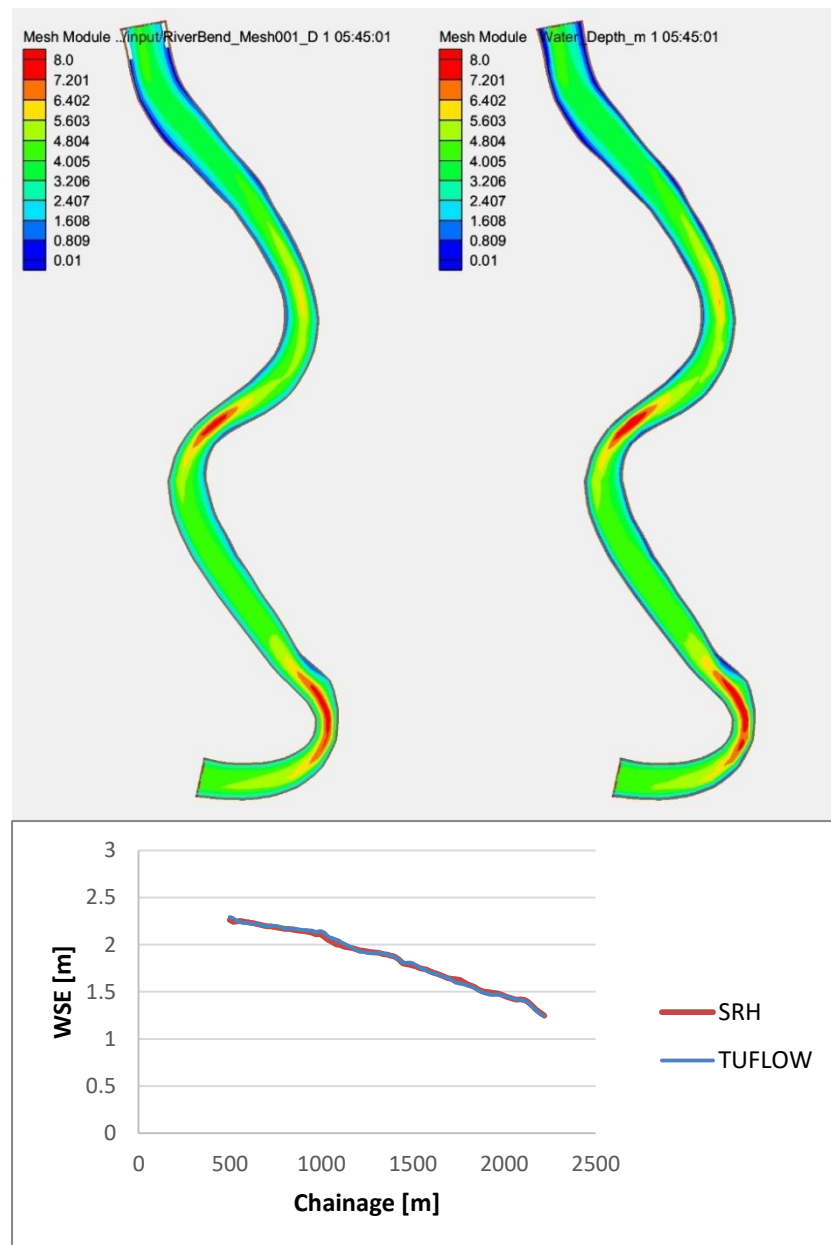


Figure 11 Map of water depth for "River Bend" case: TUFLOW FV top left and SRH-2D top right, with centreline water levels plotted at bottom.

² River Rom Restoration modelling review, Eden Vale Young, September 2020

A.2 Updated frictions

The initial 2019 hydraulic model used a general floodplain friction of $n = 0.06$. As part of the 2020 model update, the surface frictions for the model were refined as shown in the map and table below. Friction values were obtained using experience and the UK CES roughness advisor software.

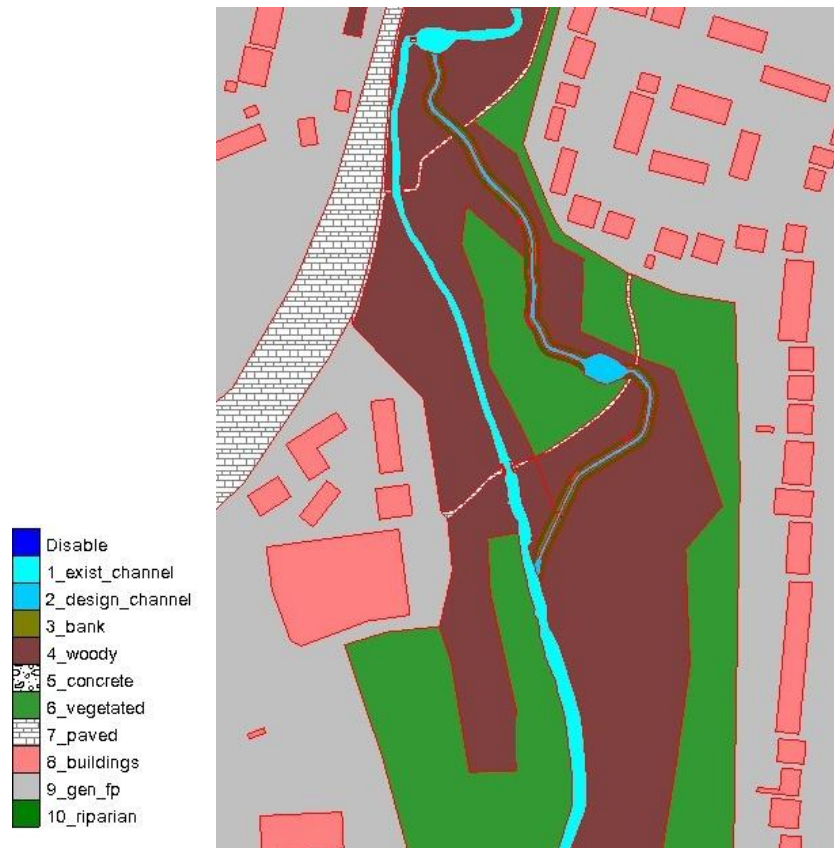


Figure 12 Updated friction map, 2020 model

Table 5 Manning frictions used in the baseline and design 2D 2020 model update (source CES roughness advisor)

Material	Zone	Manning n
1	Existing channel	0.03
2	Design channel	0.025
3	Design banks	0.036
4	Wooded	0.1
5	Concrete	0.025
6	Grass	0.035
7	Paved	0.02
8	Buildings	0.3
9	Urban floodplain	0.032
10	Riparian	0.055

There is one point 30 m downstream of the design channel tie in where the flow is constricted by two prominent Alder trees. The cells associated with these tree trunks are disabled in the model (i.e. they are set as solid obstacles) rather than assigning a friction. Velocity therefore more realistically increases between the trees, rather than reduces as would be the case for an increase in friction.

A.3 Updated boundary conditions

One of the difficulties of setting upstream boundary conditions is the unknown flood flow routing upstream of the site (i.e. some of the flow is in the channel, but other flow routes may enter the domain over the floodplain at the upstream end).

All sub-critical hydraulic models require the downstream water level, for a specific discharge. For the downstream boundary, and in the absence of other data, initial modelling (see³) used a normal depth assumption with friction 0.06 and slope of 0.002 to set the water level.

However, as part of the 2020 model update, the Lower River Loddon flood model held by the Environment Agency was used to set upstream flow extents and downstream water level more robustly, with less uncertainty.

The flow extents and levels for the 5, 20, 100 year and 100 year plus 25% climate change return periods, and their corresponding discharges, were extracted from the larger 1D ESTRY/2D TUFLOW Lower Loddon flood model. The flood extents are shown for the design area (blue line polygon) in Figure 13.

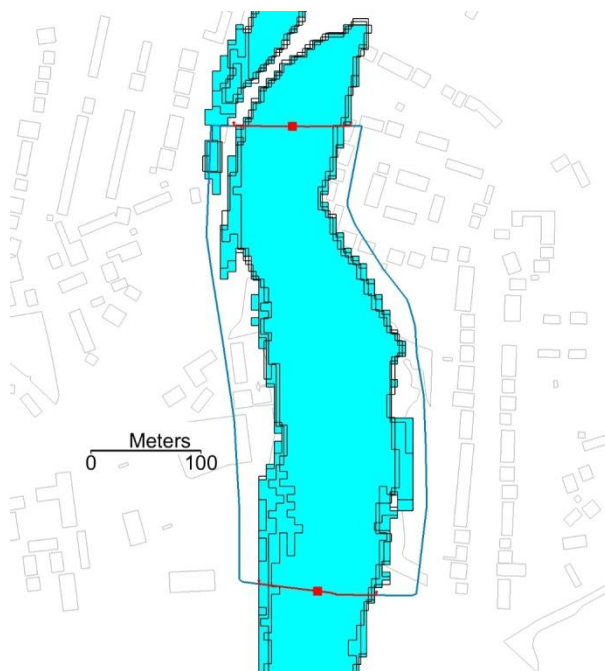


Figure 13 Design 2D model domain: blue line. Filled blue polygon and black polygons: flood extents extracted from the Lower Loddon 1D ESTRY/2D TUFLOW model (EA). Red lines model inlet and outlet nodestrings. Map contains OS shapefile data, Crown Copyright 2019.

³ cbec report 19-1001 EMM BROOK WOOSHILL FISH PASSAGE IMPROVEMENTS, cbec 2019

The downstream boundary condition was developed into a rating curve (Table 6):

Table 6 Rating curve for 2D design model boundary extracted from Lower Loddon flood model (EA)

Return period [years]	Discharge [m ³ /s]	Water level at 2D model boundary [mAOD]
5	6.18	45.15
10	7.44	45.27
100	10.78	45.45
100CC	14.55	45.58

A.4 Structures

There are five bridge structures on the Emm Brook: two on the existing channel and three design bridges on the design channel. See the schematic in Figure 14. The bridges were modelled in 2D with pressure flow (see⁴) when water levels reach the soffit, combined with broad crested weir flow (a 1D unit) once levels overtop the structure deck/parapet. Additional manning friction of $n = 0.1$ was added to account for the friction of the soffit, additional form losses, and the effect of railings on the structure. This method has the advantage of computing full momentum transfer and velocities through the bridge for both normal flow and pressure flow conditions. Velocity outputs were a specified deliverable for the work.

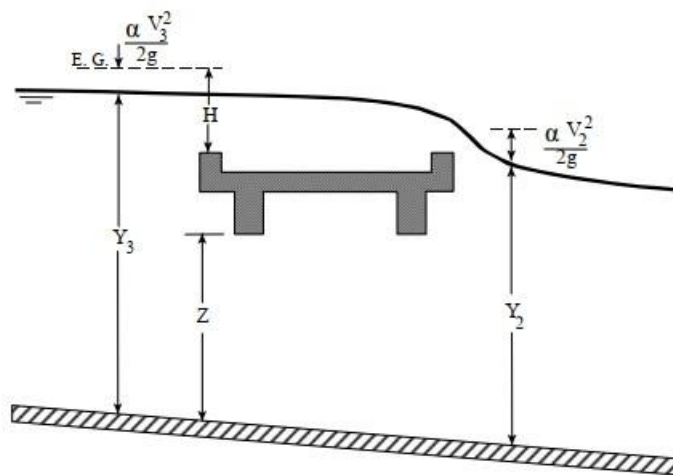


Figure 14 Schematic of bridge model. There is pressure flow (a cap on water levels) under the soffit, with extra manning friction at soffit, and broad crested weir flow over the structure. (source HEC-RAS training manual).

In addition to the bridge structures, there are three pipe crossings modelled. Each pipe crossing is modelled as a solid obstruction with a drag coefficient, and the model adds the shear stress

$$\tau_{\text{total}} = \tau_{\text{bed}} + \frac{1}{2} \rho (U^2 + V^2) C_D \frac{A_{\text{wet}}}{A_{\text{cells}}}$$

to the cells containing the obstruction. Free surface drag coefficients tend to be higher than the infinite flow drag coefficient (≈ 0.8 to 1.3 for a 2D cylinder), and there is some evidence that they approach 2.0 where the obstruction is a significant proportion of the total flow area. To provide conservative results, $C_D = 2.0$ was used in the model. This method has the advantage that no changes to resistance are computed for water levels below the pipe soffit.

⁴ D. Robinson et al, TWO-DIMENSIONAL HYDRAULIC MODELING FOR HIGHWAYS IN THE RIVER ENVIRONMENT REFERENCE DOCUMENT, FHWA-HIF-19-061, October 2019

The following are the specifications of each bridge structure and pipe crossing in the model. Note that the structure widths are the widths of the pressure unit and overtop weir.

Kingfisher Bridge

479873E,

169056N.



Figure 15 Kingfisher bridge, existing channel. Note that the pipe obstruction beneath the bridge is also modelled.

Data as appearing in model (all dimensions in [m]):

SOFFIT	45.45
TOP OF DECK (PARAPET WALL)	45.87
SOFFIT SHAPE	FLAT
SOFFIT FRICTION	0.10
WIDTH	3.38
OVERTOP WEIR TYPE	BROAD CRESTED

Pipe crossing within structure (all dimensions in [m]):

SOFFIT	45.1 (estimated)
PIPE DIAMETER	0.15 (estimated)
DRAG COEFFICIENT	2.0

Un-named crossing, existing channel

479815E, 169195N



Figure 16 Un-named bridge across existing channel

Data as appearing in model (all dimensions in [m]):

SOFFIT	45.50
TOP OF DECK	46.20
SOFFIT SHAPE	FLAT
SOFFIT FRICTION	0.10
WIDTH	4.45
OVERTOP WEIR TYPE	BROAD CRESTED

Bridge and baffle control at upstream end of design tie in

479889E, 169015N

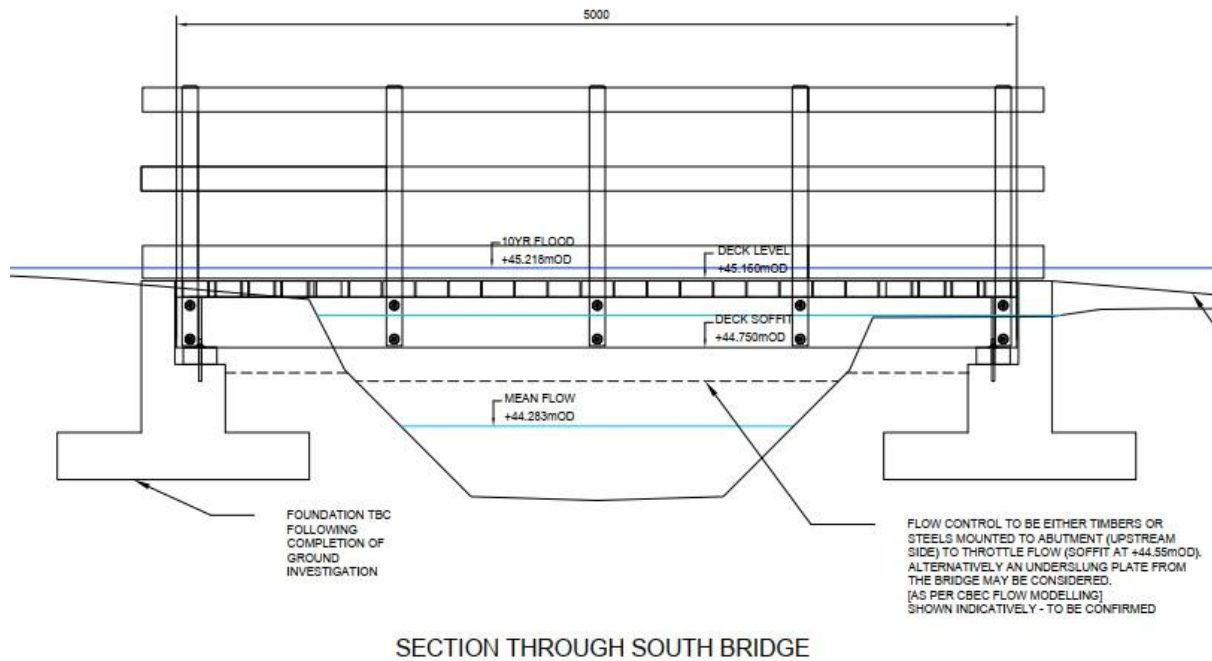


Figure 17 Proposed bridge/control at upstream end of design channel. Extract from drawing A120099-TGEE-ZZ-XX-DR-C-0005 P02 (T. Gee and Partners)

Data as appearing in model (all dimensions in [m]):

SOFFIT	44.55 (baffle)
TOP OF DECK	45.16
SOFFIT SHAPE	FLAT
SOFFIT FRICTION	0.10
WIDTH	3.0
OVERTOP WEIR TYPE	BROAD CRESTED

Immediately downstream of this structure is a suspended foul sewer pipe (all dimensions in [m]):

SOFFIT	44.35 (estimated)
PIPE DIAMETER	0.15
DRAW COEFFICIENT	2.0

Bridge upstream of design pond

479935E, 169105N

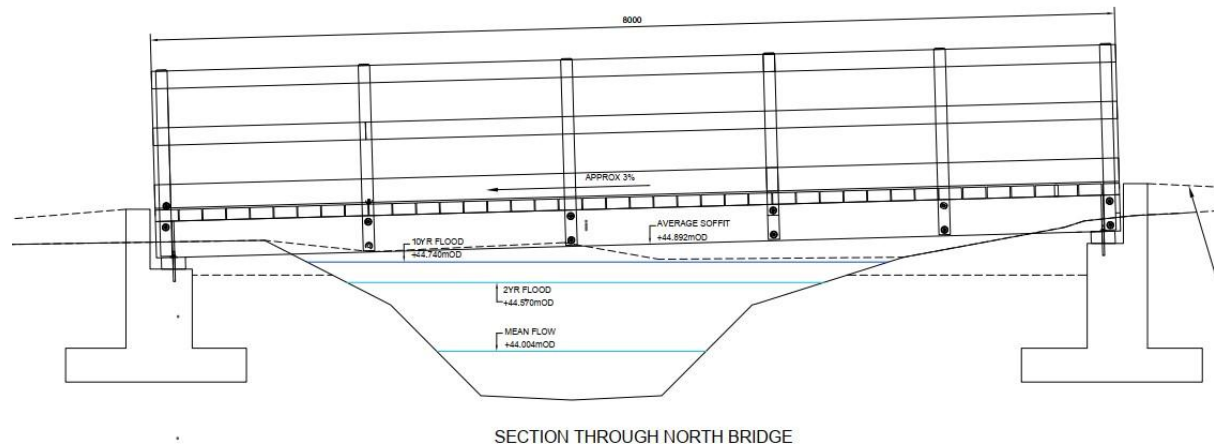


Figure 18 Proposed bridge upstream of pond. Extract from drawing A120099-TGEE-ZZ-XX-DR-C-0005 P02 (T. Gee and Partners). Bridge is modelled with average soffit and deck height.

Data as appearing in model (all dimensions in [m]):

SOFFIT	44.892 (average)
TOP OF DECK	45.30 (average)
SOFFIT SHAPE	FLAT
SOFFIT FRICTION	0.10
WIDTH	6.23
OVERTOP WEIR TYPE	BROAD CRESTED

Pipe crossing and concrete supports

479877E, 169188N



Figure 19 Pipe crossing

Pipe crossing and concrete supports (all dimensions in [m]):

SOFFIT	44.17
PIPE DIAMETER	0.15
DRAG COEFFICIENT	2.0

North bridge (TBC)

479848E, 169225N

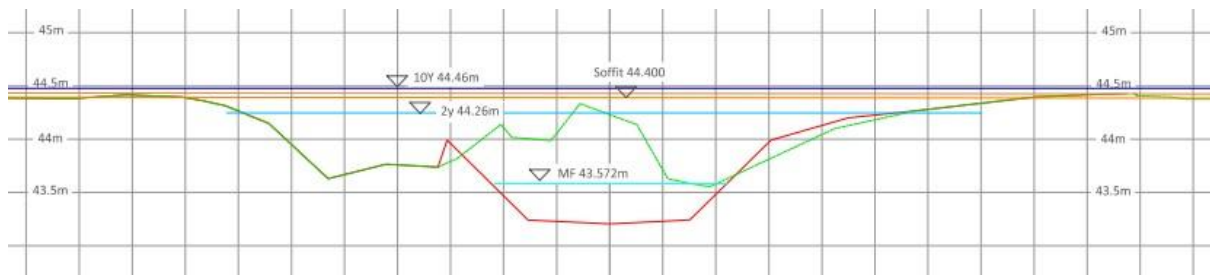


Figure 20 North bridge (cbec specification to architect, from drawing Emm Brook - Design - 09 - BRIDGE 2 - 10052019). At the time of modelling, these were the only available data.

Data as appearing in model (all dimensions in [m]):

SOFFIT	44.40 (specified)
TOP OF DECK	45.30 (assumed)
SOFFIT SHAPE	FLAT
SOFFIT FRICTION	0.10
WIDTH	7.6
OVERTOP WEIR TYPE	BROAD CRESTED

Weirs

There are two weir structures in the model, both modelled using topography, directly in 2D. This method automatically computes drowned flow and there is no need to specify weir coefficients.

Foul sewer pipe crest

479881E, 169020N



Figure 21 Weir formed from embedded foul sewer pipe.

There were 10 mesh cells laterally across this structure with a longitudinal spacing of 0.25 m over the crest.

Weir and pool at downstream end of the site

479825E, 169272N



Figure 22 Weir and pool at downstream end of the site.

There were five mesh cells fitted laterally across this structure and five over the crest and glaxis.

A.5 Mass loss

The mass loss in all model runs was low (< 1% of inflow). Mass loss is tabulated Table 7 and is acceptable for all runs.

Table 7 Computed mass loss, 2D model

RP		FLOWS	MASS LOSS
5	inflow	5.360	
	outflow	5.351	0.17%
20	inflow	7.690	
	outflow	7.670	0.26%
100	inflow	10.840	
	outflow	10.838	0.01%
100CC	inflow	13.550	
	outflow	13.548	0.01%

APPENDIX B

MODEL SENSITIVITY TESTS

All sensitivity testing was undertaken using the 100 year return period design model.

B.1 Flow sensitivity

Sensitivity to flow may be assessed by comparing the 100 year design water levels with the 100 year plus 25% flow water levels as shown in Figure 23. Water levels increase by approximately 0.09 m.

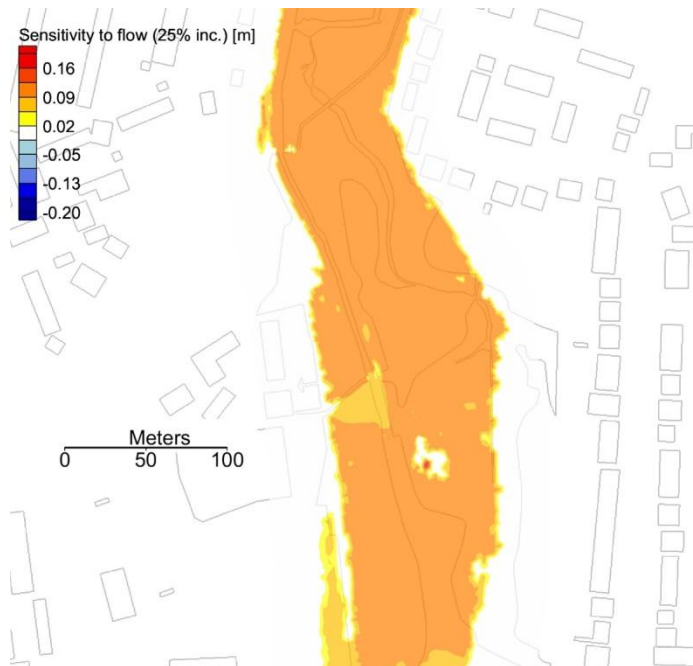


Figure 23 Sensitivity of water levels to an increase in 100 year flow by 25%. Map contains OS data, Crown copyright.

B.2 Friction sensitivity

Sensitivity to friction may be assessed by considering a 40% increase in Manning value over and above baseline, as shown in Figure 24. The model is most sensitive to uncertainty in friction at the upstream end - the insensitivity to friction at the downstream end is likely due to the fact that water levels here are dominated by the boundary condition value.

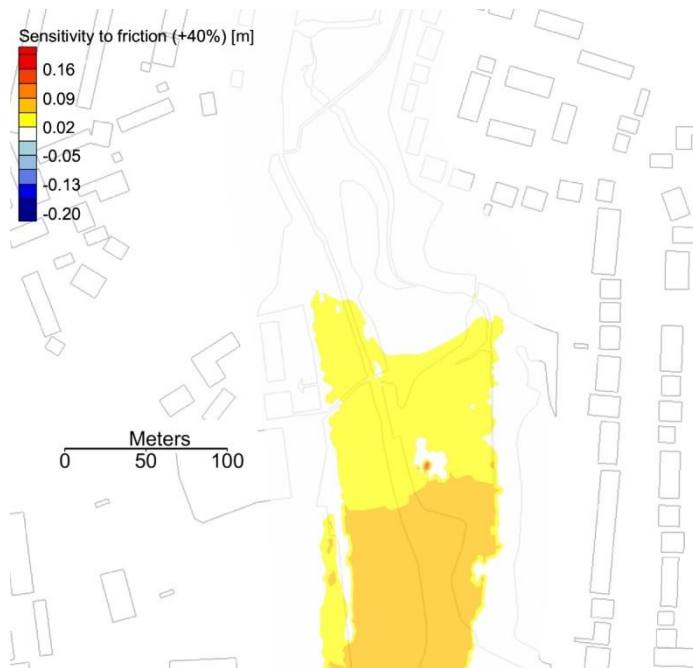


Figure 24 Sensitivity of the model to an increase in Manning friction by 40%.

B.3 Boundary condition sensitivity

Sensitivity to boundary condition is assessed by adjusting the downstream rating curve. For this test, the downstream boundary level was raised 0.25 m above baseline. Levels at the bottom of the model therefore increase by 0.25 m. Levels at the model inlet increase by 0.13 m, showing that the boundary condition increase affects the entire domain, as shown in Figure 25. An uncertainty of the downstream boundary condition by +0.1 m is not likely to affect the upstream end of the model.

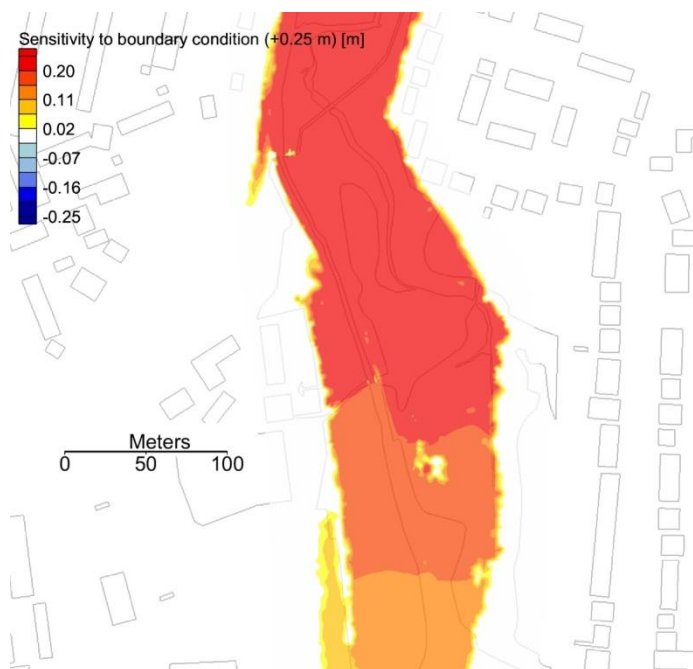


Figure 25 Sensitivity of the model to a 0.25 m increase in downstream boundary level.

B.4 Sensitivity to hydrograph

Model runs were undertaken for steady flow, as there is little flood storage at the site. However to compare the effects of steady/unsteady computations a hydrograph was computed using ReFH v2.2 software and catchment descriptors for the site. The peak water level for the 100 year unsteady flow and the 100 year steady flow is shown in Figure 26. The effects for this site are small, confirming that steady state computation of peak flows and flood extents is acceptable. There are only minor level differences immediately upstream of two of the bridge structures.

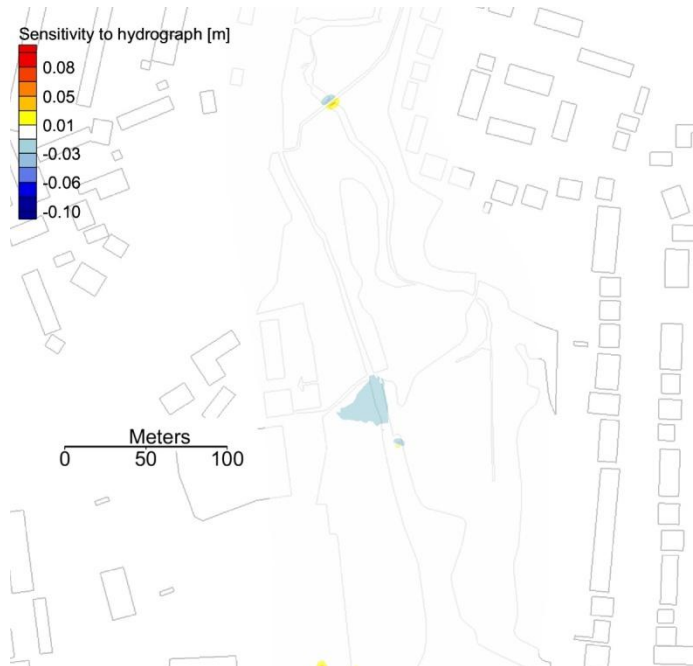


Figure 26 Sensitivity of water levels to hydrograph. Plotted is the peak water level for an unsteady computation minus that for a steady computation.

B.5 Sensitivity to structure modelling and resolution

To determine the sensitivity of the flood results to how the bridge and pipe structures are modelled, the model was also run without these structures in place. The flow extent is wide (≈ 100 m) and the structures themselves are narrow (≈ 7 m max) so are easily bypassed by water- bridge afflux is therefore quite small. The peak afflux is upstream of the South Bridge/control structure and is 4 cm for the 100 year flood, and is 2 cm upstream of the Kingfisher bridge. The afflux is less than 1 cm upstream of the other structures.

Mesh dependency was also studied- resolution was increased by a factor of two around all structures, but there was no appreciable difference in the computed water levels.