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On the Dynamic Modelling of the Flooding Phenomena in Boscastle

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Date: 27th May 2020

I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.

Signed 林叶侠 date 27 / 05 / 2020

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Abstract

Boscastle is situated on the north Cornwall coast, at the confluence of the Rivers Jordan and Valency. It is very prone to flooding. The 2004 flood was one of the most devastating floods the UK has experienced in the past few decades. It is a case of great interest in terms of research on flooding and will hence be used as the case study in this project.

For most floods, the horizontal length scale is much greater than the vertical length scale, and they can thus be mathematically described by Shallow Water Equations(SWEs).

The Alternating Direction Implicit scheme is the more conventional flood modelling scheme. It is widely adopted in commercial software, examples include Flood Modeller Pro and HEC-RAS. However, it does not allow shock capturing and spurious oscillations which exist in numerical predictions of trans-critical flows. Numerical treatment such as tuning the Boussinesq coefficient is required to stabilise computation in the cases of trans-critical flows. Usually, the tuning consists in reducing the β to be below 1.

However, the value of β should be slightly above 1.0 for all most all cases of flooding. By running simulations with a range of β values, therefore, this project aims to assess the degree of inaccuracy associated with the common practice of reducing the β value when using the commercial flood modelling software. Tests are run using the more

robust, shock-capturing TCD-MacCormack scheme.

First, tests were run to compare modelled and surveyed water levels at the 37 gauging points for the 2004 flood event. Next, tests were run for five different riverbed conditions under four flooding scenarios of increasing severity (1-in-25, 1-in-50, 1-in-75, and 1-in-100), for which graphs of the centreline water level estimations are plotted.

It was found that there is a positive correlation between the value of β and the simulation results of water levels, both for the 37 gauge points as well as centreline comparisons. In addition, at higher inputting flow rates, the influence of varying beta increases. This is shown by the greater spread of green plots as scenario changes from 1-in-25 (least rare among the four cases) to 1-in-100 (rarest among the four cases).

Therefore, imagine a water engineer attempting to make predictions on rises in water level during future floods. If he or she applies an excessively low β value in order to stabilise the simulations based on the ADI scheme, the results would be underestimated. If the discrepancy becomes large enough, it could potentially lead to inadequate flood defence schemes being designed. Even worse, the error in estimation grows as input flow rates increase, that is to say, when predictions are being made for more severe floods.

In fact, the final part of analysis presents plots of 3D views showing contours of water levels for 4 β values 1.6, 1.0, 0.8, and 0.6 for the RCLW scheme in the 1-in-75-a-year flooding scenario. RCLW scheme was the final mitigation implemented in Boscastle after the 2004 flood, it was designed to protect against a 1-in-75-a-year flood. It was reconfirmed through clear visualisation that we would indeed underestimate the water levels if β values were reduced. At β to be 0.6, the underestimation is such that the 3D contour plots do not show any overflow to the river bank. However, at $\beta=1.0$, which should give the most accurate prediction, it was shown that water would overflow into the river bank and hence the RCLW flood mitigation scheme would be insufficient in the 1-in-75 case.

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1 Introduction

1.1 Project background

Boscastle is situated on the north Cornwall coast, at the confluence of the Rivers Jordan and Valency, which outflow eventually to the Celtic Sea. In addition, the narrow valley geological conditions would mean increased risks as water is less able to spread and would drain rapidly through the steep valleys in case of a storm. Due to these hydro-geological conditions, Boscastle is very prone to flooding. See Figure 1 for an aerial view of the region.



Figure 1: Aerial View of Boscastle [1]

Notably, the 16 August 2004 flood is the most severe recorded, being rarer than a 1-in-200 year event. [2] During the flood, significant overland flow took place and there was flooding in both the minor watercourses and the main rivers in the area. As a result, damage was experienced by the highways, bridges, road surfaces as well as the local utility network. [1]

The likelihood of this type of flooding event occurring more frequently in the future is of concern to urban drainage engineers, policy-makers, and inhabitants in flood-prone regions. Changes in land use and the spread of impermeable surfaces, and planning policy for new developments are factors that will influence a region's exposure to flooding. [3]

1.2 Project aim

Given that future precipitation will increase both the flooding frequency and duration, the need for well-planned drainage systems and protocols in case of emergency becomes increasingly important. In this regard, rainfall models in both temporal and spatial resolution would be the most appropriate for use in urban drainage model simulations. [4]

There were various mitigation measures proposed by Halcrow in association with the Environmental Agency. There are high investment costs associated with the implementation of such measures. Hence, computational modelling of flood mitigation measures, under flooding scenarios of interest, has the potential to offer a relatively quick and accessible method of testing the suitability before the actual implementation.

The value of β should be around 1.0 for most cases of flooding. However, most commercial flood simulation software is based on the ADI scheme, often require tuning the β value below 1.0 in order to ensure stable running of simulations. By running simulations with a range of β values, the aim of this project is to assess the degree of inaccuracy associated with this practice.

Due to the high susceptibility to future floods, Boscastle is a case of great interest and will be used throughout this project.

1.3 Literature review

Olofsson (2007) emphasised the importance of rainfall models in both temporal and spatial resolution for urban drainage model simulations. The application of climate data in urban drainage models would contribute greatly to the prediction of future flooding disasters.

Liang, Falconer and Lin (2014) made a comparison between TVD-MacCormack and ADI solvers of shallow water equations which are used to describe many flooding events, using dyke-break as an example. It is demonstrated that the ADI model is unable to predict trans-critical flows correctly, and artificial viscosity has to be introduced to remove spurious oscillations. This is often done by tuning the Boussinesq coefficient to eliminate the fluctuations and increase computational stability.

Yang, Liang and Xiao (2018) analysed the consequences of modifying the β value in modelling rapid flows, assessing a combination of floods and riverbeds. It was found that for

steady flow over the frictional bed in particular, the amplitude of the spurious oscillations decreases with the decrease of the Boussinesq coefficient, whereas the steady water surface elevation reduces with a smaller β value. However, stable results were only achieved for small β values of 0.0, 0.4 and 0.6 under the ADI model.

2 Computational Model

2.1 Shallow water equations

The shallow water equations (SWEs) are a set of partial differential equations that describe the flow below a pressure surface in a fluid. For most floods, the horizontal length scale is much greater than the vertical length scale, and they can thus be mathematically described by SWEs.

During the derivation from the depth-integration of the three-dimensional incompressible Navier–Stokes equations, it is assumed that the velocity in the vertical cross-section direction is uniform. Hence, the Boussinesq Coefficient (β), or the moment correction factor, is added to correct for this assumption by quantifying the non-uniformity of moment over depth. Ignoring the Coriolis, viscous and wind forces, the SWEs are expressed as:

$$\frac{\partial \eta}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \quad (1)$$

$$\begin{aligned} \frac{\partial q_x}{\partial t} + \frac{\partial(\beta q_x^2/H)}{\partial x} + \frac{\partial(\beta q_x q_y/H)}{\partial y} = & -gH \frac{\partial \eta}{\partial x} - \frac{g q_x \sqrt{q_x^2 + q_y^2}}{H^2 C^2} \\ & + \nu \left[2 \frac{\partial^2 q_x}{\partial x^2} + \frac{\partial^2 q_x}{\partial y^2} + \frac{\partial^2 q_y}{\partial x \partial y} \right] \end{aligned} \quad (2)$$

$$\begin{aligned} \frac{\partial q_y}{\partial t} + \frac{\partial(\beta q_y^2/H)}{\partial y} + \frac{\partial(\beta q_x q_y/H)}{\partial x} = & -gH \frac{\partial \eta}{\partial y} - \frac{g q_y \sqrt{q_x^2 + q_y^2}}{H^2 C^2} \\ & + \nu \left[\frac{\partial^2 q_y}{\partial x^2} + 2 \frac{\partial^2 q_y}{\partial y^2} + \frac{\partial^2 q_x}{\partial x \partial y} \right] \end{aligned} \quad (3)$$

where t represents time; η is the water surface elevation above the still water datum; q_x and q_y are the volumetric discharge components per unit width in the x and y directions respectively; $H(=h+\eta)$ is the total water column depth, in which h is the depth below the still water datum; g is the gravitational acceleration; C is the Chézy roughness coefficient determined from the Manning formula; ν is the viscosity coefficient. Finally, β is defined as the integral of momentum along the depth divided by momentum from depth-averaged

velocity(U), it quantifies the momentum effect of the non-uniform velocity distribution over the depth. See equation (4) below, where u is a function of the vertical coordinate z :

$$\beta = \frac{\int_{z_b}^{\eta} u^2 dz}{U^2 H} \quad (4)$$

According to Goldstein (1938) [5], for a logarithmic velocity profile, the momentum correction factor can be expressed as below:

$$\beta = \left(1 + \frac{g}{C^2 \kappa^2}\right) \quad (5)$$

The value of β should be slightly above 1 for most flow cases.

2.2 The Alternating Direction Implicit (ADI) Scheme

With the appropriate initial conditions and boundary conditions, these nonlinear equations can be combined into matrices and solved numerically. The Alternating Direction Implicit is an iterative implicit method that can be implemented on structured grids. It is widely applicable in solving matrices arising in system and control, it is also a classic method used for modeling heat conduction and solving the diffusion equation in two or more dimensions.

It is also a conventional scheme used in flood modelling and is widely adopted in most commercial software, such as Flood Modeller Pro or HEC-RAS. However, it does not allow shock capturing and spurious oscillations exist in numerical predictions of trans-critical flows. In addition, the ADI scheme can become unreliable when the Boussinesq coefficient becomes too high for it to yield stabilised results. See Figure 2 below for the simulation of the trans-critical flows during a dam-break using the ADI scheme, note the spuriousness of the result due to the trans-critical nature of the flow. [6]

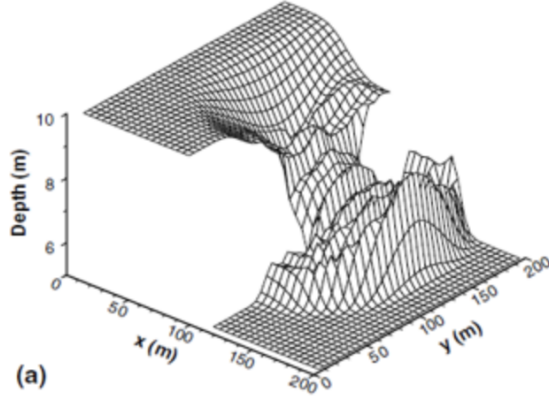


Figure 2: Dam-break simulation using an ADI model [6]

Numerical treatment such as tuning the Boussinesq coefficient is required to stabilise computation in the cases of trans-critical flows. This process of tuning can affect prediction accuracy. Physically, the value of β should never be smaller than unity according to the Eq.(5), however, in these numerical treatments β is often set at values much smaller than 1. [6]

2.3 The TVD-MacCormack scheme

On the other hand, there is the total variation diminishing modification on the conventional MacCormack (TVD-MacCormack), which is not so widely applied as the ADI scheme.

The MacCormack method is a discretization scheme in computational fluid dynamics, for the numerical solution of hyperbolic partial differential equations. Total variation diminishing (TVD) is a property of certain discretization schemes, it is employed to capture sharper shock predictions. The TVD-MacCormack scheme provides increased accuracy of simulation results as it allows shock-capturing. At the same time it is still relatively simple and efficient, and would not incur excessively high computational costs. [6]

In this scheme, the 2-D equations are solved as two orthogonal 1-D problems based on the operator-splitting technique. [6, 7] See Figure [3] below for the simulation for the same dam-break using the TVD-MacCormack scheme, the result is much more smooth and stabilised.

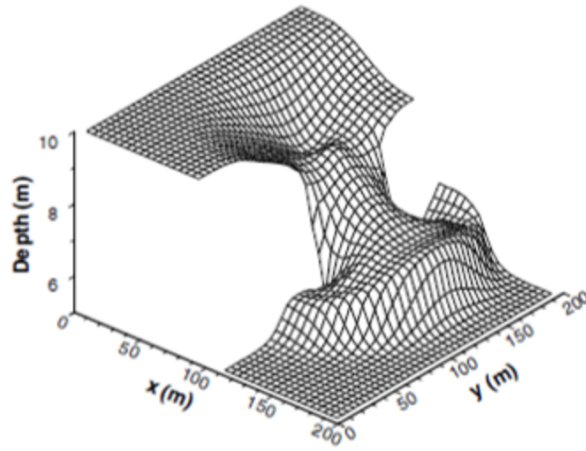


Figure 3: Dam-break simulation using an TVD-MacCormack model [\[6\]](#)

The aim of the project is to evaluate the extent of prediction inaccuracy involved in varying the Boussinesq coefficient, by applying the more robust TVD-MacCormack scheme on the example of Boscastle.

3 Simulations of the 2004 flood

3.1 Actual flooding event

Below is a summary of the flooding event in 2004. [\[8\]](#)

On the 16th August 2004, due to a series of convective storms located along the north Cornwall coast, Boscastle experienced heavy rainfall from 13:00 to 17:00 BST.

The soils in the area are thin and overlay impermeable rock, and hence became saturated rapidly. Reduced infiltration led to increased surface run-off. The high land slopes in many parts of the Valency catchment meant that floodwater moving towards the river was rapid.

The flow in the Valency increased from approximately 13:00 BST onwards, with water discharging from the channel at the downstream bridge at about 15:30 BST. Soon thereafter, Both the B3263 road bridge and the lower bridge became blocked due to debris. This resulted in increased water levels upstream of both bridges. The overall flow rate down the B3263 also increased.

The flood then resulted in the collapse of the 9-foot wall separating the car park from the B3263 and other buildings. The collapse of the wall would have resulted in a surge of water downstream the B3263, as well as cars being flushed out of the car park, which worsened the blockage and contributed to further water level buildup. The flow reached a peak at about 17:00 BST. See Figure [\[4\]](#) for a selection of photos during the flooding event.



(a) Flooding on the River Valency (Stollery, 16:05 BST)



(b) B3263 bridge blocked on the Valency(18:13 BST)

Figure 4: A selection of Photos from the Studies Report on the 2004 flood by HR Wallingford

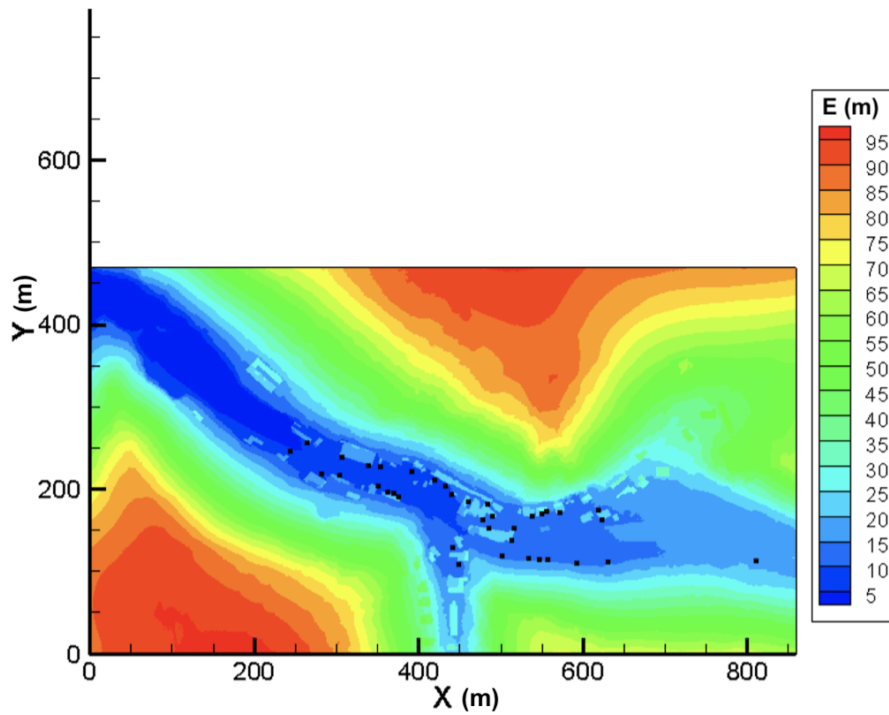
The flood can be attributable to the limited ability of the Valency to carry water downstream due to low channel capacity. For example, next to the lower bridge, the channel capacity was such that the annual probability of out-of-bank flow was as high as 10%. The flooding event also coincided with the high tide in the bay, which placed a restriction on the exit of water into the Celtic Sea.

3.2 Simulation results

The raster named 'ActualEl', which represents riverbed conditions before any flooding mitigation, is used. Tests are run for 72,000s (3 hours) by inputting constant flow rates of $160\text{ m}^3/\text{s}$ and $19\text{ m}^3/\text{s}$, which are the estimated peak discharges during the actual flood at the upstream ends of the River Valency and River Jordan respectively.

3.2.1 Model calibration against the 2004 flood

Model calibration is made possible thanks to data collected from the 37 gauging points during the 2004 flood. Simulations are run and river elevation data from 37 points are extracted using Tecplot. See Figure 5 for the 37 points on the Digital Elevation Model as well as on the Tecplot plot. These points correspond to the 37 points for which real elevation data during the 2004 flood is available.



(a) Locations of the 37 points to be extracted using Tecplot, contours being the elevations

Figure 5: Location of the 37 gauge points

Only β values equal to or below 1.0 were computed using a time step of 0.02 s, water level predictions become increasingly underestimated as β the value decreases. Results for β values above 1.0 are missing as the simulations would not stabilise, even at much reduced time steps (for example, 0.0001 s).

A comparison of prediction results with surveyed data in the 2004 flood is shown in Figure 6. Note how the overall accuracy of prediction is quite high as the results lie quite close to the red diagonal line representing the perfect match between predictions and measurements.

Furthermore, for most location points, lower the β value, lower the predicted water level.

As lighter blue colours are used for smaller β values, it is possible to see the transition from darker shades of blue to lighter shades of blue in most locations. However, there are some outstanding data points where this relationship is not so strictly followed.

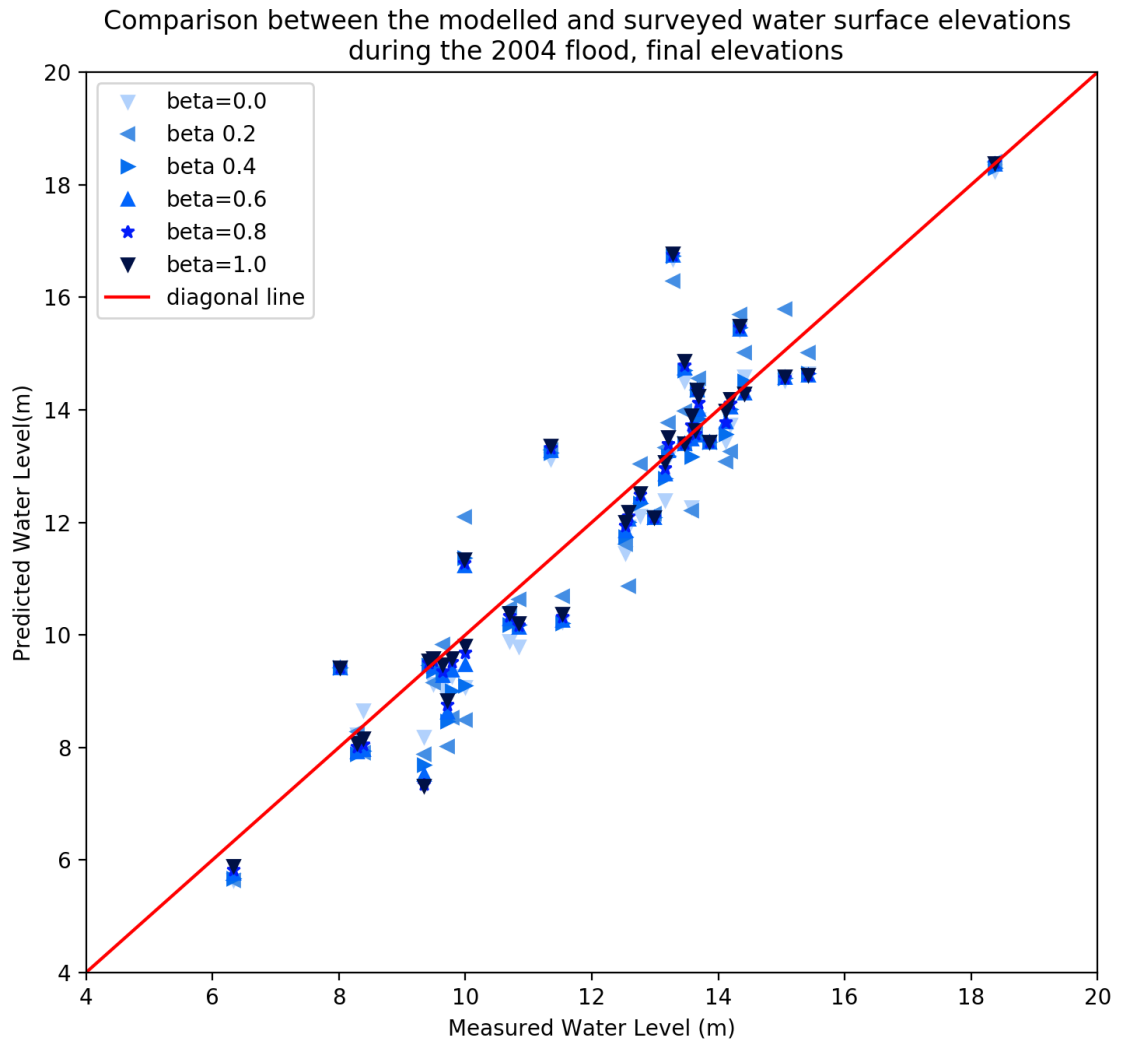


Figure 6: Model calibration tests against actual data from gauge points during the 2004 flood

3.2.2 Comparison of centreline elevations

Using Tecplot, the centreline is drawn (along the centre of the green region). Data from about 200 points on the line are extracted, see Figure 7. The plotted centreline will be used for all data extractions.

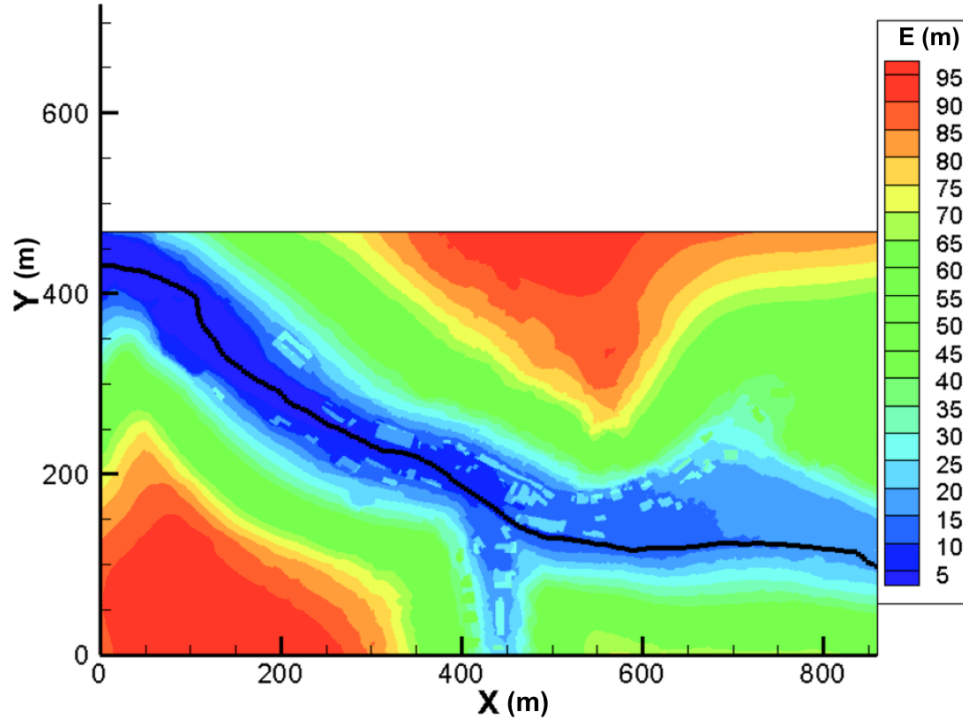


Figure 7: Countour of elevation with the centreline plotted

The elevations of riverbed and water surface are plotted against horizontal distance in the x-direction is shown in Figure 8. Note that the difference between water surface elevation and riverbed elevation would be the actual water depth. The same correlation between estimate and β value is observed. However, there is some instability in the results especially in the region around the estuary.

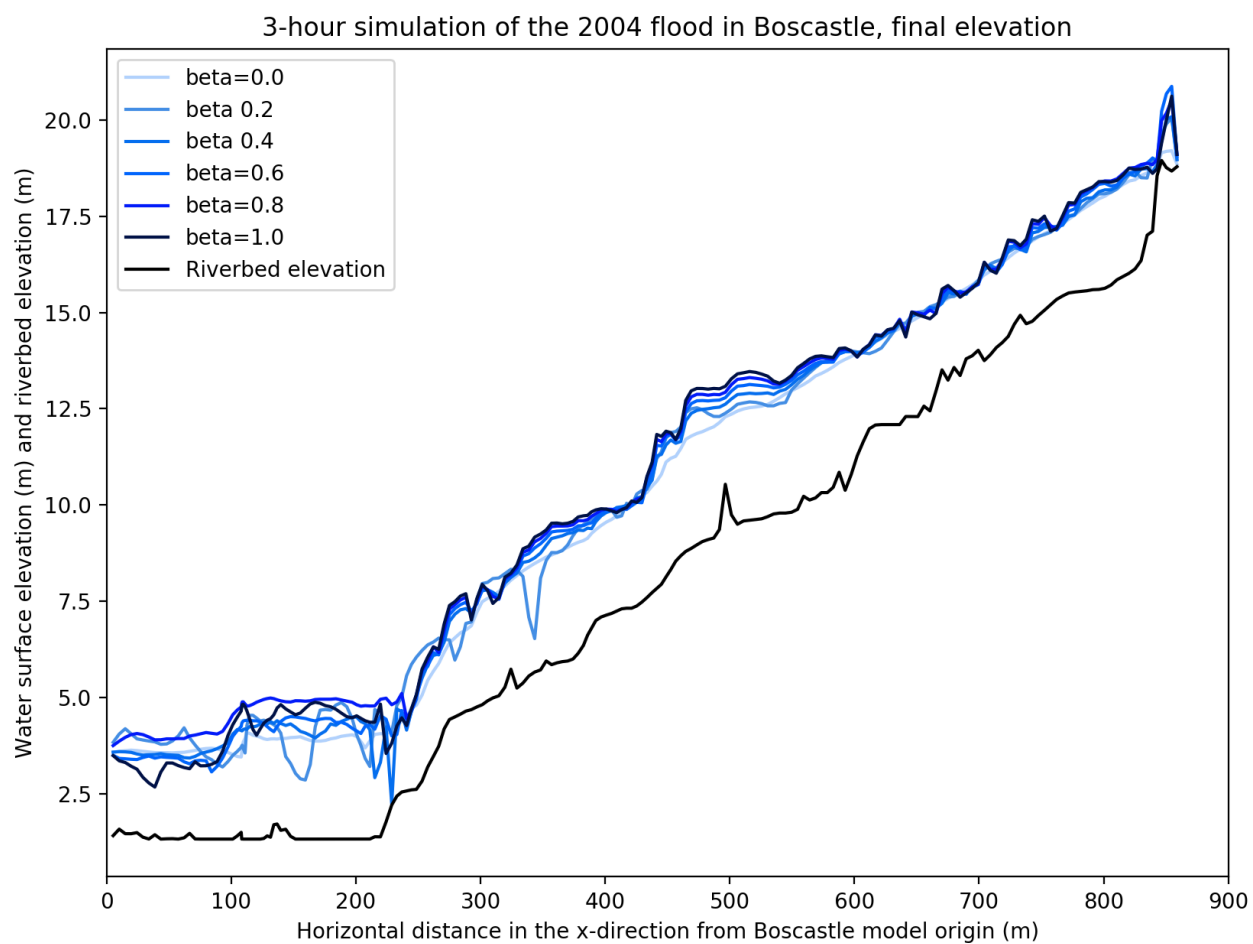


Figure 8: Centreline results

4 Numerical predictions of potential floods

4.1 Flooding scenarios

There are four flooding scenarios to be considered in this Project. Each river will be modelled as an inlet of constant flow, and the magnitudes of these flows vary depending on the flooding scenario. See Table [1](#) below.

Table 1: Expected flow rates in the two rivers for various floods scenarios in Boscastle

Chance of flood occurring in any one year	Constant volumetric flow rate (m ³ /s)		
	Valency	Jordan	Combined
1 in 25	21	4.8	25.8
1 in 50	32.6	7.4	40.0
1 in 75	42.3	9.6	51.9
1 in 100	51.6	11.8	63.4

4.2 Riverbed conditions

4.2.1 Mitigation measures

The channel capacity was certainly too low during the 2004 flood. After the flooding event, the Environmental Agency commissioned Halcrow Group Ltd for mitigation measures that can protect against floods up to 1-in-75-a-year severeness. This presents a relatively low exceedance probability, 1.33%, and ensures minimal impact on local landscape. The following mitigation measures were proposed in stages. [2](#)

1. Deepened and widened river:

Before the modification works, the channel capacity was lowest around the Lower Bridge, where there was a 10% chance of out-of-bank flooding occurring in any one year. Widening sections of the river and deepening the river between 0.5m and 1m is designed to increase the channel flow capacity and improve the conveyance of debris.

2. Redeveloped car park:

During the 2004 flood, over 100 cars were washed out of the car park onto the B3263 road and down the river. This imposed severe hazards, creating extensive damage to property from impact forces, and blocking up the lower bridge. To prevent this in the future, bollards were installed and the car park site was raised.

3. Modified ground elevations:

In order to minimise the impact of sedimentation on the channel capacity, a large braided channel section is excavated upstream of the car park. By reducing river energy and by allowing the deposition of sediment before reaching the central and lower reaches, the flow of sediment and debris would be more controlled.

In addition, a two-stage channel has been incorporated alongside the car park to replace the pre-existing low flow river channel. This is to avoid trapping of sediment and thus allow free movement to the lower reaches.

Downstream of the B3263 road bridge, a section of the road on the north bank is being lowered to maintain the height difference between the bank and the channel bed in this area in order to reduce the potential for harm caused by falls.

4. Flood defence wall:

A flood defence wall built by the river on the lower reaches of the car park, it can help prevent river water overflowing into the car park.

4.2.2 Elevation rasters

Influences in factors such as urbanisation, topography, and the spread of impermeable surfaces can influence the severity of flood events. The four different stages of flooding mitigation schemes proposed, together with the unmodified conditions before modification, will result in a total of five different riverbed conditions. All five will be considered in this project, they are described in Table 2 below.

Table 2: Description of the five input riverbed conditions

Raster	Description
ActualEl	Represents real ground conditions before any mitigation measures are applied.
R	'ActualEl' but with the river deepened and widened.
RC	'R' but with the car park modifications.
RCL	'RC' but with the ground elevation next to the river modified.
RCLW	'RCL' but with the additional wall in the lower reaches of the car park.

See Figure 9 below to visualise the proposed mitigation works and their corresponding Raster.

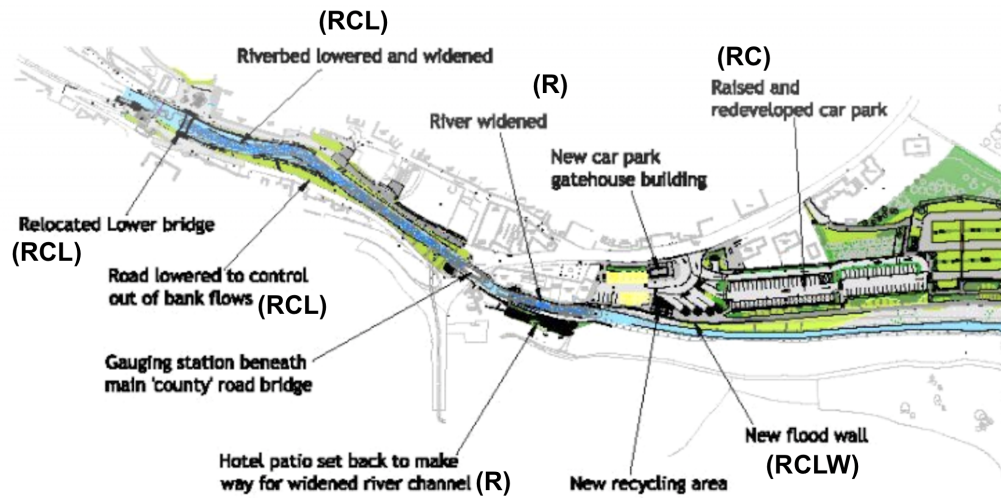


Figure 9: Proposed flood risk improvement works and their associated raster

All the simulations were ran for 18,000 seconds. A range of β values ranging from 0 to 2 at intervals of 0.2 were used.

4.3 Results

Riverbed elevation and results for estimated total river elevation are plotted against horizontal distance in the x-direction from Boscastle model origin, where $x=0$ represents the estuary. Plots of the complete simulation results can be found in Appendix A. Note that water depth would be the difference between river ground elevation, Z_b , and total river elevation, E .

Taking the RCLW riverbed scheme for the four flooding scenarios as an example, see Figure

10

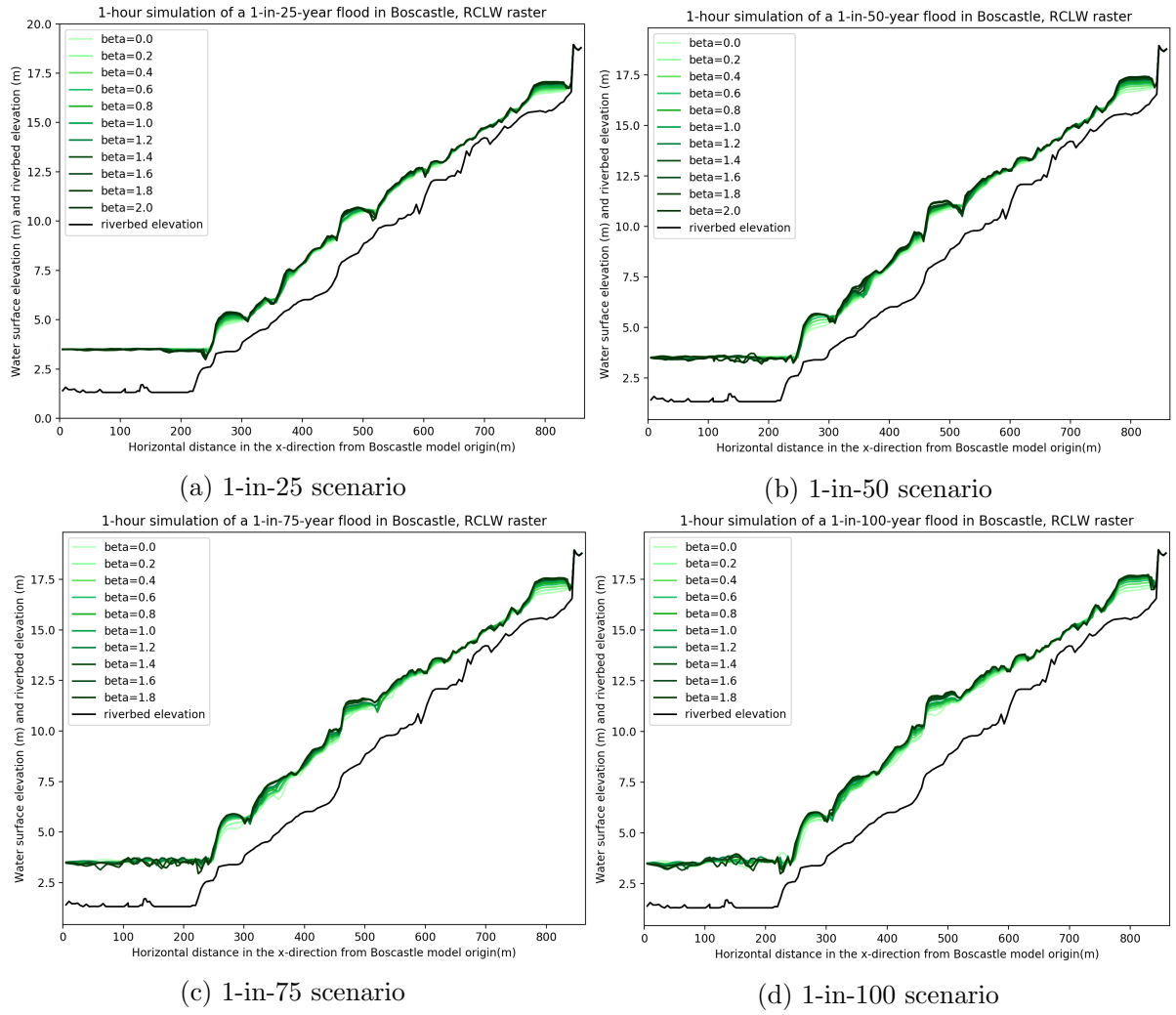


Figure 10: Simulation results for 1 in 75 year chance

The main observations are summarised below:

1. There is a positive correlation between the value of β and the simulation results of water surface elevations, except for a few small regions. Notably, in the region near the estuary (x below around 220m), the results are significantly unstable.
2. The discrepancies are in general quite small with the exception of the following three regions: $x \approx 220$ to $x \approx 310$, $x \approx 460$ to $x \approx 520$, $x \approx 780$ to $x \approx 850$. In these regions, the influence of the varying β value are greater, as can be noted by the greater spread of the curves with different shades of green.
3. As flow rates increase (when a flood becomes rarer), the influence of varying beta increases. This is shown by the greater spread of green plots as flow rates increases. The most clear contrast can be made by comparing sub-figure 10(a) (1-in-25, the least rare flooding scenario among the four cases) and sub-figure 10(d) (1-in-100, the rarest scenario among the four).
4. As flow rates increases (when a flood becomes rarer), the results are overall more unsteady. Note that in the 1-in-100 scenario, it was not possible to finalise simulations for β at 2.0. For the 1-in-100 scenario, it was possible to finalise simulations for up to $\beta=2.0$ with the exception for the RCLW scheme. See Appendix A for the full results.

5 Effectiveness of the flood defence schemes

To evaluate the effectiveness of the different stages of mitigation schemes, a selection of simulation results are loaded to Tecplot, this time, we use a 3D view which shows contours of DP, the actual water level, which is given by the difference between the total elevation and ground elevation. The dark blue represents no water, whereas DP values ≥ 3 are categorised under red. Refer to the legend in each figure for details.

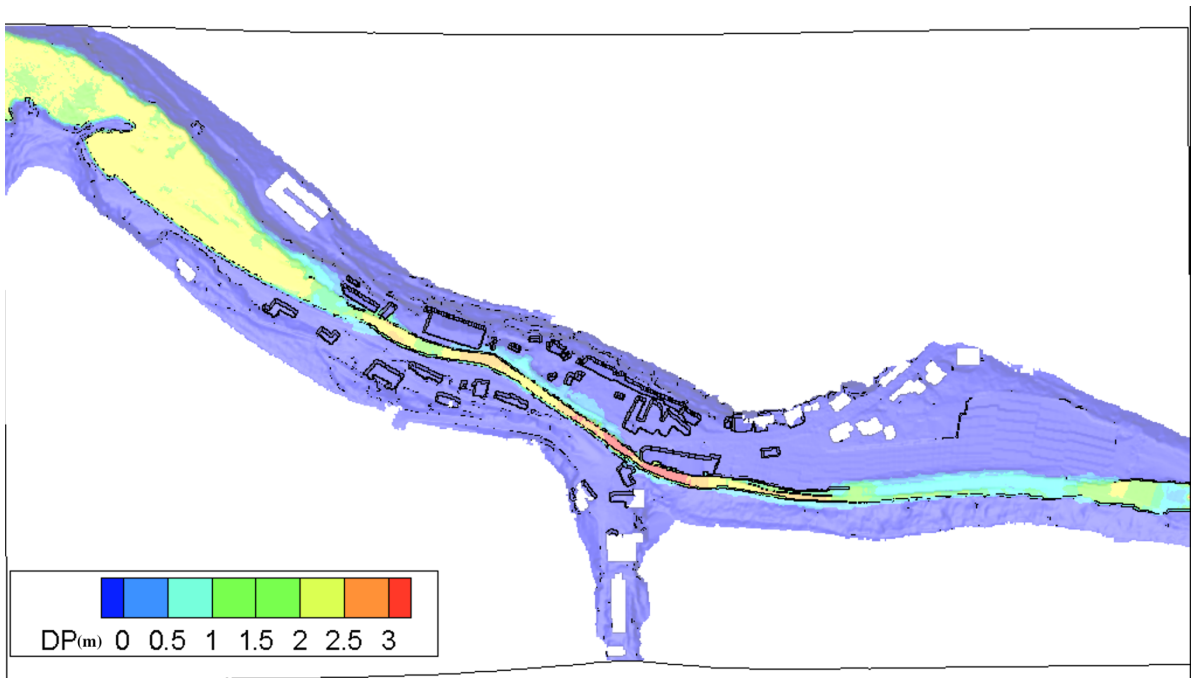
5.1 RCLW

5.1.1 1-in-75-a-year scenario

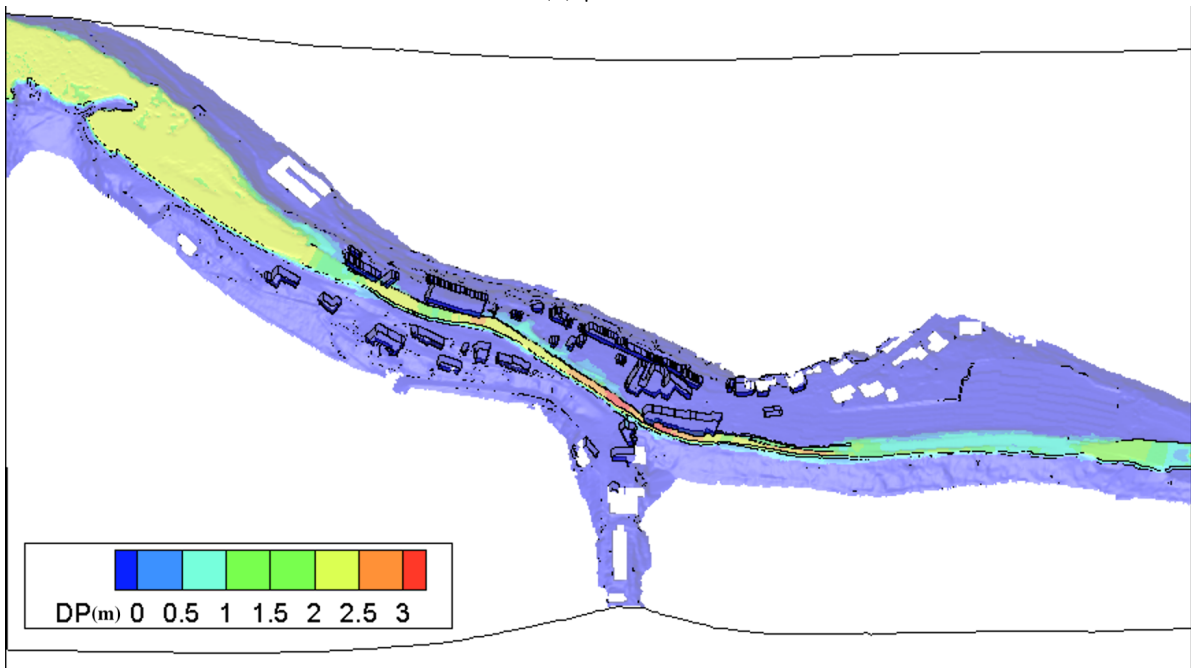
RCLW was the final flood protection scheme implemented, it was designed against the 1-in-75 in any year chance flooding scenario. It is therefore the mitigation scheme of most interest and will be analysed first. See Figure 11, where plots are presented in descending order of β values of 1.6, 1.0, 0.8 and 0.6.

Firstly, by comparing sub-figures 11(a) and 11(b), we can reconfirm our previous observation that as β value increases, we overestimate the total elevation, as ground elevation should stay constant, the DP becomes overestimated. This is shown by the greater area of orange/red in sub-figure 11(a) and how the floodwater submerges a greater area on the land.

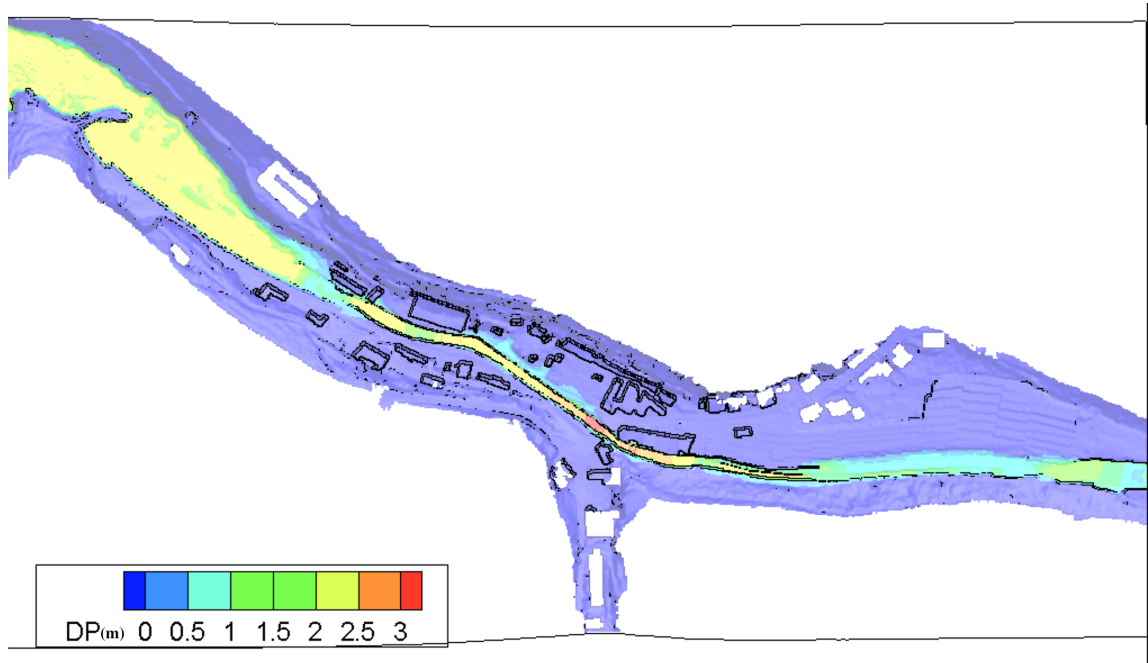
Next, by comparing sub-figures 11(b), 11(c) and 11(d), it is possible to observe how areas in orange/red continues to decrease, as expected. Most importantly, the submerged light-blue area on the land decreases as β decreases.



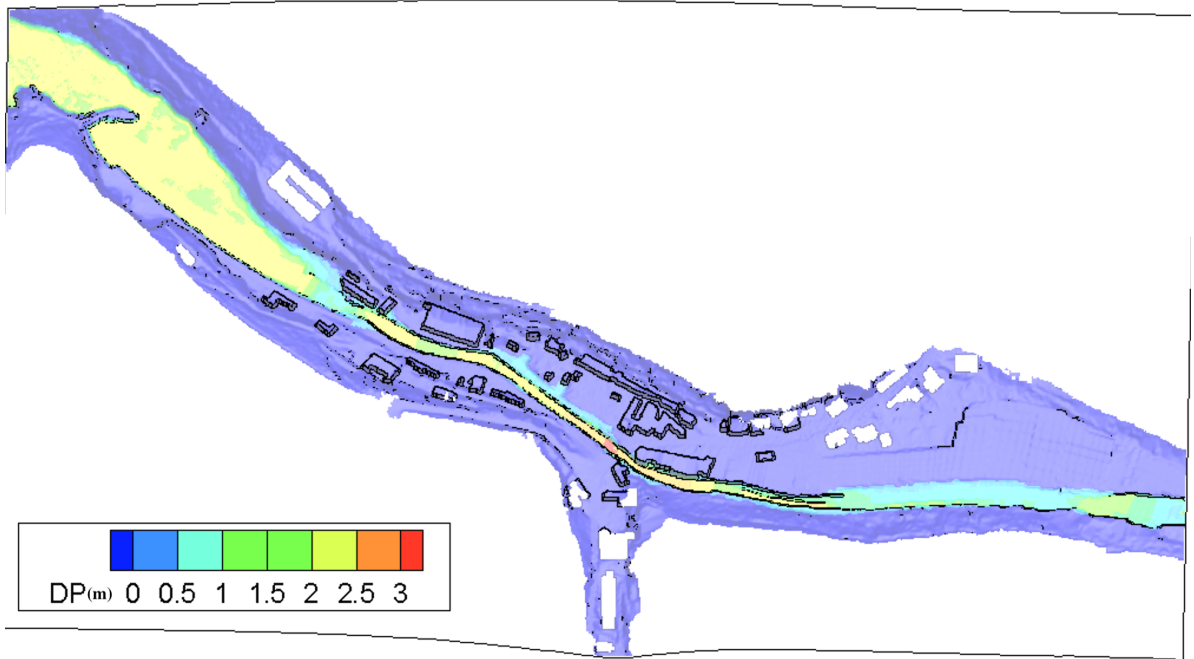
(a) $\beta=1.6$



(b) $\beta=1.0$



(c) $\beta=0.8$



(d) $\beta=0.6$

Figure 11: RCLW raster under 1-in-75-a-year flood simulations.

See a detailed zoom-in comparison in Figure 12 below. It can be observed clearly that when β reaches 0.6, there is almost no overflow of river water to the ground shown, see sub-figure 12(d)

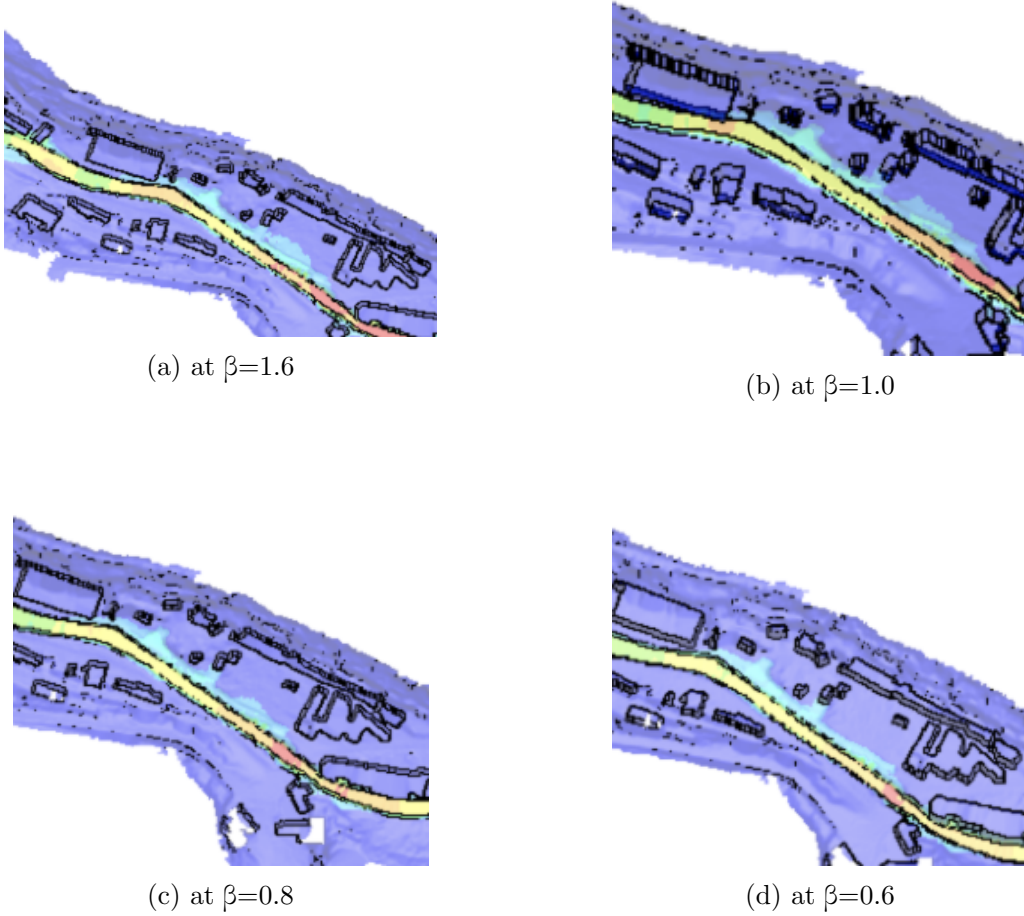


Figure 12: Detailed view for RCLW raster under 1-in-75-a-year flood simulations.

Recalling that results from β at 1.0 would most accurately predict water levels and hence the severity of the flooding event. It can be concluded therefore, that the defence scheme RCLW is insufficient for a 1-in-75-year flood, because spillage is observed for beta=1.0. However, if during simulations β was set to 0.6 or below, water engineers may reach the wrong conclusion that the proposed defence scheme is adequate.

5.1.2 1-in-50-a-year scenario

To allow comparison, a plot of the 1-in-50 scenario at $\beta=1.0$ is shown below in Figure 13. It can be observed that the edges are clearly defined, implying that there is no overflow.

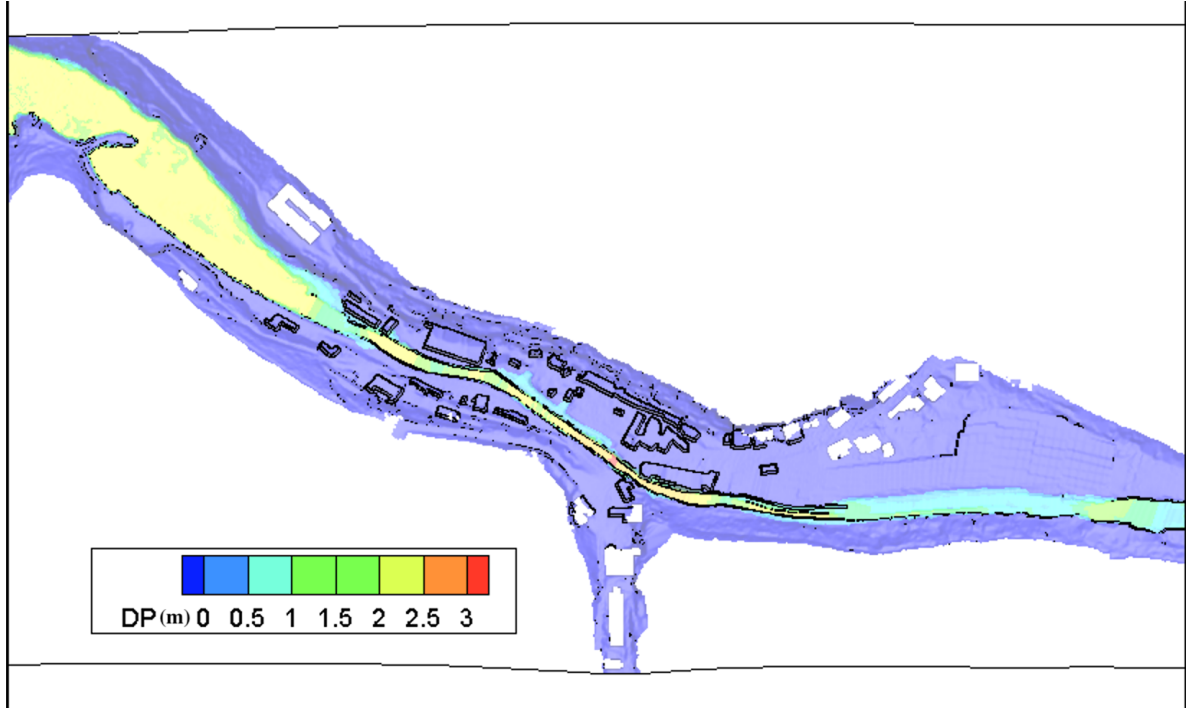


Figure 13: Flood simulation on the RCLW scheme under the 1-in-50 a year flood scenario, at $\beta=1.0$

Therefore, the flood defence schele RCLW should be sufficient for a 1-in-50-a-year flood, but not a 1-in-75-a-year one.

5.2 RCL raster

As RCLW has been deemed insufficient to withstand a 1-in-75-a-year flood, RCL would be so as well for the 1-in-75 case, as Therefore, 1-in-50 case at $\beta=1.0$ is analysed straightaway. It was found that the RCL scheme would be adequate for the flood with 1-in-50-a-year severity. See Figure 14 below.

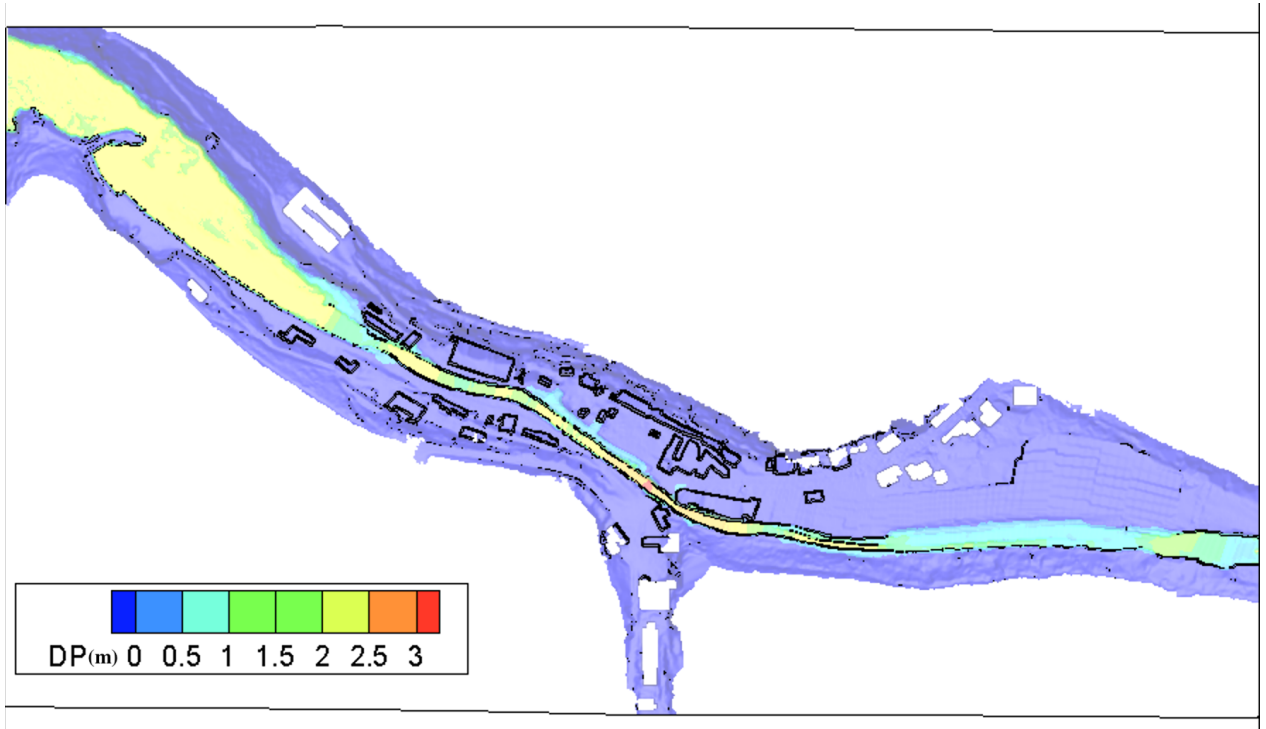
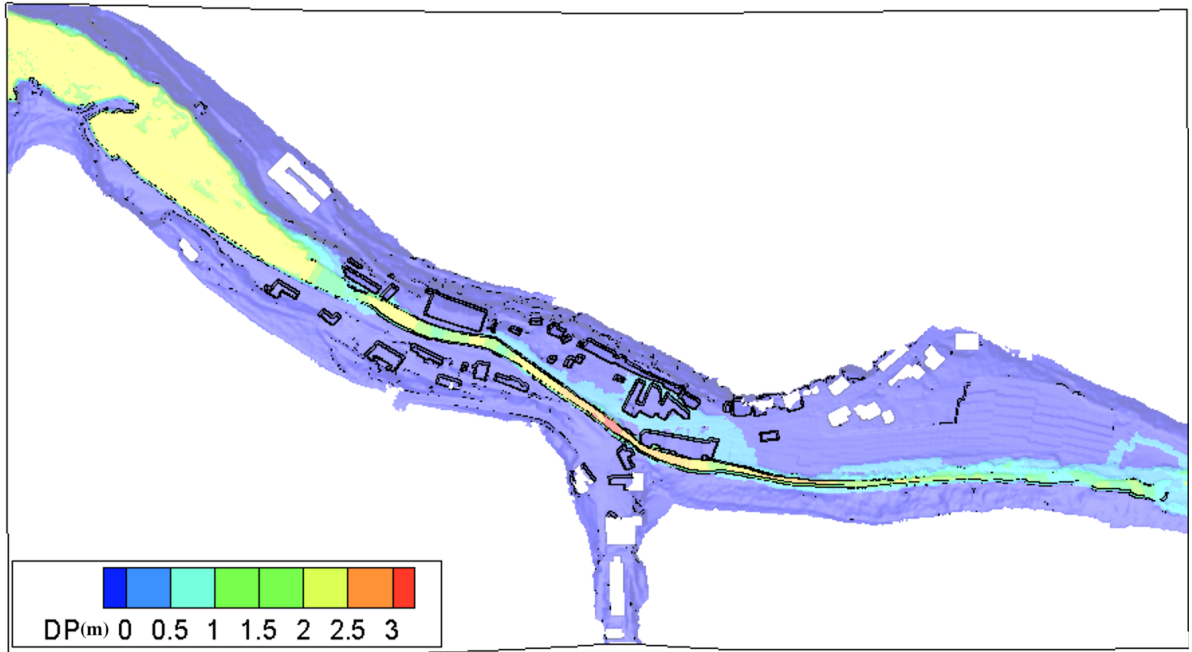


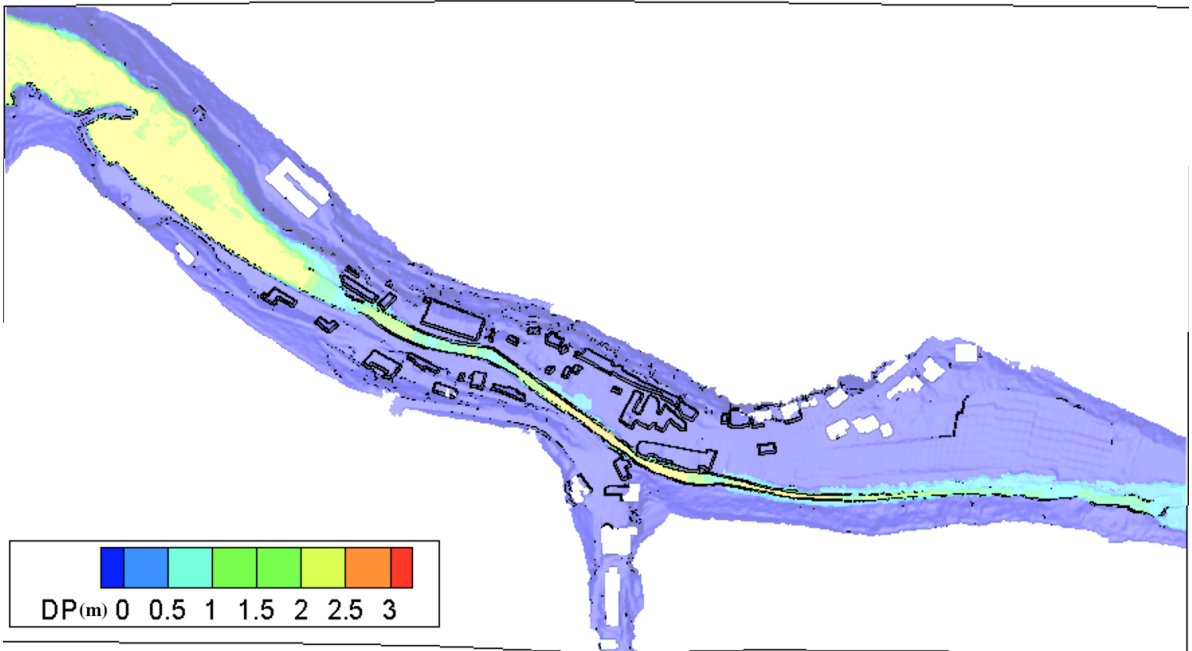
Figure 14: Flood simulation on the RCL scheme under the 1-in-50 a year flood scenario, at $\beta=1.0$

5.3 RC raster

For the RC raster, the 3D view plot was first run for the 1-in-50 case. As can be seen in Figure 15(a) there is a considerably large extent of overflow to the river bank. Next, the simulation was run for the 1-in-25 case. It was found that there is some minimal overflow. See Figure 15(b)



(a) 1-in-50 flooding scenario



(b) 1-in-25 flooding scenario

Figure 15: Flood simulation on the RC scheme at $\beta=1.0$

5.4 R raster

As RC proved to be insufficient for the 1-in-50 scenario, R would be so as well. Hence, simulation results for the 1-in-25 was considered directly. It was also found that there would be some overflow. Despite being of low severity, the extent of inundation is greater than the case of RC raster, which is expected as R contains fewer mitigation schemes than RC. See Figure 16 below.

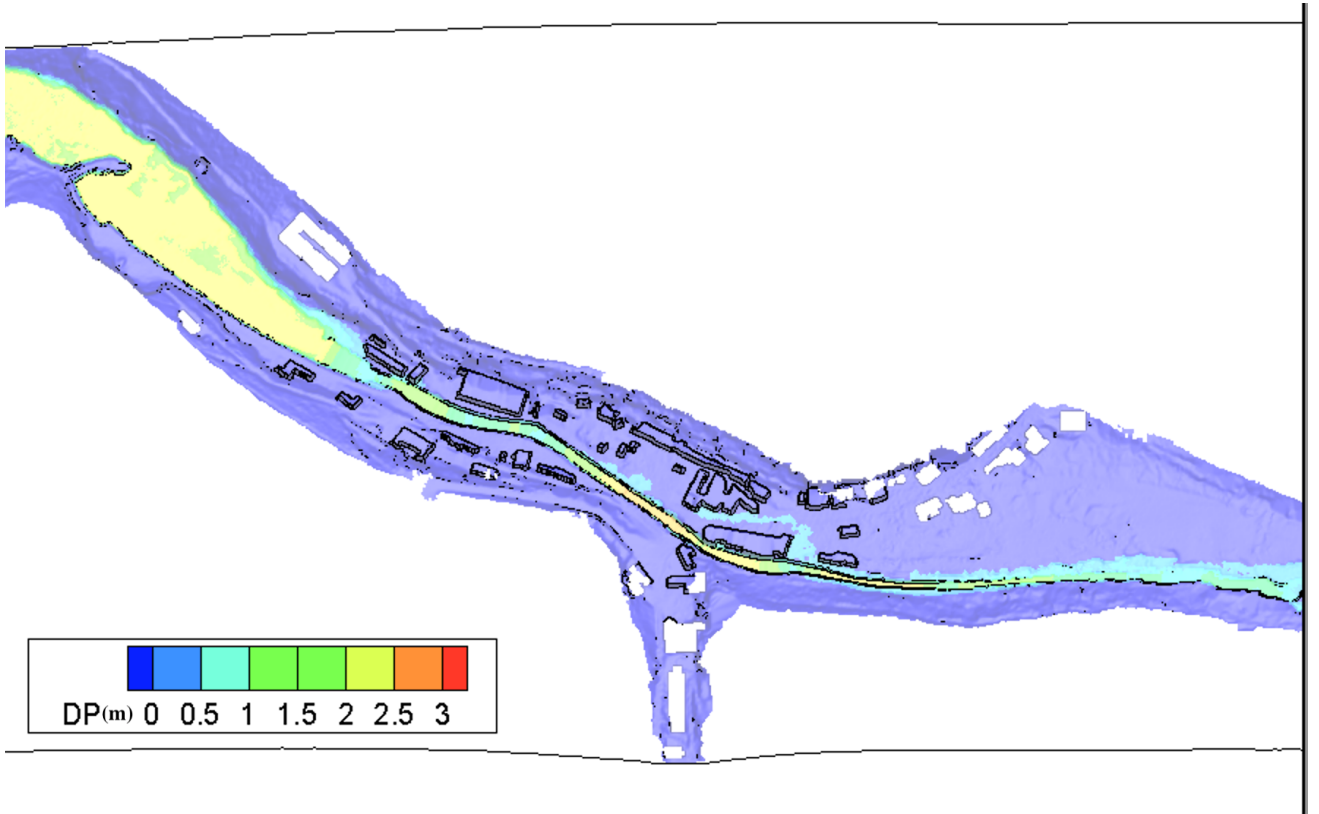


Figure 16: Flood simulation on the RCL scheme under the 1-in-50 a year flood scenario, at $\beta=1.0$

5.5 ActualEl raster

Overflow to river banks would naturally be expected for the ActualEl raster. For the sake of completeness, simulation results of the ActualEl raster at $\beta=1.0$ in the 1-in-25 scenario was also plotted to allow comparison. The severity of inundation is visibly worse, see Figure [17](#) below.

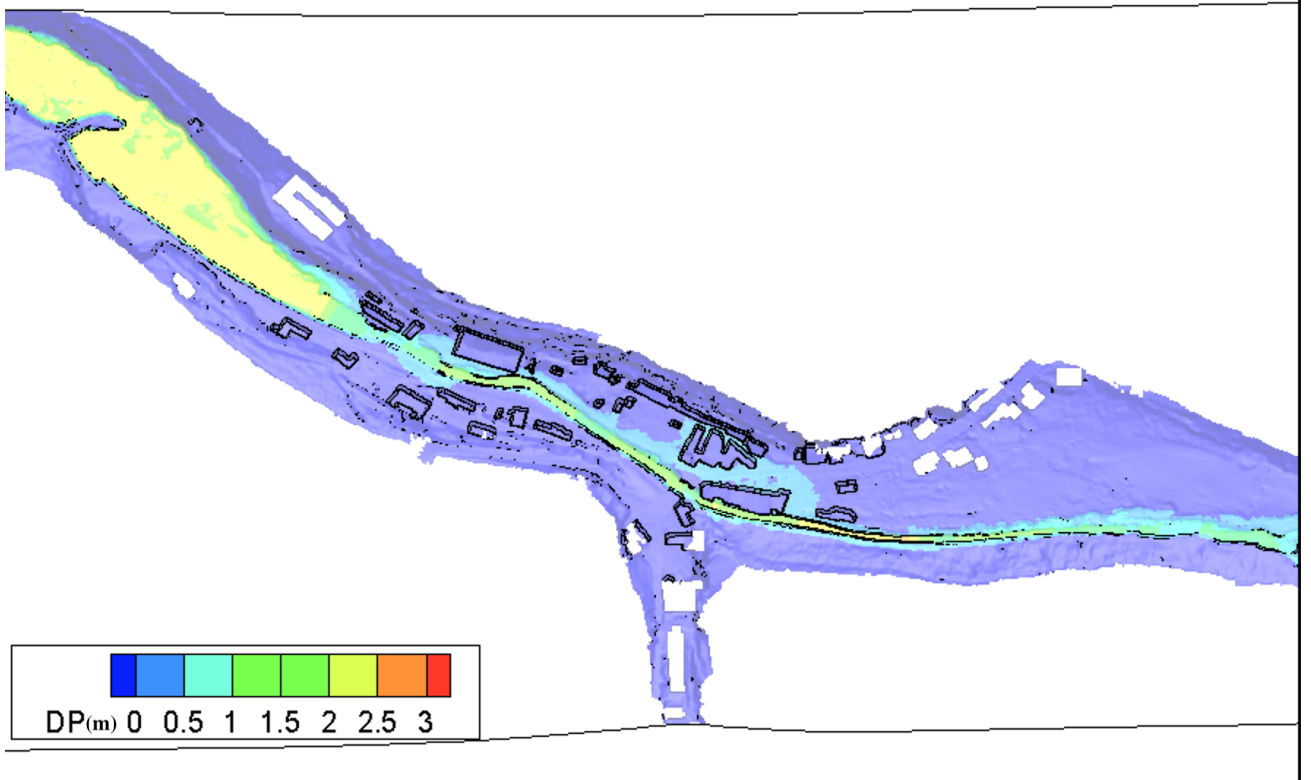


Figure 17: Flood simulation on the RCL scheme under the 1-in-25 a year flood scenario, at $\beta=1.0$

6 Conclusion

Boscastle suffered one of the major flooding disaster in 2004. Due to its hydrogeological conditions, it is a case of great interest and was used throughout this project. For most floods, the horizontal length scale is much greater than the vertical length scale, and they can thus be mathematically described by Shallow Water Equations (SWEs). These equations can be assembled into matrices and be solved. The Alternating Direction Implicit is the most widely adapted scheme in most commercial software, it requires numerical treatment of the Boussinesq coefficient to stabilise computation in the cases of trans-critical flows. Usually, the tuning consists in lowering the β below 1. The project aimed to understand better the scale of inaccuracy in simulation results involved with this practice by running tests on the more robusts TCD-MacCormack scheme, as this would allow stable simulations under a greater range of β values. First, tests are run to compare modelled and surveyed water levels at the 37 gauging points. It was found that simulation results match well with the measured data from the gauging points. Moreover, for most locations, the positive correlation between the Boussinesq constant and the the water level estimations stands true.

In the second part of the project, tests were run for four flooding cases of increasing severity (1-in-25, 1-in-50, 1-in-75, and 1-in-100). Graphs of the centreline water level estimations are plotted. The positive correlation between the Boussinesq and the simulation result for the stabilised regions of the river was reconfirmed. Furthermore, at higher inputting flow rates, this positive correlation becomes even more noticeable. This is demonstrated by the greater spread of plots as flow rates increases. Therefore, if a water engineer applies an excessively low β value in order to stabilise the flood simulations when using a commercial software based on the ADI scheme, the results would be underestimated. This could potentially lead to the design of inadequate flood defence schemes. Even worse, the error in estimation is greater for high input flow rates, that is, when more severe floods are in question.

In the final part of the project, it was concluded that the mitigation measures implemented in Boscastle after the 2004 flood were insufficient to prevent a 1-in-75-a-year flood. One potential reason could be that the flood engineers might have applied low Boussinesq constants when performing the simulations and underpredicted the severeness of flood flows. It was also found that RCL would also be sufficient for 1-in-50. The 1-in-25 flooding scenario, would result in some minimal overflow to the riverbanks for the RC raster, however, for the R raster the overflow is much more widespread.

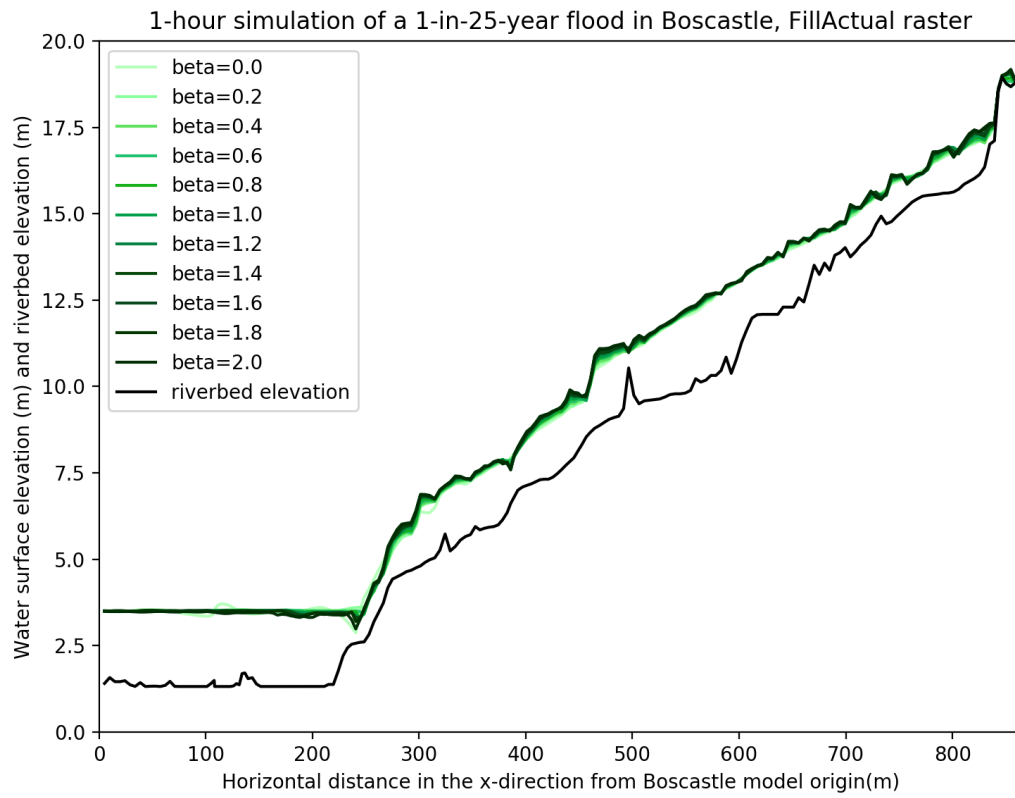
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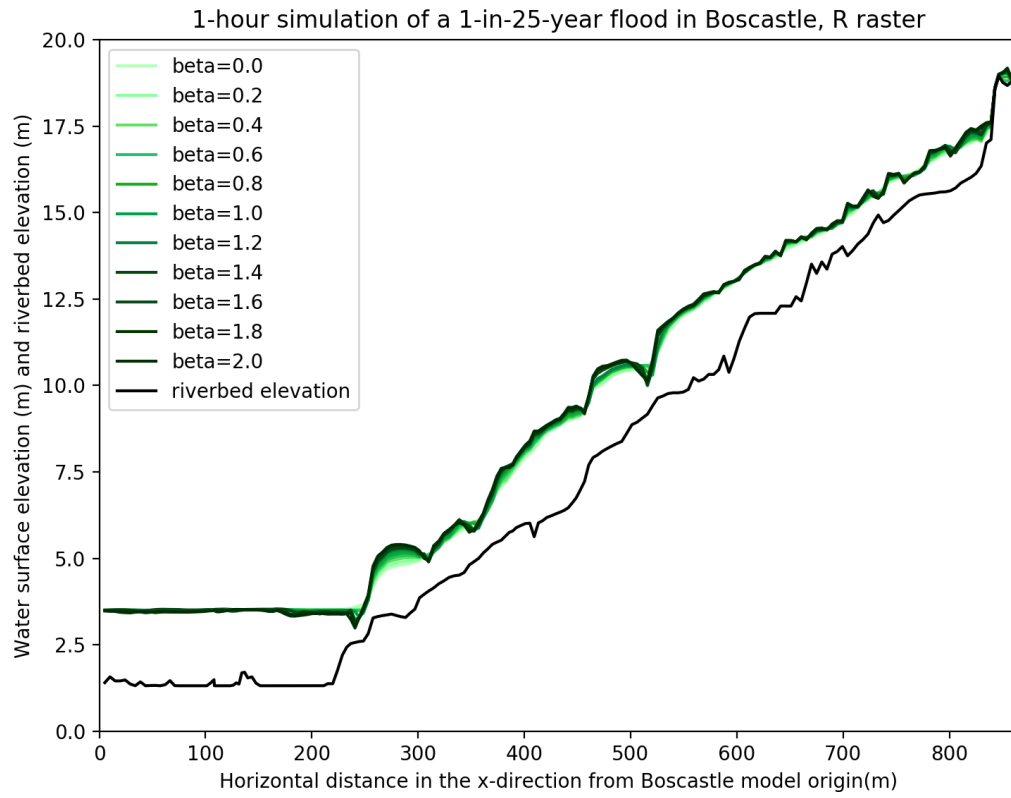
Appendix A Complete simulation results

A.1 Results for 1-in-25 chance

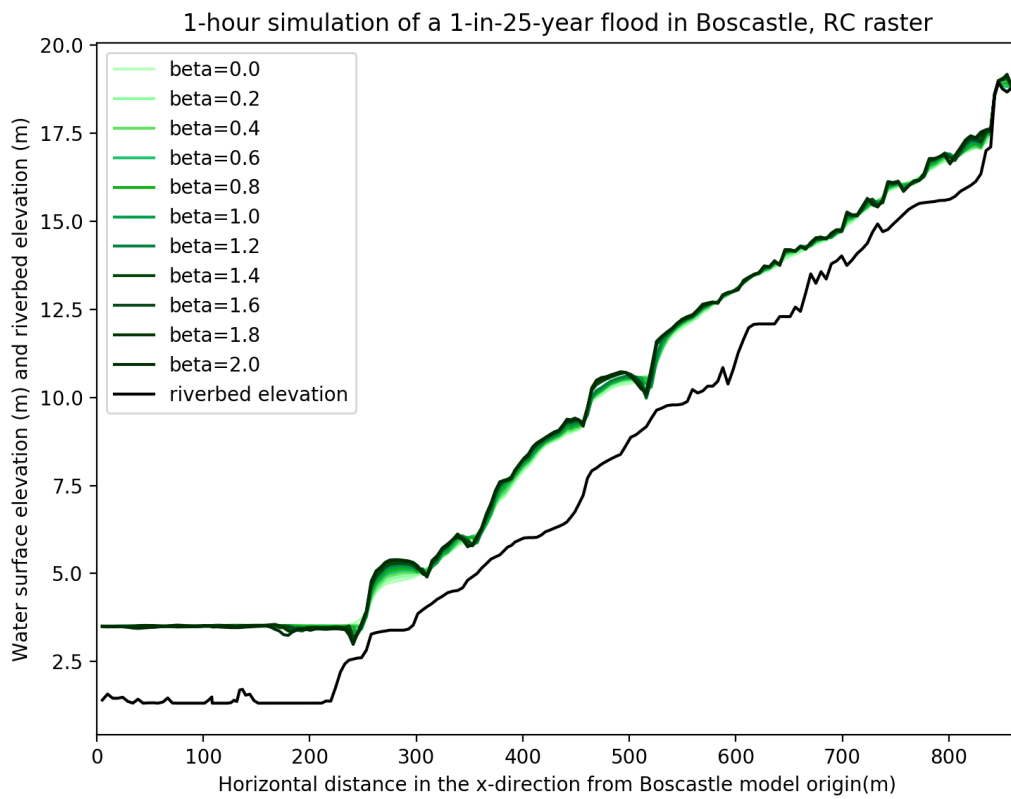
-



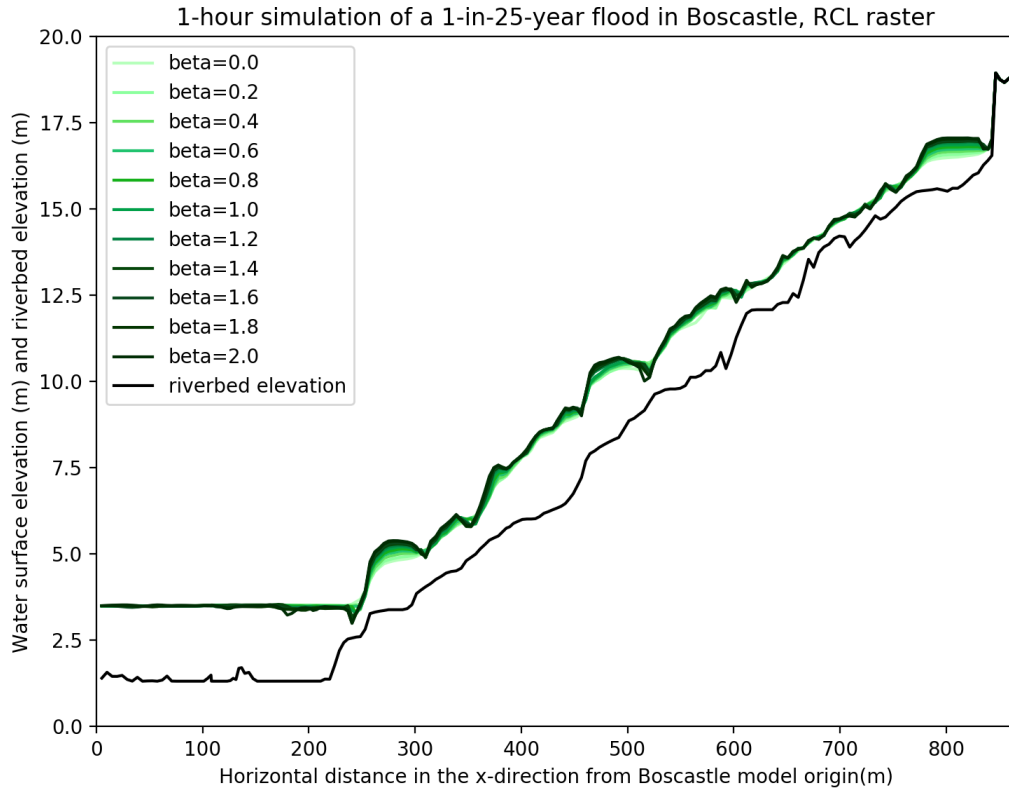
(a) FillActual Scheme



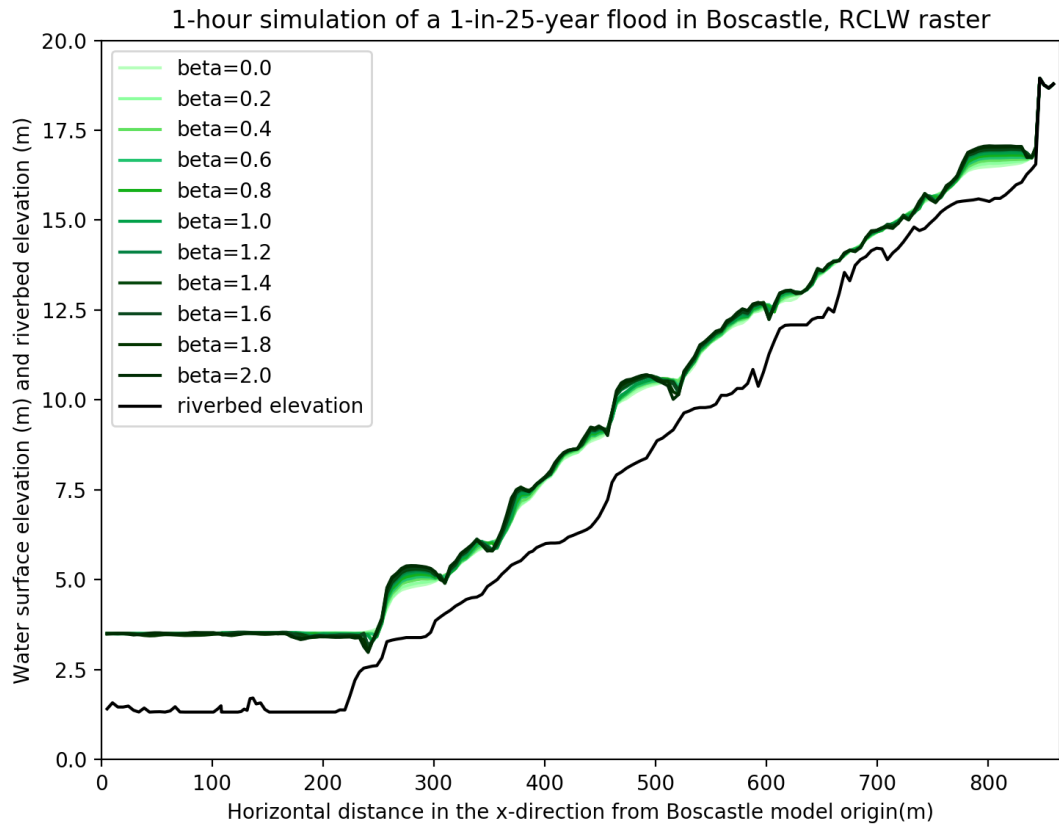
(b) R Scheme



(c) RC Scheme



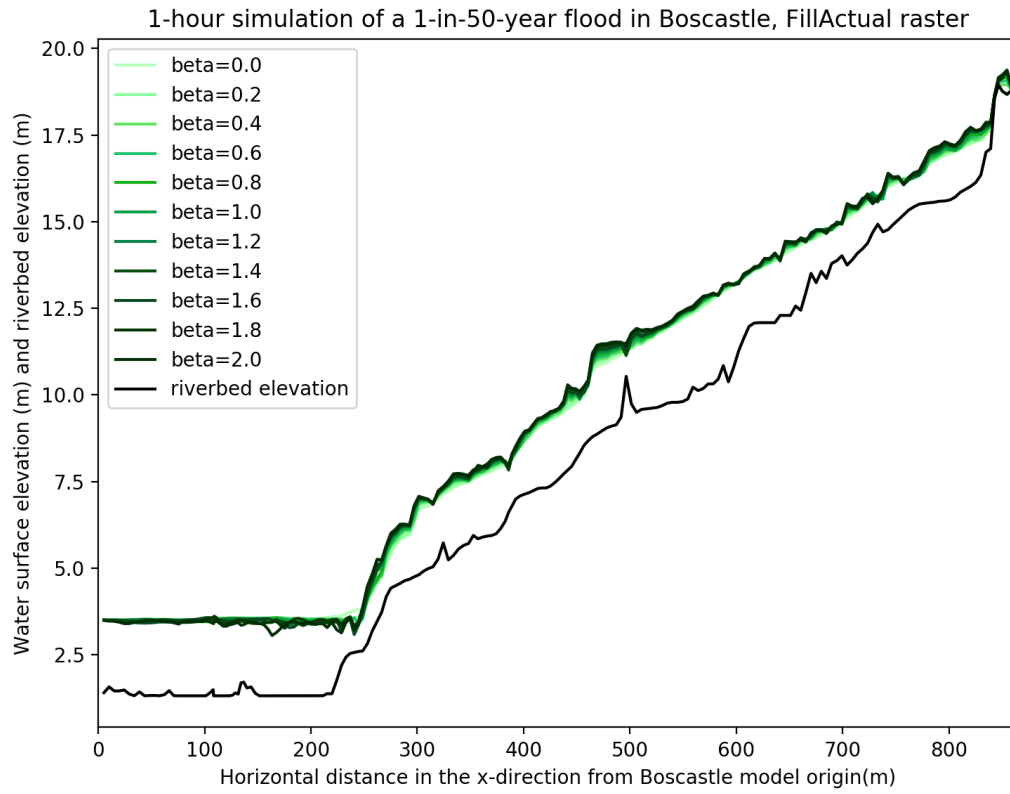
(d) RCL Scheme



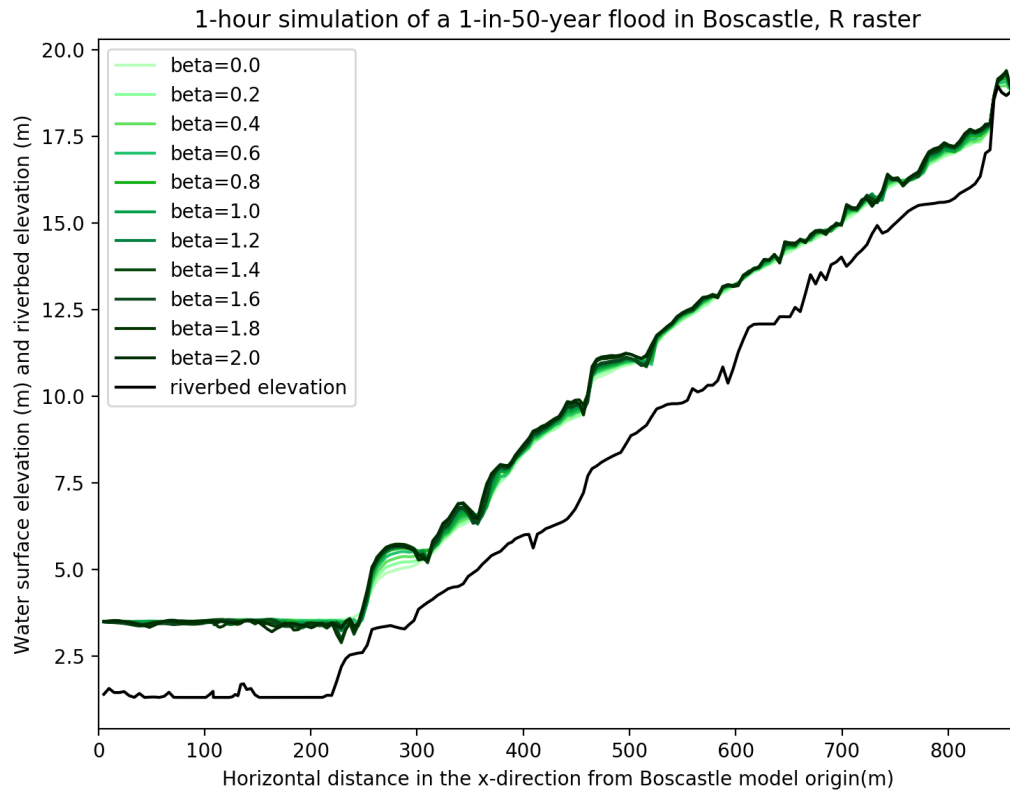
(e) RCLW Scheme

Figure 18: Simulation results of elevation for the 1-in-25-year chance flood

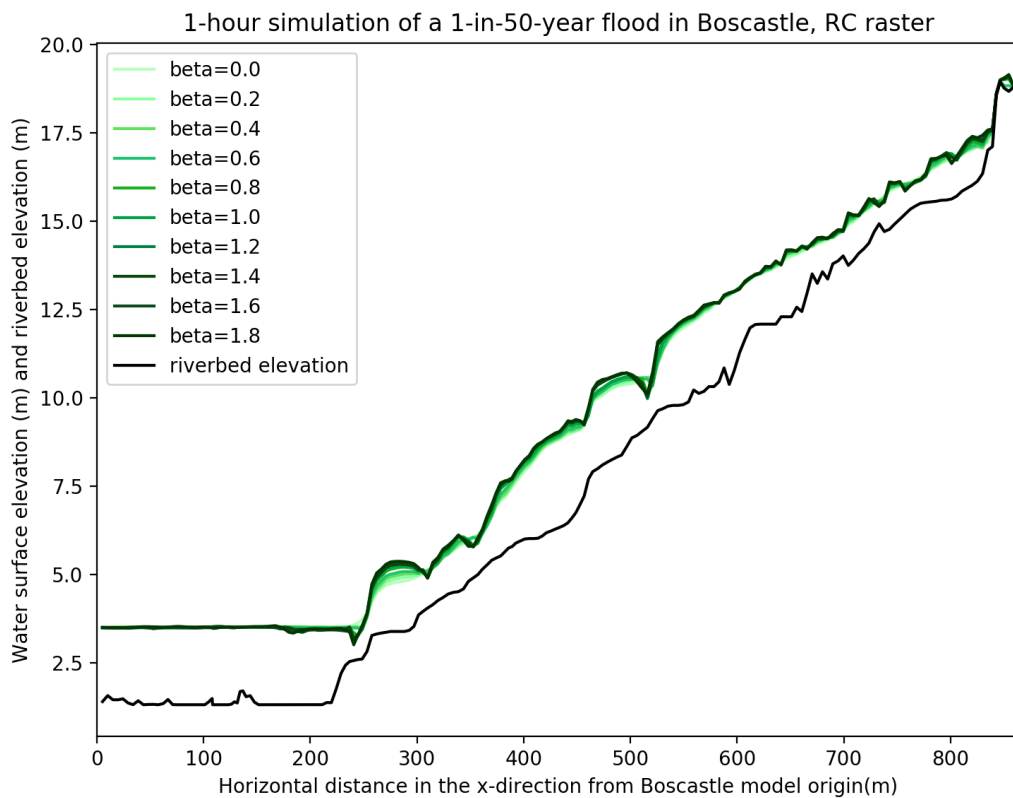
A.2 Results for 1-in-50 chance



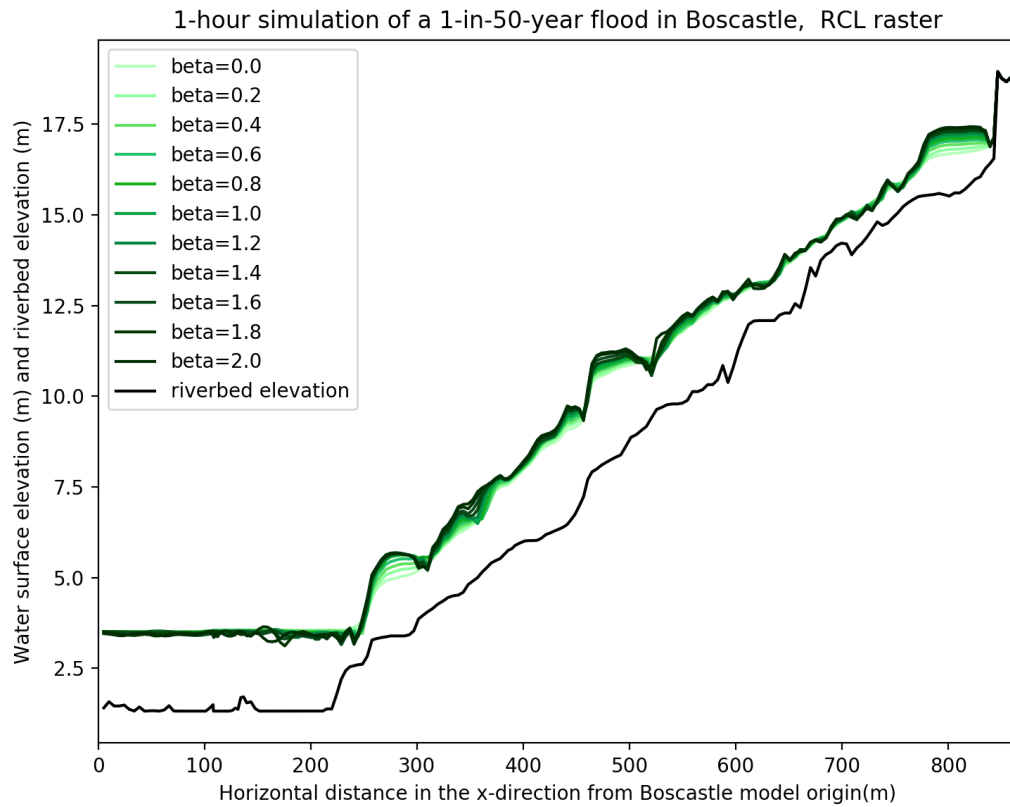
(a) FillActual Scheme



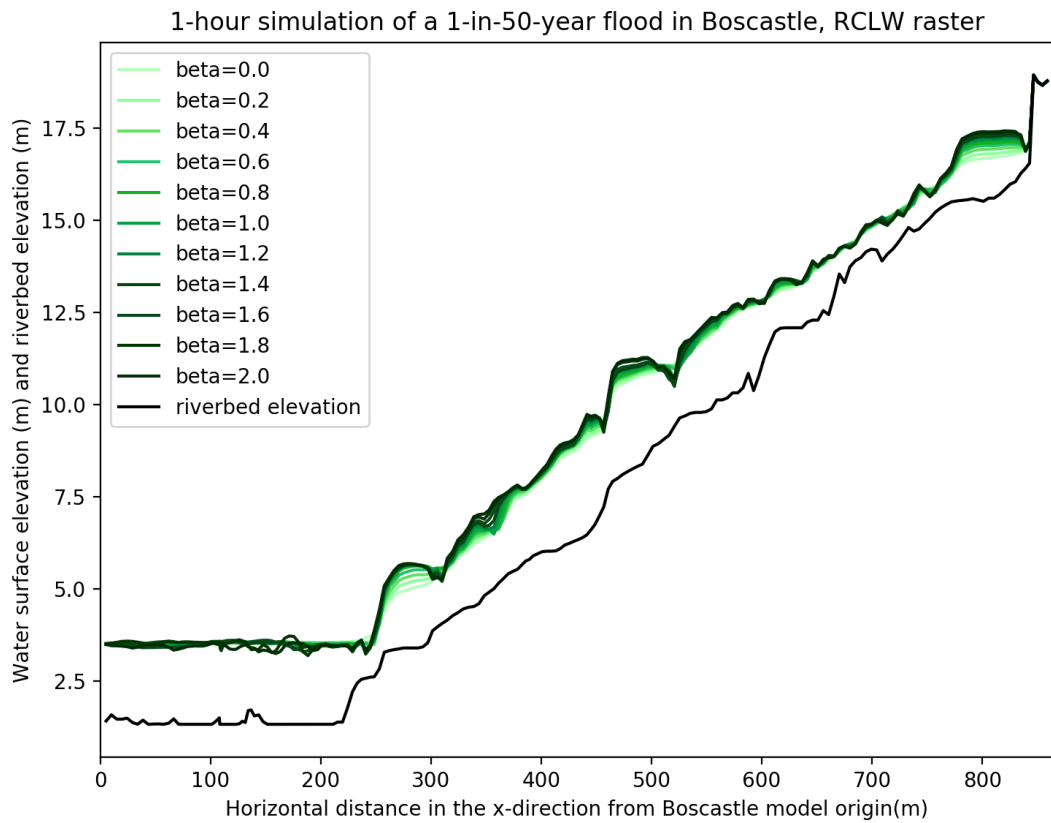
(b) R Scheme



(c) RC Scheme



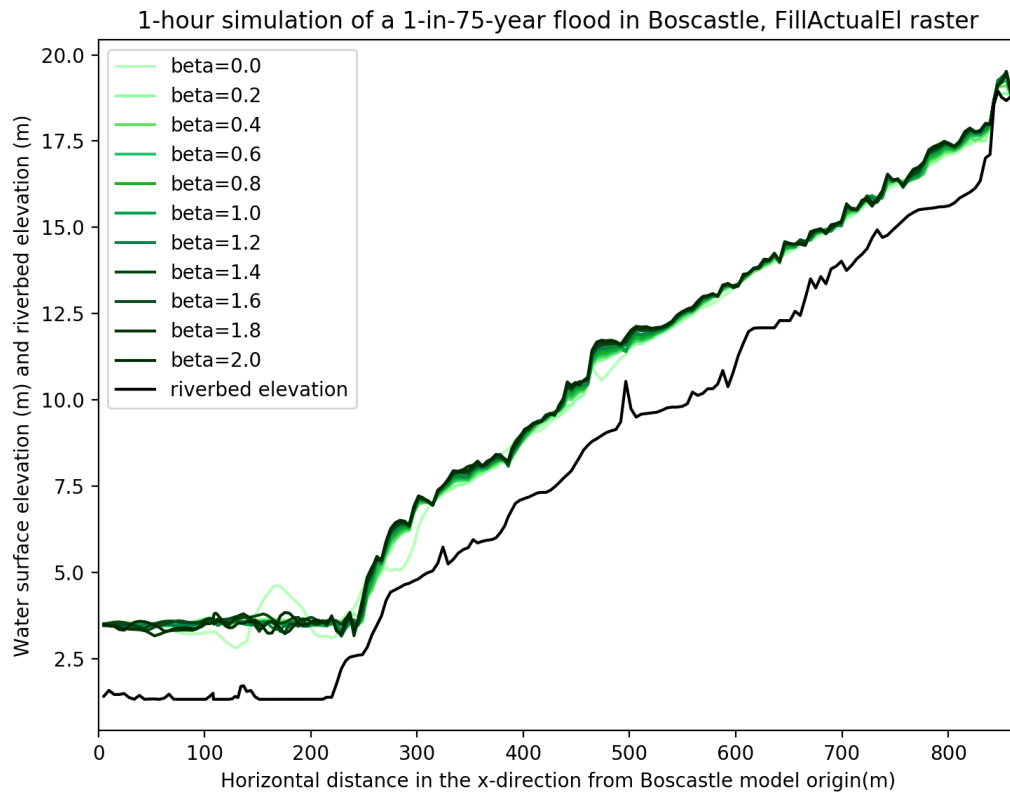
(d) RCL Scheme



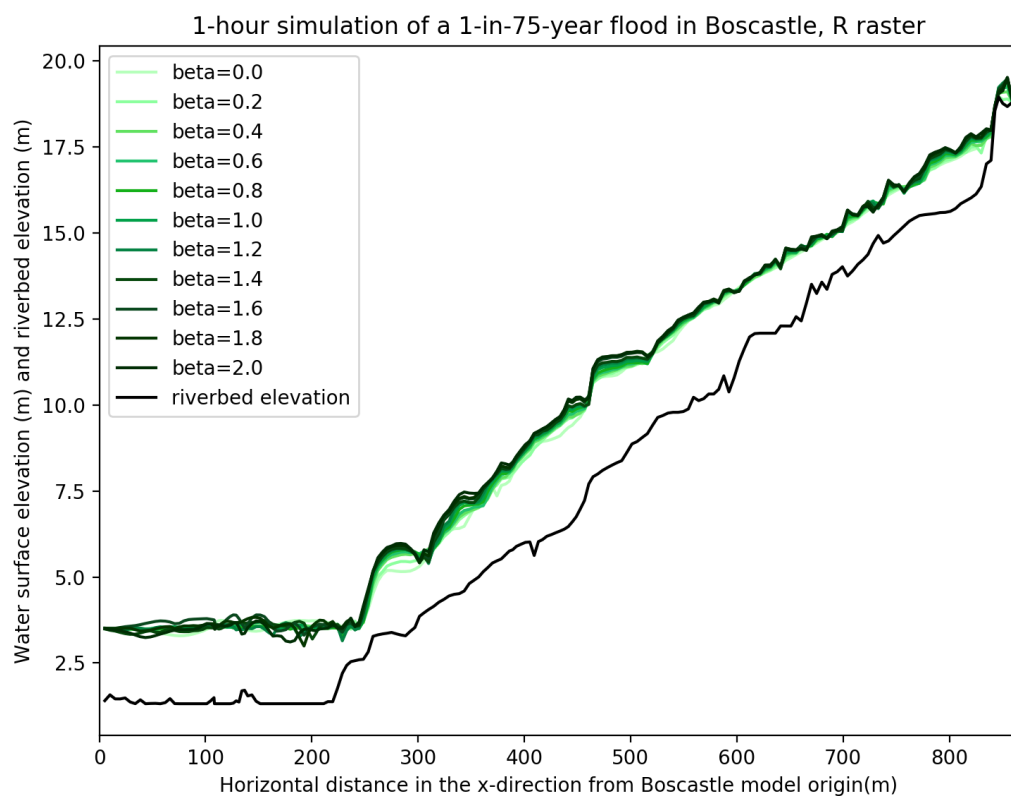
(e) RCLW Scheme

Figure 19: Simulation results of elevation for the 1-in-50-year chance flood

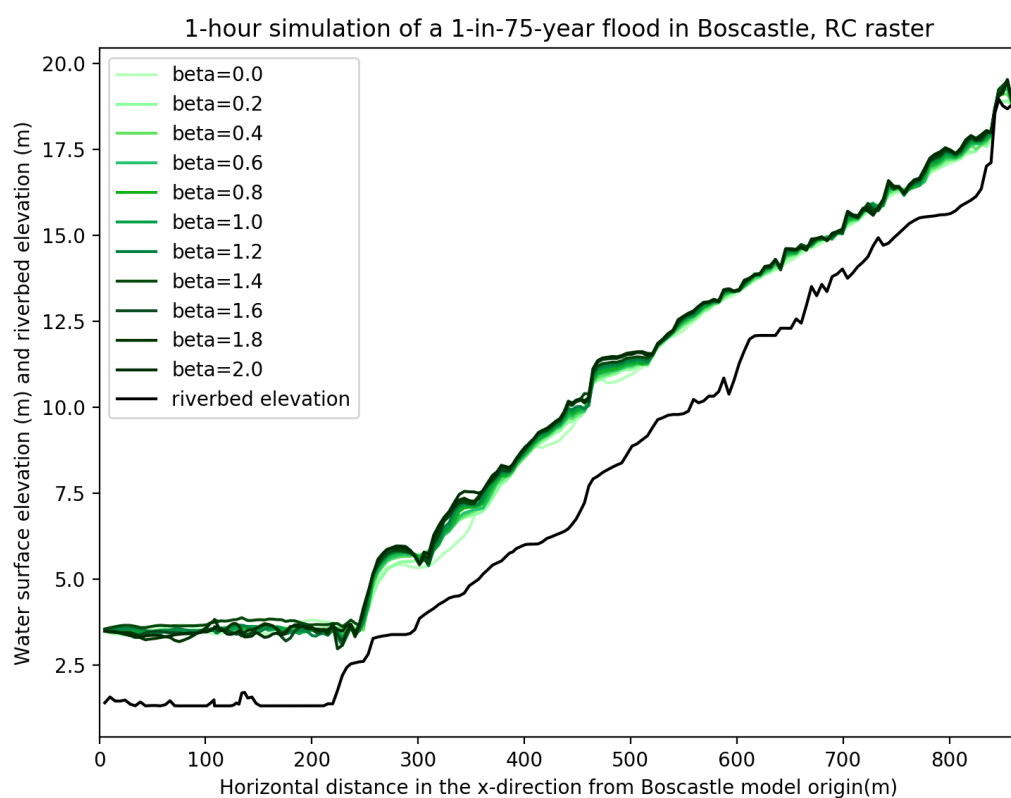
A.3 Results for 1-in-75 chance



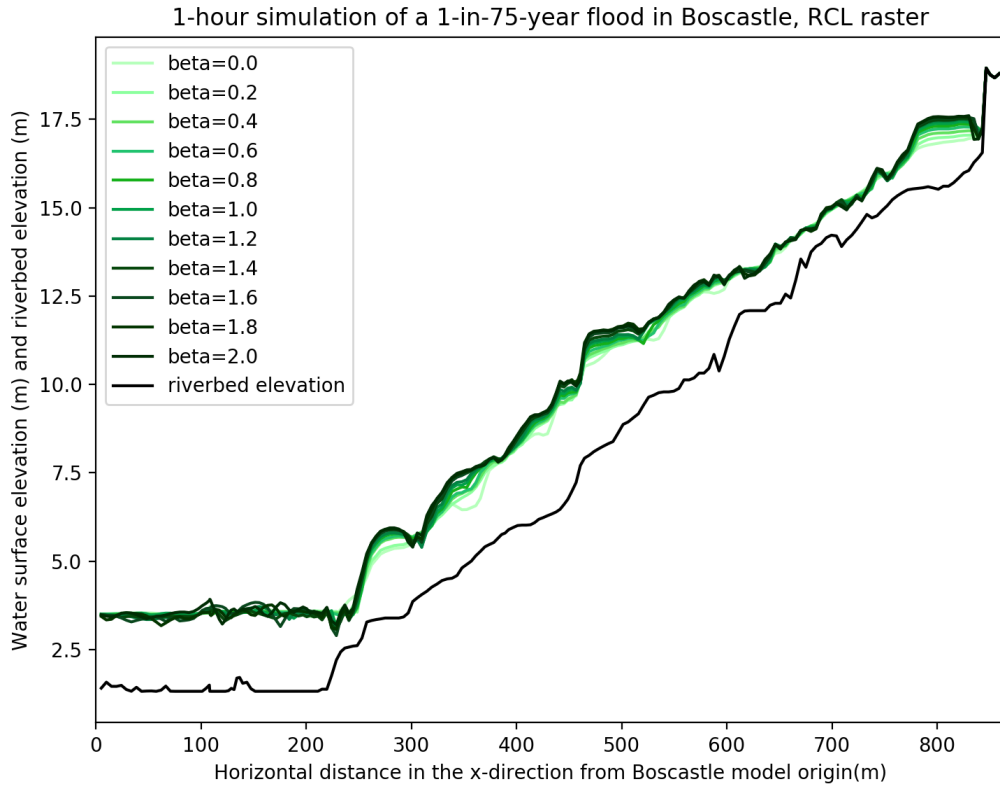
(a) FillActual Scheme



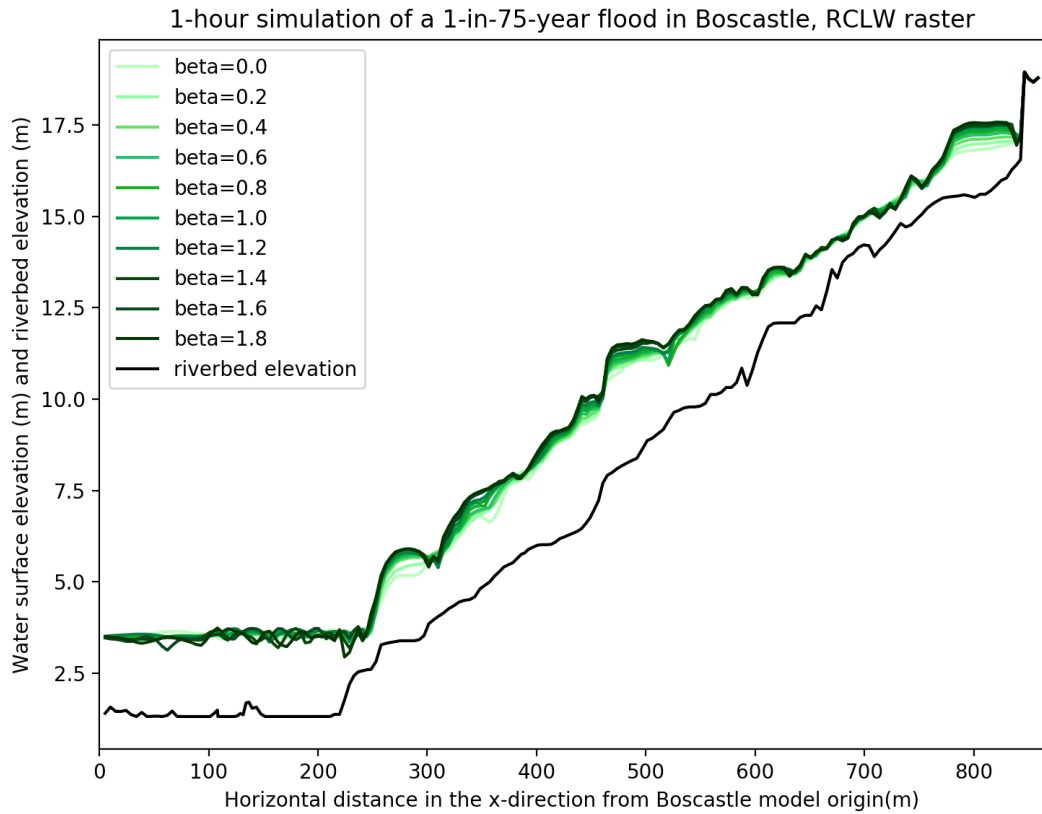
(b) R Scheme



(c) RC Scheme



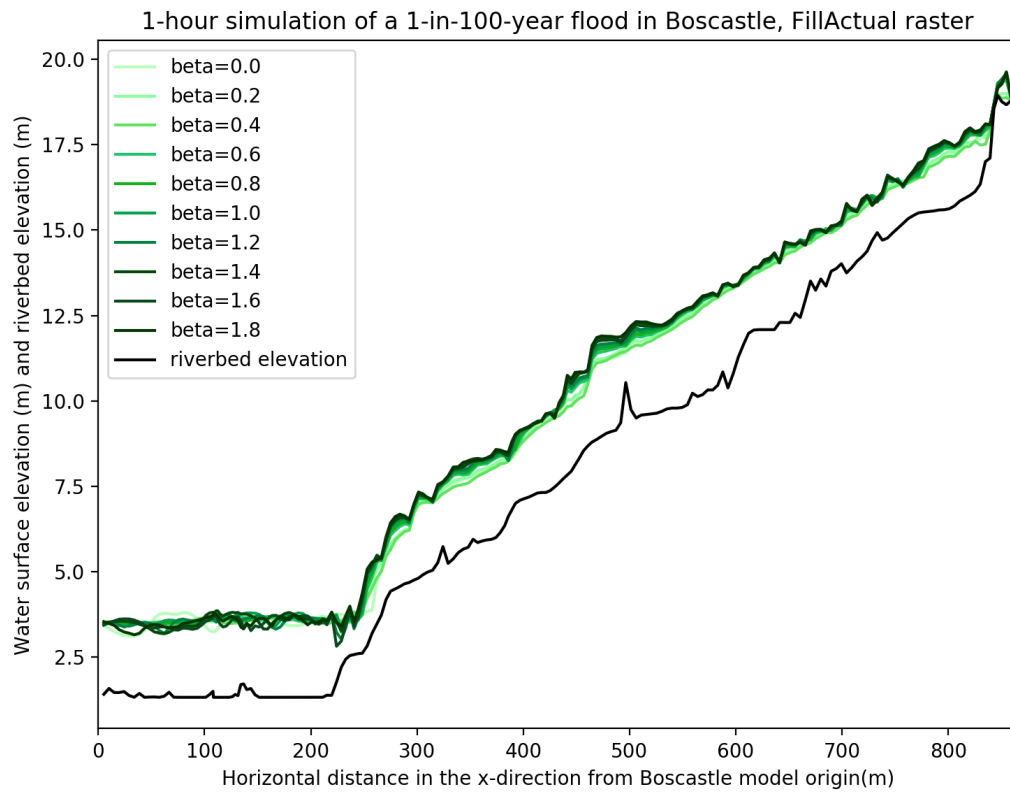
(d) RCL Scheme



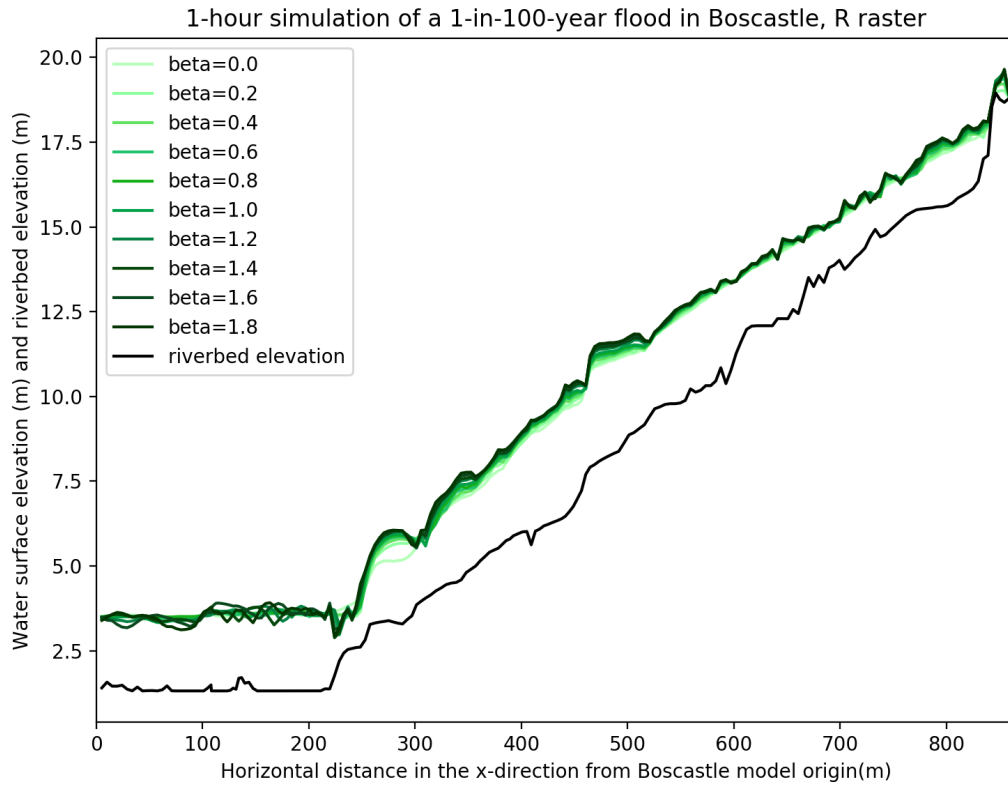
(e) RCLW Scheme

Figure 20: Simulation results of elevation for the 1-in-75-year chance flood

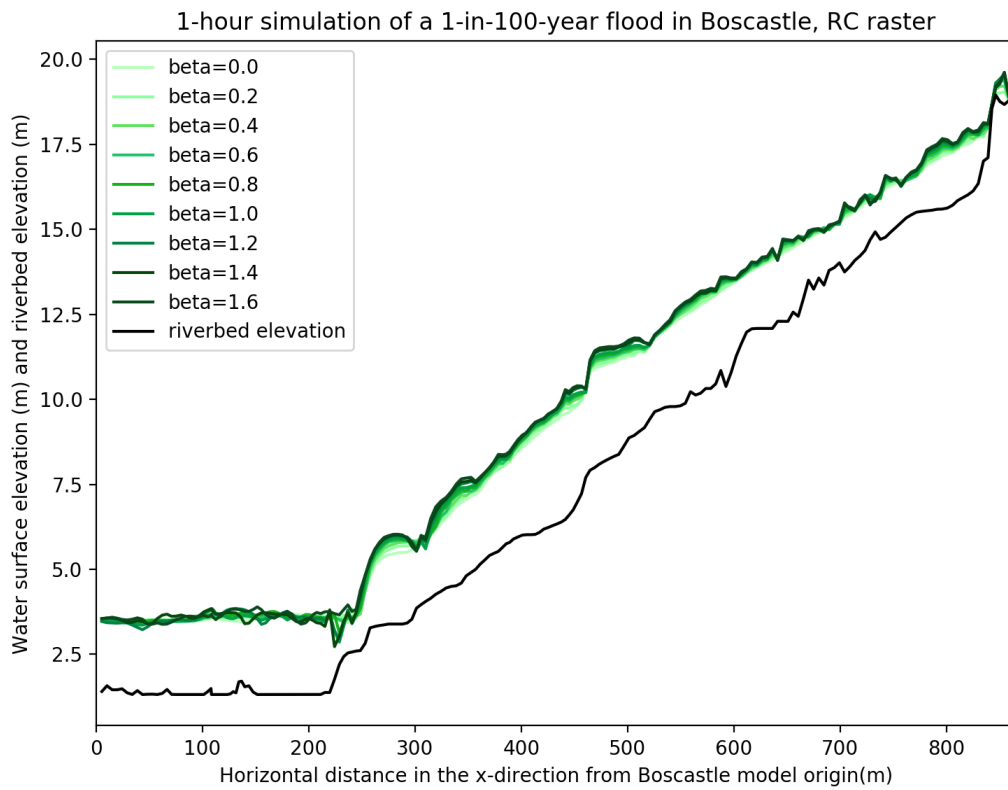
A.4 Results for 1-in-100 chance



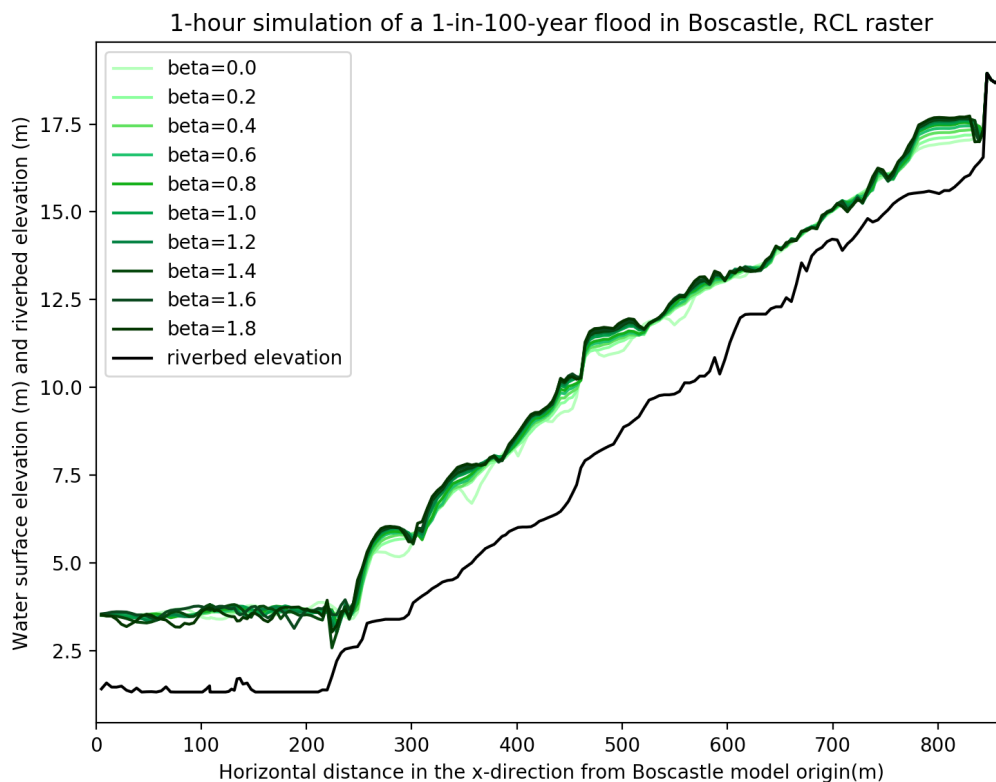
(a) FillActual Scheme



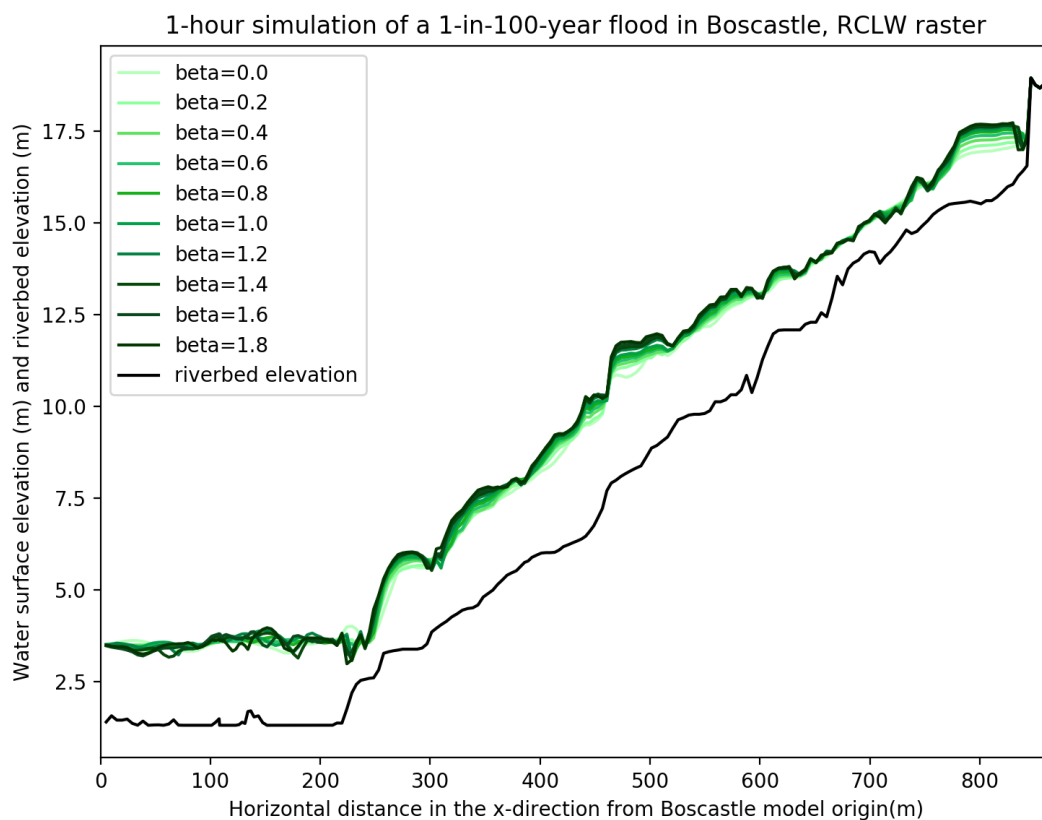
(b) R Scheme



(c) RC Scheme



(d) RCL Scheme



(e) RCLW Scheme

Figure 21: Simulation results of elevation for the 1-in-100-year chance flood

Appendix B IIB Project risk assessment retrospective

A risk assessment was performed at the start of the project to ensure that the safe execution of the project. The risks associated with this project were low due to the computational nature of the work. The main risks identified were Repetitive Strain Injury and eye strain due to significant amounts of time being spent on computers. These risks were minimised by taking regular breaks and maintain correct posture as suggested in the risk assessment.

In addition, the use of desktop computer in the Geotechnical Research Office (GRO) at the Department of Engineering was largely involved. To minimise risks associated with the use of the desktop computer, drinks and food were to be avoided as much as possible near the computer. Also, to reduce any fire hazards, all sort of plugs for equipment such as phone charger or electric kettle should be unplugged when leaving the GRO.

No further risks arose during the project and the risk assessment was appropriate to understand and prevent any potential risks.

Appendix C Covid-19 disruption

Remote access to the computer used in the Geotechnical Research Office was possible even before the Covid-19 situation, thanks to TeamViewer. Also, backup was made for the simulation results that were completed before the Covid-19 lockdown. However, the password for the TeamViewer would reset periodically by itself and a permanent password had to be set-up, which was made possible thanks to my supervisor. This somewhat delayed the process of running some final sets of simulations and caused some inconvenience when transferring some final parts of the simulation data to my personal computer. However, thanks to the computational nature of the project, the overall impact has been relatively small.