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Modelling Boscastle floods with different numerical techniques

Steven Tilbury, St John's College

Dr Dongfang Liang

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Technical Abstract

Flash flooding is a dangerous phenomenon that has caused and continues to cause immeasurable damage to countries and communities worldwide. The impact that it has on infrastructure can be catastrophic and it has claimed many lives. Therefore the need for accurate modelling software for design against these types of floods cannot be overstated. Flash flooding is widely analysed in industry using an Alternating Direction Implicit method of solving the shallow water equations in a generic two-dimensional model. This method however requires stability parameters to specify a maximum Froude number and a correction coefficient, both of which affect the values of the water surface elevations produced by the model. Moreover, improper conditioning of these parameters can lead to spurious oscillations and the numerical software failing to produce intelligible results. An alternate means of analysis developed by Dr Dongfang Liang, Cambridge University Engineering Department, provides a Total Variation Diminishing property to the well-established MacCormack method of analysis of the equations, which separates the analysis into two orthogonal, one-dimensional problems. This method is expected to be more accurate in its predictions of flash flooding, with its stabilising Total Variation Diminishing properties having a smaller impact on the accuracy than the parameters used by the Alternating Direction Implicit method. However it still requires a higher computational cost.

The two means of analysis are tested firstly through a hypothetical one-dimensional system to exaggerate the discrepancies between the models. The effect of flow criticality is examined for both models by varying the Manning coefficient of the flow for fixed stability parameters,

before the correction coefficient for the ADI model is varied to assess its effects. The models are then tested in two-dimensional analysis to simulate flash flooding in Boscastle. The models assess both a widely-recorded historical flood recorded within the British village and the effect of implemented mitigation measures within Boscastle on future design storms. The effects of the correction coefficient for the ADI model is again investigated for several of the cases.

The results within this document correlate with the initial expectations: that the newer method developed by Dr Liang produces more accurate and consistent results whereas the Alternating Direction Implicit method produces in general a larger water elevation and is less sensitive to the implementation of mitigation measures. Moreover the improper conditioning of the parameters has led to oscillations, with several simulations producing unintelligible results. However, the effect of a low correction coefficient has been determined to reduce the water level and any oscillations, which can therefore be utilised to produce a more accurate simulation. In spite of all of this, the discrepancies between the Alternating Direction Implicit method and the MacCormack Total Variation Diminishing method or measured values are small in comparison to the scale of the flooding event and have little effect on the predicted extent of flooding. Furthermore two-dimensional and hence real-life flooding events do not widely experience significant supercritical flow, especially during the initial stages, and as such the Alternating Direction Implicit model only gives a slight overestimate of the event. The positive benefits that the Alternating Direction Implicit method provides such as its large stability domain through the manipulation of the parameters and the lower computational cost outweigh this slightly conservative estimate and this therefore provides evidence as to why this method has prevailed within the analysis of flash flooding.

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1.1 – Flooding as a global hazard

Flash flooding is the phenomenon in which short, intense storms or infrastructural collapse cause a rapid flow of water into low-lying areas. Flash flooding remains a prevalent and pressing issue worldwide; the high velocities of the flows involved, coupled with the destructive nature of the floods, pose a serious risk to both infrastructure and human life. The ever-imposing threat of severe storms due to climate change has helped to heighten public perception and spending on revised infrastructure to prevent or reduce the potential impacts. For example, in Britain at the start of 2016 many of the existing flood defences were deemed to be inadequate and the Secretary of State for the Department for Environment, Food and Rural Affairs confirmed a commitment to a '£2.3 billion … six-year programme'[1] to reduce the flood risk facing the island. Many other countries have similar commitments in place in an effort to reduce the impact these future storms may have.

It is therefore extremely important that such flooding events are predicted and designed against as accurately as possible. Following the December 2015 winter floods in Britain, the Secretary of State for the Department for Environment, Food and Rural Affairs called into question the need to review the modelling software that is currently being used to analyse flooding events: 'in light of the extreme weather, we must ensure that it is fit for purposes for future decisions'[1].

1.2 – Flood modelling methods and the shallow water equations

The basis for analysis of flash flooding is established from what are known as the shallow water equations. These equations are derived through integration of the Navier-Stokes equations over a shallow water (vertical) depth and reflect mass conservation and conservation of linear momentum in the two horizontal directions respectively. Neglecting Coriolis and wind forces, the shallow water equations can be simplified as outlined in Appendix 7.1 Equation 1. Even with these simplifications, full analysis of these equations proves difficult and requires a high computational cost.

1.3 – Traditional numerical model (Alternating Direction Implicit scheme)

These equations can be solved directly and implicitly in a two-dimensional generic model. Alternating Direction Implicit (ADI) analysis uses stability conditions to achieve this, by defining a maximum Froude number within the flow and a stabilising parameter Beta, which alters the flow at values other than 1.00. However this can still prove to be unstable if the conditions are not optimally defined. Moreover the analysis offers no shock-capturing capability required for the analysis of supercritical flows, which can be common in the high-velocity nature of flash flooding.

However this means of analysis is widely used to model flash flooding, due to its large stability domain through the manipulation of the relevant parameters. It also provides a favourable balance between computational cost and accuracy. However its fundamental inability to correctly predict trans-critical and supercritical flows that can occur in flash flooding make it highly unsuitable. Secondly the simplifications and corrections that it introduces to analyse the shallow water equations are potentially unjustifiable. Finally the lack of any shock-capturing capability, particularly over steep flows such as hydraulic jumps and dam-breaks that can occur regularly in flash floods, results in spurious oscillations.

1.4 – Shock-capturing model (MacCormack Total Variation Diminishing scheme)

An alternative method of analysing the shallow water equations (TVD-MacCormack), which involves the manipulation of a well-established MacCormack scheme, has been developed by Dr Dongfang Liang, Cambridge University Engineering Department. The diffusion (second order) terms present in the shallow water equations are also ignored, which allows for the three equations to be solved as two orthogonal one-dimensional problems. The equations for each orthogonal direction are listed in Appendix 7.1 Equation 2.

This scheme is additionally altered by introducing a stabilisation term to remove numerical oscillations near any steep gradients. This provides the scheme with a Total Variation Diminishing property and greatly reduces the required computational cost at only a small expense of the accuracy.

1.5 – Aims of the study

Despite requiring a still higher computational cost than the ADI method, the TVD-MacCormack method's predicted ability to more accurately analyse both subcritical and supercritical flows without producing erroneous oscillations makes it far more theoretically suited to the analysis of flash flooding. While the ADI method is already well-established in most analyses and as the benefits of the TVD-MacCormack method are yet to be widely proven, this study aims to assess the two different methods in a variety of different one-dimensional and two-dimensional situations in order to provide further evidence and conclusions as to a preference for either analysis.

2 – One-Dimensional Analysis

The two methods of analysis (ADI and TVD-MacCormack models) are first tested for flow over the bed profile shown in Figure 1. The bed profile is kept constant in one direction to provide a one-dimensional simulation of the flow. Each model specifies a certain Manning coefficient and boundary conditions for the flow rate at the top of the profile and the water depth at the bottom of the profile. Each simulation is run until steady flow is achieved over the profile.

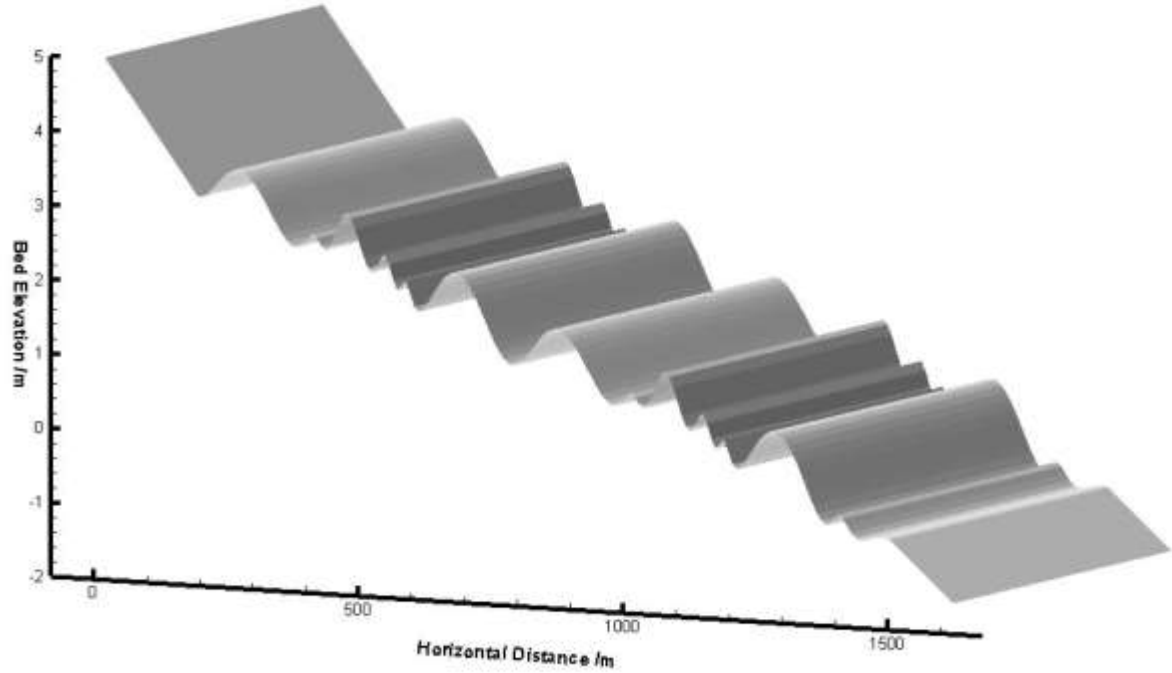

Figure 1 - One-dimensional bed profile.

2.1 – Flow criticality (maximum Froude number)

To achieve stability for this means of testing, the ADI software was limited to subcritical flow by setting the maximum Froude number to be 1.00 and the Beta coefficient was effectively removed by setting it to be equal to 1.00 as well. In order to investigate the performance of both models in both subcritical and supercritical flows with the set parameters, the input conditions were varied so as to change the Froude number of the flow. Definitions for the Froude number and flow criticality are defined in Appendix 7.1 Equation 3 and Appendix 7.1 Equation 4 respectively.

For the specified bed slope profile as shown in Figure 1 and set boundary conditions on the water depth and flow rate, the criticality of the flow has been manipulated by varying the

Manning coefficient for each model. Figure 2 and Figure 3 show the modelled water surface elevations and respective Froude numbers for each model and for Manning coefficients of 0.030 and 0.029 respectively.

For the first case (Figure 2), the flow rarely experiences supercritical nature and as such the models predict almost identical flows. The only notable discrepancies are at the few supercritical locations, where the ADI model produces a slightly larger water depth and hence a lower Froude number, effectively rounding off the "spikes" at each hydraulic jump. This reflects the ADI model's inability to model the nature of any supercritical flows, whereby it generates a larger water depth and reduces the Froude number closer to subcritical conditions and below the maximum Froude number set initially.

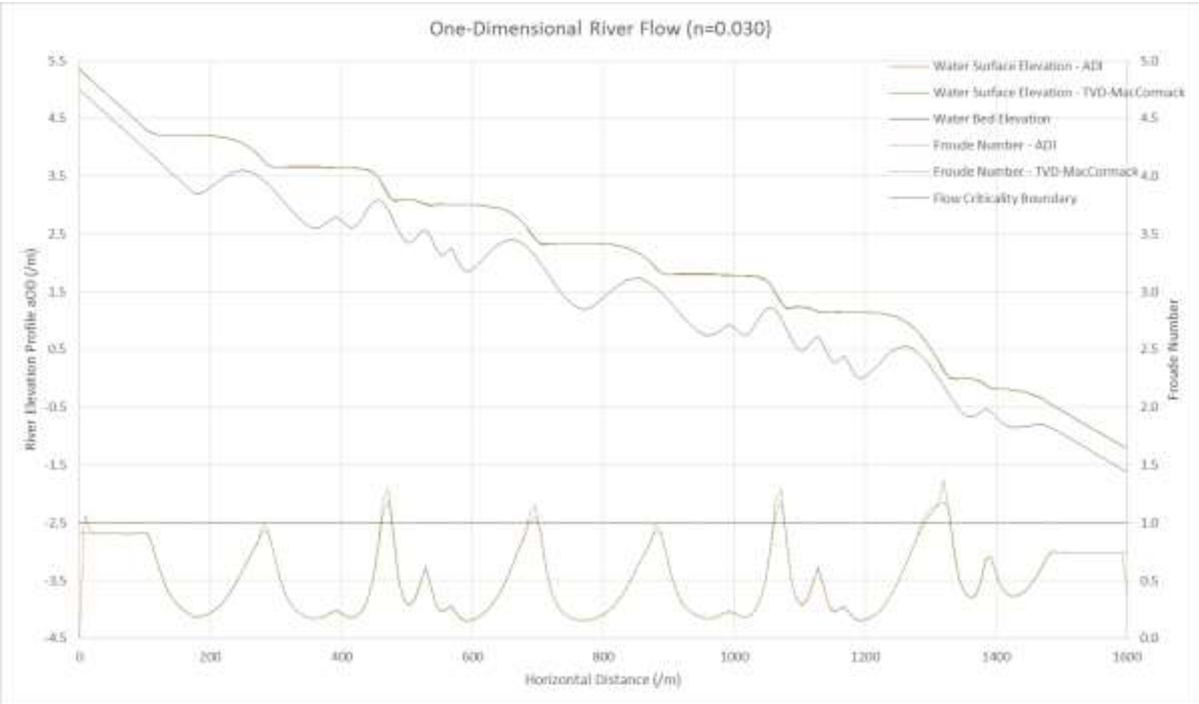

Figure 2 - One-dimensional analysis for a Manning coefficient of 0.030.

This is far more noticeable for the case in which the Manning coefficient is lowered (Figure 3). The flow should now more frequently and more significantly exceed a Froude number of 1.00 and the TVD-MacCormack model models this change effectively; however the ADI model is far less capable at modelling the event and so produces a significantly higher water depth across the entire domain. This again has the effect of a reduction in Froude number to near subcritical levels, which results in a great expense in accuracy.

Figure 3 also demonstrates oscillations downstream of the lower hydraulic jumps, showing the ADI's lack of shock-capturing capability. This is even more apparent for a Manning coefficient defined as 0.031 (Figure 4); as the flow progresses down the slope it becomes progressively more unstable and gives rise to spurious oscillations. The TVD-MacCormack model does not ever experience these oscillations.

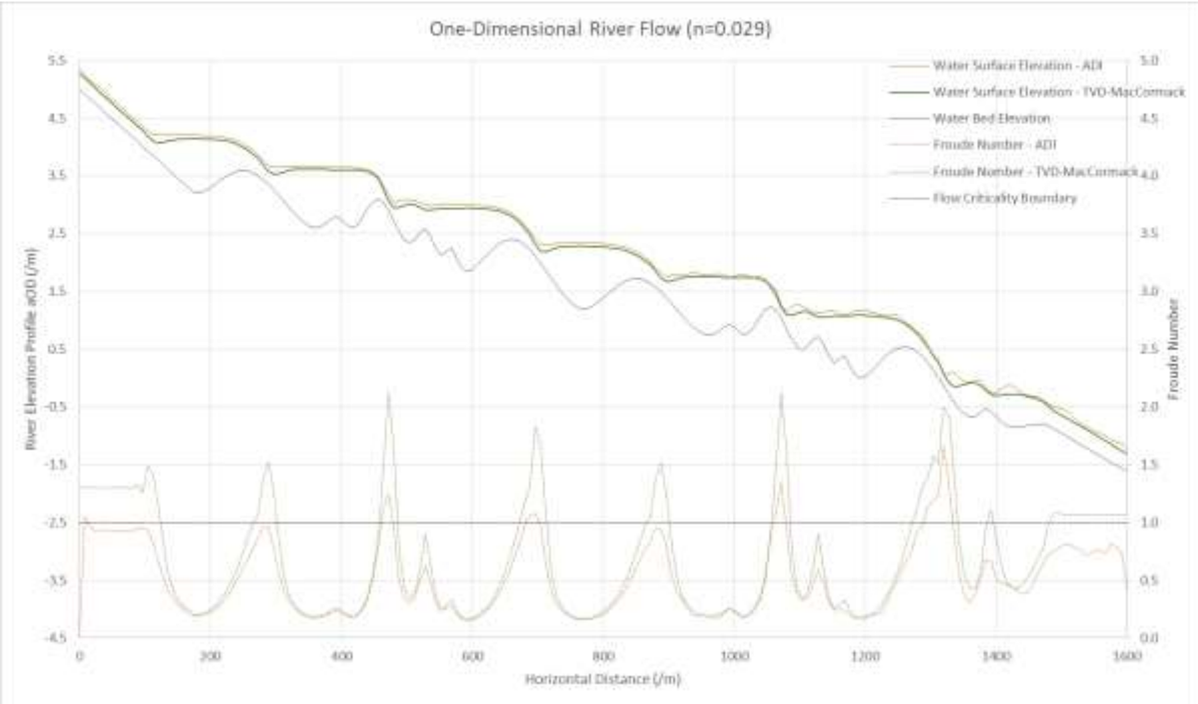

Figure 3 - One-dimensional analysis for a Manning coefficient of 0.029.

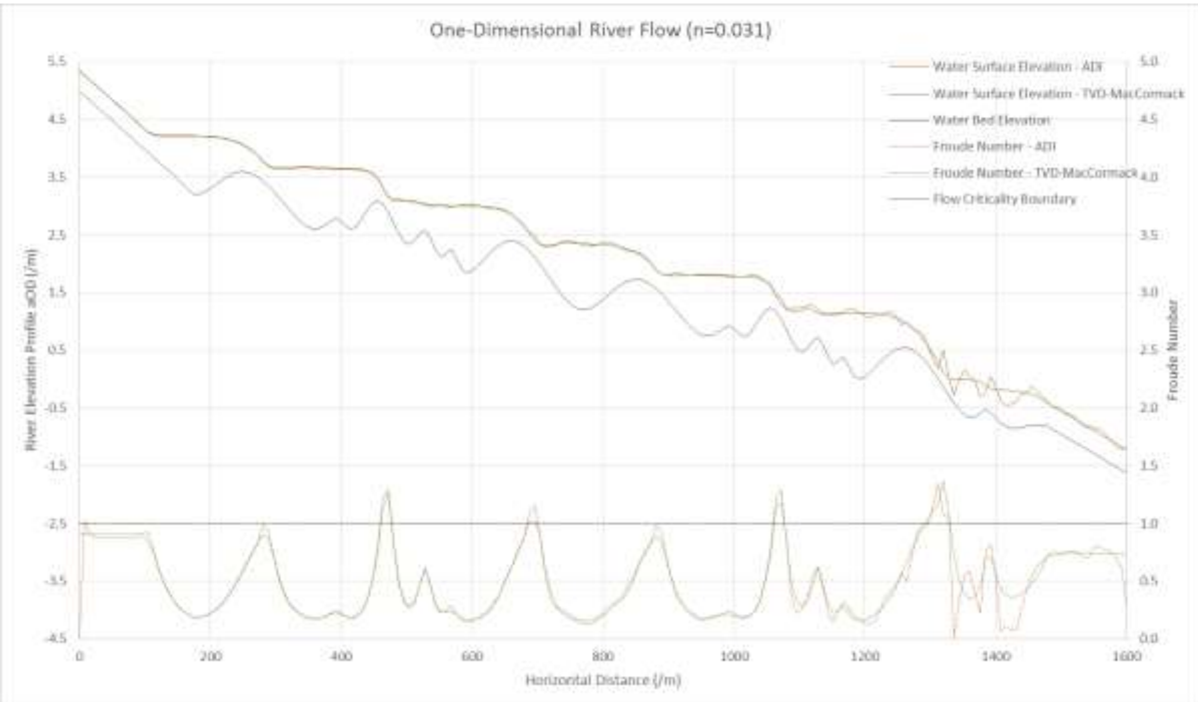

Figure 4 - One-dimensional analysis for a Manning coefficient of 0.031.

2.2 – Correction coefficient (Beta)

It is expected that these oscillations can effectively be removed not only through the modification of the maximum Froude number, but also by varying the correction coefficient, Beta. However, to observe the sole effect that varying Beta to be values other than 1.00 has on the accuracy of the water surface elevation, the ADI model must be tested for the case in which the maximum Froude number is fixed at an extremely large value (Fr=100). Figure 5 shows the results from the ADI model for this case in which the Manning coefficient is specified as 0.031. For steady flow, as Beta increases the water surface elevation rises for the most part, however the gradients become more extreme across the hydraulic jumps and the flow has larger oscillation. For the one-dimensional case the model becomes unstable above values of 1.50 and crashes. Therefore despite the ability of the Beta parameter to stabilise the flow, it does have an impact on the accuracy of the water surface elevation profile.

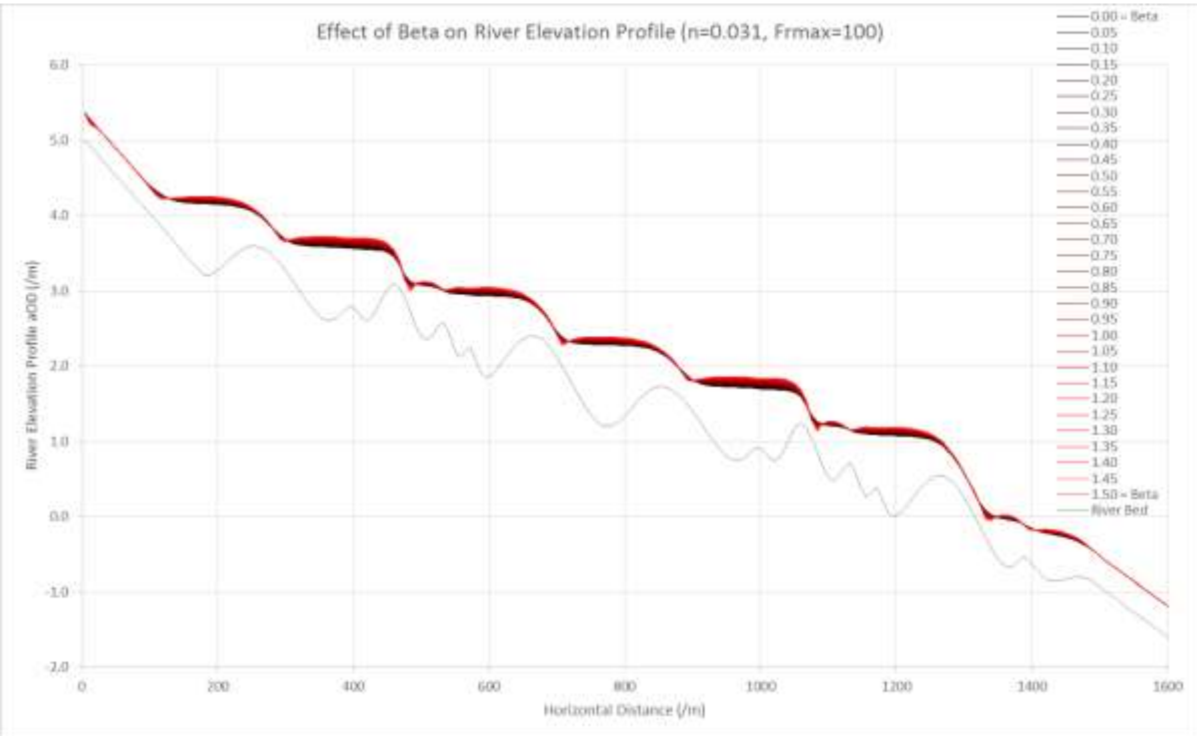

Figure 5 - Effect of Beta on the ADI model for large allowable Froude number.

3 – Two-Dimensional Analysis – Historical Flooding

While the one-dimensional analysis has provided clear differences between the performances of each model, the outputted results are rarely useful in the comparison against real two-dimensional flash floods. The two models are therefore extended to the simulation of a well-documented historical flooding event in Boscastle.

3.1 – Boscastle 2004

Boscastle is a small village and fishing port in Cornwall in Britain. The village is situated around the intersection of River Valency and River Jordan, which continue into the Celtic Sea, as shown in Figure 6. Boscastle is inherently vulnerable to flash flooding, with notable floods occurring in 1950, 1958 and 1963, 2004 and 2007.

Figure 6 - Aerial view of Boscastle village[3] and the centrelines of River Valency and River Jordan.

The 2004 flood is the most severe flood ever recorded in the village. The flood was devastating enough by the sheer magnitude of flow that entered the catchment: modelling suggested that a combined peak discharge of the order of 180 m^3s^{-1} flowed through the two main rivers and measurements suggested up to 200 mm of rainfall fell within a 24 hour period[2]. It was estimated that on balance the event had only a 1 in 400 chance of recurring in any one year[2] and so the flood defence measures that were already in place had little ability to protect the village against these flows and were easily overwhelmed.

The flood was further exacerbated by multiple other factors:

- Existing River Valency capacity – at the downstream end of River Valency, the channel capacity was designed only against a 1 in 10 chance of recurring storm and so the flow quickly and detrimentally exceeded the river boundary in this location, flooding the downstream end of the village.
- Channel and bridge obstructions – due to the severity of the storm, many trees upstream of the village and cars from a car park at the eastern end of the village were washed away and carried into the river channel. These filled the river with debris and became jammed at the central bridge, located where the River Jordan outfall meets River Valency. The debris prevented any further flow underneath the bridge, which was therefore diverted over the top of the bridge and into the village centre.
- Geomorphology – the storm also resulted in the deposition of a large amount of sediment from upstream. The high flows deposited the sediment downstream of the River Jordan outfall, which further reduced the channel's limited capacity.
- Catchment characteristics – the Boscastle catchment itself has very steep topography. Moreover the catchment has a very thin layer of soil above impermeable rock. Together these result in very large and fast runoffs of any rainfall, making the village very susceptible to flash flooding, particularly in short, intense storms.

3.2 – Data collaboration

HR Wallingford, an engineering consultancy, undertook extensive data collection from photos, observations and residual evidence of the flood. The wealth of evidence available has allowed for the 2004 flood to be an excellent case study against which flooding software can be assessed.

LiDAR, a laser-based technology, has been used by a previous fourth-year student to determine elevations for the area of Boscastle and produce a digital elevation model (DEM) that can be used for analysis. The Boscastle site has been separated into a grid of 860 elements by 470 elements, with each element specifying an elevation as provided by LiDAR. Due to low water levels in both rivers during normal conditions, the DEM specifies an initial water depth and flow velocities only at the mouth of River Valency by the Celtic Sea (see Figure 7). Further conditions state initial discharges at the upstream ends of River Valency and River Jordan flowing into the model (see Figure 7). During the flood the flow through River Jordan exceeded the underground outflow and passed through a hotel (located at X=450m; Y=80m). The initial flow of River Jordan is therefore specified at the edge of this hotel to simulate the conditions of the flood. Moreover, since the blocking of the central bridge was critical to the severity of the impacts of the 2004 flood, the topography of this part has been raised to simulate a blocked bridge from the start of the flood. Following initial testing of the ADI model, the maximum Froude number and Beta were set at 1.00 and 0.80 respectively to achieve stability.

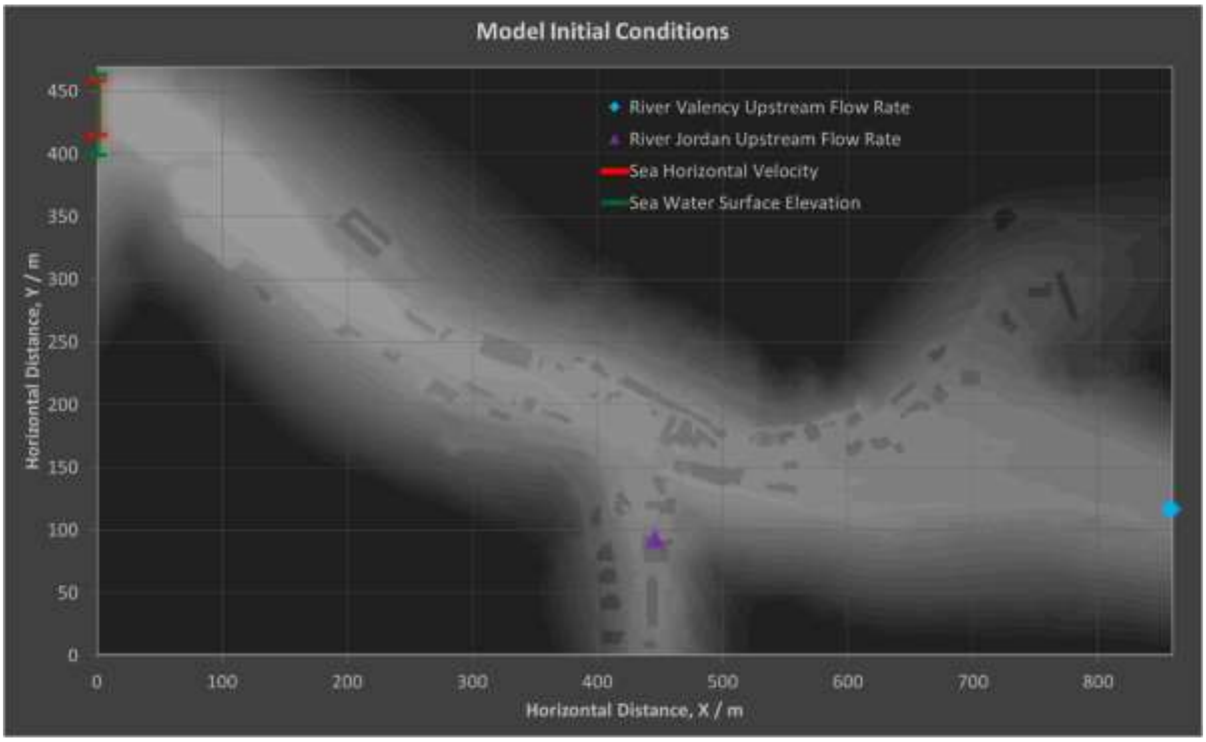

Figure 7 - Grid model of Boscastle and the specified initial conditions for the 2004 flood.

From observations alone HR Wallingford determined the hydrograph for Boscastle on the day of the flood as shown in Figure 8. This hydrograph has been idealised as shown in Figure 9 in order to simplify the calculations required for each software. This idealised hydrograph is inputted into each software by separating the flow between the two rivers, as shown in Figure 10.

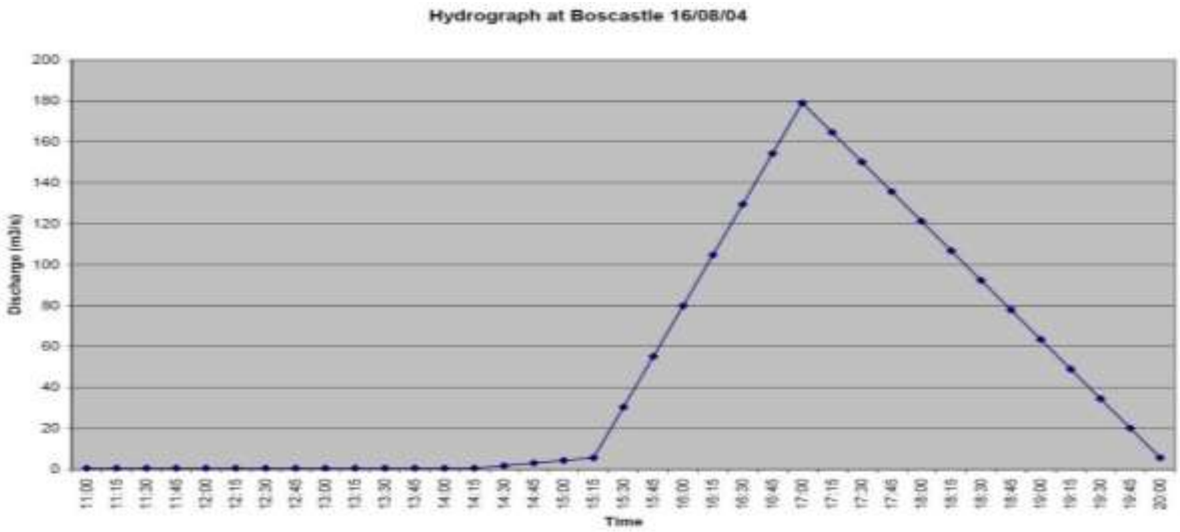

Figure 8 - Reconstruction of the hydrograph shape at Boscastle[4].

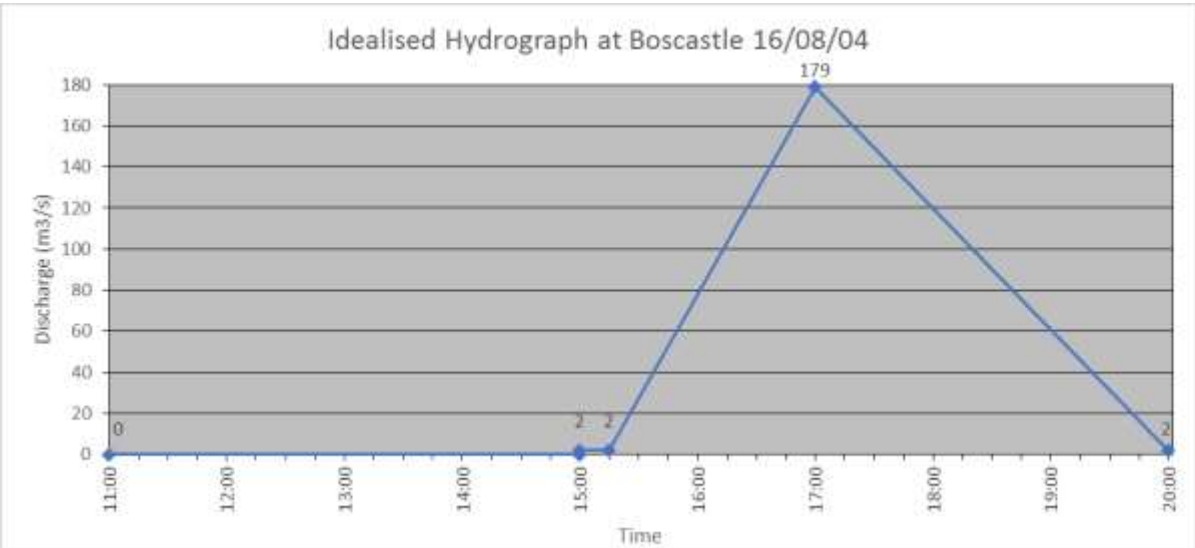

Figure 9 - Idealised hydrograph used for analysis.

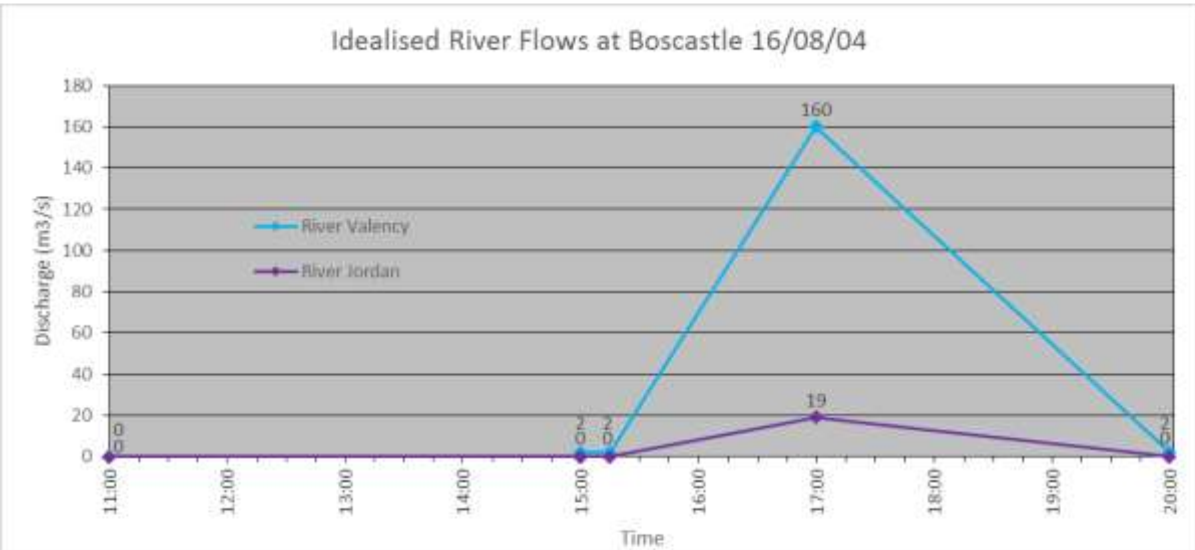

Figure 10 - Specified hydrographs for River Valency and River Jordan in analysis.

HR Wallingford also used the collected data to determine the maximum water surface elevations at ninety distinct locations within the catchment, as shown on Figure 11. The coordinates of each of these points have been determined on the numerical grid (to a precision of 0.5m).

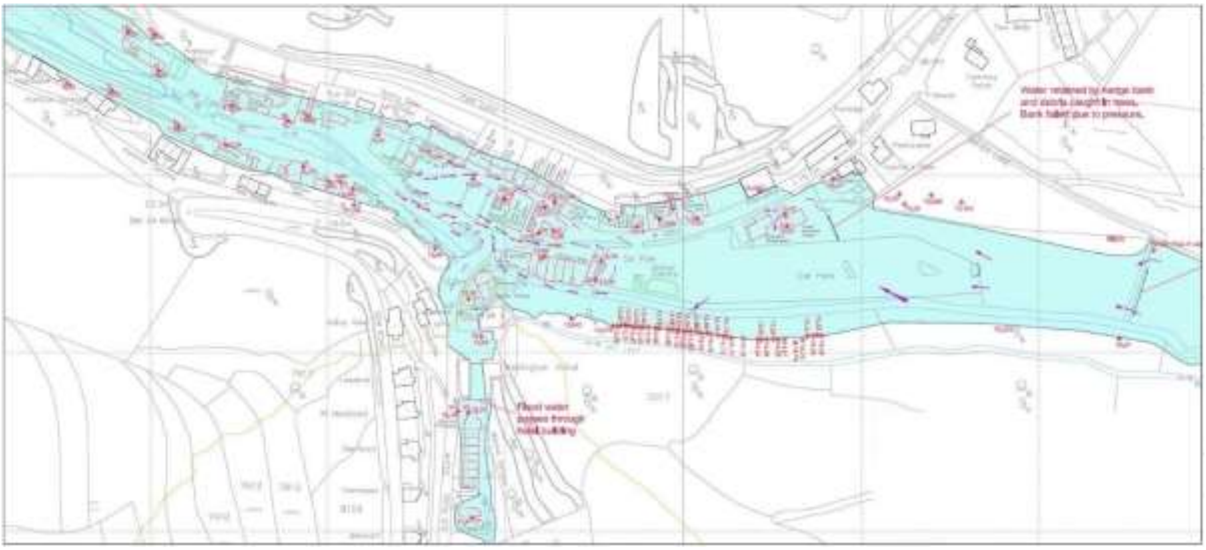

Figure 11 - Maximum water surface elevations at ninety distinct locations[4].

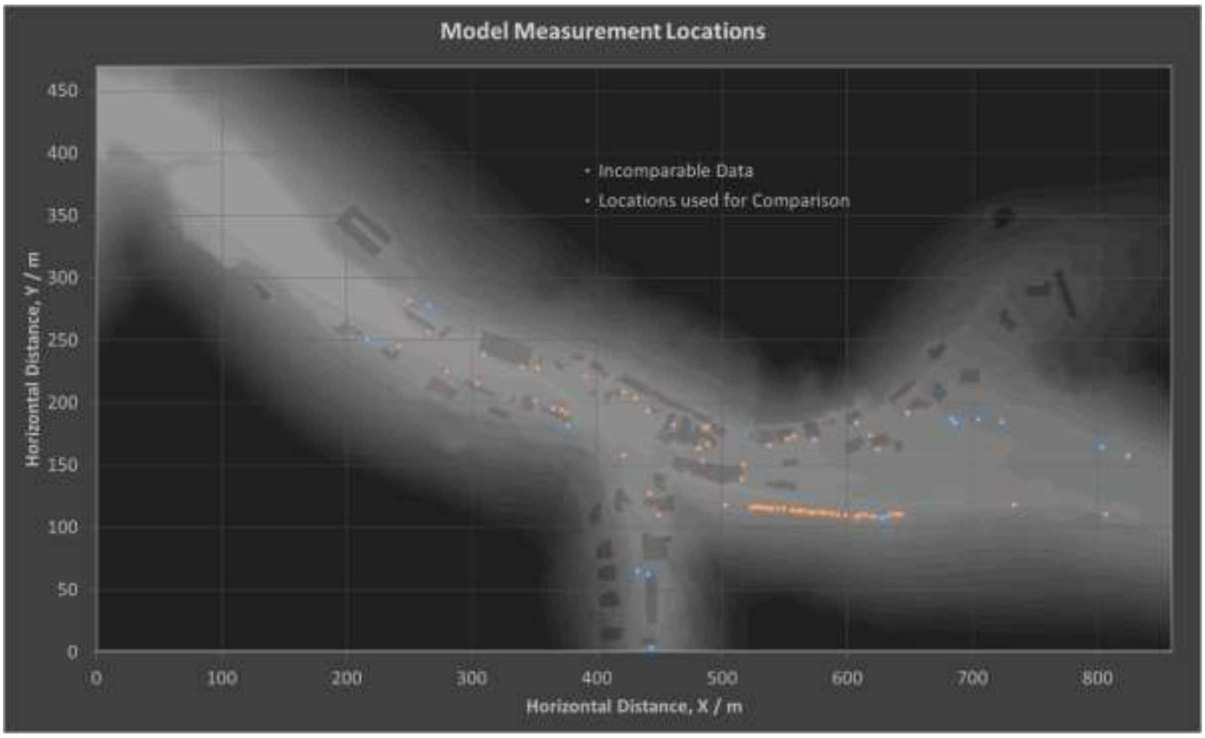

Figure 12 - Elevations located on the DEM and chosen for comparison.

Figure 12 shows these locations on the numerical grid. Despite showing evidence of residual water, ten of these locations appeared dry on both HR Wallingford's flood extent diagram (Figure 11) and for both numerical models; as such these points were excluded from any analysis. Moreover, three additional locations were upstream of the model's specified input for River Jordan and so also couldn't be used for any analysis. This therefore provided seventy-seven points of comparison.

3.3 – Measured locations

These seventy-seven locations have been tested on each model for the maximum water surface elevation that they experienced throughout the flood duration. These elevations are plotted on Figure 13 against the values determined by HR Wallingford. The central line indicates any values for which either model is correct; any results to the left of this line are conservative and overestimate the water level experienced; any values to the right of this line under-predict the actual water level. As witnessed for the supercritical flow in the one-dimensional simulation for a low Manning coefficient, the ADI software produces higher water levels, with the majority of results falling to the left of the line of correct values. The difference in water surface elevation is small in comparison to the absolute values for the elevations and thus the errors in values are not excessive. While the TVD-MacCormack software also produces a selection of inaccurate data, the values correlate far better to the line than the results of the ADI software.

These same results are plotted against horizontal distance on Figure 14. While there is a notable inability of both models to accurately predict the points downstream of the blocked bridge (at X=450m), the TVD-MacCormack model far better predicts the values upstream. The conservative predictions of the ADI software are again apparent, with the elevation profile plotting consistently higher than the TVD-MacCormack profile and the measured values. Moreover, Figure 14 shows that the ADI software has far smaller changes in water level across the upstream region and thus is far less able to correctly model the topographical changes in this region. While the TVD-MacCormack results have not unanimously produced highly accurate values, the simulation appears to model the water elevation and hence topographical changes across the region well.

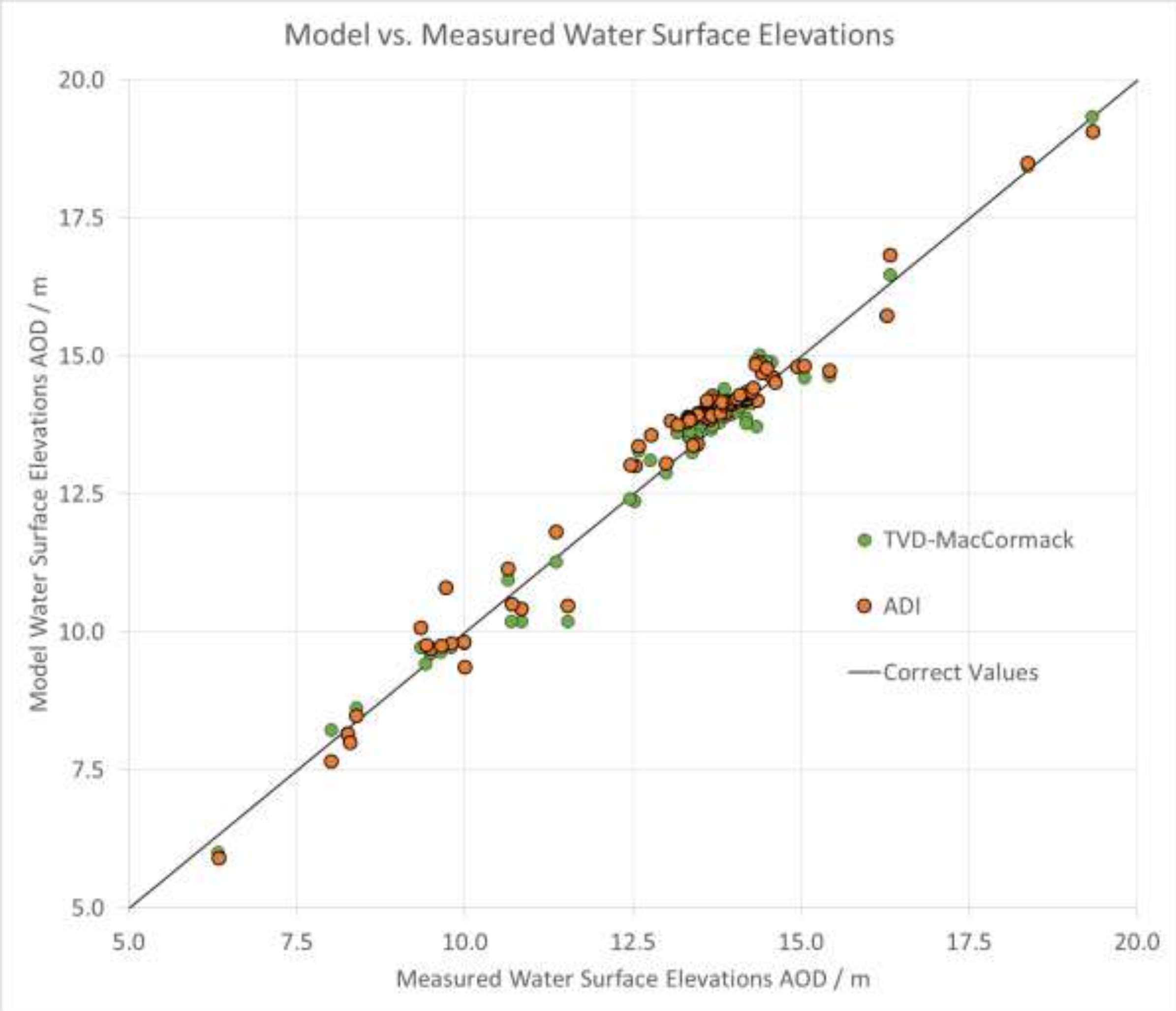

Figure 13 - Comparison of modelled and measured maximum surface water elevations.

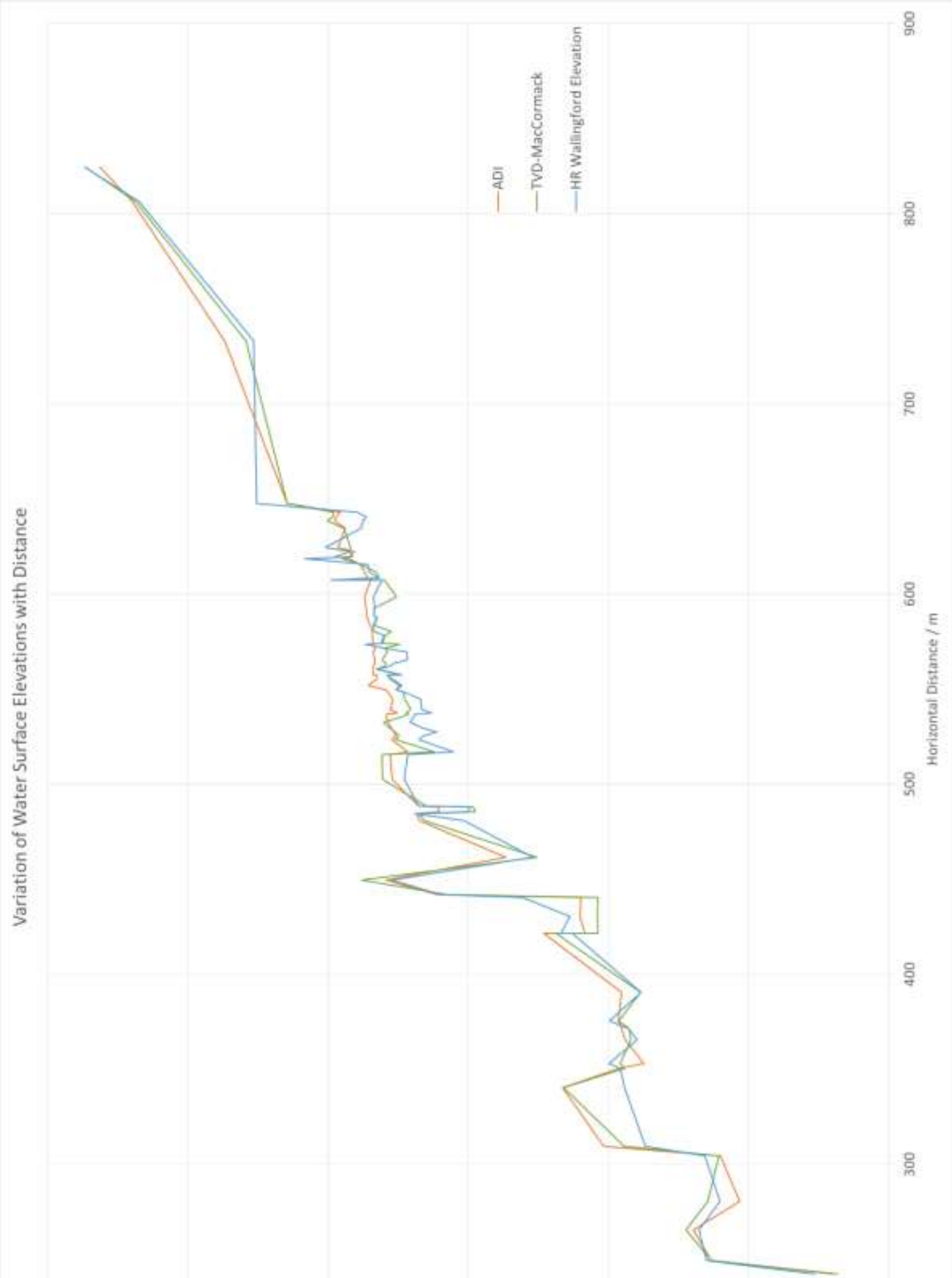

Figure 14 - Variation of water surface elevation with horizontal distance.

3.4 – Flood extent

The extent of the flood for each software have been overlaid (in dark blue) on top of the extent provided by HR Wallingford in Figure 15 and Figure 16 respectively. Despite differing water surface elevations as discussed above, both models produce fairly synonymous profiles that correlate with the measured extent, with the only notable discrepancies being at the car park towards the upstream northern bank of River Valency.

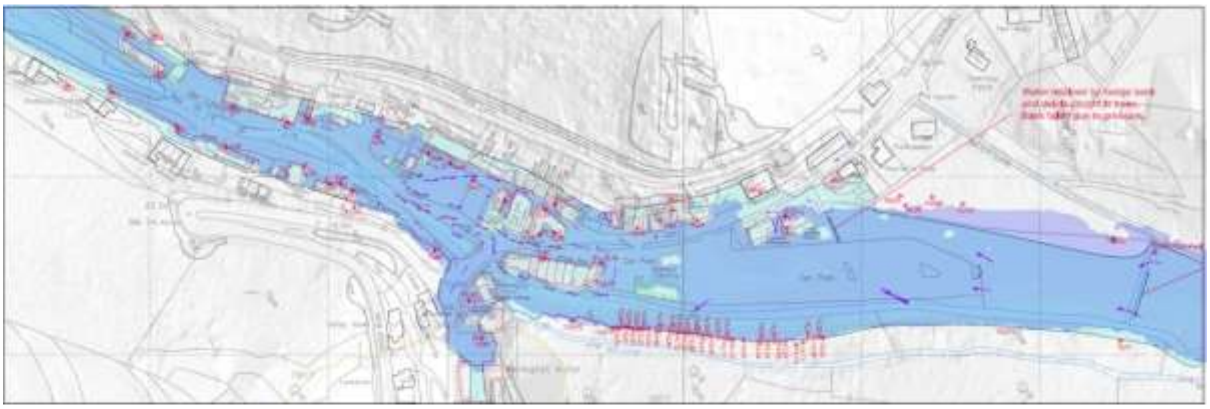

Figure 15 - Measured (HR Wallingford[4]) and modelled ADI flood extent.

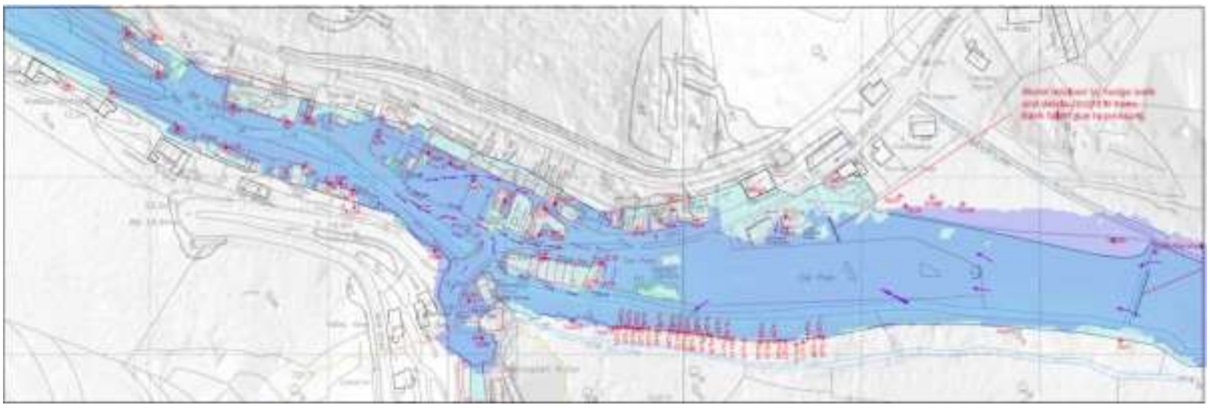

Figure 16 - Measured (HR Wallingford[4]) and modelled TVD-MacCormack flood extent.

As both models have experienced a similar discrepancy it is likely due to simplifications within the software. For example, the extreme flooding of the car park caused a large amount of debris, including submerged cars, to move through the village. These cars had a large effect on the flow paths of the flood and so likely caused blockages that resulted in the flood water receding to the north-easternmost houses that were determined to be flooded by HR Wallingford. Other than the blocking of the bridge, the obstructions resulting from these cars have not been modelled in either simulation.

3.5 – River progression

In order to assess the effect of the extent of supercritical flow on the ADI values, the progression of flow through River Valency itself has been modelled at different times during the storm. For each of these times, the Froude number of the flow through the entire village has also been assessed. Measurement locations have been selected on the grid along the river centreline, at X=10m spacing as shown on Figure 17. As no data has been recorded for the levels within the river, the ADI data is compared solely against the TVD-MacCormack data, which after the previous comparisons is assumed to accurately resemble the historical conditions.

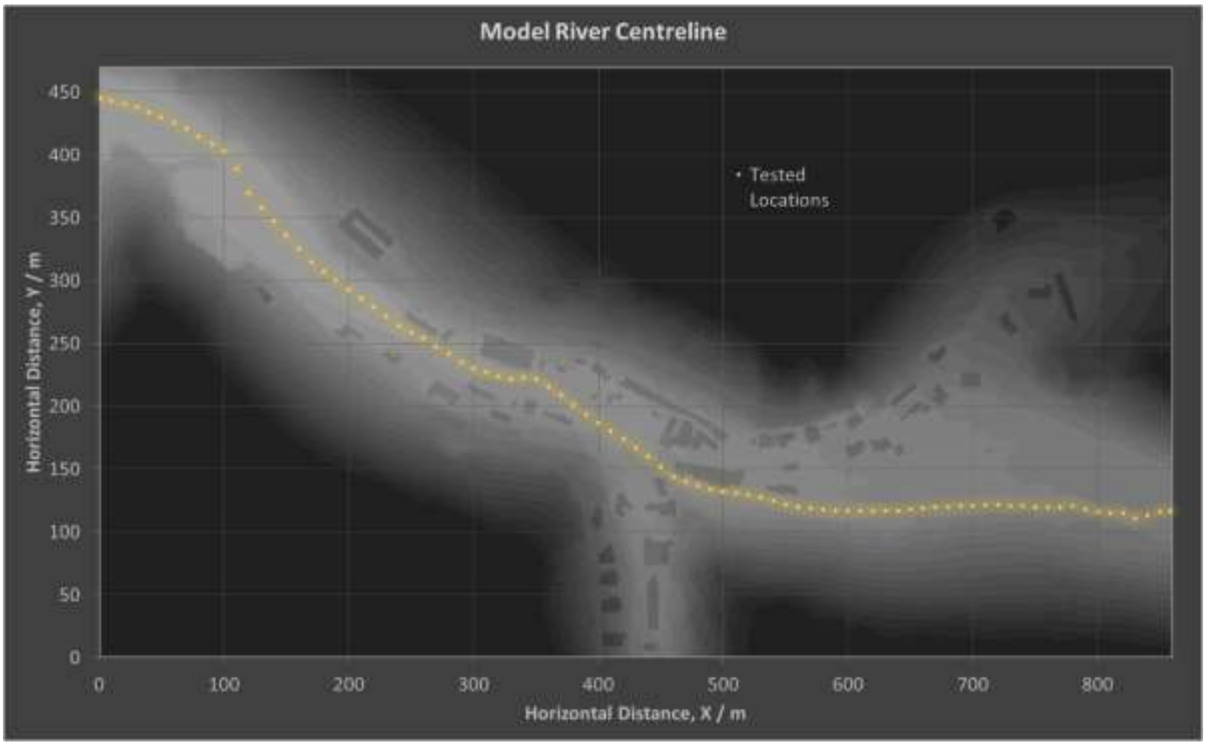

Figure 17 - Tested locations along the river centreline.

Figure 18, Figure 19, Figure 20 and Figure 21 show the simulated water surface elevations along the river centreline at separate times during the storm. These figures also show the locations within the catchment for which the TVD-MacCormack model experiences (and hence what is assumed to be where the actual flood experienced) supercritical flows that the ADI model is expected to have difficulty in predicting.

The first case that is considered is at 15:20, 20 minutes into the storm, for which the majority of the flow is still contained within the River Valency boundary. Figure 18 shows very little difference between the water surface elevations across the two models. Correspondingly,

Figure 18 also shows only scattered locations at which the Froude number starts to exceed 1.00, with predominant locations at only the blocked bridge and at the upstream end of the river. Thus the accuracy of the ADI software is not largely affected by flow criticality at this stage in the flood and produces a relatively accurate simulation, with the only notable discrepancy (at X=550m) likely due to the supercritical hydraulic jump over the blocked bridge slightly downstream.

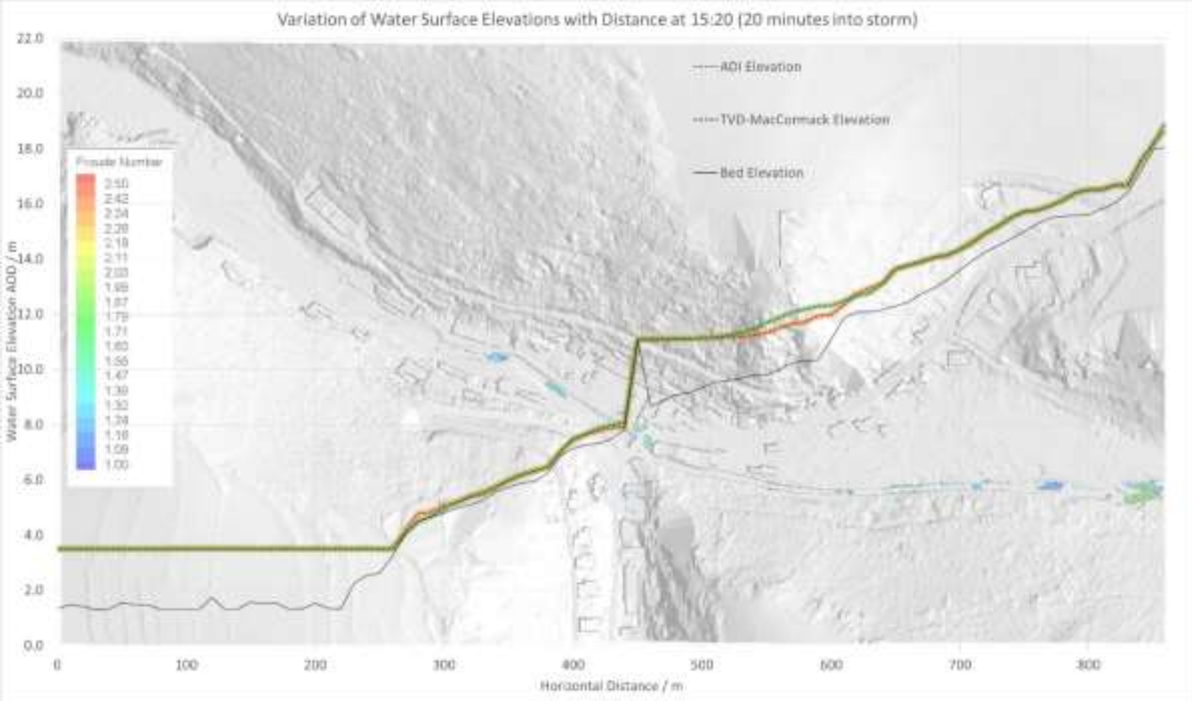

Figure 18 - River Valency water surface elevation and Froude number at 15:20.

Shortly afterwards at 15:30, supercritical flow becomes more prevalent at the banks of the river (see Figure 19). The ADI simulation still remains fairly accurate in spite of this, however it does produce a consistently and marginally higher water surface elevation towards the upstream end of River Valency.

At 60 minutes into the storm, the flow greatly exceeds the river boundary and is widely supercritical (see Figure 20). The discrepancies that were only marginally visible in Figure 19 are more prevalent, with the ADI simulation providing a notably higher water surface elevation at all points other than at the upstream end of the blocked bridge. Moreover the ADI software's inability to model the variation of surface water elevation along the river with the changing topography is again notable, with much of the profile maintaining a relatively constant gradient.

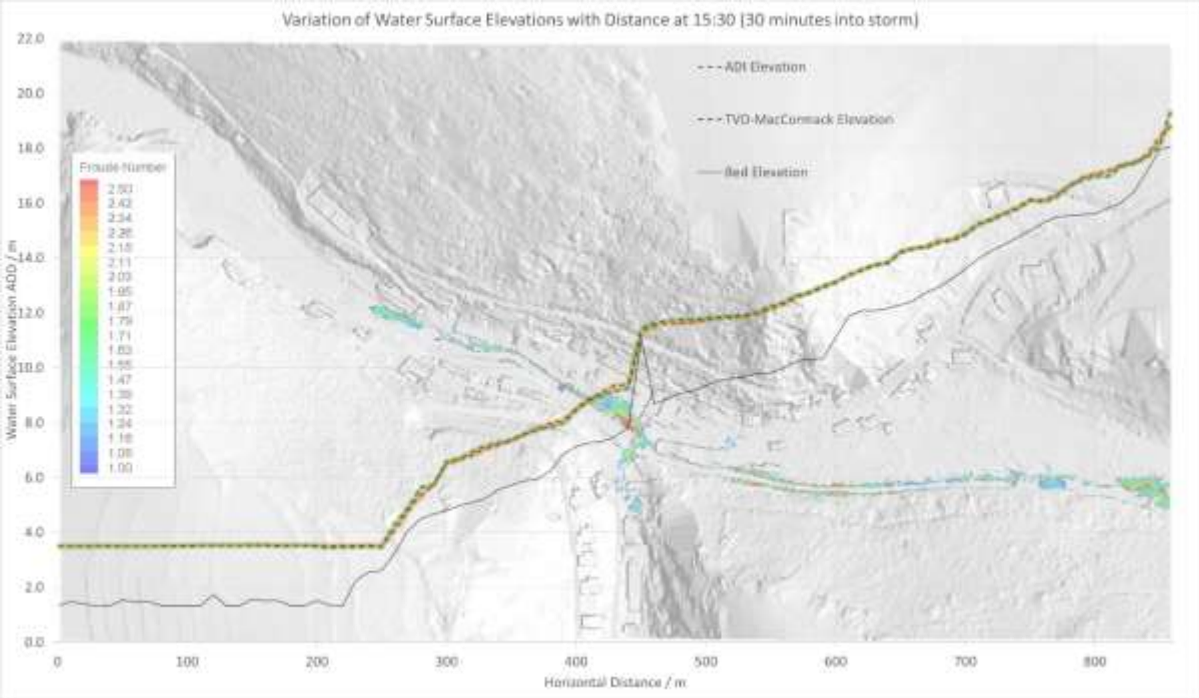

Figure 19 - River Valency water surface elevation and Froude number at 15:30.

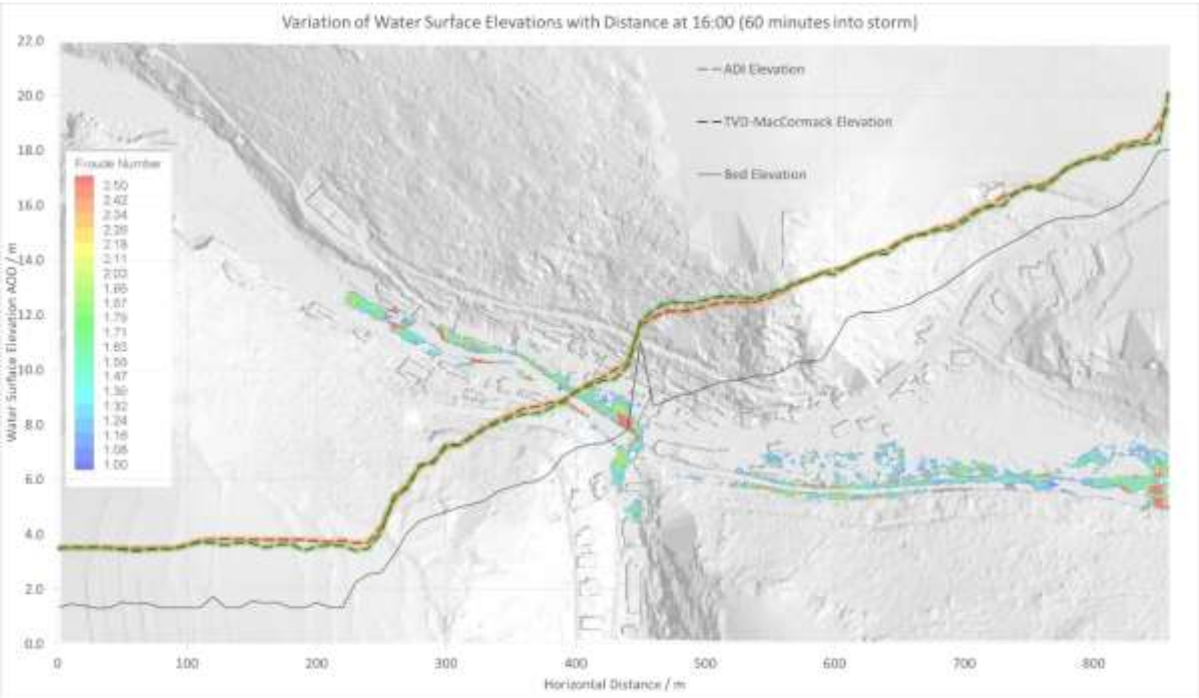

Figure 20 - River Valency water surface elevation and Froude number at 16:00.

This is further emphasised at the peak of the storm. After 120 minutes, the river profile starts to vary significantly across the catchment, as can be seen by the TVD-MacCormack plot on Figure 21. The ADI model is again unable to model this variation and once more produces a larger averaged water surface elevation as a result. For the peak of the storm the locations at which supercritical flow is reached has also spread, which is the expected cause of the heightened inaccuracies in the results of the ADI software.

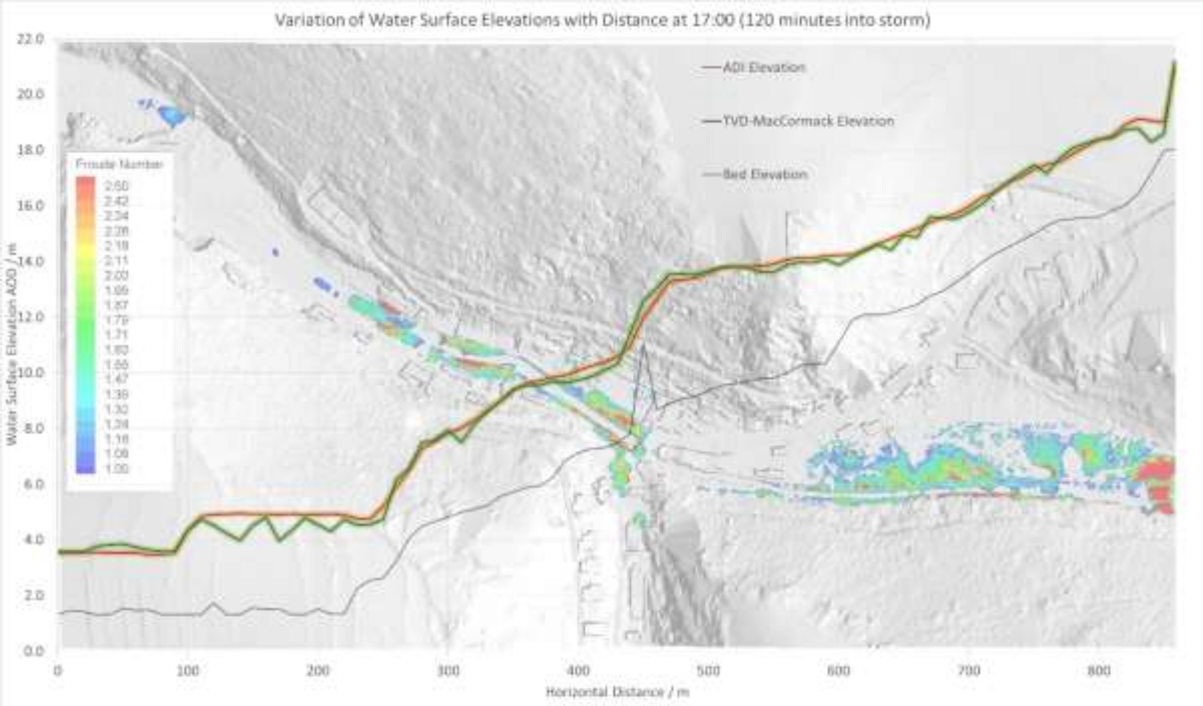

Figure 21 - River Valency water surface elevation and Froude number at 17:00.

4 – Two-Dimensional Analysis – Design Flooding

The previous two sections have provided many means of comparison for the two separate models of analysis in order to determine their effectiveness at modelling flash flooding. However the most important use of any flooding simulation lies within the prevention of flooding, in particular in analysing certain potential mitigation measures that could be put in place.

4.1 – Boscastle post-2004

Boscastle once again provides an excellent case study in this regard, as many separate mitigation measures were put in place following the 2004 flood. These mitigation measures were used collectively to design against the severity of a 1 in 75 year storm at the time. The corresponding flows in River Valency and River Jordan for this design flow are shown in Table 1.

Table 2 Scheme Design flows ($m^3 s^{-1}$)

Chance of flood occurring in any year	Valency	Jordan	Combined
1 in 5	8.5	1.9	10.4
1 in 10	12.2	2.8	15.0
1 in 25	21.0	4.8	25.8
1 in 50	32.6	7.4	40.0
1 in 75 (Design Flow)	**42.3**	**9.6**	**51.9**
1 in 100	51.6	11.8	63.4
1 in 200	82.9	18.9	101.8

Table 1 - Scheme design flows[2].

The main mitigation measures that were implemented are shown on Figure 22. The measures include:

- <u>River bed lowering and widening</u> – towards the downstream end of River Valency, the village was very restrictive against many mitigation techniques so as to maintain the aesthetic characteristics of the historical surroundings. As such there was little option but to deepen and widen the river bed close to and downstream of the central bridge. This has enabled a greater flow capacity within the river boundary and improved sediment and possible debris transportation out of the village and into the Celtic Sea.
- <u>Raised and redeveloped car park</u> – A major contribution to debris and flow obstructions in the 2004 flood were cars from the car park upstream, which during the storm washed out and into the village. The redevelopment of the car park has raised the entire elevation and built it further away from the river to reduce the risk of it flooding. Moreover, bollards have been introduced to prevent the movement of cars out of the car park in the event of another storm of a similar magnitude.

- <u>Modified river banks</u> – the banks of the river upstream of the village have been excavated to allow for deposition of sediment and trees that may become caught in an increased river flow. In addition to increasing the flow capacity of the river this prevents debris flowing further downstream and causing either damage or flow obstructions within the village similar to the events of the 2004 flood. Furthermore, downstream of the central bridge, the road was lowered as a means of sacrificial infrastructure; in the event of any out of bank flows or large river flows this road floods to prevent the flood water extending into the village.
- <u>Flood defence wall</u> – the existing flood defences isolate an area by the downstream end of the car park that is designed to flood before any other flooding is experienced within the village. Following the previous mitigation measures for other parts of the village, a flood defence wall has been introduced to delay the onset of flooding in this area.
- <u>Bridge redesign</u> – the bridge openings were widened to accommodate a larger flow capacity and reduce the likelihood of future blockages that could recreate the situations of the 2004 flood. As the river was widened downstream, one bridge required lengthening and relocating.
- <u>Landscape clearing</u> – at the upstream end of River Valency, trees were coppiced and limited from further growth in an effort to reduce debris in any future storm.
- <u>Domestic mitigation</u> – small flood defence walls were also introduced in private gardens and in confined spaces within the village.

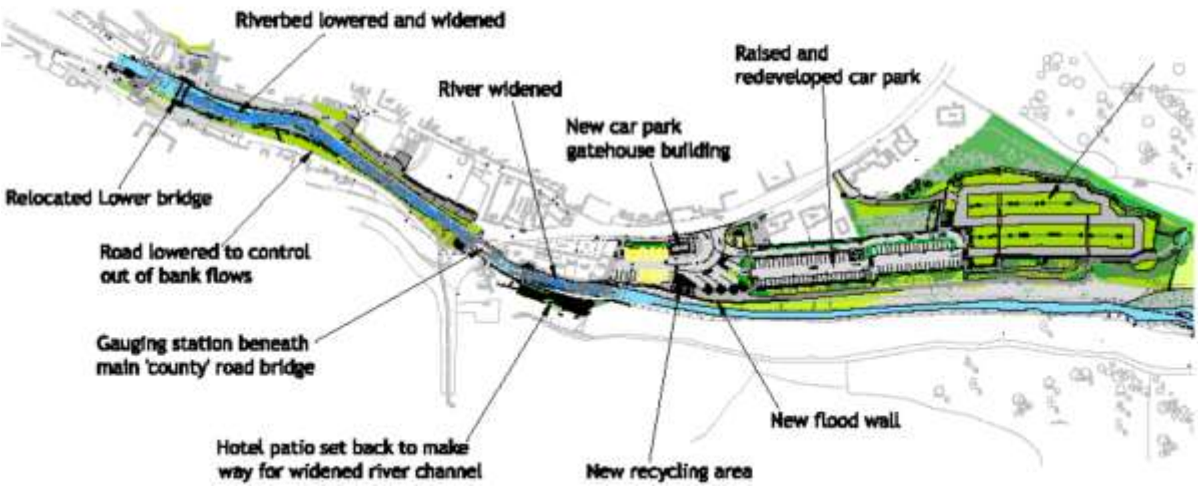

Figure 22 - Implemented flood risk improvement works in Boscastle post-2004 flood[2].

4.2 – Flood simulation

The 1 in 75 year storm corresponds to the hydrograph shown in Figure 23, which was once again separated between the two rivers as shown in Figure 24 (according to Table 1). For the constant flow inputs for the future scenario, each simulation was run until steady flow conditions were reached before any analysis was undertaken.

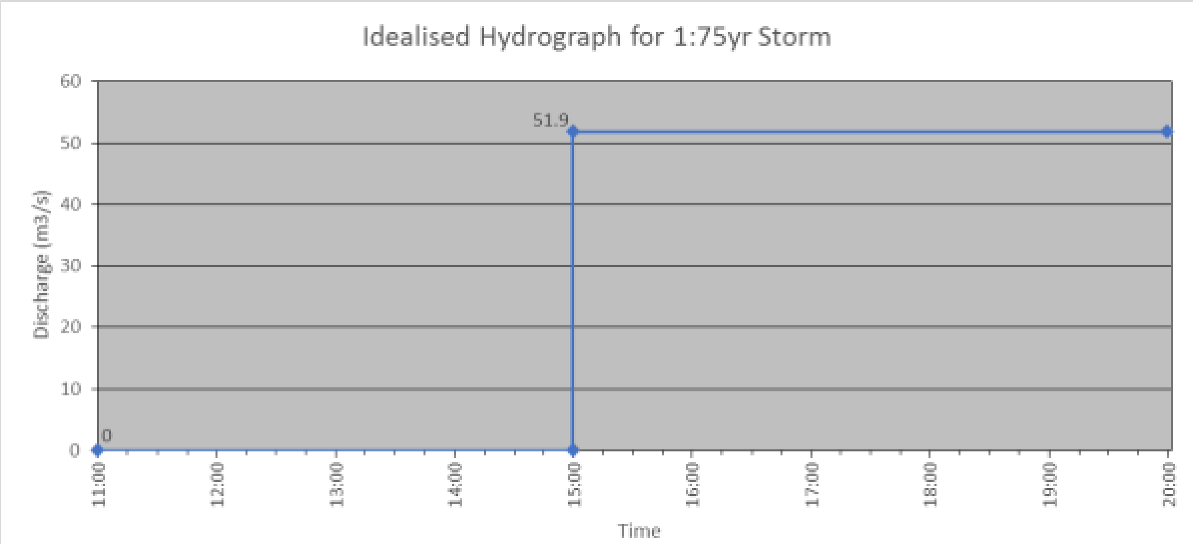

Figure 23 - Hydrograph for future 1 in 75 year storm in Boscastle.

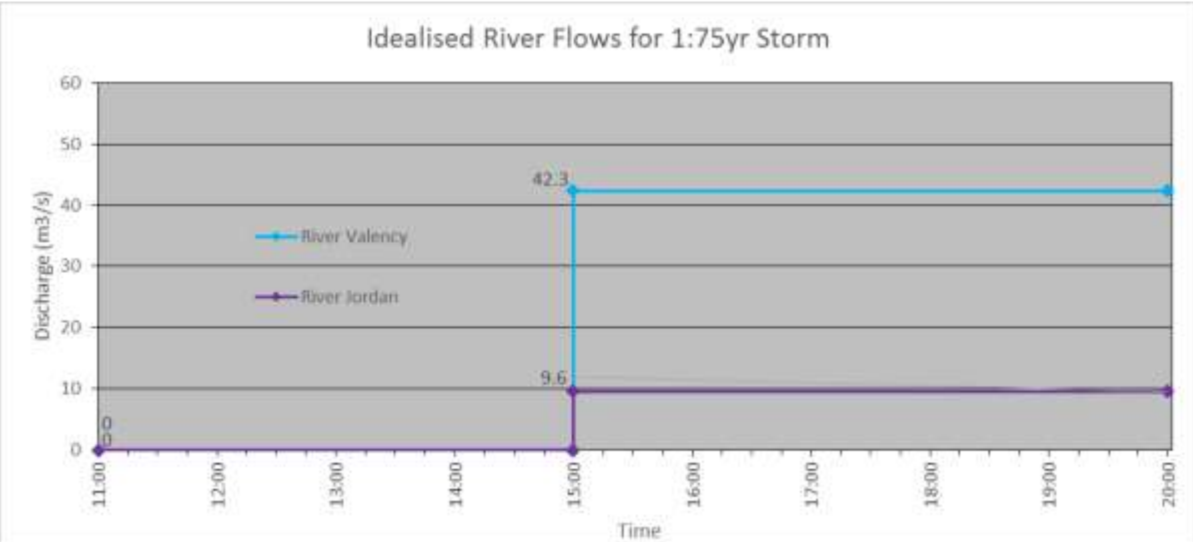

Figure 24 - Specified hydrographs for River Valency and River Jordan in analysis.

Several changes had to be made to the inputted data for both models: firstly the bridge was no longer blocked and so the topography of the river bed at this location was adjusted back to the original values. Secondly the River Jordan would not overtop the buried outflow far upstream and so would not be flooding through the hotel; as such the input of the River

Jordan flow was relocated to the end of the buried outflow downstream of the central bridge, as shown on Figure 25.

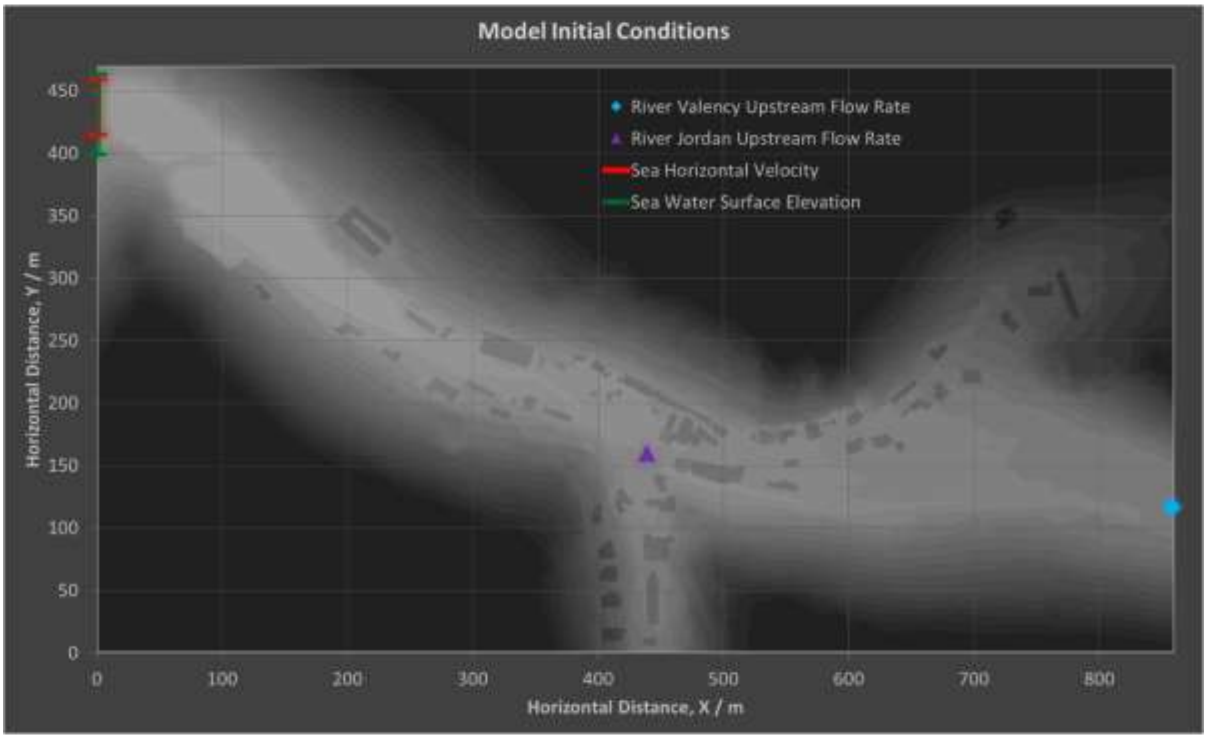

Figure 25 - Specified initial conditions for the future flooding scenario in Boscastle.

While a wide range of mitigation measures were introduced into Boscastle following the flood, the ability to implement some of these measures into the input files is difficult (due to simplifications within the DEM); however four of the mitigation measures can be effectively modelled by changing to topography of the input files. These four mitigation measures are outlined on Figure 26 and include the deepening and widening of the river bed, the raising of the car park, the lowering of the river banks and the installation of the flood defence wall. The topographical changes due to these mitigation measures were determined by a previous student and implemented into multiple input files for which each measure is added in turn.

Both the TVD-MacCormack and the ADI software have been used to model the future flooding scenario for each of these input files. As a control case, both modes of analysis were first used to simulate the future storm in Boscastle in its unmitigated state following the 2004 flood. The steady flow water surface profile following the centreline of River Valency has been plotted for both models on Figure 27. In line with the conclusions from the previous section, while both models produce similar profiles, the ADI software often overestimates the water level of the river and cannot model the variation along the river effectively. In addition, Figure

27 shows the extent of the flooding and the water depths due to the flooding scenario as modelled by the TVD-MacCormack software. While the flood is far less extreme than the historical case previously, the flood still exceeds the old flood defences and causes several buildings to be surrounded by water.

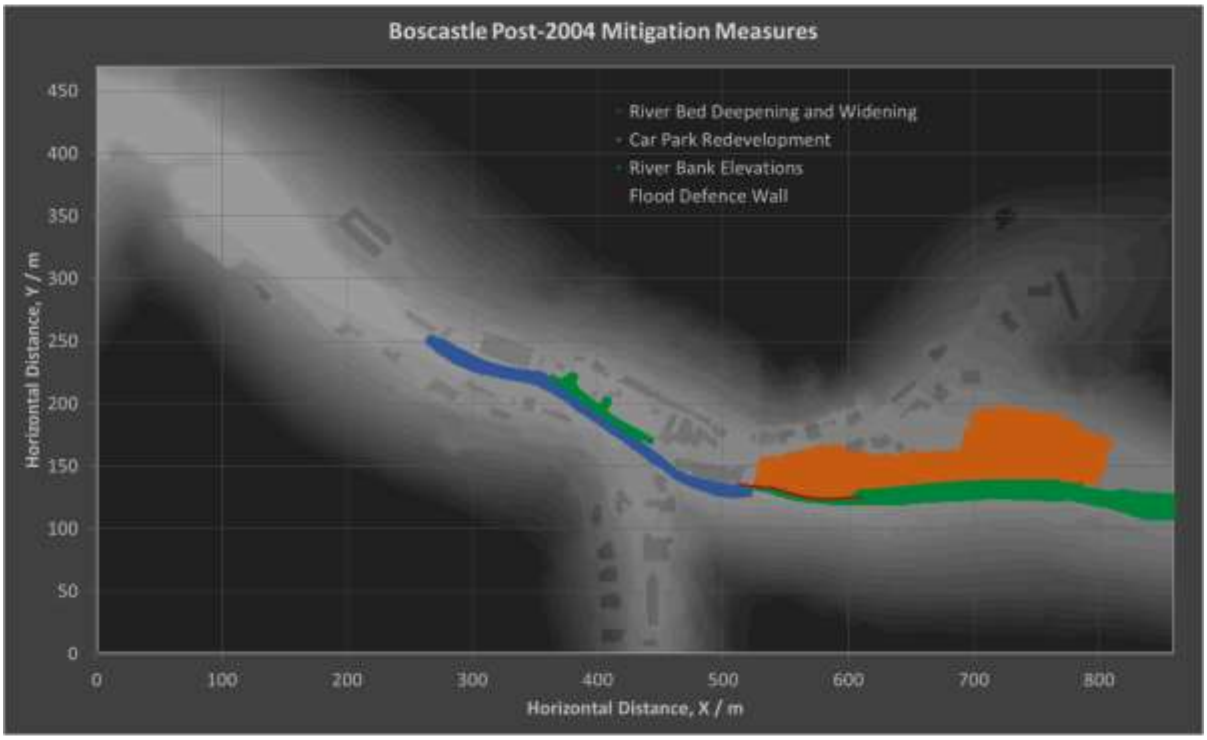

Figure 26 – Modelled Boscastle post-2004 mitigation measures.

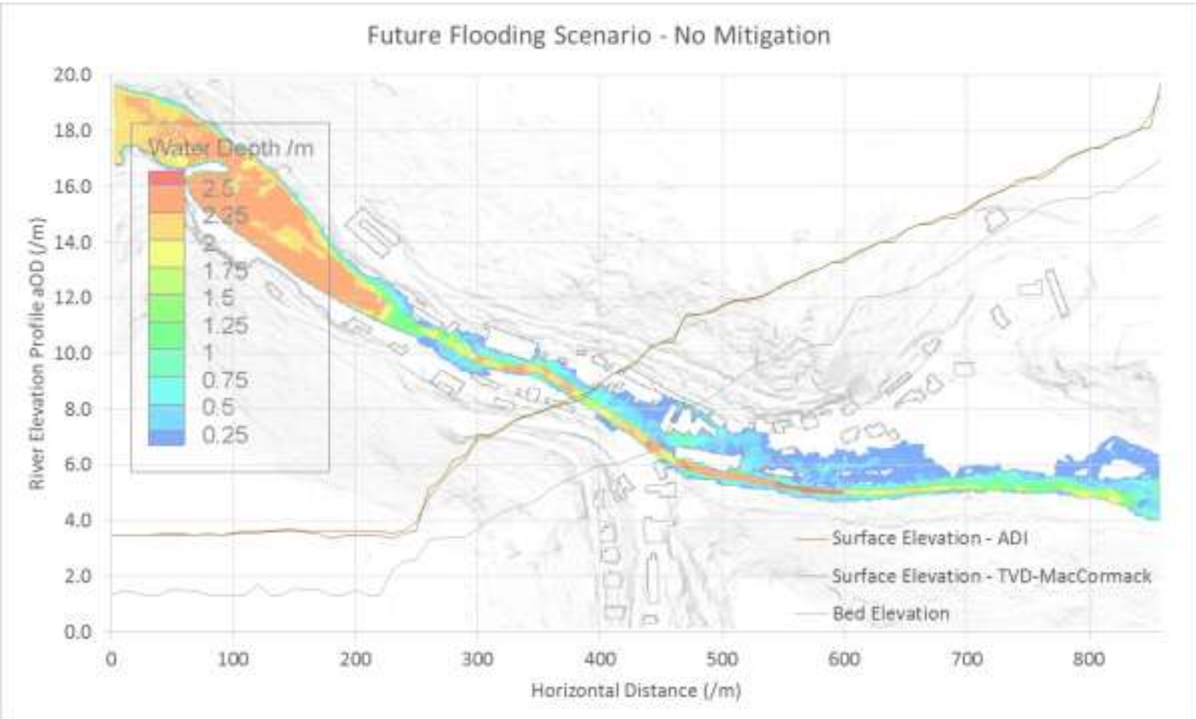

Figure 27 - River Valency water profile for a 1 in 75 year storm with no mitigation measures.

4.3 – River bed deepening and widening

Figure 28 shows the water surface profile for River Valency with this mitigation measure implemented and the flood extent and water levels across the catchment (as modelled by the TVD-MacCormack software). The contour plot shows a much larger water depth in the river channel corresponding do the lowering of the bed depth. As a result, the water depth within the village has been reduced and more notably several properties downstream of the mitigation measure are no longer flooded.

A significant difference between the two models is apparent between X=100m and X=400m which almost directly corresponds to the range over which the river topography changes. There is a significant change in water elevation profile for the TVD-MacCormack case as a result of the deepening and widening of the river, however the change for the ADI case is more subtle.

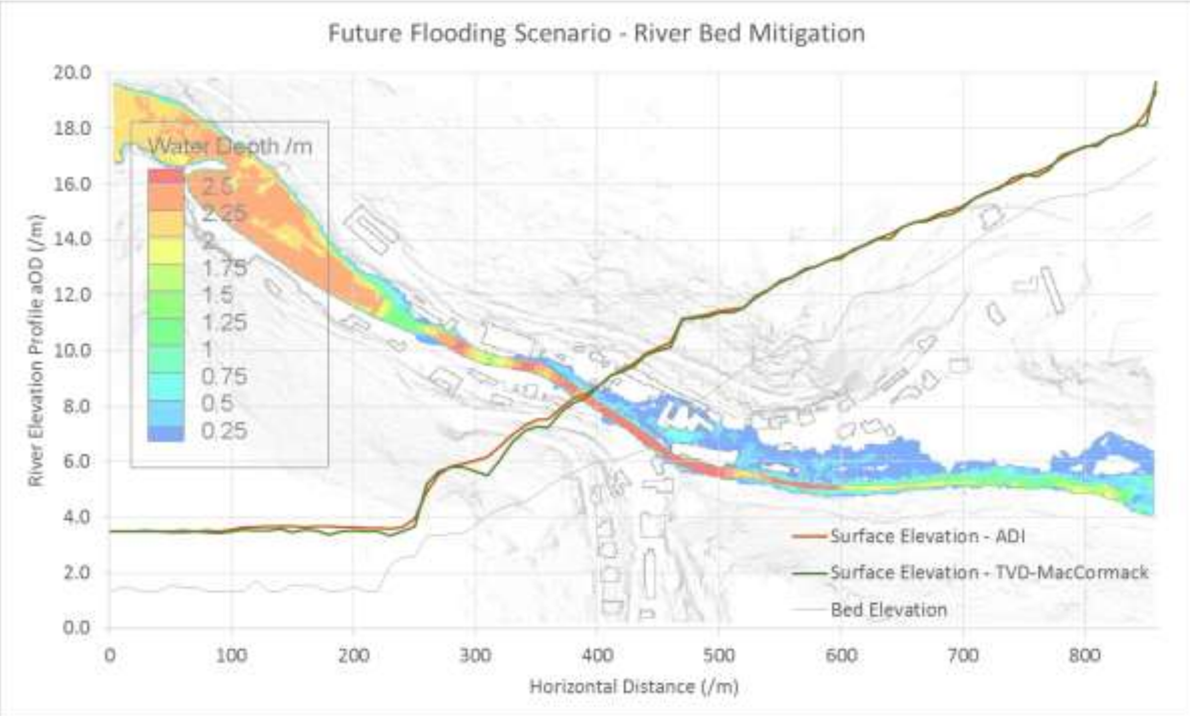

Figure 28 - River Valency water profile for a 1 in 75 year storm with the river bed deepened and widened.

Figure 29 shows the difference in water surface elevations between Figure 27 and Figure 28. For the TVD-MacCormack model this mitigation measure results in a dramatic reduction in water surface elevation across the length over which the river has been deepened and widened. This is to be expected, as the widening of the river allows for a lower water depth to maintain the same cross-sectional area of flow; moreover the deepening of the river reduces the water surface elevation above the fixed datum. Figure 29 also shows more clearly that the ADI software is less sensitive to the change in topography due to this mitigation measure, most notably at X=300m, where the ADI software predicts a water surface elevation change of almost half that of the TVD-MacCormack software, and at the downstream end of the river, where the ADI software predicts almost no change in water surface elevation whatsoever.

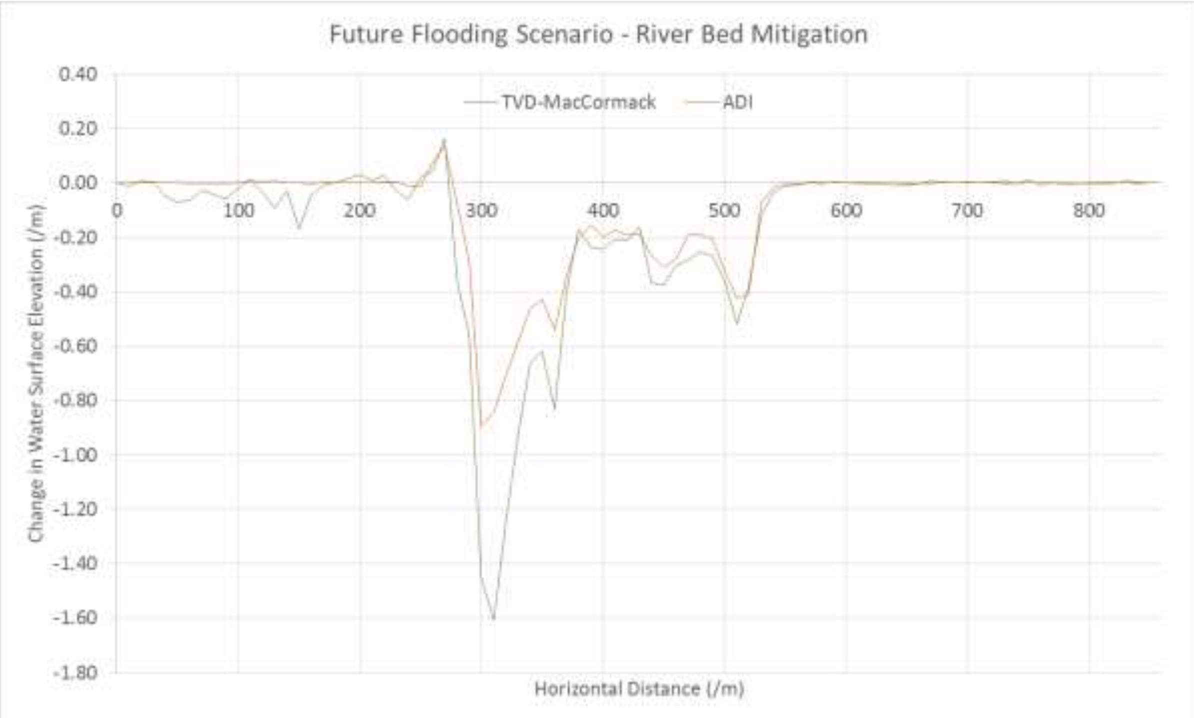

Figure 29 - Change in water surface elevation due to the deepening and widening of the river bed.

4.4 – Car park redevelopment

Figure 30 shows the modelled river water surface elevation profile, the flood extent and water depths for the case in which additionally the car park has been raised. Very little difference within the river channel is notable, as can be seen on Figure 31, which shows the corresponding change in water surface elevation due to the addition of the car park redevelopment (the difference in elevation between Figure 28 and Figure 30). However, what cannot be modelled here is the reduction of debris escaping the car park as a result of the flooding and causing obstructions downstream. Moreover the contour plot in Figure 30 shows a significant reduction in flood extent around the car park as a result of the mitigation measure.

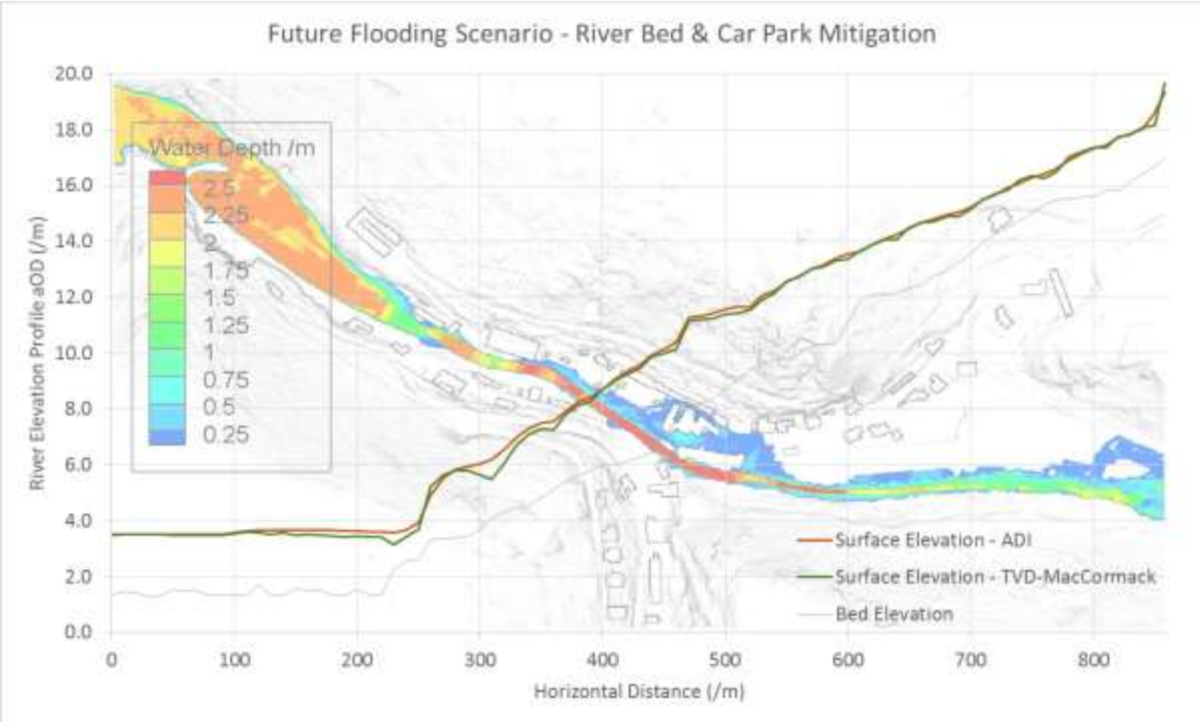

Figure 30 - River Valency water profile for a 1 in 75 year storm with the river bed deepened and widened and the car park redevelopment.

Figure 31 - Change in water surface elevation due to the raising of the car park.

Again, the ADI software appears to be unable to accurately model the effect of the mitigation measure and models an overall increase in water surface elevation as a result of its implementation (Figure 31).

4.5 – River bank elevations

Figure 32 shows the river elevation profile, the flood extent and the water depths for the additional lowering of the river banks. This produces a large change in the upstream river profile that is modelled by both the ADI and the TVD-MacCormack software (see Figure 33). The additional impact this mitigation measure has on reducing the dynamic accumulation of sediment during a flood cannot be effectively modelled.

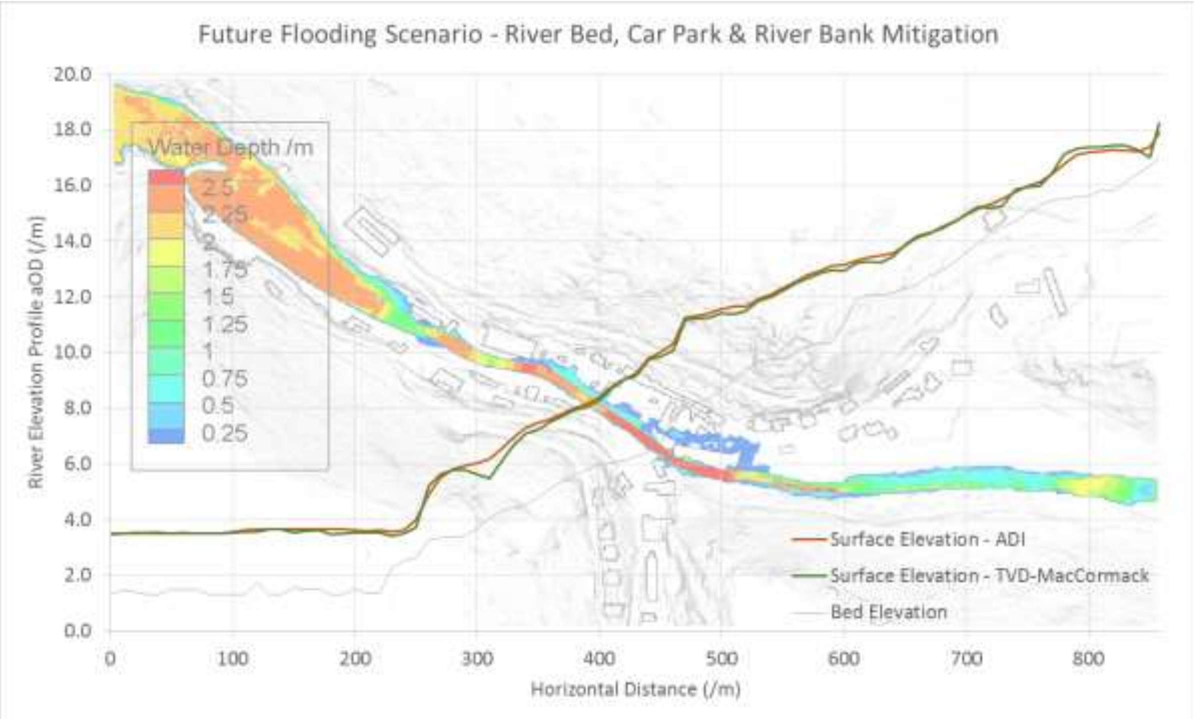

Figure 32 - River Valency water profile for a 1 in 75 year storm with the river bed deepened and widened, the car park redevelopment and the river bank lowered.

Nonetheless, the contour plot shows a dramatic reduction in flooding within the village as a result of this mitigation measure. The only two locations at which the flow is no longer contained within the channel correspond to the two locations at which the flow is designed to first exceed the river boundary in the event of a severe storm: firstly downstream of the central bridge, the road that has been lowered to act as sacrificial infrastructure is flooded, but keeps the overtopped flow contained within it; secondly upstream of the central bridge the flow overtops the lower end of the car park as previously designed prior to the 2004 flood.

The ADI software provides a more accurate representation of the effect of this mitigation measure than it has done for the previous measures. However, the software still is unable to model steep changes effectively and only produces values on Figure 33 that follow an averaged profile of the true TVD-MacCormack values.

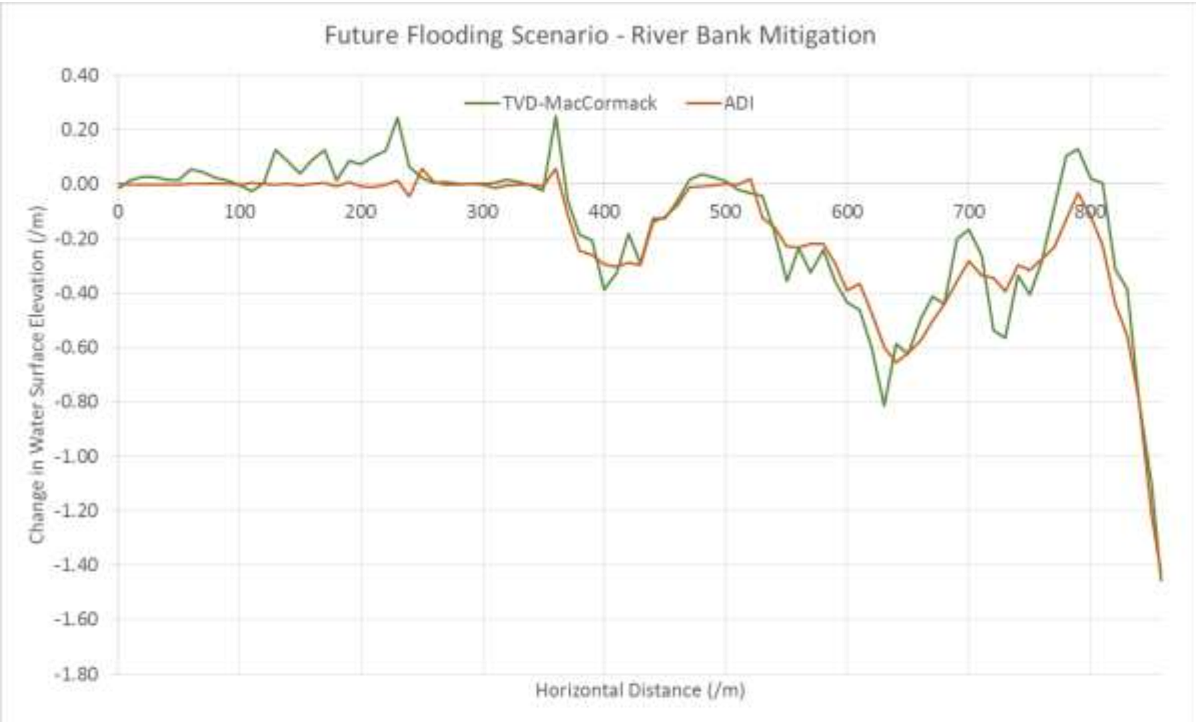

Figure 33 - Change in water surface elevation due to the lowering of the river banks.

4.6 – Flood defence wall

With the final mitigation measure implemented, the flow no longer escapes the river boundary upstream of the central bridge and is entirely contained within the channel and lowered road (see Figure 34). As a result, the implementation of all four mitigation measures successfully protects the village against the design storm event and therefore the TVD-MacCormack method produces the expected simulation for each mitigation measure.

The flood defence wall has little impact on the water surface elevation within the river, however Figure 35 once again shows the ADI model's lack of sensitivity to the change in topography and therefore the marginal positive effect of this mitigation measure.

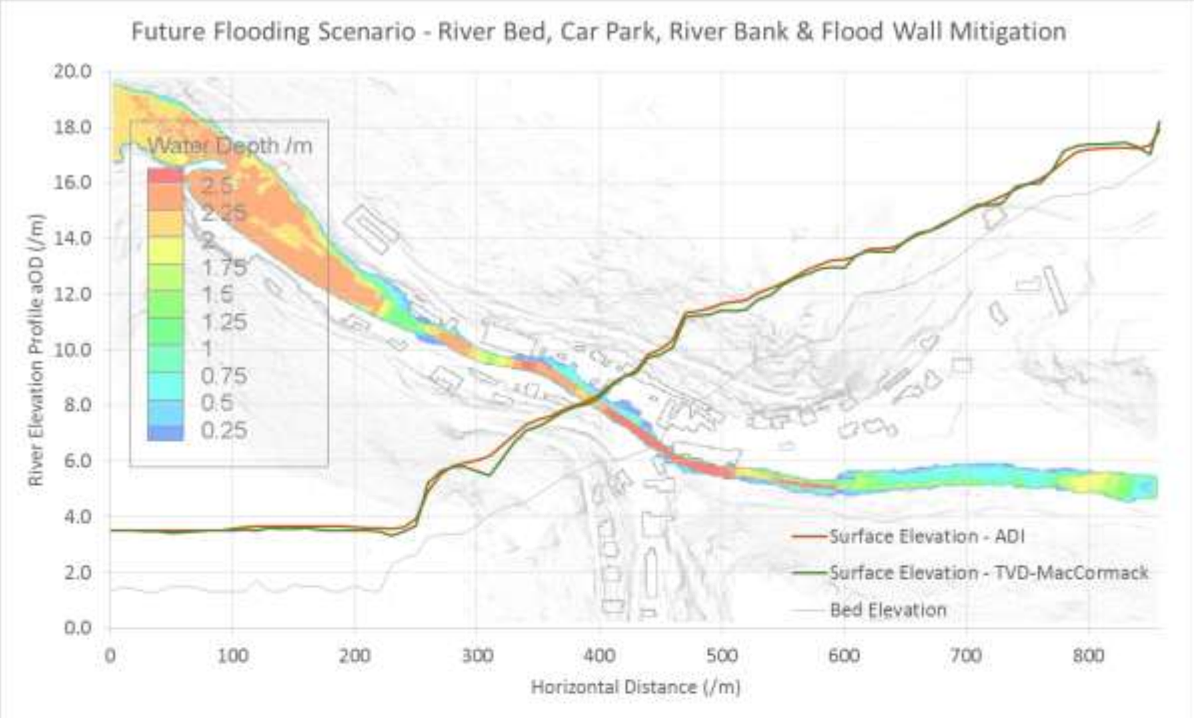

Figure 34 - River Valency water profile for a 1 in 75 year storm with the river bed deepened and widened, the car park redevelopment, the river bank lowered and the flood defence wall introduced.

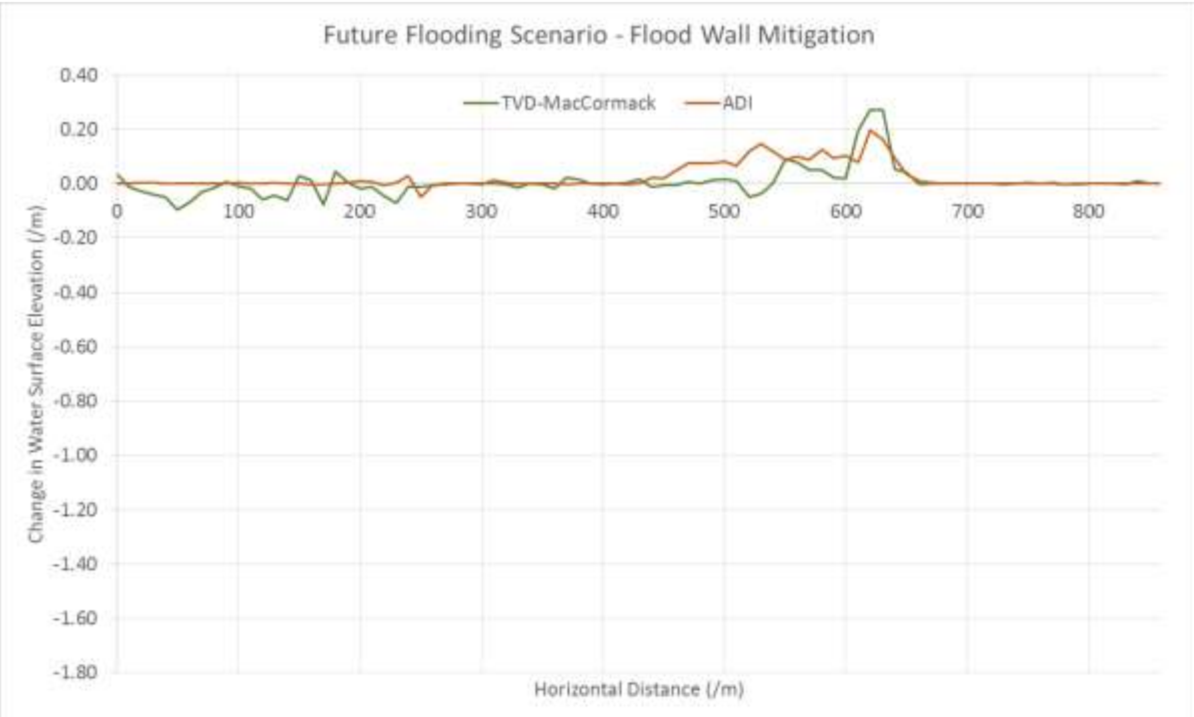

Figure 35 - Change in water surface elevation due to the installation of the flood defence wall.

This lack of sensitivity that has been observed throughout each of these mitigation measures would suggest that the ADI model is likely to lead to further conservative design of mitigation measures, to ensure that they have the desired effect on containing the flow.

4.7 – Correction coefficient (Beta)

Similar to the one-dimensional case, the Boscastle case was run again using the ADI analysis for a large maximum Froude number (100) and varying values of Beta. Firstly for the case in which no mitigation measures are implemented, the water profiles along the River Valency centreline have been determined and are plotted on Figure 36.

As was noted for the case in Section 2, as Beta increases, the water surface elevation also increases. However, for the two-dimensional case in question, the ADI simulation becomes unstable and crashes above values of 0.90, with results at this value starting to oscillate (as seen on Figure 36). Therefore in order to successfully run the ADI software for the Boscastle case, the Beta value must be reduced lower than 1.00 and therefore is expected to produce a lower water surface elevation than the expected profile for Beta equal to 1.00.

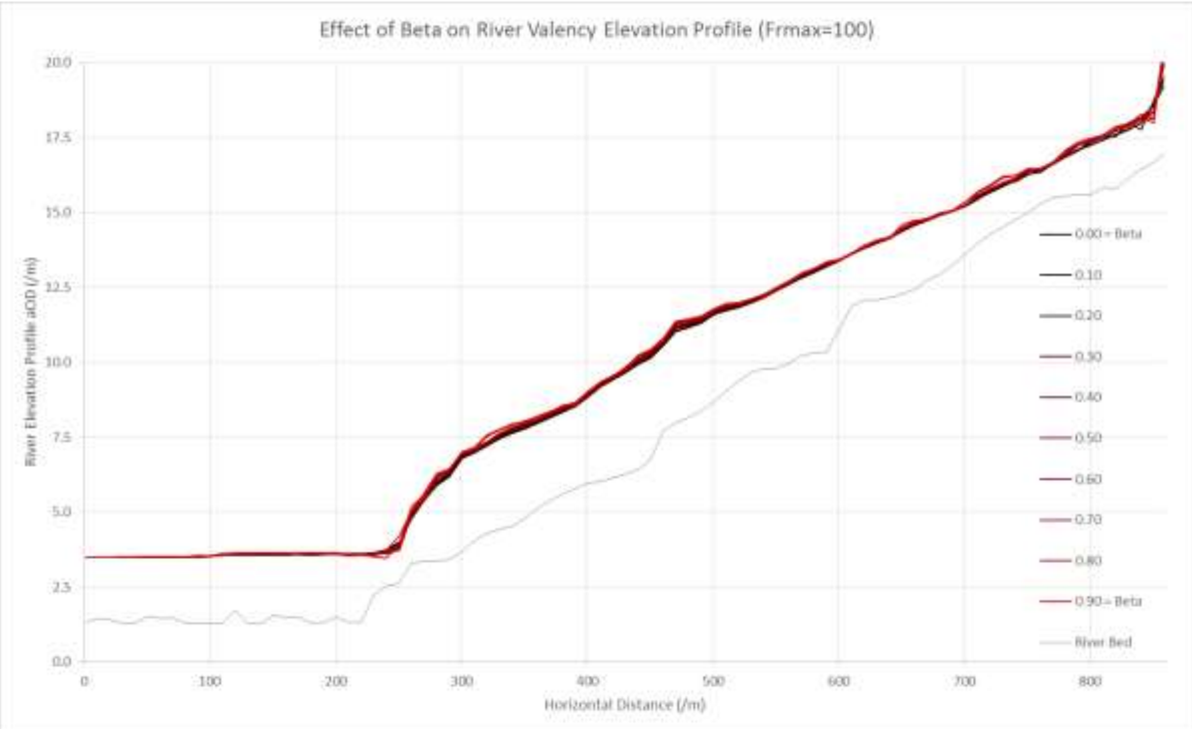

Figure 36 - Effect of Beta on the elevation profile of River Valency with no mitigation measures in place.

This is again true for the case in which all four mitigation measures are in place; Figure 37 shows that as the value of Beta is increased, the water surface elevation also increases. Moreover the simulation becomes unstable for values of Beta greater than 0.90 and small localised regions of oscillation for the maximum plotted value, for example at X=240m and X=750m.

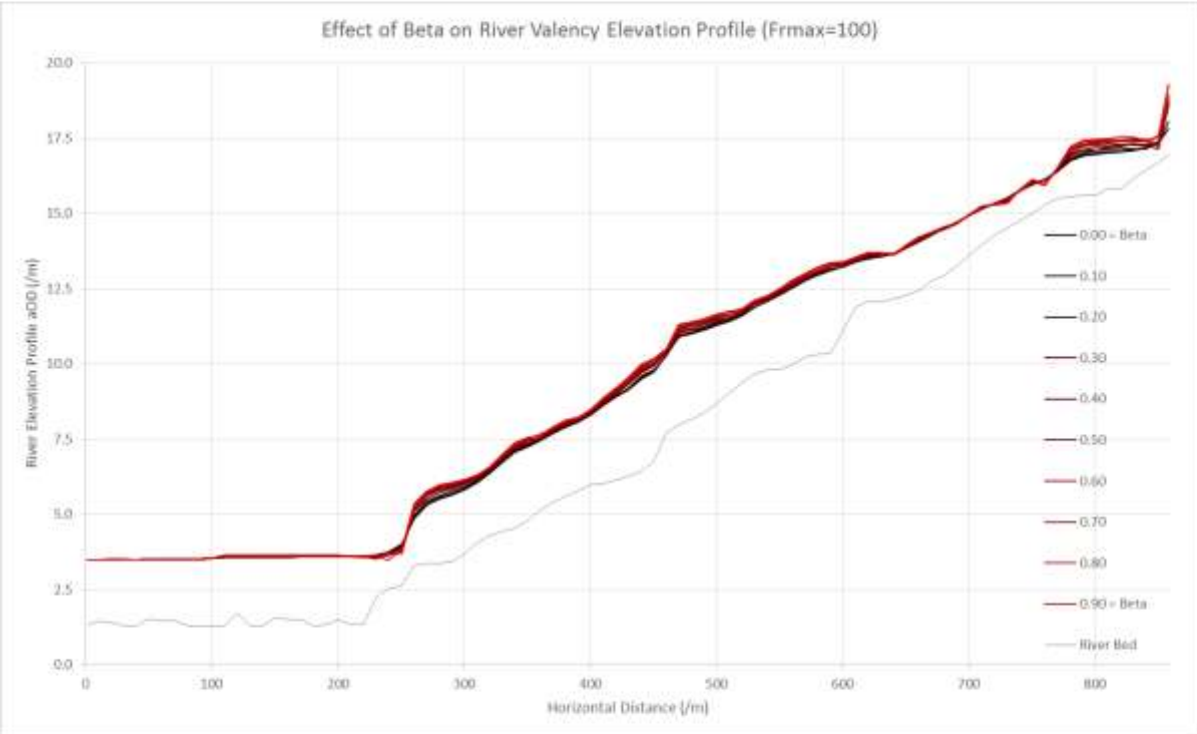

Figure 37 - Effect of Beta on the elevation profile of River Valency with all four mitigation measures implemented.

To compare the impact that the value of Beta has on the sensitivity of the implementation of these mitigation techniques, the elevations on Figure 36 were subtracted from those on Figure 37 to provide the difference in elevation profile shown on Figure 38. The plot shows that as Beta is increased, the change in river elevations are smaller for the majority of the river profile and for the highest values of Beta the plot becomes spurious due to the oscillations in the river profiles across the two cases. Therefore, as the Beta term must be set to be less than 1.00 to achieve a stable simulation, it is expected that this also produces a simulation that is slightly less conservative and slightly more sensitive to any changes in topography due to the introduction of the mitigation techniques.

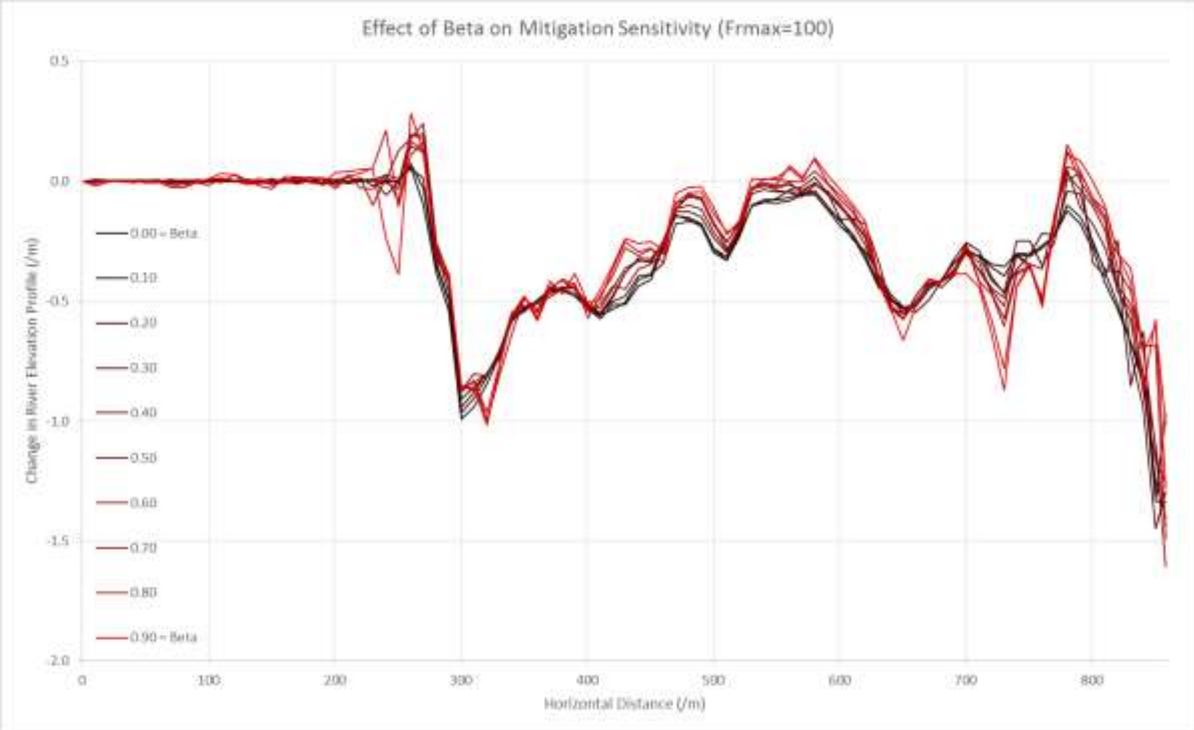

Figure 38 - Effect of Beta on the sensitivity of the implemented mitigation measures.

5 – Conclusions

While the ADI analysis has been widely implemented into the analysis of flash flooding, the theoretical simplifications that it makes yields the expectation that it is unsuitable and will produce notably inaccurate results. In particular, the ADI software is expected to produce larger water surface elevations so as to reduce the Froude number to below its specified maximum value. Moreover the analysis uses a correction coefficient that can help to stabilise the simulation but has a notable effect on the simulated water surface elevation. On top of this, if the stabilising parameters are not adjusted to suit the specific situation that is being analysed, spurious oscillations can occur leading to highly inaccurate and infeasible results. In contrast, the MacCormack method is expected to produce far more accurate results and the Total Variation Diminishing property that can be implemented into it can prevent any spurious oscillations at only a small expense in accuracy.

These initial expectations are very noticeable within the one-dimensional case and the ADI model appears to be far less suitable for this mode of analysis. Moreover the effects of the maximum Froude number and the correction coefficient Beta can be made distinctly clear. However the one-dimensional situation over which the models have been tested is highly conceptual and so less suitable from which to draw any strong preferences for the use of either means of analysis for real-life flash flooding.

For the two-dimensional case, the same discrepancies were noted throughout. Again, the ADI software overestimated the water surface elevation across the entire domain and also has been determined to be less sensitive to topographical changes made to the input files to simulate the mitigation techniques. However, for the two-dimensional case, the discrepancies were more subtle. This is likely due to the fact that across a real two-dimensional domain, the Froude number does not widely exceed the flow criticality limit, especially during the initial stages of the storm (as can be seen on Figure 18 and Figure 19). Moreover, the difference in water elevation is often small relative to the scale of the flood and the channel depth. The ADI model also has no difficulty in simulating an accurate extent of flooding (as can be seen from Figure 15). Despite the inaccuracies, the ADI software consistently provides only a slightly conservative estimate of the river flows and the effect of the mitigation measures, which would only lead to a slight overdesign of any flood defence scheme. These conservative estimates can be made more accurate by reducing the value of Beta below 1.00, but due to the severity of flash flooding events on infrastructure and human life, a slight overdesign of flood defence schemes can be deemed to be beneficial. With the ADI software's reduced computation time and cost and its wide stability domain, the marginal effect these discrepancies have on the total design and the wider impact of this slight overdesign lends to the suggestion that the ADI software still deservedly maintains its widespread use in industry.

6 – References

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3 – GOOGLE MAPS, 2016. Available at: https://www.google.co.uk/maps/@50.6904224,-4.6937386,776m/data=!3m1!1e3 [Last Accessed 23 May 2016].

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7 – Appendices

7.1 – Flow theory

7.2 – Risk assessment retrospective

7.1 – Flow theory

$$\frac{\partial \eta}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0$$

$$\frac{\partial q_x}{\partial t} + \frac{\partial(\frac{\beta q_x^2}{H})}{\partial x} + \frac{\partial(\frac{\beta q_x q_y}{H})}{\partial y} = -gH\frac{\partial \eta}{\partial x} - \frac{g q_x \sqrt{q_x^2 + q_y^2}}{H^2 C^2} + v\left[2\frac{\partial^2 q_x}{\partial x^2} + \frac{\partial^2 q_x}{\partial y} + \frac{\partial^2 q_y}{\partial x \partial y}\right]$$

$$\frac{\partial q_y}{\partial t} + \frac{\partial(\frac{\beta q_x q_y}{H})}{\partial x} + \frac{\partial(\frac{\beta q_y^2}{H})}{\partial y} = -gH\frac{\partial \eta}{\partial y} - \frac{g q_y \sqrt{q_x^2 + q_y^2}}{H^2 C^2} + v\left[\frac{\partial^2 q_y}{\partial x^2} + 2\frac{\partial^2 q_y}{\partial y} + \frac{\partial^2 q_x}{\partial x \partial y}\right]$$

$where$

$t \text{ is time}$

$\eta \text{ is the water surface elevation above a datum}$

$x \text{ and } y \text{ are the horizontal displacements in orthogonal directions}$

$q_x \text{ and } q_y \text{ are the corresponding discharges per unit width in each direction}$

$\beta \text{ is a correction factor for the non} - \text{uniform vertical velocity profile}$

$H \text{ is the total water column depth}$

$g \text{ is the gravitational field constant } (9.81 ms^{-2})$

$C \text{ is the Chezy bed roughness coefficient}$

$v \text{ is the kinematic eddy viscosity}$

Equation 1 - Shallow water equations.

$$\frac{\partial \eta}{\partial t} + \frac{\partial q_x}{\partial x} = 0 \qquad \frac{\partial \eta}{\partial t} + \frac{\partial q_y}{\partial y} = 0$$

$$\frac{\partial q_x}{\partial t} + \frac{\partial(\frac{\beta q_x^2}{H})}{\partial x} = -gH\frac{\partial \eta}{\partial x} - \frac{g q_x \sqrt{q_x^2 + q_y^2}}{H^2 C^2} \qquad \frac{\partial q_x}{\partial t} + \frac{\partial(\frac{\beta q_x q_y}{H})}{\partial y} = -\frac{g q_x \sqrt{q_x^2 + q_y^2}}{H^2 C^2}$$

$$\frac{\partial q_y}{\partial t} + \frac{\partial(\frac{\beta q_x q_y}{H})}{\partial x} = -\frac{g q_y \sqrt{q_x^2 + q_y^2}}{H^2 C^2} \qquad \frac{\partial q_y}{\partial t} + \frac{\partial(\frac{\beta q_y^2}{H})}{\partial y} = -gH\frac{\partial \eta}{\partial y} - \frac{g q_y \sqrt{q_x^2 + q_y^2}}{H^2 C^2}$$

Equation 2 - MacCormack-simplified shallow water equations.

$$Fr = \frac{U}{\sqrt{gh}} = \frac{\frac{1}{n}\,S_b^{\frac{1}{2}}h^{\frac{2}{3}}}{g^{\frac{1}{2}}h^{\frac{1}{2}}} = \frac{h^{\frac{1}{6}}}{n}\,\sqrt{\frac{S_b}{g}}$$

$where$

$U\ is\ flow\ velocity$

$g\ is\ the\ gravitational\ field\ constant\ (9.81ms^{-2})$

$h\ is\ the\ water\ depth\ above\ the\ bed\ elevation$

$\sqrt{gh}\ is\ the\ velocity\ of\ disturbance\ introduced\ into\ the\ flow$

$n\ is\ the\ Manning\ coefficient\ of\ the\ flow$

$S_b\ is\ the\ bed\ slope$

Equation 3 - Froude number definition.

$$Subcritical\ flow\ is\ defined\ for\ cases\ in\ which\ Fr\ < 1.00$$

$$For\ these\ cases, \sqrt{gh}\ > U$$

$$Supercritical\ flow\ is\ defined\ for\ cases\ in\ which\ Fr > 1.00$$

$$For\ these\ cases, \sqrt{gh} < U$$

Equation 4 - Flow criticality definition.

7.2 – Risk assessment retrospective

The main hazard that was experienced in this project was extended use of computational software, specifically on a laptop, which provided this risk of repetitive strain injury and eye fatigue. Before the project work was undertaken, careful attention was given towards proper setup of display screen equipment, including position, reach and height of the screen and keyboard. Furthermore the seat was adjusted to ensure it was at the optimum height and provided sufficient support and regular breaks were taken from any extended periods of computational work to avoid any injuries. As a result no injuries of these kinds were experienced.

In addition, the laptop on which all of the work and many of the simulations were undertaken experienced notable overheating, particularly due to multiple background simulations running whilst other work was being carried out. The risk of injury due to this overheating was reduced by firstly using departmental computers for the running of any simulations where possible. In addition, a laptop cooler was used to reduce the temperature the laptop and again regular breaks were ensured to prevent any heat-related injury to fingers. As a result no injuries were experienced for the entire duration of the project.

If the project were to be repeated, the only additional risk to be included would be the concern regarding overheating due to the running of the flooding simulations.