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SIMULATION AND ANALYSIS OF 2D FLOODED AREA OF RIVER SPREE

(TECHNICAL REPORT)

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Abstract

River renaturation can be a helpful management tool to restore the natural capability of floodplain therein reducing the impacts during high flow period. A study was carried out to simulate the flooding in the restored reach of the Spree River using a 1D-2D HEC-RAS model where the flood plain was treated as 2D and the main channel was treated as 1D. The computation in this model is done using Preissmann scheme for finite difference approximation for 1D model and finite volume approximation for 2D model. Several scenarios were simulated using various time steps and grid sizes to understand the sensitivity of the parameters and model. In addition, flood mitigation strategies has been simulated by the inclusion of dykes, dredging and change of vegetation pattern (Manning's n). In most of the scenarios without the mitigation measures, the model showed flooding mainly in the downstream part of the channel whereas with mitigation measures, the inundation in the floodplain decreased significantly. The restored area needs to be maintained by sustaining the ecological balance in the floodplain of the channel. Therefore, considering the possibility of flooding in the area, mitigation strategies that balances the environmental as well as economical aspect of the area needs to be adopted.

Acknowledgement

Firstly, we would like to thank our Professor, Dr. Michael Nones, for his persistent support throughout this study.

The 2D model of Spree river floodplain for this study was provided by Dr. Michael Nones. This study would not have been possible without this model.

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Abbreviation

1D	One-Dimensional
2D	Two-Dimensional
FEMA	Federal Emergency Management Agency
USACE	United States Army Corps of Engineers
HEC-RAS	Hydrologic Engineering Center-River Analysis System
GIS	Geographic Information System
FVM	Finite Volume Method
WSEL	Water Surface Elevation Level
USGS	U.S. Geological Survey
DEM	Digital Elevation Model
XS	Cross-Section

1 Introduction

1.1 Background

Naturally, rivers show dynamic behavior that results in alteration of its morphology over time. However, this is a slow process that gives the river time to adjust to its new status while maintaining its capability. On the other hand, things are different when there is artificial manipulation of river and its surrounding areas. One of such example is the Spree River in Germany. Over the years, the river was changed for the navigation purpose (Leag, n.a.). The river was straightened and dykes were constructed along the river stretch. This led to the decrease in the flood retention areas and as a result the river could not cope naturally with the high water conditions.

Flood is a condition when the water reaches a high level in the river and overtops the banks deviating from its natural course and causing inundation in the floodplains. One of the main reasons of flooding is when the river cannot route the incoming water due to low conveyance capacity (Popescu, 2014). It is one of the great natural disasters causing the destruction of anything on its way. This has been evident from history when the flood in various parts of the world caused great destruction of life and property leading to about two-third of destruction in terms of number and economic loss. These are generally a natural phenomenon but the impacts and effects can be exacerbated by human interventions.

A Floodplain is an area of low-lying ground adjacent to a stream or river which stretches from the bank of its channel to the base of enclosing valley walls, and which experiences flooding during the period of high discharge (Anon., n.d.). Floodplains are formed as the rivers erode their own banks. As the river flows, it washes material downstream of the bank, and when a flood occurs, this material or sediment largely composed of a layer of silt, sand and mud, is suspended in water and added to the floodplain (Anon., 2018). Floodplains constitutes a valuable part of the environment as they filter, store, and release floodwaters, recharge aquifers, store a variety of sediments, and also provide a habitat for a diversity of wildlife (Anon., 2008). Floodplain soils absorb water during the wet season, then slowly release moisture to plants and in to the streams. This lessen the impact of peak runoff and keeps plants growing and streams flowing longer in to the dry season. Also, the streambank vegetation helps to cool the surface water temperature, and

create important habitat for fish, waterfowl, and other wildlife species. River renaturation can be helpful to some extent in the flood reduction in the floodplain. A study carried out in Floha river in the border of Germany and Czech Republic showed that the afforestation in the floodplain reduced the peak flow by 4% than when applied on barren floodplain. Though this study did not show a significant impact of renaturation on the flood reduction and may not be a major tool for flood defense, these method contributes on management of flood.

The impacts of flood and the significance of floodplains to maintain the naturality of rivers made it necessary to carry out studies to understand the behavior of river systems for its better management. The concept of river models started during 1970s and later on hydraulic models for rivers, channels and pipe systems emerged during the 1980s for flow prediction, finding the travel time and water level variations in rivers, channels and canal systems (Popescu, 2014). In simple terms, modeling is a process of creating a simulation of reality by different computational process in order to understand the complex phenomena of nature. These days modeling has been an important aspect in every field of research, water resources being one of them. The governing equations is generally same for most of the models but the approach will vary depending on the research topic.

1.2 Objective

The purpose of the study is to simulation of 2-D flooded area (clear water) for the entire reach (about 11 km), sensitivity analysis on time step and grid dimensions and management strategies to reduce the flood risk (Nones_presentation). This hydro-engineering study project provide opportunity to apply the HEC-RAS code to simulate flooding events after the main restoration measures and to gain a greater understanding of the theoretical basis and practical application of two-dimensional floodplain modeling. The objectives of this project are accomplished by simulation of one and two dimensional model for different flooding scenario in the entire reach along with implementation of some flood reduction strategies.

2 Literature Review

2.1 Hydraulic Modeling

The dynamics of river has been assessed using various techniques over the years. Hydraulic model has been developed as a means of simulating flooding events to understand their potential effects and come up with a feasible solution. Conventionally, 1D models were used as a means of modeling the flood events. This approach provides a reasonable results for the flow within the channel even though it is one dimensional and assumes uniform velocity and consistent water level across the cross section (Tayefi et al., 2007). It should also be taken into account that the lateral movement of flooding in the river with complex topography is not addressed through this approach. On the other hand, 2D models are structured in a way where the lateral flow over the floodplains can also be modeled even though the computational process might be more complex. The irregular topography in the rivers leads to a mixed flow regime which leads to complex computation due to wider range of boundary and initial conditions to be considered (Popescu, 2014).

Any model is a function of the underlying governing equations which are solved using numerical approximations (Popescu, 2014). The first stage of any model is the preparation of the base such as schematics of the river (geometry) which is basically created by interconnecting nodes and allocating the upstream and downstream boundaries for the river section under study. The computation of parameters are done in these nodes. It should be noted that not all the nodes correspond with the available cross section measurements and interpolations are carried out in nodes where the data for cross sections are not available. Among the various hydraulic modelling tools, HECRAS is one of the widely used tools and it has been explained in the section below.

2.2 Modeling in HECRAS

Hydraulic Engineering Center's River Analysis System (HECRAS) is a software developed by the US Army Corps of Engineers. This software provides a wide range of applications in the area of water resource management such as steady and unsteady flow simulations, sediment transport as well as water quality analysis. Initially the unsteady flow was only capable to 1D analysis but the developers installed a new feature in the system for the 2D flow analysis in Rivers. The overview of the components of HEC-RAS has been described in the following section.

2.2.1 Geometry

The geometry in 1D and 2D model is created in a different manner. In 1D, the geometry is basically described by the connectivity between river reach cross section data and the hydraulic structures. The division between main channel and the floodplain is done by using different roughness coefficient for the cross sections. In case of 2D model, the main channel has a 1D geometry whereas the floodplain has geometry as a computational mesh created by interconnecting cells with no more than 8 faces. The mesh is connected to the 1D channel and the boundary condition by connecting the cell points to the cell 1D structure. The flow between the faces is calculated during the computation and a single water depth for each cell is calculated.

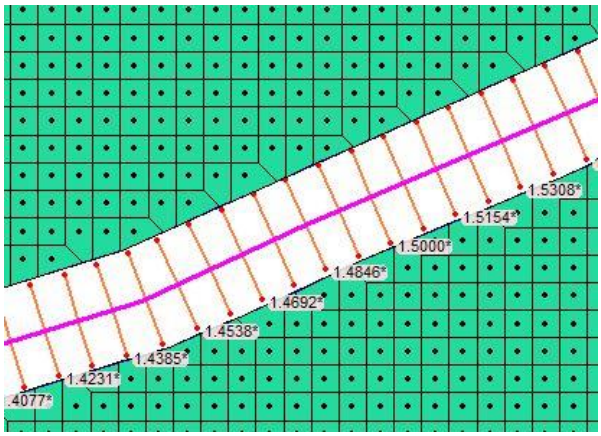


Figure 1: Example of 2D mesh with 1D channel.

2.2.2 Boundary Condition

The boundary condition that can be applied in the model are flow hydrograph, stage hydrograph, rating curve, normal depth and various other inflow (lateral/uniform). The type of boundary condition and its locations are based on the type of modeling to be carried out. For instance, for an unsteady flow, the B.C. should be specified in both the boundaries and only flow or stage hydrograph, can be used in the upstream boundary whereas flow hydrograph, stage hydrograph, both or normal depth assumption can be used in the downstream boundary. There is an additional calculation carried out using the Manning's value when the normal depth is taken.

2.2.3 Lateral Structures

Lateral Structures can be used as an option to model the overtopping of levees or other major infrastructure along the river. These structures can be weirs, culverts, gate, embankment, etc. The

flow of water above these structure can be determined when the water level in the 1D section or the 2D cell in the mesh becomes greater than the elevation of the lateral structure. These flows can be calculated using the 2D equations or weir equation.

2.2.4 Governing Equations

HEC-RAS uses the St. Venant Shallow Water Equations as basis for the analysis of the flow in the rivers. Both 1D and 2D unsteady flow works on the basis of conservation of mass (Continuity Equation) and momentum (Momentum Equation). The computations are carried out through Preissmann Scheme using Finite Difference Method. In the continuity, the idea is that the net mass in the control volume is equal to the change in storage. On the other hand, the momentum equation builds up in the idea that the rate of change of momentum is equal to the total force acting on a component. The basic idea is similar for both 1D unsteady and 2D unsteady models. However in 2D, the flow varies along two spatial dimension with time which makes the solution more complex relative to the 1D flow. The governing equations for each of the 1D and 2D flow has been described in the following sections.

1D Unsteady flow: Continuity equation

The continuity equation for 1D flow is written as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + q = 0$$

Where, Q is the flow rate, A is the cross sectional area and q is the lateral inflow.

1D Unsteady flow: Momentum Equation

The momentum equation for the 1D flow is given as:

$$\frac{\partial v}{\partial t} + g\frac{\partial}{\partial x}\left(\frac{v^2}{2g} + h\right) = g(S_0 - S_f)$$

Where, v is the velocity, g is the gravitational acceleration, h is the water depth, S_0 is the bed slope and S_f is the friction slope.

2D Unsteady Flow: Continuity Equation

The continuity equation for 2D flow is written as:

$$\frac{\partial H}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} + q = 0$$

Where H is the water surface elevation, h is the water depth, u and v are the depth averaged velocities in the x- and y-direction, and q is the source term

2D Unsteady Flow: Momentum Equation

The momentum equation for the 2D flow is given as:

In x-direction:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = -g\frac{\partial H}{\partial x} + \nu_t\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) - c_f u + fv$$

In y-direction:

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = -g\frac{\partial H}{\partial y} + \nu_t\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) - c_f v + fu$$

Where H is the water surface elevation, h is the water depth, u and v are the depth averaged velocities in the x- and y-direction, ν_t is the eddy viscosity coefficient, c_f is the friction coefficient, f is the Coriolis parameter. In the above equations, the first term denotes the local acceleration and the second and third term denotes the convective acceleration and the remaining terms denotes the forces due to gravity, eddy viscosity, bed friction and Coriolis force respectively.

The friction coefficient can be determined by using the Manning's formula which is expressed as:

$$c_f = \frac{n^2 g|u|}{R^{\frac{4}{3}}}$$

Where, where n is Manning's n, g is the acceleration due to gravity, u the velocity in the x-direction and R the hydraulic radius.

2.3 Numerical Schemes for Computation

Combination of a hybrid discretization scheme with finite differences and finite volumes is used in HEC-RAS to take advantage of orthogonality in grids. Using a Newton-like solution technique. The discrete solution for the hydraulic equations is computed.

2.3.1 Finite Difference Approximation

A finite difference scheme expresses a derivative as the difference of two quantities. This technique has already been tacitly used in the equation mentioned below, to discretize the volume derivative in time as the difference of the volumes at times *n* and *n+1* and divided by the time-step Δt.

$$\frac{\partial y}{\partial x} = \frac{\Omega(H^{n+1}) - \Omega(H^n)}{\Delta \mathrm{t}}$$

Finite differences in space work identically. Given two adjacent cells j1 and j2 with water surface elevation H1 and H2 respectively, the directional derivative in the direction n′ determined by the cell centers is approximated by the equation mentioned below. (HEC-RAS). Δn′ is the distance between the cell centers.

$$\nabla H.n' = \frac{\partial H}{\partial n'} \approx \frac{H_2 - H_1}{\Delta n'}$$

2.3.2 Finite Volume Approximations:

The finite volume method is a robust method extensively used in computational fluid dynamics (Moukalled, 2016). The finite volume method is commonly addressed as FVM. Two methods of finite volume methods are used. The first is cell vertex and second is cell-centered.

The cell vertex method uses a secondary mesh made up of a cell for every vertex. The grid cell is shown in black color in Figure below. The HEC-RAS generate a new cell for every vertex by joining the centroids of the grid cell, which is drawn in a blue color. The finite volume formulation is applied on these newly generated cells. The advantage of the cell vertex method is that the application of boundary condition becomes easy because the cell centroids lie on the boundaries.

However, the construction of secondary mesh and data structures associated with it can get a little messy, if not handled properly.

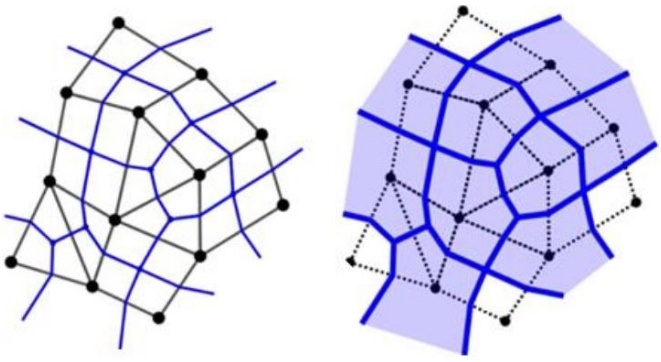


Figure 2: Cell Vertex Method and Secondary Mesh

HEC-RAS uses a hybrid method to compute the ∇H gradient term (Brunner, 2016). When the direction of the hydraulic gradient is normal to the cell face as shown then HEC-RAS uses a finite difference solution to find ∇H as shown in the equation mentioned below. In this case, cells are said to be orthogonal. H2 and H1 are the water surface elevations at cell 2 and cell 1 respectively. ΔL is the distance between the center of cell 2 and cell 1

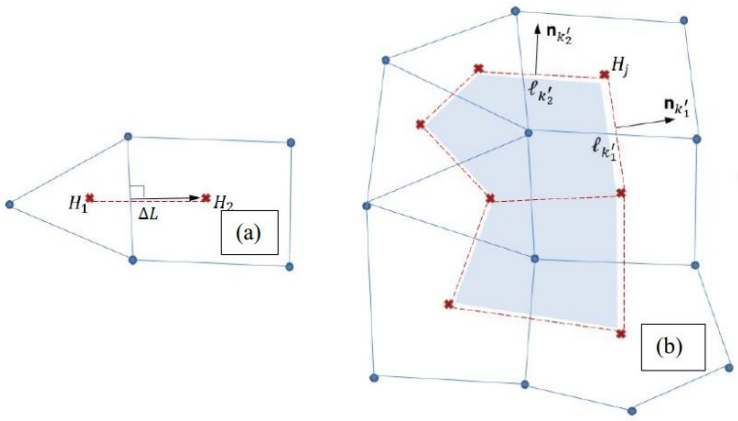


Figure 3: Dual Cell Finite Difference and Volume Formulation (Brunner, 2016)

The partial differential equation shown below, which is representative for a general governing equation is considered to derive the finite volume formulation. This equation is unsteady as shown by the ∂t term where t is time and is two dimensional as evidenced by the ∂x and ∂y terms where x and y represent spatial coordinates. The quantities m, f, g and S can be scalars and vectors (Hirsch, 1989).

$$\frac{\partial m}{\partial t} + \frac{\partial f}{\partial x} + \frac{\partial g}{\partial x} = S$$

Integrating the equation over a single cell, we can write the equation as shown in Equation below and A is the area of the cell and Ω is the volume of the cell.

$$\iint_{\Omega} \frac{\partial m}{\partial t} dA + \iint_{\Omega} \left(\frac{\partial f}{\partial x} + \frac{\partial g}{\partial y}\right) dA = \iint_{\Omega} S dA$$

The cell can be of any shape like a triangle, quadrilateral or some other polygon. HEC-RAS limits the grid cell to no more than eight sides. Now, the simplification is the next step, which leads us to the finite volume formulation. We simplify the first term by using the average of m, which is defined as the integral of m divided by an area of the cell. Similarly, the integral of the source term gets simplified by using the definition of the average of the source term as follows.

$$\iint_{\Omega} \frac{\partial m}{\partial t} dA = A \frac{\partial m'}{\partial t}$$

$$\iint_{\Omega} S dA = S'A$$

However, the most interesting term is the area integral of the space derivative terms. These terms get simplified by using the Gauss-Divergence Theorem, which converts the area integral over the cell footprint to a line integral over the closed boundary of the cell (Hirsch, 1989). The lines

enclosing the cell open up and then a line integration is out by summing the contributions of these integrals over all faces as shown in Equation 3.9

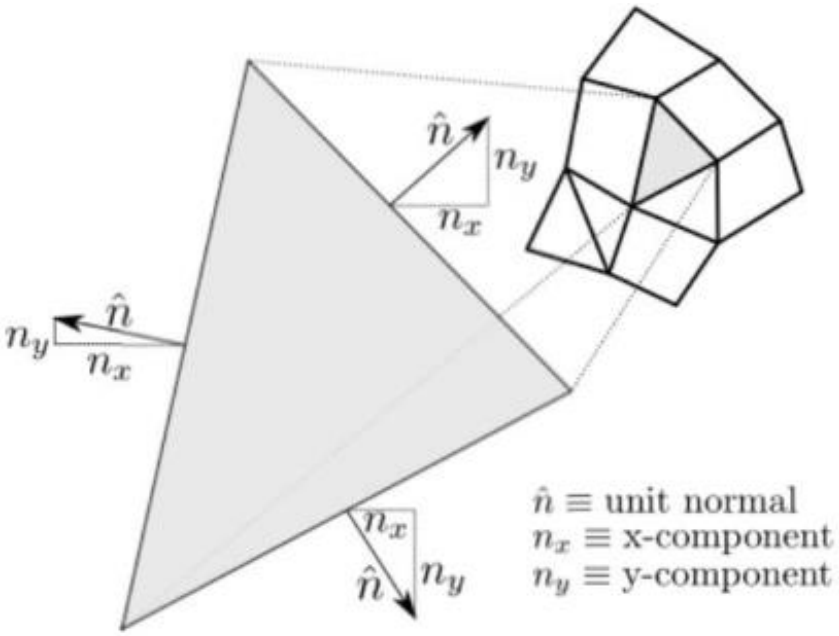


Figure 4: Unit Normal ṅ Components

The unit normal ṅ has components (n_x and n_y), which are going to be different for each of the faces as shown above in Figure 8. The unit normal vector points outward of the cell (and not inward). Now application of the integrals to each of the faces can be obtained in the following equations. Using the definition of average, the expressions can be further simplified.

$$\oint (\mathrm{fn}, \mathrm{x} + \mathrm{gn}, \mathrm{x}) \mathrm{dL}$$

$$\sum_{faces} \int_{face} (\,(\mathrm{fn}, \mathrm{x} + \mathrm{gn}, \mathrm{x}) dL$$

$$\sum_{faces} (\mathrm{fn}, \mathrm{x} + \mathrm{gn}, \mathrm{x}) \Delta L$$

The Finite Volume formulation is the sum of the fluxes exchanged between neighboring cells, when the source terms is absence (Hirsch, 1989). The steps of FVM depend on the scheme used for calculation of fluxes and time integration.

$$A\frac{\partial m'}{\partial t} + \sum_{faces} (\mathrm{fn}, \mathrm{x} + \mathrm{gn}, \mathrm{x})\Delta L = S'A$$

We use a simple Cartesian mesh to solve a 2D equation as shown in Figure 9. This equation looks like the governing equation for which have just obtained a finite volume formulation.

$$\frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} + q = 0$$

Where t is time, h is the water depth, q is a source or sink term and u and v are the velocity components in the X and Y direction. Now, continuing the procedure of FVM as shown earlier. Then, ΔXΔY refers to the area of the rectangular grid cell.

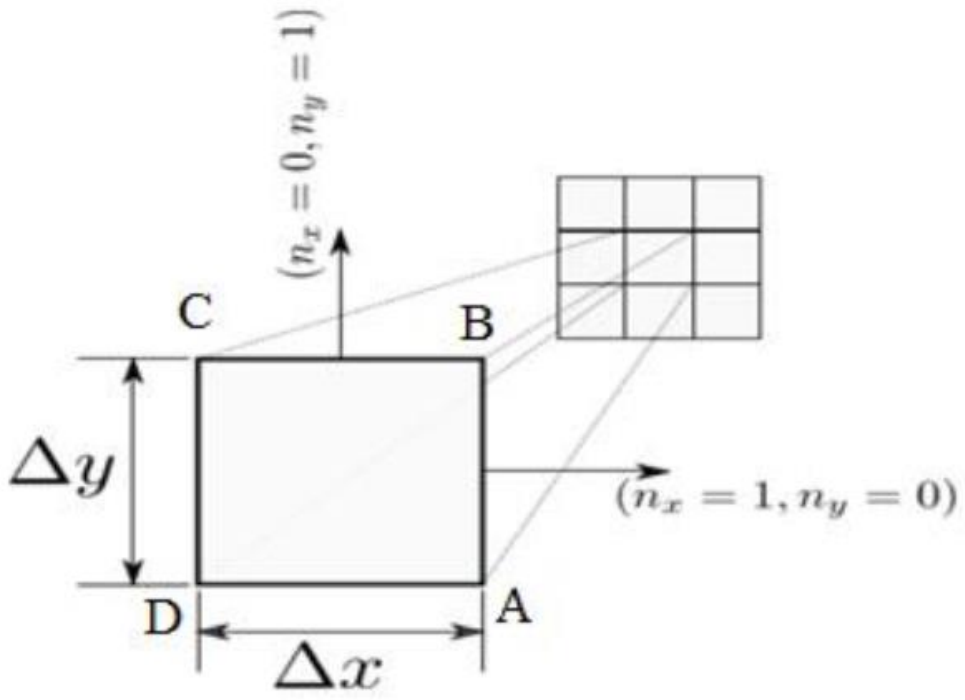


Figure 5: Cartesian Mesh

$$\frac{\partial h}{\partial t} + \frac{\partial f}{\partial x} + \frac{\partial g}{\partial y} = 0$$

$$\iint_{\Omega} \frac{\partial h}{\partial t} dxdy + \iint_{\Omega} (\frac{\partial f}{\partial x} + \frac{\partial g}{\partial y}) dxdy = 0$$

$$\Delta X \Delta Y \frac{\partial h'}{\partial t} + \oint (\mathrm{fn,x} + \mathrm{gn,x}) \Delta X \Delta Y = 0$$

For a Cartesian mesh, it easily simplifies further to as single term inside the line integral. In Cartesian mesh, the vertical faces have $n_y = 0$, therefore the term $g_{n,y}$ goes away and for horizontal faces $n_x = 0$, therefore, $f_{n,x}$ becomes zero. Also simplifying this further by using the average values of flux (flowrates) over the faces.

$$\Delta X \Delta Y \frac{\partial h'}{\partial t} = -\oint ((\mathrm{fn,x}) \Delta X) - \oint ((\mathrm{gn,y}) \Delta Y) = 0$$

$$\Delta X \Delta Y \frac{\partial h'}{\partial t} = -\sum_{faces} (\mathrm{f'n,x}) \Delta X - \sum_{faces} (\mathrm{g'n,y}) \Delta Y$$

2.4 Flood mitigation measures

The attempts of man to protect himself from flooding are as old as the history of civilization. Flood control measures may be divided into the engineering measures (e.g. the construction of reservoirs, dikes, diversion of flood flows, improvement of river channel) or administrative measures (flood forecasting, flood plain zoning, flood insurance) (Edward, 1965).

Reservoirs:

Storing flood water in the upstream part of a drainage basin is the most direct way to reduce the flood hazard in the downstream part of the basin. There are three places where water can be stored: in the ground, in small reservoirs on creeks and minor streams and in large reservoirs on the major stream channels of the river system.

Dikes:

The oldest, most common and often most economical means of flooding protection is by constructing a system of dikes. Dikes should be kept at a good distance away from river channels. The very need for dikes indicates that we are probably dealing with an alluvial river flowing in a flood plain. Such rivers have a natural habit of continuously eroding their banks.

The great hazard involved in a diking system is that it provides full protection up to a certain flood stage and no protection at all for higher stages. A long period of absence of extreme flood stages will create a feeling of security for inhabitants of dike-protected areas, leaving them unprepared for an eventual failure of the dike (Edward, 1965).

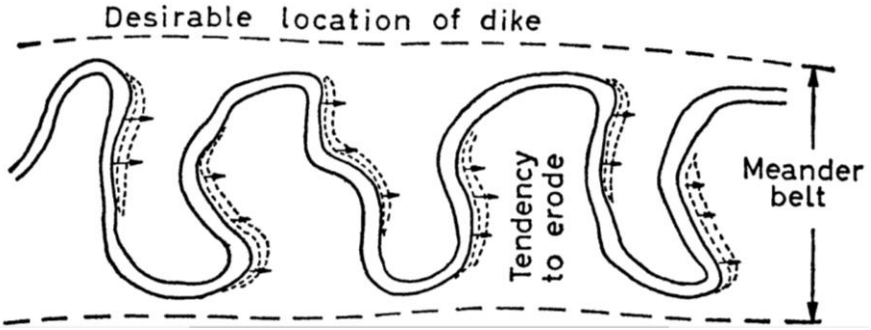


Figure 6: Dikes along the meanders of the river

Flood Diversion:

The most direct and effective way to cope with a flood situation is to take water away from the river channel. A primitive way of accomplishing this is to breach a dike on purpose, in an area where the resultant damage is relatively small, in order to save the dikes that protect another area where the damage would be relatively large. Some requirements are that the area into which the floodwaters are diverted is free of habitation. A diversion channel will be most effective in lowering water levels, if the diverted water can be taken away from the river, without returning it farther downstream.

Dredging

Dredging is the operation of removing material from one part of the water environment and relocating it to another (EuDA, 2018). The overall goal of most dredging activities is to reduce the extent of flooding and perform as a flood management tool. It is sometimes effective because

nature always fills the river bed with sediments. For this reason, for long-term solution, sometimes dredging is not a good concept.

Renaturation of flood plains

By increasing the channel roughness, flood can be mitigated. However, if the channel roughness is increased, some inferior ecological effects might be experienced.

3 Study Area

3.1 Description

The Spree River originates in the Lausatian Mountains in the border of Czech and Germany. It flows over a stretch of 380km before meeting the Havel River in Berlin. (Nones and Gerstgraser, 2015).

The study area includes approximately 11 km restored river reach of Spree River which is used to simulate flooding scenario using one and two dimensional hydrodynamic model. The study site is located approximately 10 km north (downstream) of Cottbus, near coal mining activities (Figure 1). The catchment area is about 62 km2. The longitudinal gradient of the River reach is around 0.07%. The River has a mean discharge of 7.5 m3 /s and a bankfull discharge of 35 m3 /s.

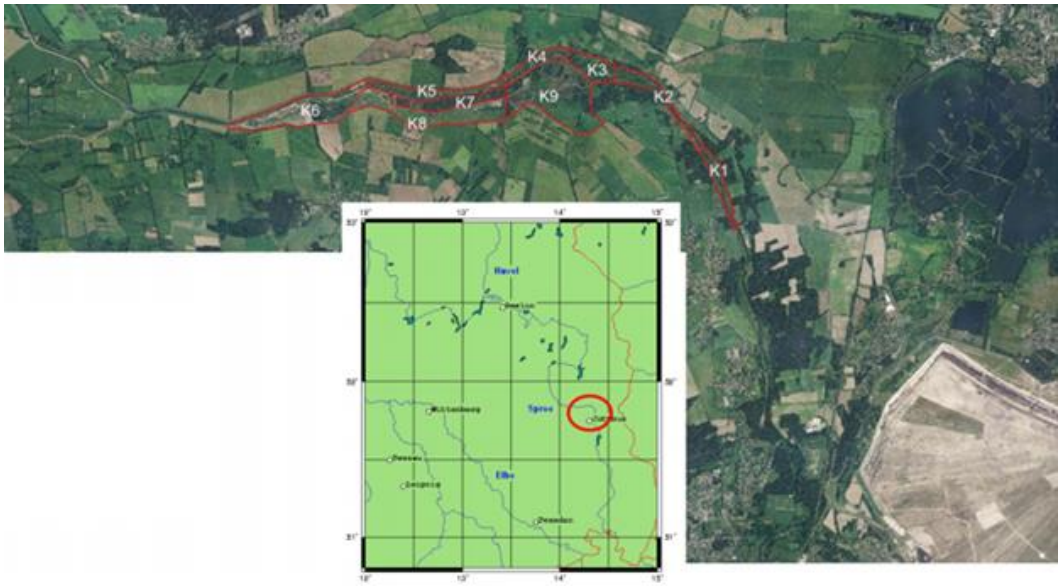


Figure 7: Satellite image of the area of study

3.2 Renaturation Project

There had been a lot of manipulation in the Spree River over the years. The river was straightened, regulated and diked. On top of that agricultural lands were created by clearing out the forests along the river. This led to the deterioration of the natural structure of the area causing the fall in regenerative capacity as well as the ecological efficiency. In the previous times, the Spree River consisted of a widely branched river system with floodplain extending up to 5km which was changed due to the involvement of human to change the landscape for their benefit. Therefore, in order to reestablish the efficient natural environment in the river, various works was carried out over the period of 2006 to 2014.

This renaturation project was one of the largest river renovations in the state of Brandenburg. The extent of the renaturation was over a length of 11km and an area of 400 ha. A large number of works were carried out such during this period. Both the structural as well as the ecological measures were carried out. New ponds were developed for the fish communities and their population monitored constantly, new habitats of plants and animals were created through extensive plantation. In addition to that, new floodplain was created by relocating the dikes and creating new gutters. This integrative approach led to a creation of good natural condition in the river and its surrounding areas.

Figure 8: Photographs show the some perspective view of river Spree

Figure 9: Some Photographs of floodplain areas of river Spree

Figure 10: Aerial photographs of restoration works at Spree river floodplain

Chapter 4: Implementation of 1D and 2D unsteady model

Set-up of 1D unsteady model:

A unsteady simulation for the basic model is created with simulation window to be starting from 1st September to 31 October. Different time step was considered such as 1s, 5s, 10s, 15s etc. minute time step. Initial conditions was chosen as initial flow 14 m3/s at reach no. 58.

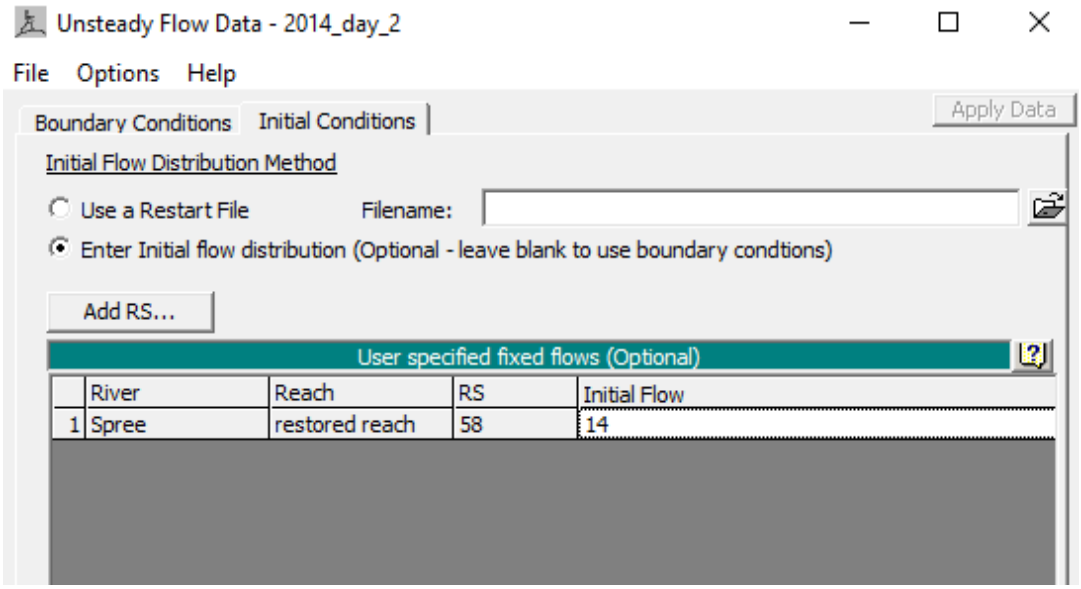


Figure 11: Initial Condition.

There are four types of external boundary conditions that may be linked directly to the 2D Flow

1. Flow Hydrograph
2. Stage Hydrograph
3. Normal Depth
4. Rating Curve

The Normal Depth and Rating Curve boundary conditions can only be used at locations where flow will leave the 2D Flow Area. The flow and stage hydrograph boundary conditions can be used for putting flow into or taking flow out of a 2D Flow Area. For a Flow Hydrograph, positive flow values will send flow into a 2D Flow Area, and negative flow values will take flow out of a 2D area. For the Stage Hydrograph, stages higher than the ground/water surface in a 2D Flow Area will send flow in, and stages lower than the water surface in the 2D Flow Area will send flow out. If a cell is dry and the stage boundary condition is lower than the 2D Flow Area cell minimum elevation, then no flow will transfer (Brunner, 2016). In this model, a flow hydrograph considered at upstream and normal depth considered at downstream as boundary condition. Friction slope of 0.0003 was considered to be in normal depth section as boundary condition at downstream cross section.

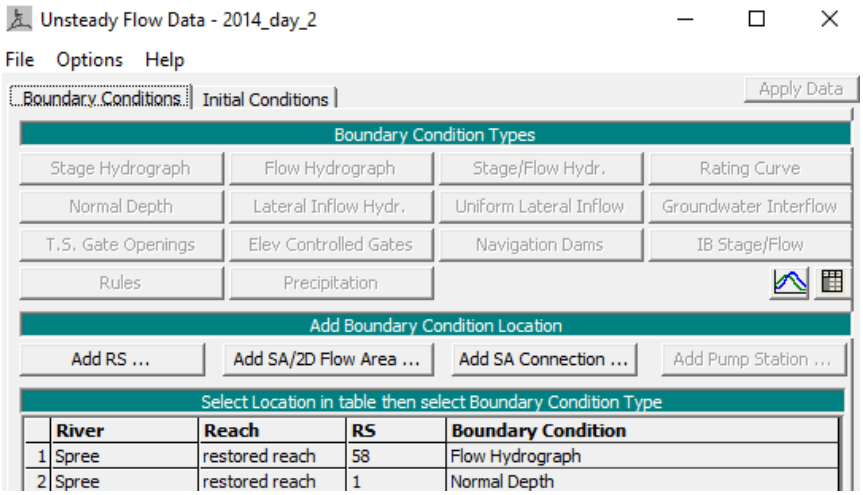


Figure 12: Boundary Condition

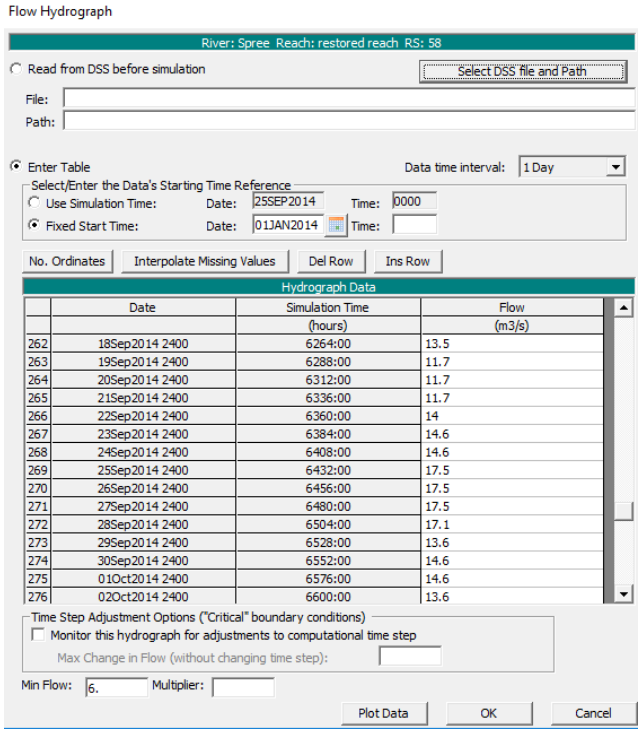


Figure 13: Flow hydrograph at upstream section shows time range of the maximum flow

Implementation of 2D floodplain model:

HEC-RAS 5.0 can perform simulation with 2D floodplain with desired mesh size. This chapter provides a detailed description for creating a 1D and 2D floodplain model using HEC-RAS for

Spree river and the floodplain. The RAS Mapper can import floating-point grid format (*.fit), GeoTIFF (*tif) and other formats (Brunner, 2016). Figure 14 shows the terrain model used in the study.

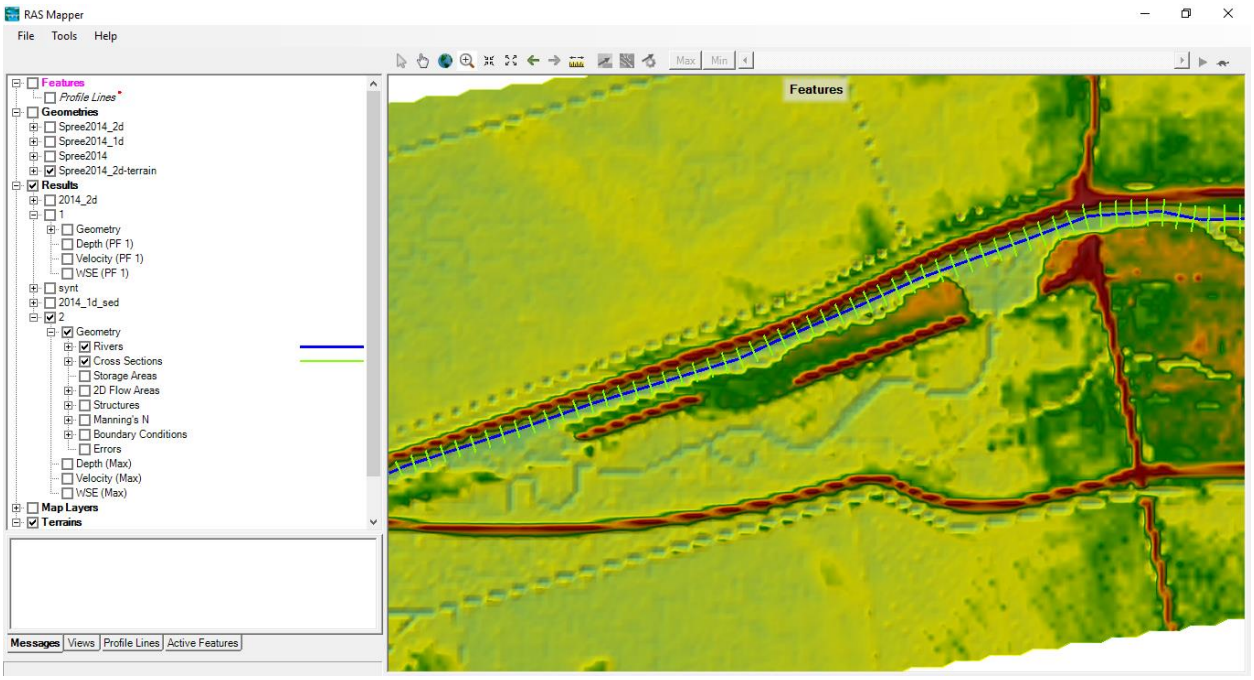


Figure 14: Terrain of floodplain

The terrain data does not often include the actual channel bathymetry underneath the water surface. RAS Mapper now can modify the terrain data to include channel bathymetry by using the individual HEC-RAS cross-sections geometry and the cross section interpolation feature. The result from this step is to generate the channel terrain layer. The channel terrain layer is created by taking the channel bathymetry data from the cross sections and using the interpolation feature to interpolate an elevation for each grid cell between any two-cross sections (Brunner, 2016).

2D Flow Area Mesh:

Main conception of 2D floodplain modeling is usage of a computational mesh. HEC-RAS uses an amalgamation of a finite-difference and a finite volume method to compute water elevation at the center of each computational cell for specific time step. 2D modeling features in HEC-RAS allow user to generate computational mesh. In the Geometric Data Editor, the modeler can specify the

limits of the computational mesh that envelopes the channel itself plus any adjacent floodplain areas.

Spatial details describing the polygon can be defined with 2D Flow Area Editor button. Spatial details include the size of the individual 2D flow cells as well as Manning's roughness values for each cell. Manning's roughness values can be defined for specified land use using GIS techniques. After the spatial details of the computational mesh, then detailed information describing the computational grid including hydraulic property table can be generated. There are tolerance input boxes that allow the user to have some control of the 2D grid. Finally, boundary conditions at the upstream and downstream ends using must be defined **(**Dewberry, 2016**).**

Connecting a 2D Flow Area to a 1D River Reach with a Lateral Structure:

The 2D Flow Area elements can be connected to 1D elements in several ways: directly to the downstream end or the upstream end of a river reach; laterally to 1D river reaches using a Lateral (s); and/or directly to another 2D area or storage area using the SA/2D Area Connection. The process for connecting the 2D Flow Area to other hydraulic elements is accomplished in the HEC-RAS Geometric editor. 2D Flow Areas can be used to model areas behind levees or overbank flow by connecting a 1D river reach to the 2D area using a Lateral Structure

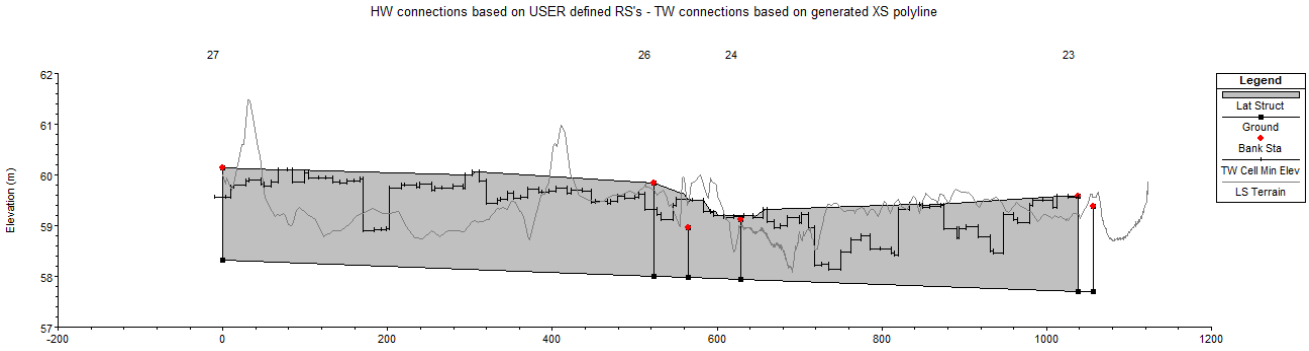


Figure 15: Lateral Structure section.

Figure 16: Highlighted lateral structure (k6) in plan view.

In general, Lateral Structure weir coefficients should be lower than typical values used for inline weirs. Additionally, when a lateral structure (i.e. weir equation) is being used to transfer flow from the river (1D region) to the floodplain (2D Flow Area), then the weir coefficients that are used need to be very low, or too much flow will be transferred. Below is a table of rough guidelines for Lateral weir coefficients under different conditions.

Table 1: Lateral weir coefficient

What is being modeled with the Lateral Structure	Description	Range of Weir Coefficients
Levee/Roadway – 3ft or higher above natural ground	Broad crested weir shape, flow over Levee/road acts like weir flow	**1.5** to **2.6** (2.0 default) SI Units: 0.83 to 1.43
Levee/Roadway – 1 to 3 ft elevated above ground	Broad Crested weir shape, flow over levee/road acts like weir flow, but becomes submerged easily.	**1.0** to **2.0** SI Units: 0.55 to 1.1
Natural high ground barrier – 1 to 3 ft high	Does not really act like a weir, but water must flow over high ground to get into 2D area.	**0.5** to **1.0** SI Units: 0.28 to 0.55
Non elevated overbank terrain. Lat Structure not elevated above ground	Overland flow escaping the main river.	**0.2** to **0.5** SI Units: 0.06 to 0.28

Chapter 5: Results

Time Step:

Case 1:

Time step: 1 Sec	Manning's n: 35	Cross-sectional distance: 10

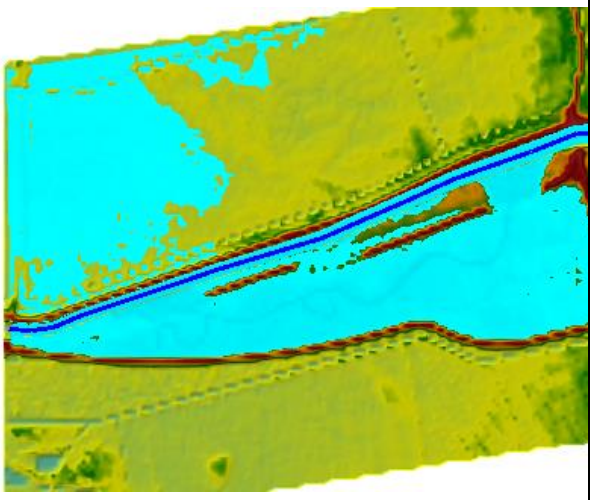
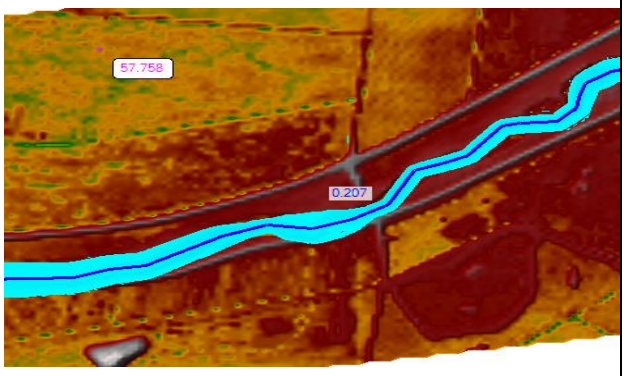
Sub-reach K6	Sub-reach K4
	

Figure 17: Flooding extent in the floodplain zones K6 and K4 (time step 1sec)

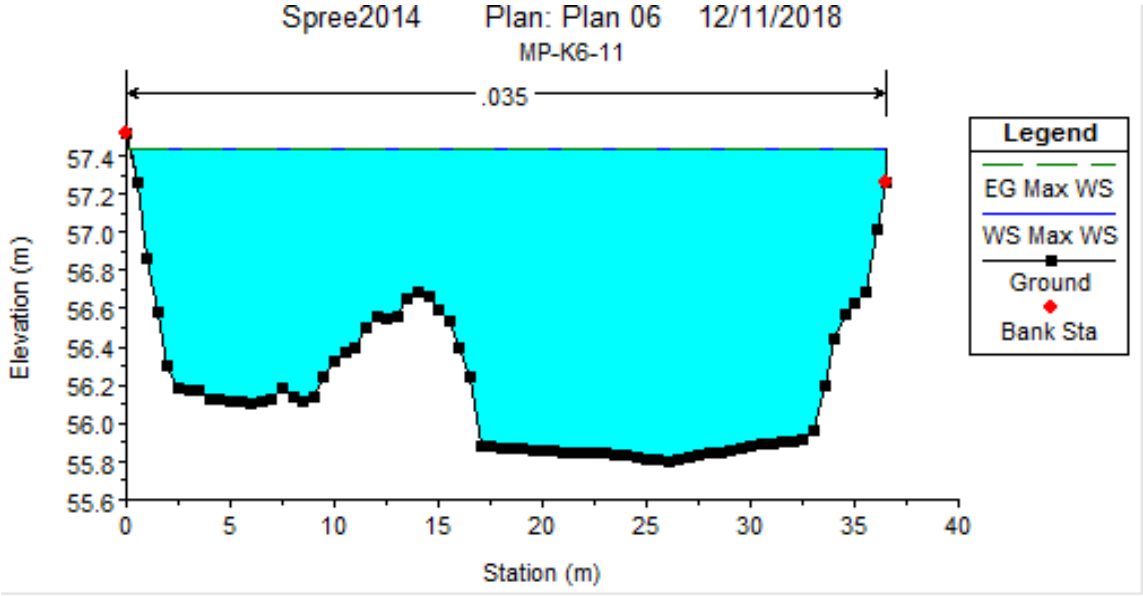


Figure 18: Cross section (XS 2) in floodplain zone K6 (time step 1sec)

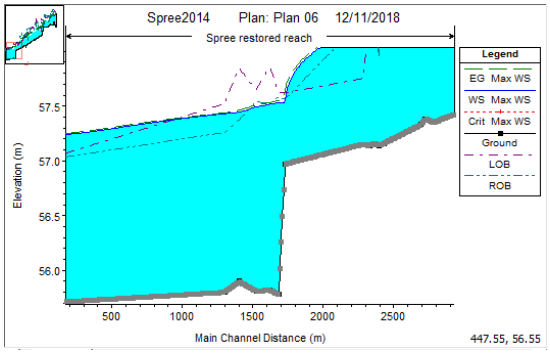
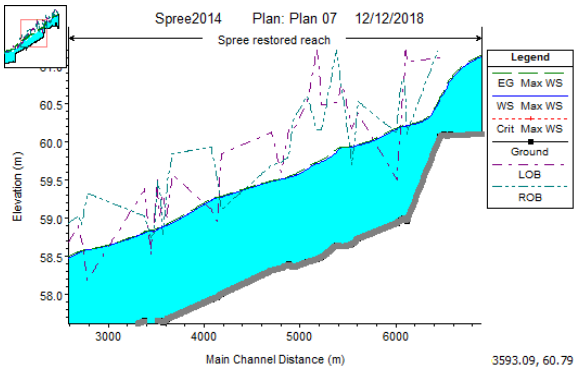
Sub-reach K6	Sub-reach K4
	

Figure 19: Longitudinal profile shows water surface level at floodplain region K6 and K4 (time step 1sec)

Case 2:

Time step: 5 Sec	Manning's n: 35	Cross-sectional distance: 10

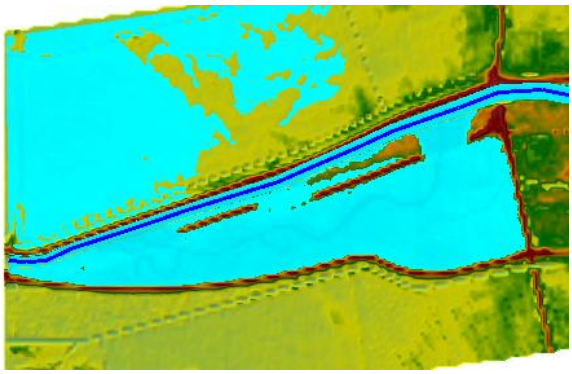
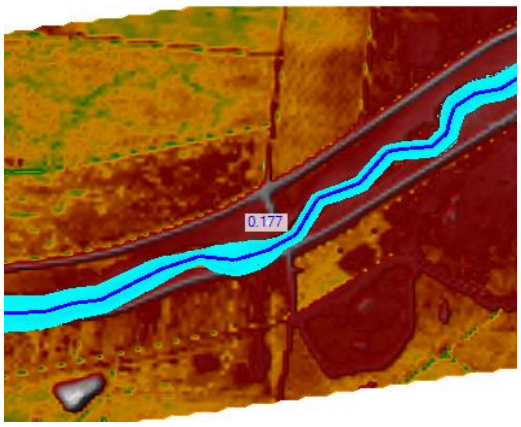
Sub-reach K6	Sub-reach K4
	

Figure 20: Flooding extent in the floodplain zones K6 and K4 (time step 5sec)

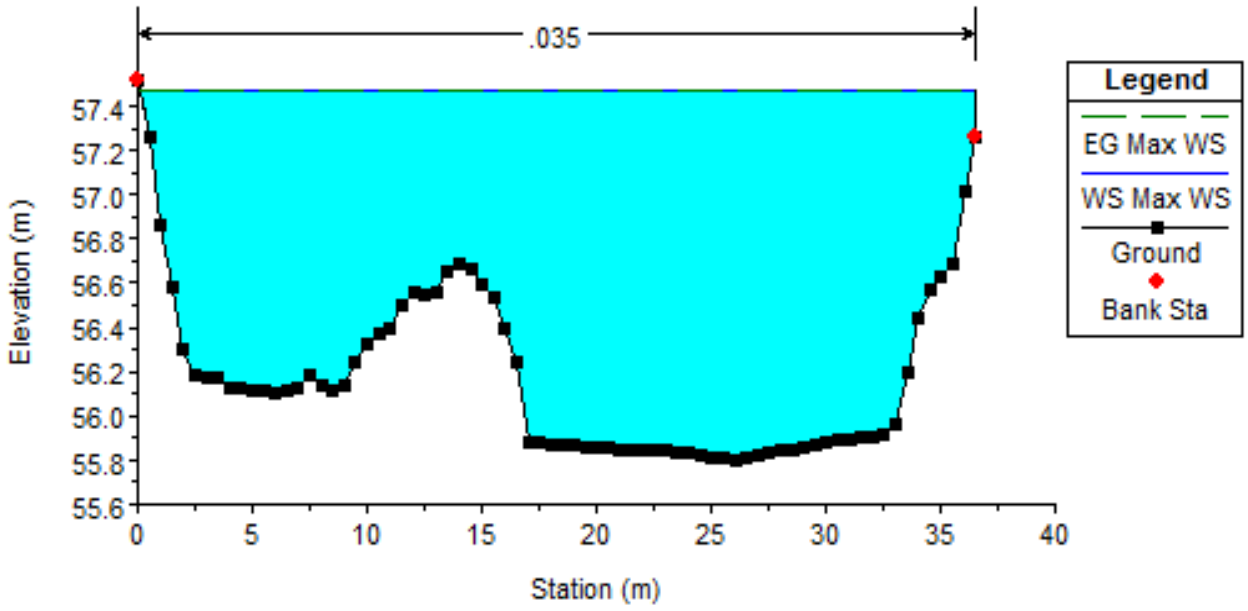


Figure 21: Cross section (XS 2) in floodplain zone K6 (time step 5sec)

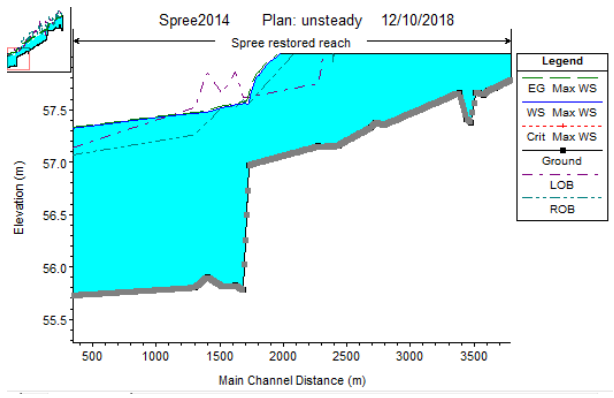
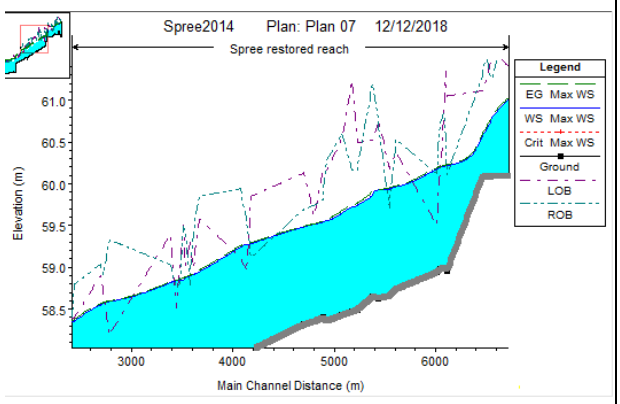
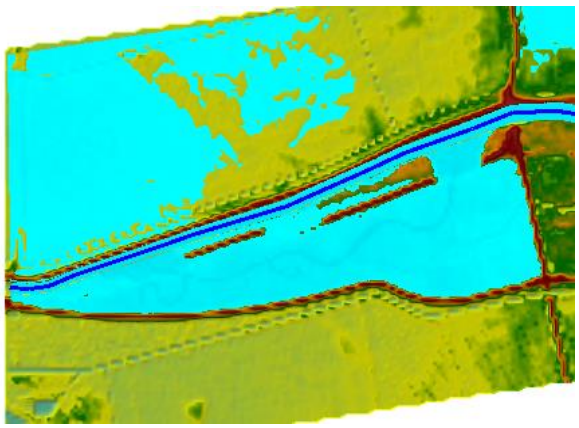
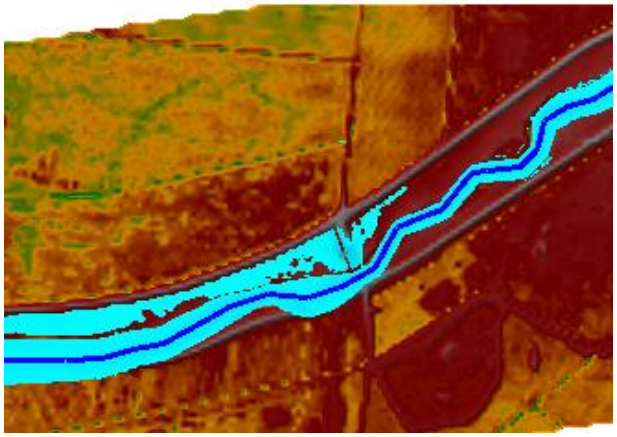
Sub-reach K6	Sub-reach K4
	

Figure 22: Longitudinal profile shows water surface level at floodplain region K6 and K4 (time step 5sec)

Case 3:

Time step: 10 Sec	Manning's n: 35	Cross-sectional distance: 10

Sub-reach K6	Sub-reach K4
	

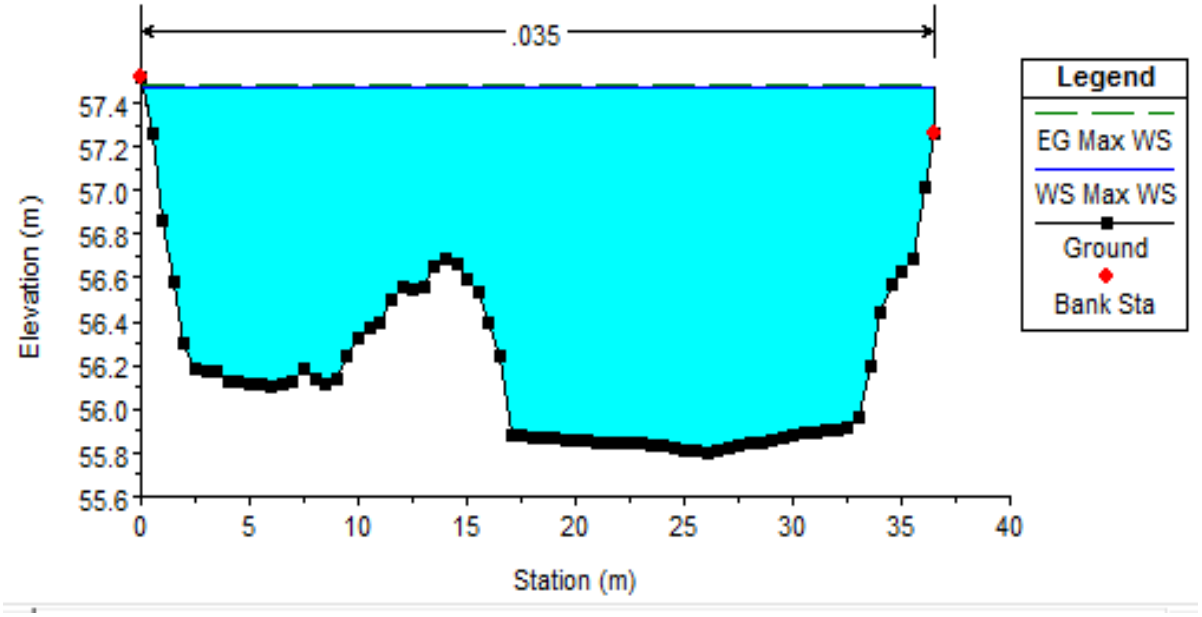


Figure 23: Flooding situation and WSE in cross section (time step 10 sec)

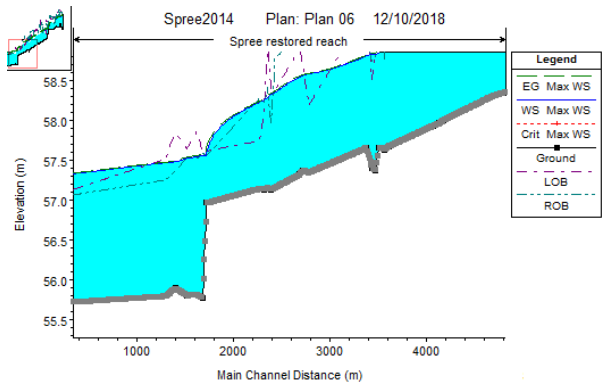
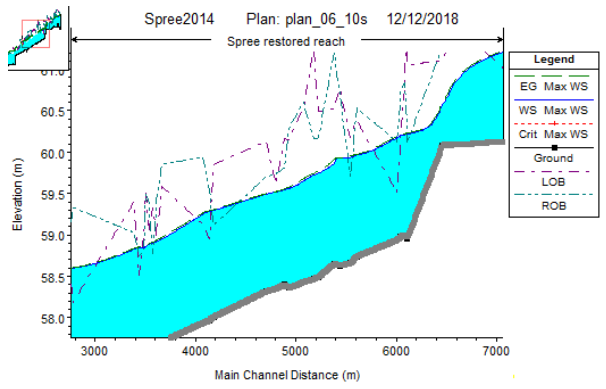
Sub-reach K6	Sub-reach K4
	

Figure 24: Longitudinal profile shows water surface level at floodplain region K6 and K4 (time step 10 sec)

Case 4:

Time step: 15 Sec	Manning's n: 35	Cross-sectional distance: 10

Flooding scenario in 2D floodplain, water surface elevation in different cross section as well as along the longitudinal profile of the river are found to be approximately similar as previous cases.

For higher time steps (e.g. 20 sec, 30 sec, 1 min etc. the simulation doesn't work and shows error message.

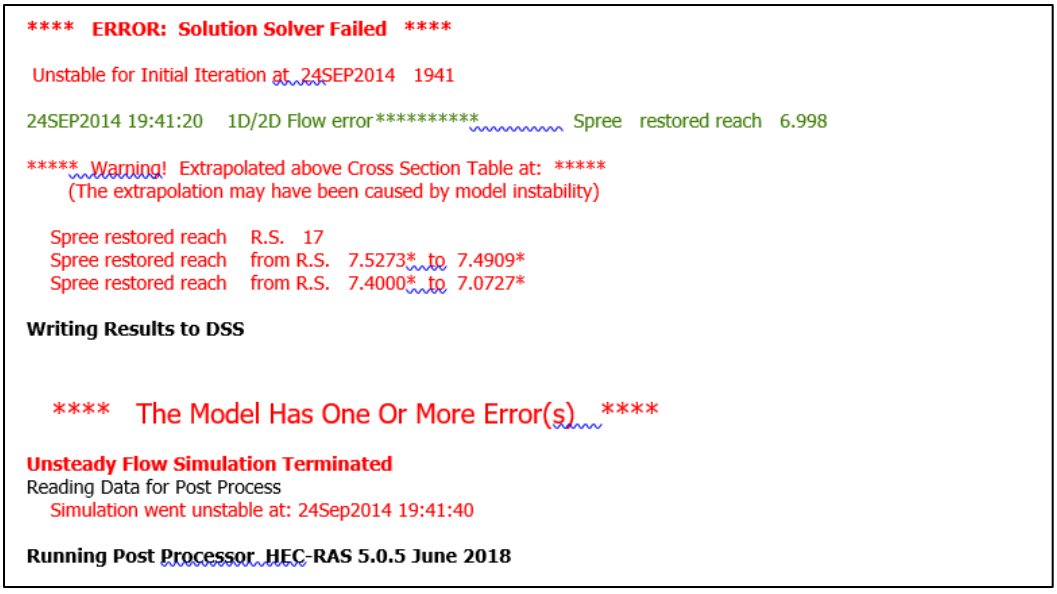


Figure 25: Error message

Cross sectional distance:

Case 1: Time step, Manning's roughness coefficient and cross sectional distance (dx) are considered to be 5 seconds, 35 and 10m for the case no. 1. The water surface level and flood scenario is almost same as previously mentioned case 2 of time step.

Case 2:

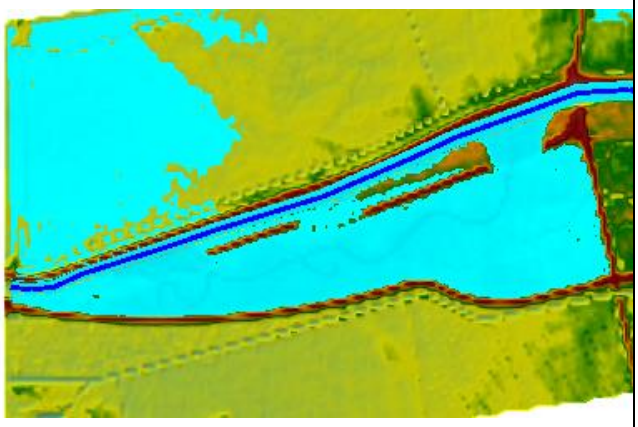
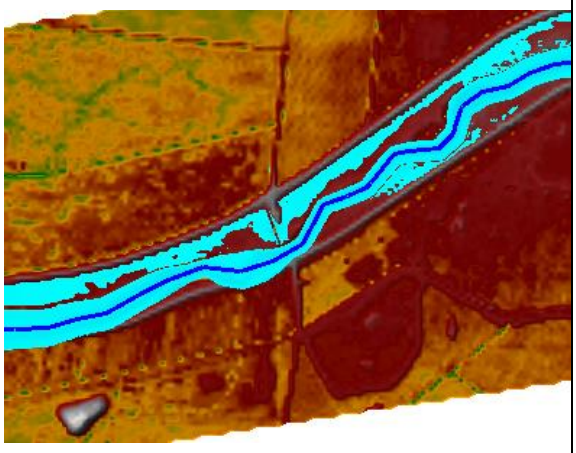
Time step: 10 Sec	Manning's n: 35	Cross-sectional distance: 20
Floodplain zone K6	Floodplain zone K4	
		

Figure 26: Flooding situation in floodplain zone K6 and K4 (dx=20m)

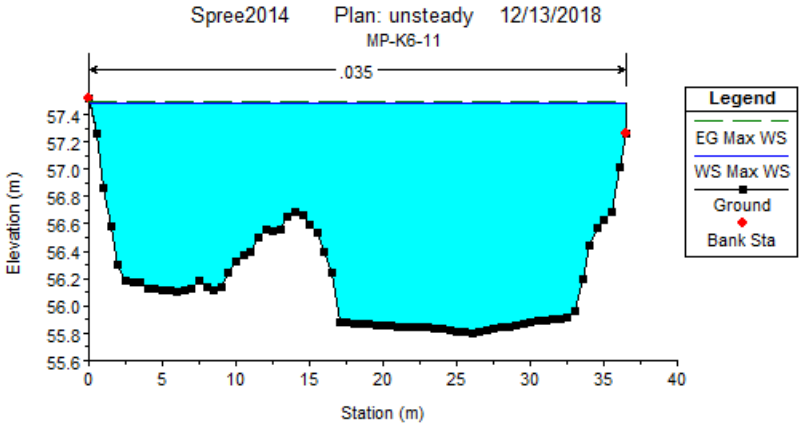


Figure 27: River cross section (XS 2) in floodplain zone K6 shows water surface (WS) elevation (dx=20m)

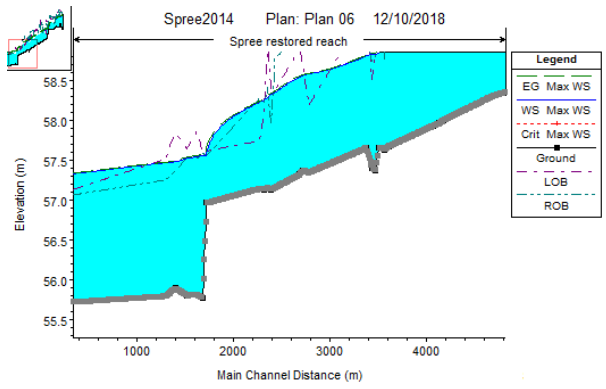
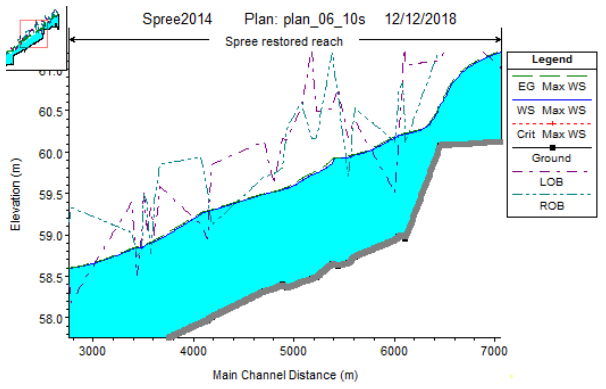
Sub-reach K6	Sub-reach K4
	

Figure 28: River cross section (XS 2) in floodplain zone K6 shows water surface (WS) (dx=20m)

Case 3:

Time step: 10 Sec	Manning's n: 35	Cross-sectional distance: 50

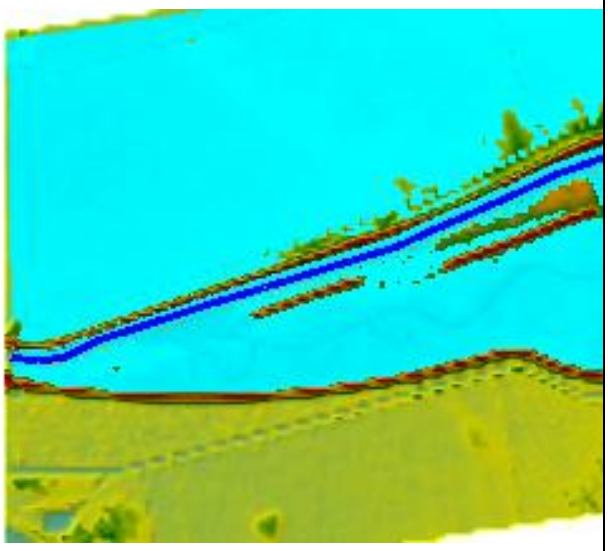
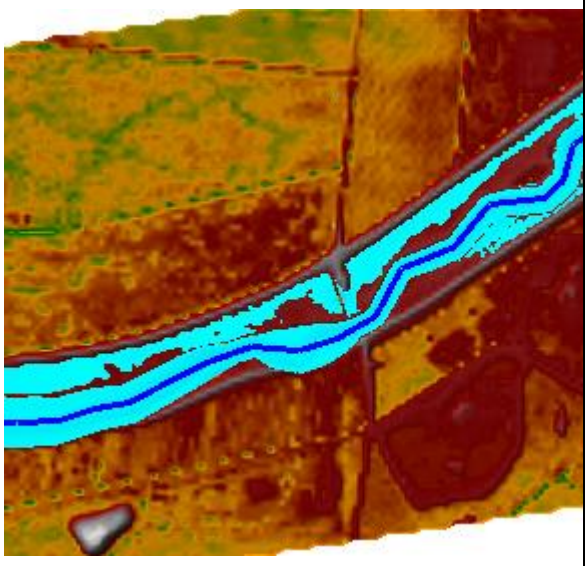
Sub-reach K6	Sub-reach K4
	

Figure 29: Flooding situation in floodplain zone K6 and K4 (dx=50m)

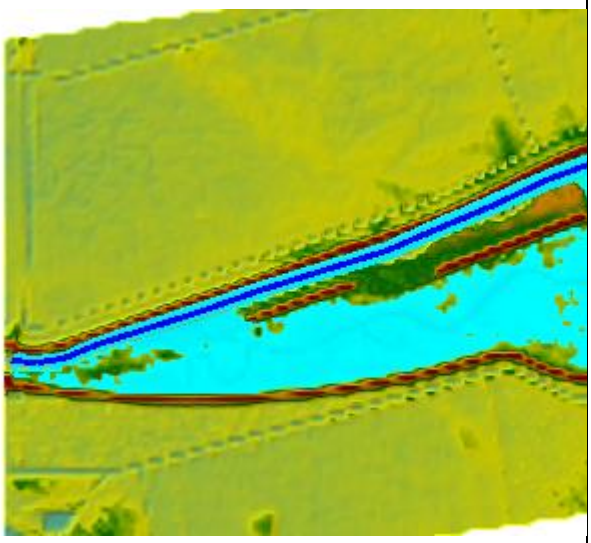
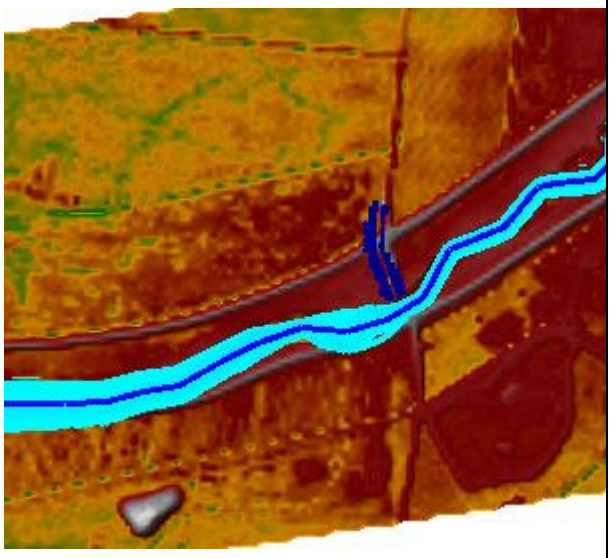
Manning's n:

Case 1:

Time step: 5 Sec	Manning's n: 35	Cross-sectional distance: 10

Water surface level and flood scenario is almost same as previously mentioned case 2 of time step.

Case 2:

Time step: 10 Sec	Manning's n: 30	Cross-sectional distance: 10
Sub-reach K6	Sub-reach K4	
		

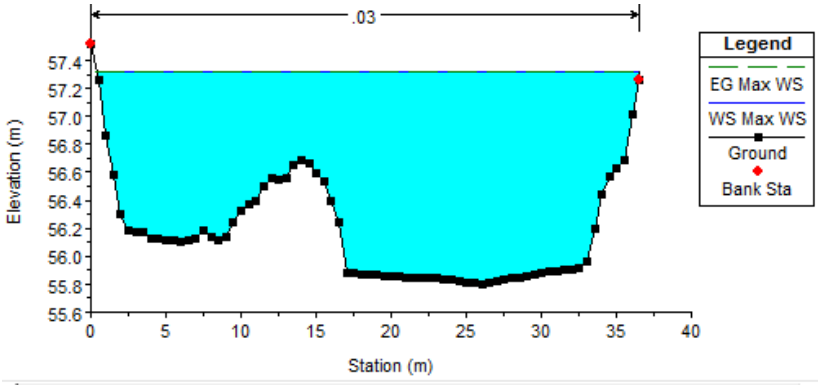


Figure 30: Flooding situation and WSE in cross section (time step 10 sec)

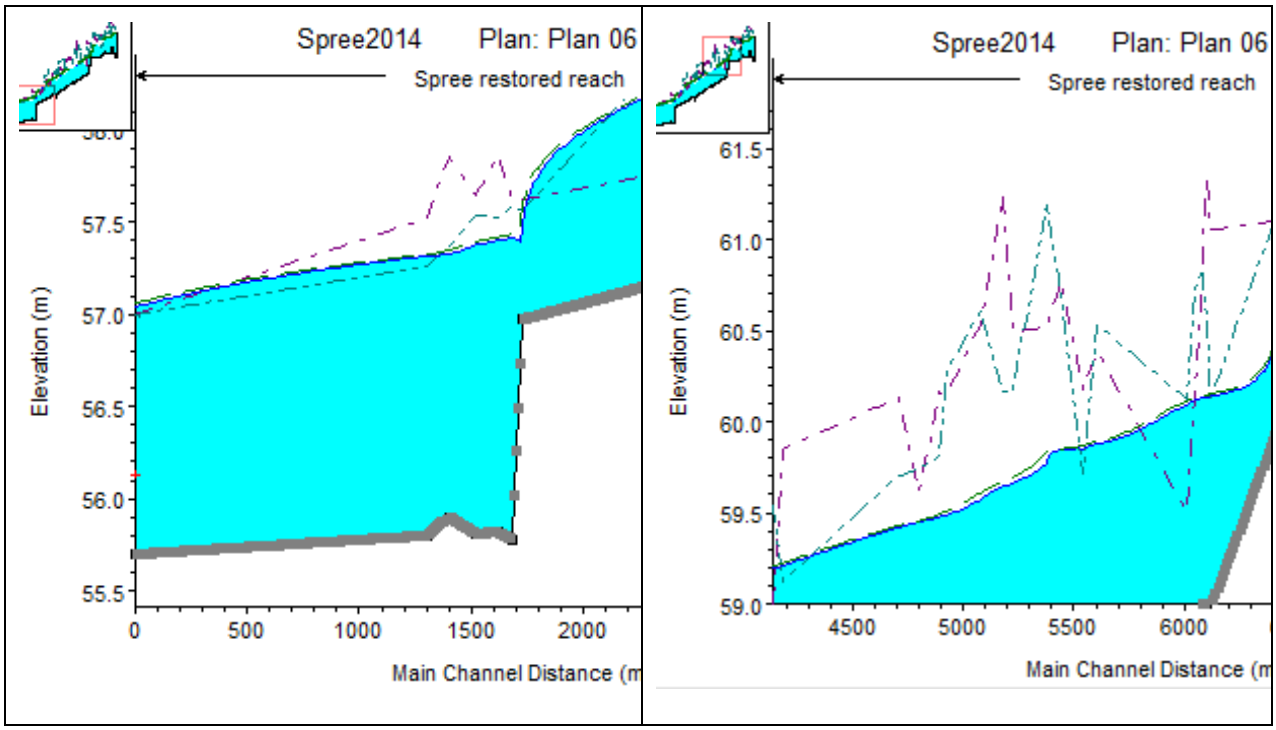
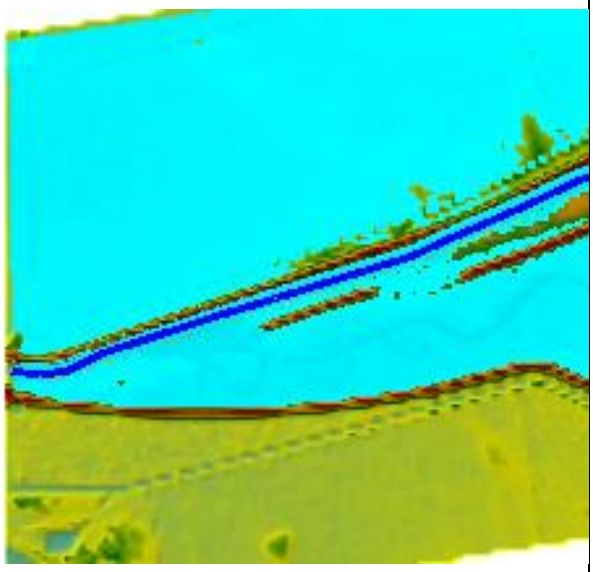
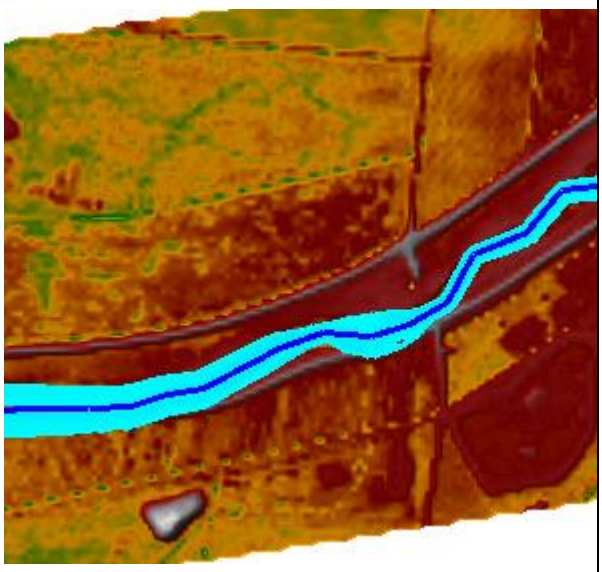


Figure 31: Portion of longitudinal profile in Floodplain zone K4 and K6 shows the water surface with changed Manning's n (30)

Case 3:

Time step: 10 Sec	Manning's n: 40	Cross-sectional distance: 10
Sub-reach K6	Sub-reach K4	
		

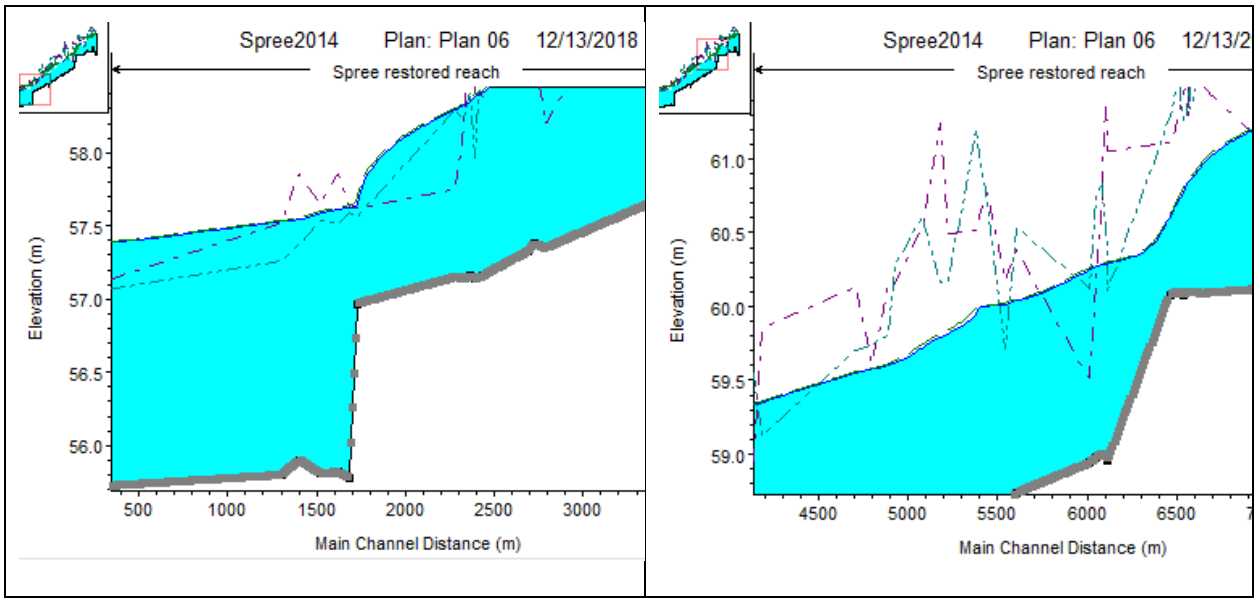


Figure 32: Longitudinal profile after adopting Manning's n to be 40

Sensitivity Analysis

Appropriate grid dimension and computational time step are very important to assess flooding in the 2D areas. The hydraulic modeling systems are powerful system to choose spatial and time variables to the desired degree of accuracy (HEC-RAS manual). In the hydro-engineering project, different computational time step and grid size were analyzed to assess flooding situation in the project area. The total project area are divided into six sub-floodplain (sub-reach).

Computational Time Step

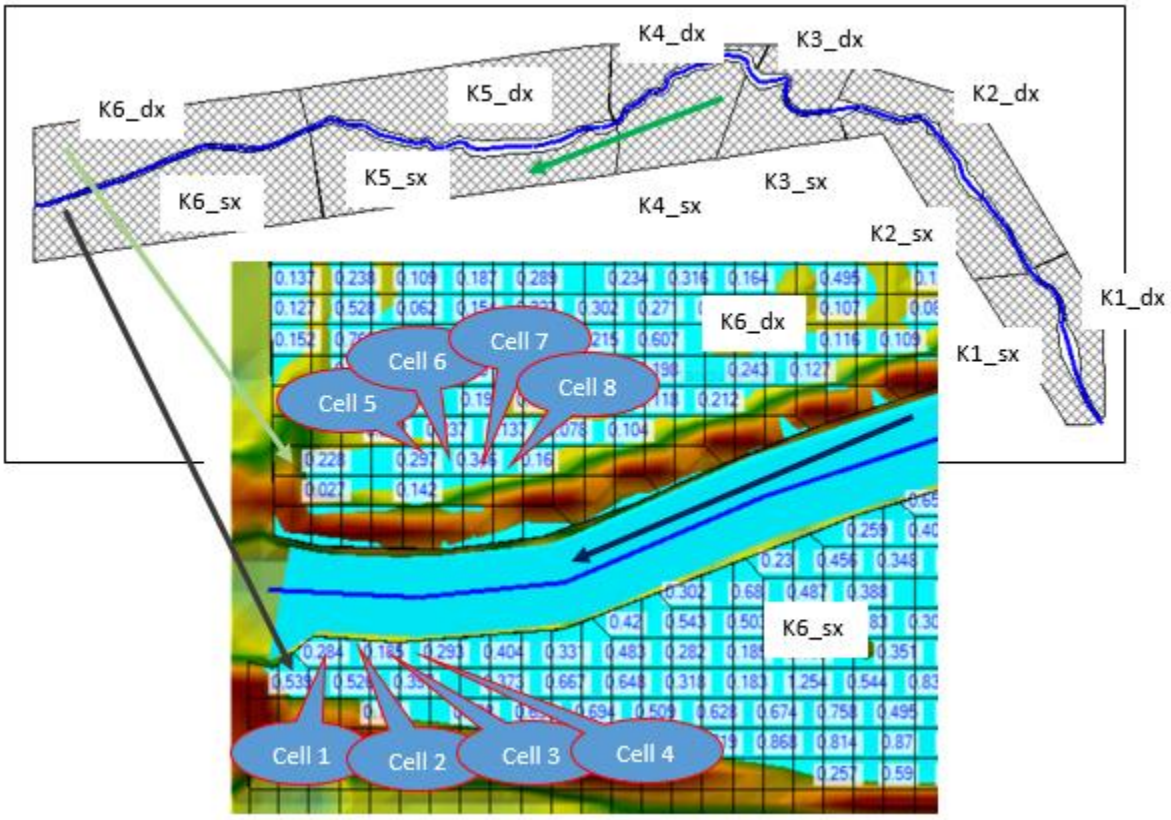


Figure 33: Project area showing different cells for flood depth comparison in different time step

For the sub-reach K6, flood was calculated in some specific area for different time step, following table shows comparison of flood depth.

Table 2: Maximum flood depth value in different cells for certain time steps in sub-reach K6

Time step (seconds)	Cell1	Cell2	Cell3	Cell4	Cell5	Cell6	Cell7	Cell8
	Flood Depth (m)							
1	1.36	1.1	1	1.2	0.37	0.3	0.26	0.2
5	1.43	1.17	1.06	1.27	0.19	0.12	0.1	0.07
6	1.44	1.18	1.07	1.28	0.14	0.08	0.06	0.05
10	1.44	1.18	1.07	1.28	0.14	0.08	0.06	0.05
15	1.38	1.12	1.01	1.22	0.47	0.38	0.34	0.28

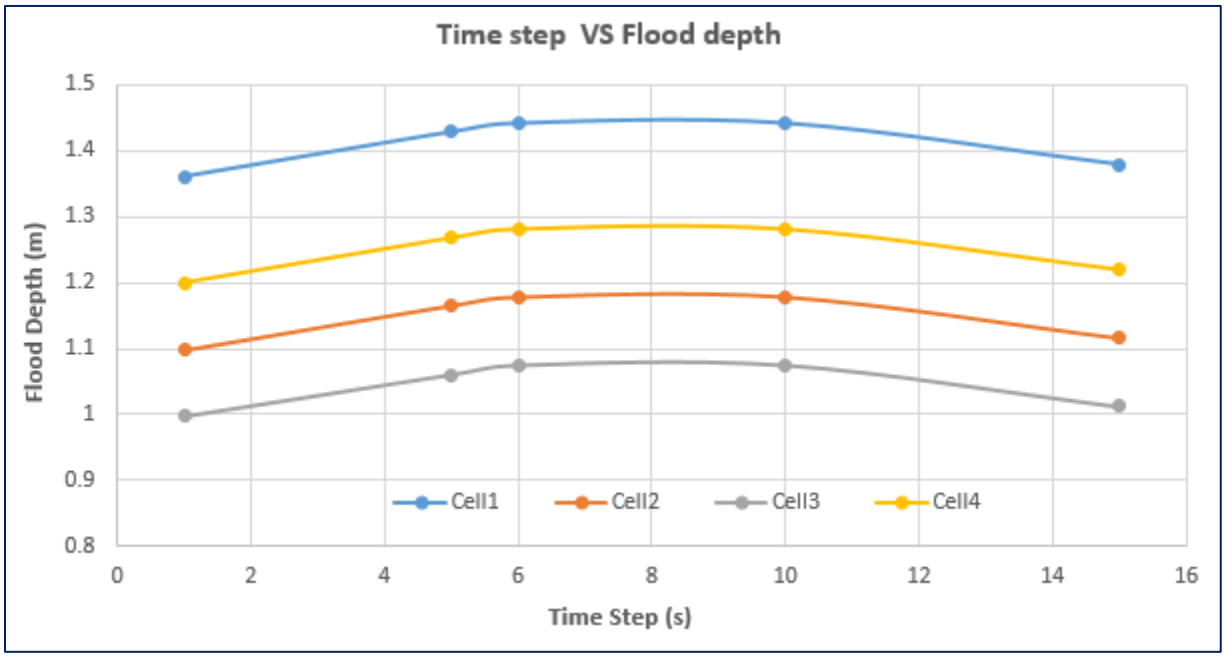


Figure 34**:** Graph shows relation between time step and flood depth at 4 (Cell 1-4) indicated cell in the floodplain

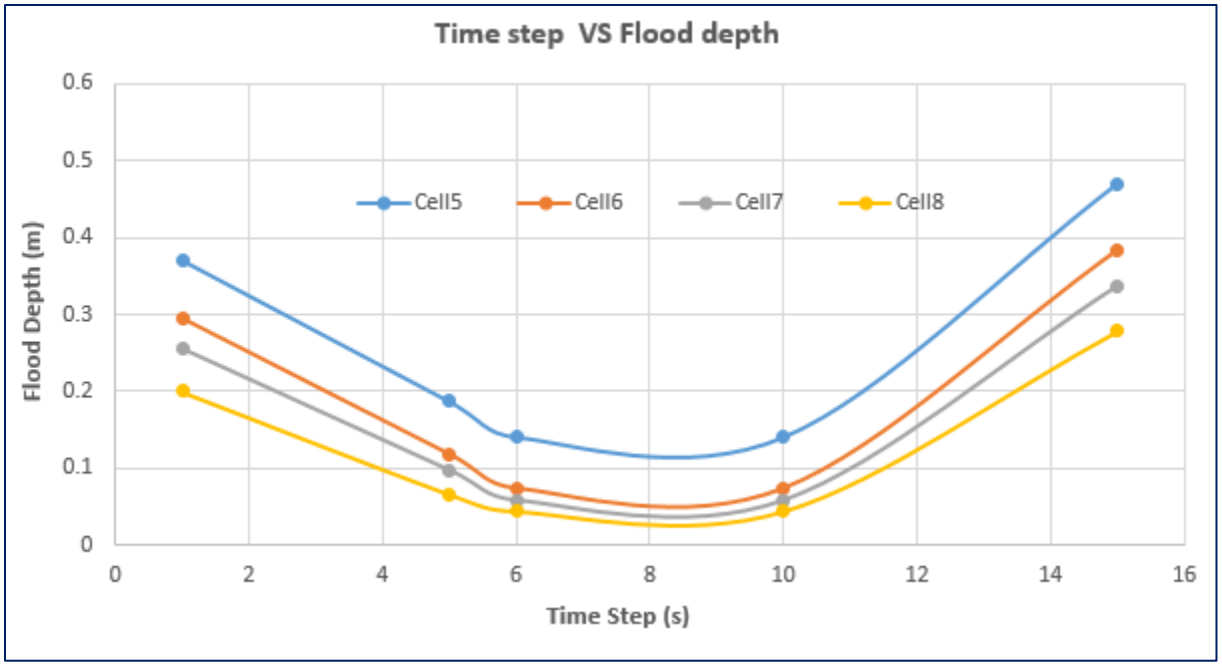


Figure 35: Change of maximum flood depth value (Cell 5-8) for different time steps in sub-reach K6

For the sub-reach K6, flood was calculated in some specific area for different cross-sections, following table shows comparison of flood depth.

Table 3: Maximum flood depth value in different cells for certain cross-sections in sub-reach K6

Cross-sections	**Cell1**	**Cell2**	**Cell3**	**Cell4**	**Cell5**	**Cell6**	**Cell7**	**Cell8**
	Flood Depth (m)							
10	1.49	1.23	1.12	1.33	0.21	0.25	0.28	0.24
20	1.39	1.12	1.02	1.23	0.48	0.40	0.35	0.29
50	1.39	1.11	1.02	1.23	0.79	0.72	0.06	0.68

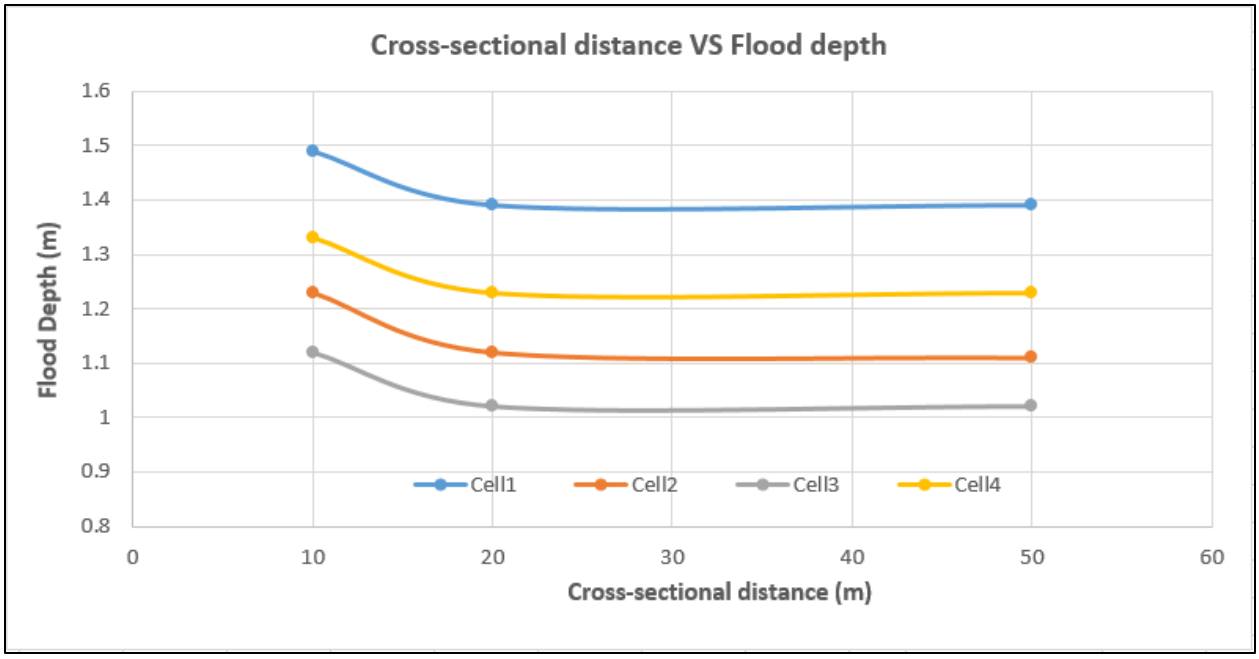


Figure 36: Change of maximum flood depth value (Cell 1-4) for different cross-sectional distances in sub-reach K6

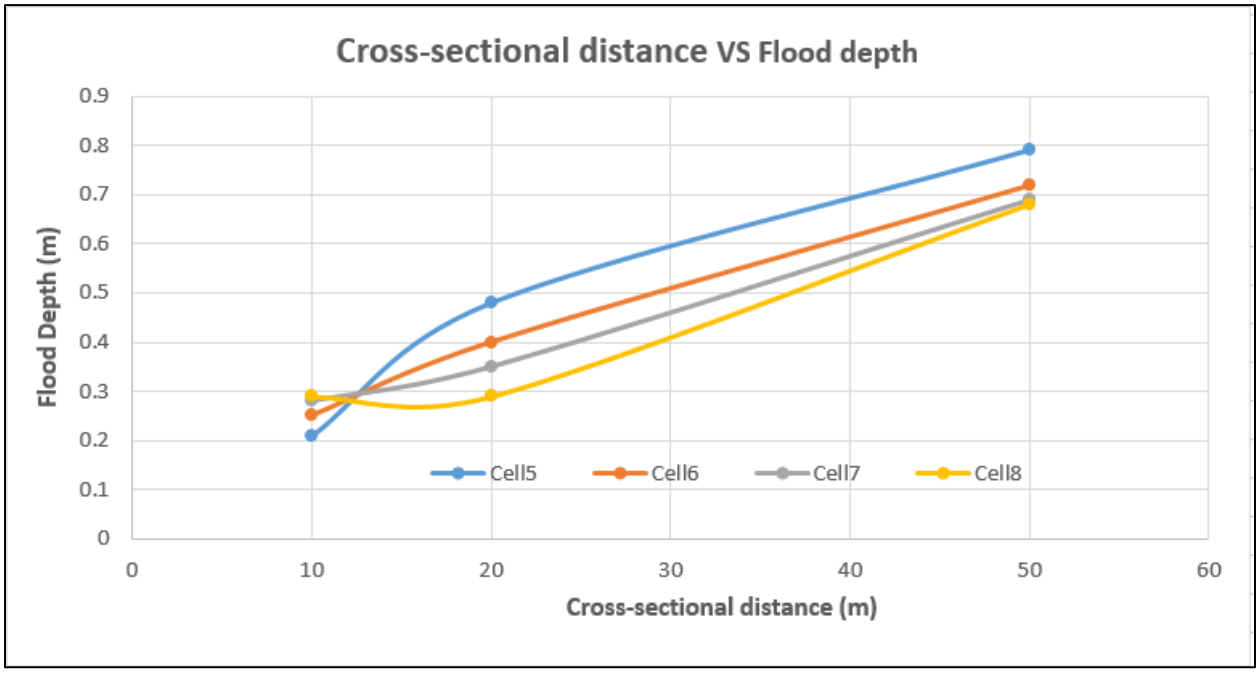


Figure 37: Change of maximum flood depth value (Cell 5-8) for different cross-sectional distances in sub-reach K6

For the sub-reach K6, flood was calculated in some specific area for different Maning's n value, following table shows comparison of flood depth.

Table 4: Maximum flood depth value in different cells for certain cross-sections in sub-reach K6 (varying Manning's n value)

Manning's n	Cell1	Cell2	Cell3	Cell4	Cell5	Cell6	Cell7	Cell8
	Flood Depth (m)							
30	1.02	0.76	0.65	0.85	0			
35	1.45	1.19	1.09	1.29	0.44	0.36	0.31	0.25
40	1.50	1.24	1.13	1.34	1.32	1.25	1.21	1.15

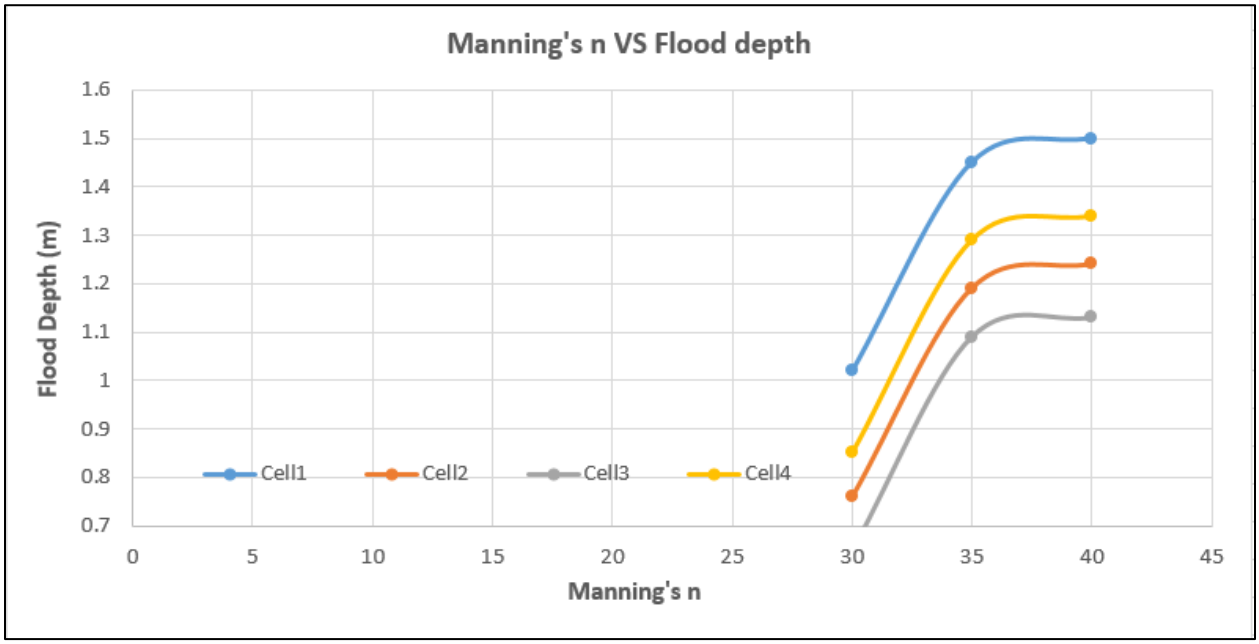


Figure 38: Change of maximum flood depth (Cell 1-4) value for different Manning's n in sub-reach K6

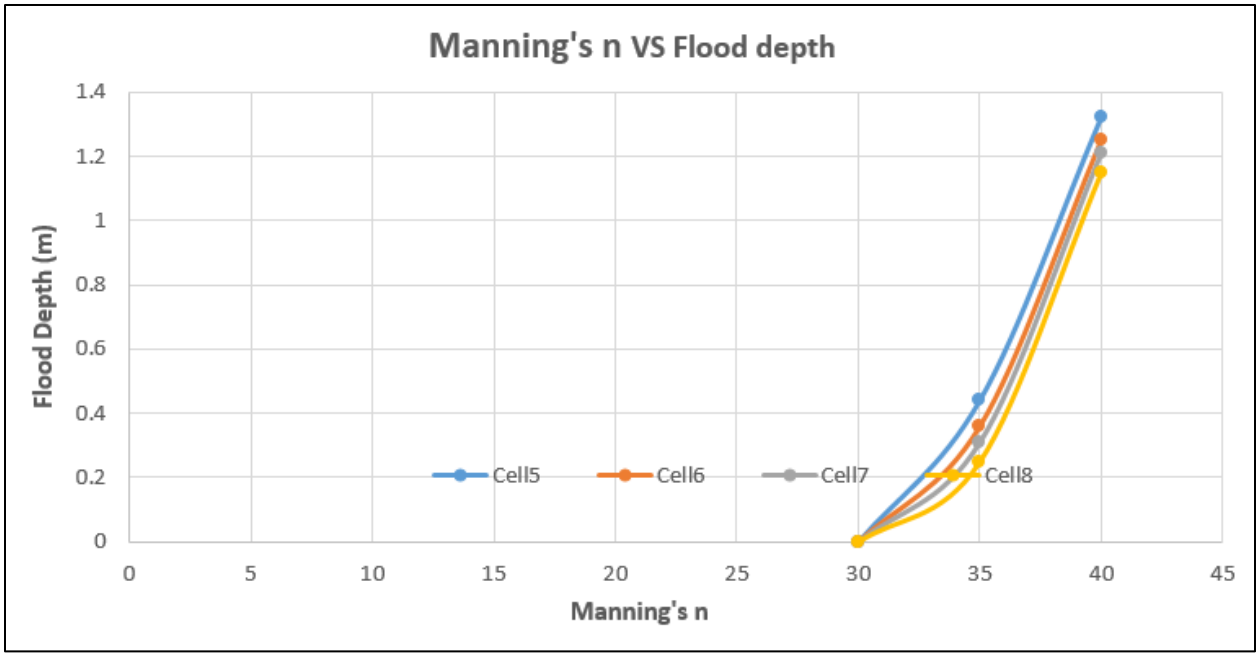


Figure 39: Change of maximum flood depth (Cell 5-8) value for different Manning's n in sub-reach K6

Flood Management Strategy

Dredging and dike construction were carried out in different flood prone locations along the Spree river to create a greater depth of water and to regulate water levels respectively with the main objective to prevent flood. At certain locations, river bed was dredged approximately 0.3m to 0.7m and dikes were constructed on the river bank with a height of nearly 1m to 3m. Table represents the detail of dredging and dike construction.

Table 5: Detail of dredging and dike construction along the river Spree

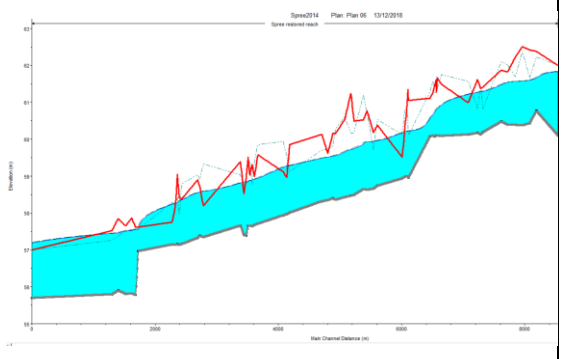
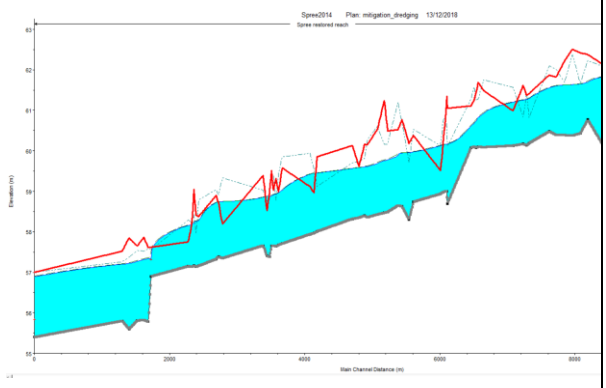
River Station	Reach Length (m)	Cross Section Number	Dredging (0.3m-0.7 m)	Dike (1m-3m)
1-3.4615	1456.772	1,2,3	0.7	3
7.0182-9	600	7,8,9	0.7	3
10.333-11.250	40	10,11	0.7	3
14.286-15.367	271.774	14,15	0.65	3

River Station	Reach Length (m)	Cross Section Number	Dredging (0.3m-0.7 m)	Dike (1m-3m)
16.667-18	67.05	16,17,18	0.65	2.5
23.854-25.2	141.149	24	0.6	2.5
26-26.396	215.706	26	0.35	2
35.818-36.143	37.589	36	0.35	2
37.415-39	279.575	38	0.35	2
42-42.086	35.179	42	0.3	1
47.705-49.667	322.777	48,49	0.3	1
50.25-51.4	155.6	50	0.3	1

Case 1:

Dredging:

Table 6: Comparison of bank inundation along the river profile between base scenario and after dredging scenario

Bank	Base Scenario	Scenario 1 (after Dredging)
LB		

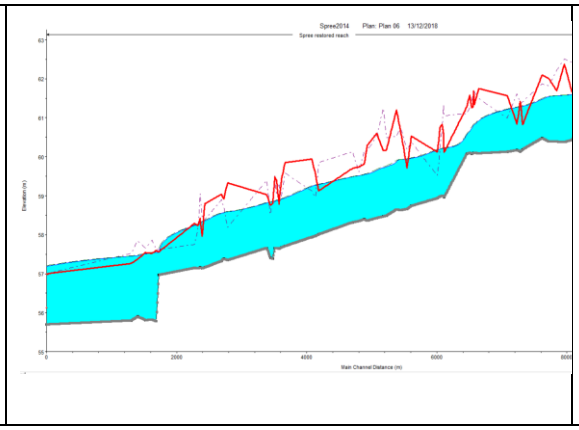
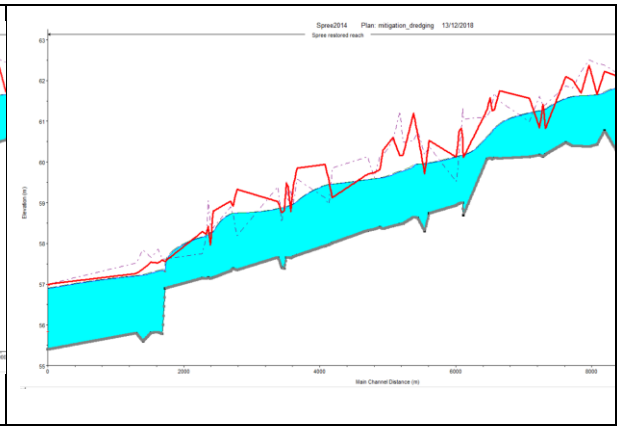
RB		

Table 7: Comparison of water stage between base scenario and after dredging scenario for different cross-sections

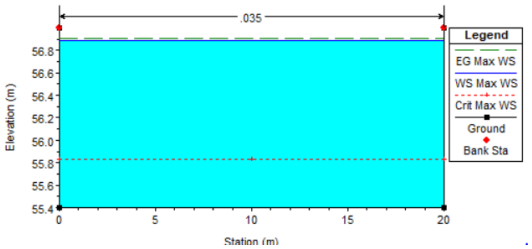
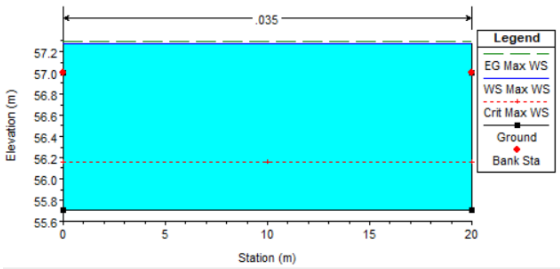
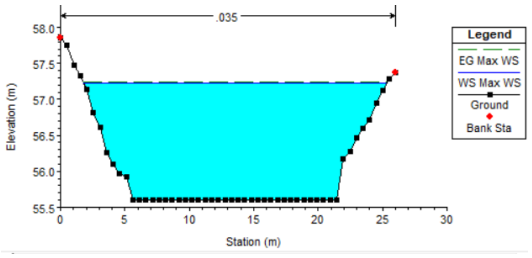
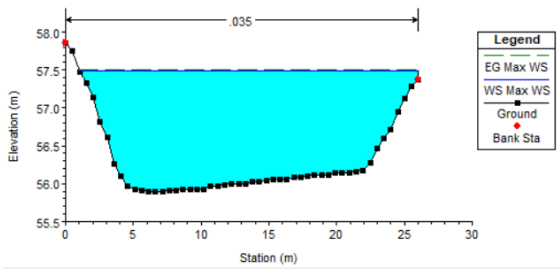
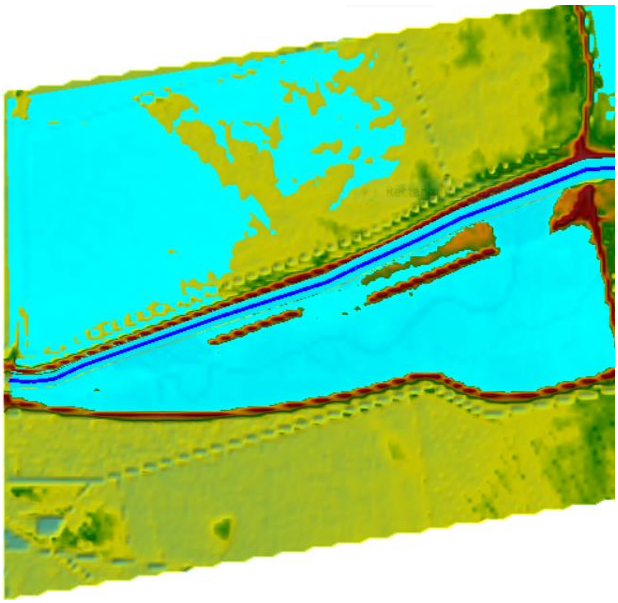
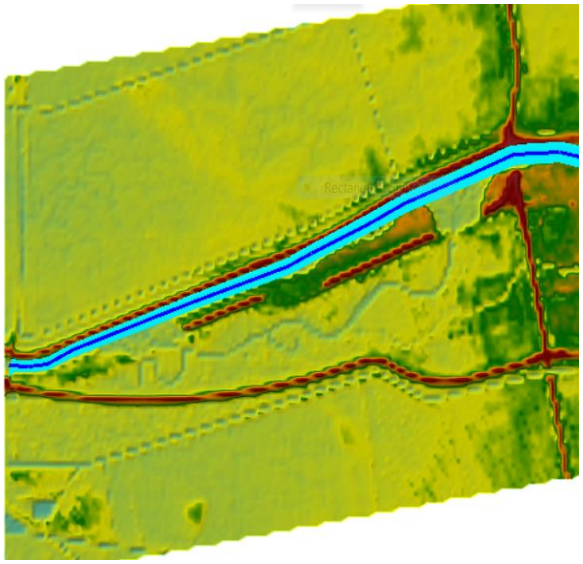
Cross-section no.	Scenario 1 (after Dredging)	Base Scenario
1		
3		

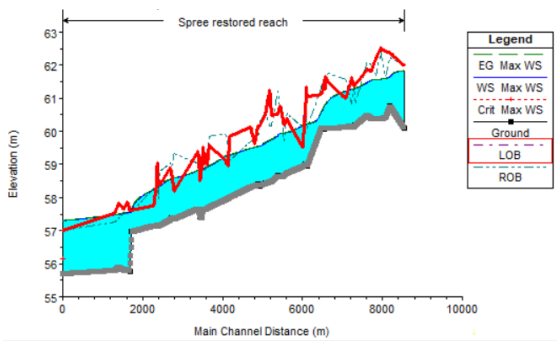
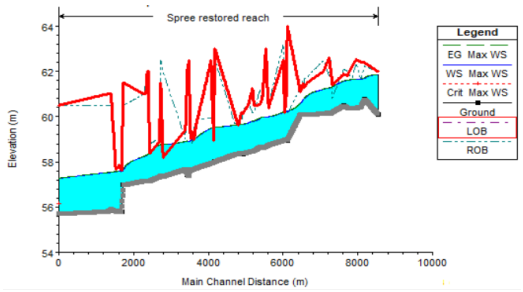
Table 8: Comparison of floodplain inundation between base scenario and after dredging scenario for sub-reach K6

Base Scenario	Scenario 1 (after Dredging)
	

Case 2:

Dike construction

Table 9: Comparison of bank inundation along the river profile between base scenario and after dike construction scenario

	Base Scenario	Scenario 2 (after Dike construction)
LB		

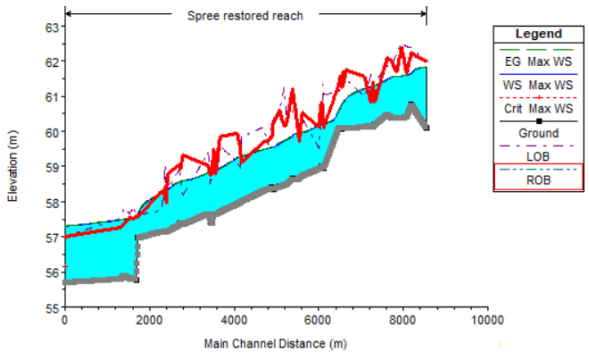
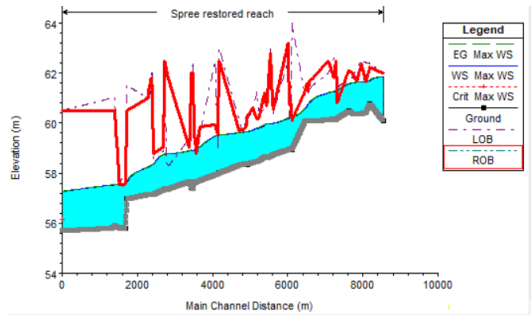
	Base Scenario	Scenario 2 (after Dike construction)
RB		

Table 10: Comparison of water stage between base scenario and after dike construction scenario for different cross-sections

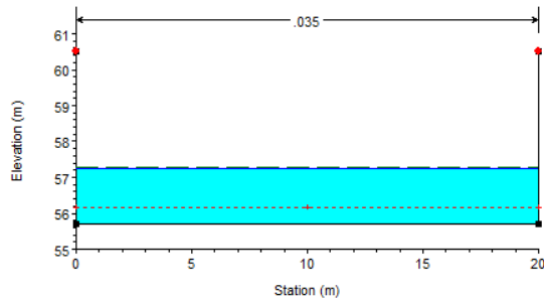
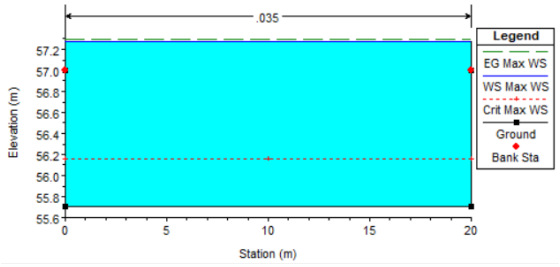
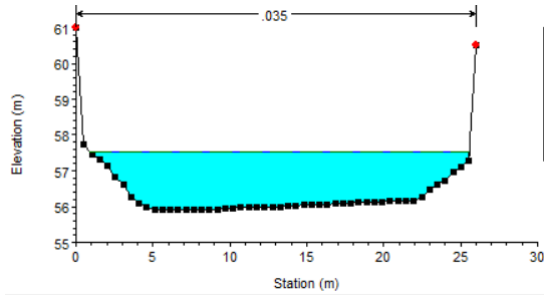
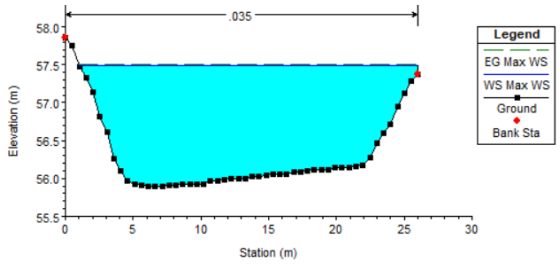
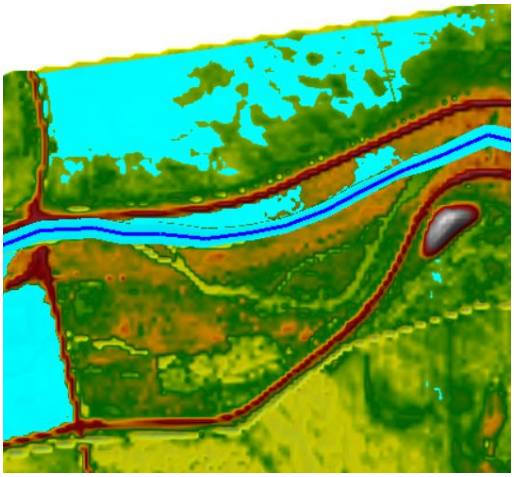
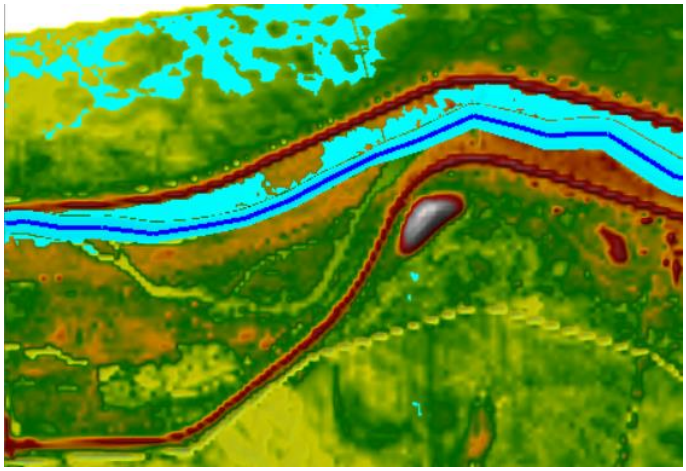
Cross -sec	Scenario 2 (after Dike construction)	Base Scenario
1		
3		

Table 11: Comparison of floodplain inundation between base scenario and after dike construction scenario for sub-reach K5

Base Scenario	Scenario 2 (after Dike construction)
	

Chapter 6: Discussion and Conclusion

The model was simulated for different time step, grid size and roughness coefficient and it was observed that the model was sensitive to these parameters. When the time step was changed keeping the same Manning's value and cross sectional distance, the overtopping of flow was prominent in the right bank of the river. This flood inundation increased with the increasing time steps. A curved pattern was observed which suggested that even though the model was stable for different time steps, more accuracy could be observed in the 6s to 10s range. This suggest that the model shows a sensitive behavior to time step. On the other hand, when different interpolated distance between cross section was taken into account, there was an insignificant change in the flood inundation pattern.

Also, the model showed a sensitive behavior to the roughness coefficient. The extent of flooding was different for different roughness coefficient which implies that the factors that increases the roughness in the floodplain is of vital importance in preventing flooding. Higher Manning's n (more roughness) led to more flooding than lower Manning's n (less roughness) which clearly suggests the importance of vegetation on the banks of the river to prevent flooding.

Different mitigation measures adopted in the model provided a change in the inundation pattern. As the results shows above, all the measures like change of vegetation patter (Manning's n), dredging and dyke inclusion in the model reduced the flood inundation significantly. This suggests that the use of such measures could be important to protect the surrounding areas which holds economical value due to its touristic appeal. However, it should also be considered that these flood prevention strategy is not the perfect way to keep the riverine area natural which is the essence of any renaturation projects. This can be achieved by creating an environment where the flora and fauna can survive and flourish. As seen in many different places, the ecological balance in the area can play a vital role in the stability of river and its floodplain. In some cases however, the presence of residential area could be more important than maintaining the ecological balance. Therefore, the measures should be taken considering the trade-off between the environmental impact due to human intervention and the natural losses.

Even though a fairly good result has been obtained from this model, there are still some areas which could be potentially be included in the further study. This model provided a good result to visualize the flood area. However, the model created a coupling of 1D and 2D. One of the approach

could be using a 2D model for the entire study area, both the channel and the floodplain. This way, the visualization could be clearer. Since, 2D analysis which uses 1D-2D coupling is a relatively new element in the HEC-RAS software, there is a room for further improvement in the approach.

In addition, this model has been carried out for fluvial flooding but not for pluvial flooding as the aspect of rainfall has not been included. A more realistic results could be obtained if the storm water could be incorporated in the model. A model that couples storm water modeling and 1D-2D coupled model could help to get a more realistic outcome.

When the underlying equations are taken into account, this model has not covered both the equations for 2D flow. The effect of fully dynamic equation for the river flow could create a different results than the diffusive wave model. The results of both the equations can be simulated and compared which could create a basis for application in another similar area.

Even with the restored land, there has been some flooding in the area. Adopting structural measures to mitigate it could neglect the integrity of renaturation. Therefore, further investigations and study could be carried out to analyze other new renaturation techniques and the structural flood mitigation measures and compare them to find the best solution which is feasible both environmentally and economically.

The parameters in the model showed a sensitive behavior to changes as indicated by different results for different scenarios in the preceding chapter. Initially, the model was unstable due to presence of lateral structures. Some of which had elevation lower than the connecting cell elevation of the 2D floodplain area. This created flow error in the model causing instability which suggests that the alignment of the lateral structure plays an important role in the model stability.

There are several ways to couple the 1D model 2D floodplain such as connecting a 2D Flow Area to a 1D River Reach with a Lateral Structure, directly connecting an upstream river reach to a downstream/upstream 2D Flow. Among them lateral structure method is more frequently used. However, connecting 1D river with 2D floodplain with lateral structure is sometimes cumbersome due to some inconsistency between terrain and lateral structures elevation. On the other hand, to predict the real extent of flooding, terrain data is needed beyond the original floodplain area. However, only a portion of floodplain is considered in the geometry file.

Design engineers and contractors alike entail to furnish projects that basically alleviate the impact of flooding. With other responsibilities and concerns, such as market situation, budget allotment and even climate change effect, it is important to stay one step ahead. Putting a scheme in place, that is built with efficacious and reliability will support to outline a concept for mitigation of flooding when remaining adaptable with environmental and regulatory requirements at the same time.

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