## Hydraulic Modelling of the Rambla de la Carrasquilla

By

## Group 322





Universidad Politécnica de Cartagena

## Hydraulic Modelling of the Rambla de la Carrasquilla

## **Multidisciplinary Project**

by



for the course CIE4061-09 Multidiscipinary Project at the Delft University of Technology

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## Preface

Before you lies the report '*Hydraulic Modelling of the Rambla de la Carrasquilla*'. This research was done to fulfill the course CIE4061 Multidisciplinary Project at Delft University of Technology and Universidad Politécnica de Cartagena. It is a collaboration between students from the master tracks Hydraulic Engineering and Geoscience & Remote Sensing.

Despite the fact we found ourselves deployed in the middle of the COVID-19 pandemic, we are thankful that during these challenging times we were still able to travel to Spain to carry out this research. During our mobility abroad, we not only gained more knowledge in hydraulics and hydraulic modelling, but also improved our skills in academic writing.

We would like to thank Dr. Antonio Rodríguez and Dr. José Maria Carillo Sánchez for welcoming us to Universidad Politécnica de Cartagena and guiding us through the project with their knowledge and kindness. Without their expertise in hydraulic modelling, weekly meetings with feedback and tips, and also your positivity, we would not have been able to conduct this research.

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Hopefully Delft University of Technology and Universidad Politécnica de Cartagena will continue working together and also provide other students with such a great opportunity as we had. We hope you enjoy reading.

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### Abstract

In the southeast of Spain heavy storms can occur in the late summer and early autumn, during which great amounts of rain pours down. During a storm that took place in September 2019, over 300 millimetres of rain fell in just five hours. Throughout the 2019 flooding event, the Rambla de la Carrasquilla has overflowed its banks and the adjacent agricultural lands flooded and large amounts of water accumulated in the streets of Los Nietos. Since the 2019 flood, improvements to the river system of the Rambla de la Carrasquilla have been made. For instance: a relatively small culvert was replaced with a bigger one in 2021, to increase the discharge capacity. Due to global warming, the probability of occurrence of the heavy storms will increase and thus the probability of flooding. However, the lack of historical data makes it hard to assess the influence of different storms and the replacement of the culvert on future floods. Numerical models can be used to make predictions of floodings and gain insight in storm impacts. The aim of this report is summarized by the question: How can the flood areas and peak water levels of the Rambla de la Carrasquilla and its catchment be obtained by simulating three different scenarios based on peak discharges using HEC-RAS models?. At first a spatial analysis of the river system was done using QGIS. Hereafter a 1D and 2D model were constructed with the help of a Digital Terrain Model (DTM). QGIS and RiverGIS were used to define the geometry of the system for the 1D model, where the 2D model geometry was constructed in HEC-RAS itself. The dimensions of the structures in the river were measured using photogrammetry, due to which dimensions could be obtained with an absolute error of around 2 cm. Fieldwork was carried out to estimate the surface roughness of the main channel and parts of the flood plains.

The results consist of different floodmaps that were made using 1D and 2D HEC-RAS models with different settings and boundary conditions. For the 1D model, one run was done using a single reach for the whole river and another including a bifurcation for the downstream part of the river. It can be concluded that the 1D model approach is not very suitable for our research as it is unable to accurately construct the flooded areas. The 2D model performed better and thus was used to further investigate the influence of different model parameters and inputs. Important take-away points are that the time step is of large influence for finding a stable solution, which is found around  $\Delta t = 1$  [second] in this report. The influence of the surface roughness of the river bed and floodplains was investigated by using Manning's n. These results showed clearly that a higher value for Manning's n leads to a larger flooded area. When investigating the influence of structures (two bridges, a pipeline and a culvert) on the flooded area, it is found that the structures do not have a great influence. The reason for this is that at critical points, the river already overflows its banks when the flow is not hindered by these structures. Adding the structures makes this only slightly worse. Furthermore, the 2021 culvert only showed a slight improvement in flooded area with respect to the 2019 culvert.

In order to further calibrate the HEC-RAS models, a comparison was made with floodmaps that were constructed by the Spanish government. Both studies show similar results, although the government used a Digital Terrain Model with a higher resolution, which can have a large impact on the outcome. However, this government study cannot be used for validation of the models in this research, as the results are based on hypothetical situations instead of empirical data as well.

For further research it is recommended to investigate more accurate hydrological boundary conditions, based on empirical rainfall data. For both boundary conditions a constant value was taken, but the behaviour over time could change the output of the results. Besides that, it could be interesting to study the exact effect of using a Digital Terrain Model with a higher resolution for the Rambla de la Carrasquilla. As this can have a direct influence on the size of the flooded area. Furthermore, the approaches used in this research can be used for investigating different flood mitigation solutions. These solutions, for example heightening of the river banks or widening of the bed, can be implemented in the 2D model before applying them in real life.

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## Introduction

#### **1.1. Problem Statement**

During the late summer and early autumn, heavy storms are common throughout Spain. The Gota Fría is a special case and an extra violent type of storm where clouds rise to great altitudes and downpours with large amounts of rain occur: in just six hours 400 millimeters of rain can fall. Consequently, flooding occurs which can cause damage to infrastructure and in the worst case even take human lives (Murcia Today, n.d.).



Figure 1.1: Region of Murcia (Wikipedia, 2021).





Figure 1.2: Flooding event Los Nietos 2019 (Murcia Today, 2019b).

Although the occurrence of these heavy weather events are not uncommon in the southeastern region of Spain (see Figure 1.1), the storm event that took place in September 2019 is said to be amongst the most dramatic ones of recent decades. This was due to the extent of the area it covered and

the amount of rain that fell: over 300 millimeters of rain was recorded within 5 hours, which led to significant quantities of accumulated water in the streets. Rivers burst their banks and irrigation and drainage channels over-flowed. This caused immense pressure on the local infrastructure. Help of well over 1200 emergency services staff was needed to rescue people and repair the damages, which took months of hard work. The estimated damages due to the 2019 flooding is estimated to be around 190 million euros, which is still on the conservative side (Murcia Today, n.d.).

Global warming predicts future peak flows and floods may become even more challenging, as the frequency of extreme weather events will increase. This means that the probability of occurrence of heat waves, heavy downpours and Gota Fría will become larger. A certainty is that the existing rivers and drainage systems in southeastern Spain are currently not able to handle these large amounts of precipitation, as seen in Figure 1.2. If the current systems are not adjusted to the changing conditions, the probability of flooding will increase as well. In order to properly adapt the system, prediction of the impact of a storm with a certain return period is needed. Unfortunately, very little data is available regarding the flooding event of September 2019. The estimated impact of this flood is based on photos and observations solely. Therefore, it is difficult to predict the influence of different storms. Numerical models and satellite images can be used to gain insight in the impact of varying storms. For example, the extent of a flooding event as a consequence of heavy rainfall can be predicted using these models.

#### 1.2. Problem Analysis

The river Rambla de la Carrasquilla is situated in the most southeastern part of Spain, see Figure 1.3. The upstream part of the river starts close to the town Atamaría. Hereafter it flows past Los Belones and into the sea close to Los Nietos, where a small delta was formed. The river is about 7 kilometres long. The coordinates place the beginning of its course at point 37°35'52.6"N 0°49'13.4"W and its end at location 37°38'57.6"N 0°46'25.6"W. The area surrounding the river mostly consists of agricultural lands. Near Atamaría a large golf complex was built in the 70s, along with several resorts. After construction, the amount of impermeable surface increased which led to an increase in runoff. In the downstream area agricultural lands are situated in the floodplain of the river. The function of the floodplain is lost here, as water cannot be stored without (economical) damage.



Figure 1.3: The river Rambla de la Carrasquilla, located between the towns Los Nietos, Los Belones and Atamaría.

The Rambla de la Carrasquilla is alternating between extremely dry and very wet. The catchment area is characterized by the water balance deficit which led to an area that is prone to erosion, especially during heavy weather events. During and after intense rainfall events the river is filled with runoff water from the municipalities of Cartagena and La Unión (Cutillas et al., 2015). The heavy flows erode the bed and an increased amount of sediment is transported downstream. The agricultural areas downstream are flooded and a large amount of (economical) damage occurs due to both the water

flow and sediment. These environmental circumstances are worsened by human interventions that were carried out in the river and its catchment area. De- and reforestation, soil degradation, various structures and dredging of the riverbed led to significant changes in the hydrology and geomorphology of the Rambla de la Carrasquilla, resulting in a dynamic bed form (Cutillas et al., 2015). In combination with the flash floods, the behaviour of the river is very unpredictable. This complicates the modelling of river flows and the prediction of floods. Furthermore, the river has a problem with overgrown vegetation. Weeds, bushes and trees litter the river bed adding to the roughness in the system. Heavy rainfall in a short amount of time can result in flushing of the vegetation in the river. The roughness of the vegetation decreases the flow velocity and increases the water level, which causes the river banks to overflow.

Currently, in the Rambla de la Carrasquilla one large highway bridge, one smaller bridge, a pipeline and two culverts are present. The two culverts provide a water drainage capacity for the roads crossing the river. During the 2019 flood, this water drainage capacity was exceeded. The increased amount of sediment supply during the flood led to the blockage of the culverts. Therefore, the water could not flow through and the river overflowed. Since the 2019 flood improvements have been made by upgrading one of the culverts. The old culvert (see Figure 1.4a) is replaced with a new and bigger one (see Figure 1.4b). The new drainage system is expected to handle more than 260 m<sup>3</sup>/s. The location of this renewed culvert near Los Belones can be seen in Figure 1.5. In the 2019 flood, the Rambla de la Carrasquilla overflowed its banks this location: the water moved downstream over the floodplains and the town of Los Nietos was flooded. Los Nietos was one of the worst affected areas in the region during this flood (Murcia Today, 2019a).



(a) Old culvert with relatively low discharge capacity.



(b) New culvert with increased discharge capacity.

Figure 1.4: In 2021 a culvert was replaced in order to increase it's discharge and decrease the risk of flooding.



Figure 1.5: Location replaced culvert with larger capacity.

#### 1.3. Objective

This report aims to simulate the flooding of the areas around the river, using different kind of peak flows as input. In order to do so, numerical models will be used to simulate the flow. These numerical models will be generated using the river modelling software HEC-RAS in combination with mapping software

QGIS. The input to these models will consist of peak discharges, the river bathymetry, a Digital Terrain Model (DTM) of the area and the structures (bridges and culverts) along the river. The data of the first three is open-sourced, but no information on the structures is publicly available. A common method to measure these large structures is laser scanning, however this is time consuming and costly. Therefore an alternative method called photogrammetry will be researched to model the structures. When the model is realized, it will also be compared to existing models produced by the Spanish government. These models were realized on a national level and require more detailed local studies (Sánchez & Lastra, 2011). The goal of this project is summarized in one main research question:

How can the flood areas and peak water levels of the Rambla de la Carrasquilla and its catchment be obtained by simulating three different scenarios based on peak discharges using HEC-RAS models?

To reach this goal, the project was subdivided into different parts through sub-questions:

- 1. What methods does HEC-RAS use for numerical computations?
- 2. Which parameters must be taken into account when designing one- and two-dimensional numerical models?
- 3. How can bridges and culverts reliably be obtained and implemented in HEC-RAS?
- 4. How does the flood model compare with previous models made by the government?
- 5. What is the effect of implementing culverts and bridges on the model outcomes?
- 6. How do the 1D and 2D models compare to each other in both approach and outcomes?

#### 1.4. Approach

The above questions will be answered using the HEC-RAS software to construct 1D and 2D hydraulic model. QGIS will be used for the spatial analysis of the river and the visualization of results produced by HEC-RAS. Field work will carried out, during which information about the river bed and floodplain roughness will be collected. Besides that, measurements of the present structures, like bridges, pipelines and culverts along the river will be taken using measurement tape and by creating 3D photogrammetry models. Photogrammetry is a process in which multiple photos from multiple angles are combined to generate a 3D model of an object. The photos will be processed using the softwares VisualFSM and Meshlab. When the 1D and 2D models are calibrated and their results are analysed, these results can be compared with previous results of the same region which are generated by the government.

#### 1.5. Reading Guide

To find an answer to the main research question, this report first elaborates in chapter 2 on the physical and numerical background of HEC-RAS for both a 1D and 2D model. In chapter 3 the most important parameters needed to set up the hydrological model are discussed, and some background information on the modelling of bridges and culverts is given. Also other aspects such as hydrological input, model calibration and comparison with a previous study can be found here. The fourth chapter presents and analyses the results of the most critical situations considered (see Appendix F for additional results). Discussion on several topics such as, limitations on the models and precision on the measurements are given in chapter 5. At last in chapter 6 conclusions and recommendations about the different models, regarding flood modelling of the River de la Carrasquilla are made.

 $\sum$ 

## **Theoretical Basis of HEC-RAS**

This chapter will explain more about the theoretical and numerical background of the HEC-RAS software. First, the difference between 1D and 2D modelling is discussed in section 2.1. In section 2.2 the theoretical background of steady 1D hydrodynamic calculations is explained, after which the same is done for unsteady 1D calculations in section 2.3. More details about the theory behind 2D modelling can be found in section 2.4. Finally, the numerical methods used by HEC-RAS are discussed in section 2.5.

#### 2.1. Difference between 1D and 2D modelling

HEC-RAS is a well known and accepted modelling program that can be used to perform 1D and 2D hydraulic calculations for a river or channel network (HEC-RAS Reference Manual, 2021). It is possible to carry out (un)steady flow simulations as well as sediment transport simulations and water quality analyses. Nowadays multiple options for river modelling are available. Whereas 1D modelling is most common to use, 2D modelling is becoming more and more popular as computers are getting more powerful and data retrieval becomes easier (Brue, 2021). A 1D model needs representative cross sections along the river or channel, while a 2D hydraulic model simply lays a mesh over the terrain data.

For both models the vertical z-direction is assumed to be constant, as can be seen in Figure 2.1. In a 1D model, the water level in the horizontal y-direction is also assumed to be constant. For a 2D model, the flow velocity and water levels can vary across the width of the channel. The St. Venant equations of Conservation of Mass and Conservation of Momentum are used by HEC-RAS to accomplish 1D and 2D unsteady flow computations. Logically, a 2D model solves the St. Venant equations along the x and y axes, whereas a 1D model solves the equations only in the x axis. In the following sections of this chapter further elaboration on the different hydrodynamic calculations can be found.



Figure 2.1: Schematization of the flow modelling for a 1D and 2D model.

#### 2.2. Steady 1D Hydrodynamic Calculations

The HEC-RAS software is able to do 1D water surface profile computations for steady gradually varied flow in rivers or channels. This concerns subcritical, supercritical and mixed flow regime water surface profiles. Basic profile calculations in HEC-RAS are carried out by solving Bernoulli's energy equation (2.1):

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e$$
(2.1)

Where:

- Z = elevation of the main channel inverts
- *Y* = depth of the water at cross sections
- V = average velocities
- a = velocity weighting coefficients
- g = gravitational acceleration
- $h_e$  = energy head loss

A graphical representation of Bernoulli's equation can be observed in Figure 2.2. The equation for energy head loss between two cross sections consists of friction losses and contraction/expansion losses, see Equation 2.2. The distance weighted reach length L is calculated using Equation 2.3. Combining these equations then gives the final water surface profiles.



Figure 2.2: Graphical representation of Bernoulli terms (Carrillo Sánchez, n.d.)

$$h_e = L * \overline{S_f} + C * \left| \frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g} \right|$$
(2.2)

Where:

L = discharge weighted reach length

 $\overline{S_f}$  = representative friction slope between two sections

 $\vec{C}$  = expansion or contraction loss coefficient

$$L = \frac{L_{lob}\overline{Q_{lob}} + L_{ch}\overline{Q_{ch}} + L_{rob}\overline{Q_{rob}}}{\overline{Q_{lob}} + \overline{Q_{ch}} + \overline{Q_{rob}}}$$
(2.3)

Where:

 $\frac{L_{lob}}{Q_{lob}}, \frac{L_{ch}}{Q_{ch}}, \frac{L_{rob}}{Q_{rob}}$  = cross section reach lengths of left overbank, main channel, right overbank  $\overline{Q_{lob}}, \overline{Q_{ch}}, \overline{Q_{rob}}$  = averages of flow between sections of left overbank, main channel, right overbank

#### 2.2.1. Subdivision of Cross Sections

In a river, the conveyance and velocity coefficient are not uniformly distributed over a cross section. However, the 1D modelling approach requires a uniform distribution of these parameters. Therefore, the cross sections must be subdivided into sections for which the flow has a uniform distribution of conveyance and flow velocity. HEC-RAS realizes this by subdividing the flow in a main channel and overbank section. The locations where the roughness values, or Manning's n, change are used as a basis for the subdivision, see Figure 2.3.



Figure 2.3: Cross sectional subdivision in HEC-RAS (Brunner, 2021)

#### 2.2.2. Calculating Individual Terms

Now that the outline for the steady 1D approach is considered, the following terms are still necessary to finish calculations: conveyance, velocity coefficients and contraction/expansion coefficients. HEC-RAS calculates the conveyance K (based on English units) for each section using Manning's equations 2.4 and 2.5.

$$Q = K \cdot S_f^{\frac{1}{2}} \tag{2.4}$$

$$K = \frac{1.486}{n} \cdot A \cdot R^{\frac{2}{3}}$$
(2.5)

Where:

K = conveyance

- n = Manning's roughness coefficient
- A =flow area
- R = hydraulic radius
- $\overline{S_f}$  = friction slope

After doing calculations for each particular section, HEC-RAS cumulatively sums all conveyances in the overbanks to obtain one value for the left overbank and one for the right overbank. The main channel is considered to have a single conveyance value. When the three different values for left overbank, main channel and right overbank are known, the total conveyance can be obtained by summing these values.

As the river cross section is subdivided into different segments, every part has a distinct kinetic energy head. HEC-RAS computes only a single water surface elevation for the entire cross section, for which it needs one mean kinetic energy head. The mean kinetic energy head can be computed by using the velocity head weighting coefficient  $\alpha$ . It is calculated by equating the mean kinetic energy head with the discharge weighted velocity head (Brunner, 2021). This leads to an expression for the velocity coefficient, see Equation 2.6.

$$\alpha = \frac{Q_1 V_1^2 + Q_2 V_2^2 + \dots + Q_N V_N^2}{0 \overline{V}^2}$$
(2.6)

Where:

 $\alpha$  = velocity head weighting coefficient of the cross section

- $Q_i$  = discharge of the subsections of the cross section
- $V_i$  = velocity of the subsections of the cross section
- Q = total discharge of the cross section
- $\bar{V}$  = average velocity of the cross section

HEC-RAS contains certain assumptions about the occurrence of contraction and expansion. For example, contraction is considered to occur when the downstream velocity head is greater than the upstream velocity head and vice versa for expansion. The degree of expansion or contraction can be characterized by the coefficient *C*. HEC-RAS calculates contraction and expansion losses by using Equation 2.7.

$$h_{ce} = C * \left| \frac{\alpha_1 V_1^2}{2g} - \frac{\alpha_2 V_2^2}{2g} \right|$$
(2.7)

Where:

- $h_{ce}$  = contraction or expansion losses
- C = contraction or expansion coefficient

V = velocity

g = gravitational constant

#### 2.2.3. Computation procedure

All unknowns in the Bernoulli equation are given in the previous sections, which means that it is now possible to run the model. HEC-RAS will calculate the water surface profile by an iterative process. A point of interest here is the determination of critical depth, especially if the water surface passes through the critical depth. In that case, the flow is located in the transition from subcritical to supercritical, which means the flow is considered to be rapidly varying. As mentioned earlier, Bernoulli's equation is only valid in gradually varying flow. In the special case of rapidly varying flow for a steady 1D model, HEC-RAS applies the momentum equation (HEC-RAS Reference Manual, 2021). However, for this report the flow is always considered to be gradually varying so there will be no elaboration on this specific method.

#### 2.3. Unsteady 1D Hydrodynamic Calculations

There are several cases in which a steady 1D model approximation is not valid, which means an unsteady 1D model must be considered. Steady flow calculations are used for kinematic waves, i.e. the discharge does not vary in space and the discharge hydrograph is unique. For dynamic flood waves the discharge will vary in space. Peak attenuation occurs as the wave moves downstream which means the discharge hydrograph is not unique. For these situations the unsteady 1D modelling approach should be considered. The unsteady 1D hydrodynamic calculations in HEC-RAS are based on the principle of conservation of mass (continuity) and the principle of conservation of momentum, which can be observed in Equation 2.8 and 2.9 respectively. When the primary direction of flow has the same orientation as the channel, it is possible to approximate a 2D flow field by a 1D representation. Finite difference equations derived by Barkau are the basis for this unsteady flow approximation in HEC-RAS. More elaboration on these equations can be found in Brunner (2021).

$$\frac{\delta A}{\delta t} + \frac{\delta S}{\delta t} + \frac{\delta Q}{\delta x} = q_l \tag{2.8}$$

Where:

x = distance along the channel

t = time

- Q = flow
- A = cross sectional area
- S = storage from non-conveying portions of cross sections
- $q_l$  = lateral inflow per unit distance

$$\frac{\delta Q}{\delta t} + \frac{\delta (VQ)}{\delta x} + gA(\frac{\delta z_s}{\delta x} + S_f) = M$$
(2.9)

Where:

Q

V g

х  $S_{f}$ М

e:  

$$Q = flow$$
  
 $t = time$   
 $V = velocity$   
 $g = gravitational acceleration$   
 $z_s = water surface elevation$   
 $x = distance along the channel$   
 $S_f = friction slope$   
 $M = momentum flux per unit distance exchanged between channel and flood plain$ 

The unsteady 1D flow equations are solved by numerical methods. For example, HEC-RAS uses the Four-Point Implicit Scheme or Box Scheme. It should be noted that for these numerical methods to be solved, the left and right overbanks are converted into a single flow compartment: the floodplain. This is different from the steady approach, where the left and right overbank are considered seperately. For the unsteady approach, the hydraulic properties of the floodplain are computed by combining the left and right overbank elevation, area, conveyance and storage into a single set. The reach length of the floodplain is found by taking the average of the left and right overbank reach lengths, see Equation 2.10.

$$L_{floodplain} = \frac{L_{leftoverbank} + L_{rightoverbank}}{2}$$
(2.10)

The floodplain reach length is used to compute the terms of the continuity and momentum equation. For the unsteady flow equations to be solved, they have to be reduced to a discrete form: the finite difference approximation. This means that there will be some built-in error, which is inconsequential for river models if calculations are carried out correctly (Goodell, 2010). More details about the computational methods can be found in Brunner (2021).

#### 2.4. Unsteady 2D Hydrodynamic Calculations

In some cases 1D modelling is difficult or even impossible. For example when the boundary between channels and overbanks are unclear or flow directions are not evident. 2D modelling enables the possibility to model locations where flow does not have uni-directional flow patterns, e.g. around structures like bridges or buildings. Or when the flow direction changes significantly in different stages, for instance due to river bends (Brue, 2021). In these examples a 2D model could be more appropriate.

The 2D hydrodynamic calculations in HEC-RAS are based on a simplification of the Navier-Stokes equations, which describe fluid motions in three dimensions (HEC-RAS Reference Manual, 2021). The simplified set of equations used by HEC-RAS for the 2D model are the Shallow Water Equations (SWE) 2.11 and 2.12.

$$\frac{\delta u}{\delta t} + u\frac{\delta u}{\delta x} + v\frac{\delta u}{\delta y} - f_c v = -g\frac{\delta z_s}{\delta x} + \frac{1}{h}\frac{\delta}{\delta x}(v_{t,xx}h\frac{\delta u}{\delta x}) + \frac{1}{h}\frac{\delta}{\delta y}(v_{t,yy}h\frac{\delta u}{\delta y}) - \frac{\tau_{b,x}}{\rho R} + \frac{\tau_{s,x}}{\rho h}$$
(2.11)

$$\frac{\delta v}{\delta t} + u\frac{\delta v}{\delta x} + v\frac{\delta v}{\delta y} - f_c u = -g\frac{\delta z_s}{\delta y} + \frac{1}{h}\frac{\delta}{\delta x}(v_{t,xx}h\frac{\delta v}{\delta x}) + \frac{1}{h}\frac{\delta}{\delta y}(v_{t,yy}h\frac{\delta v}{\delta y}) - \frac{\tau_{b,y}}{\rho R} + \frac{\tau_{s,y}}{\rho h}$$
(2.12)

Where:

*u* and *v* = velocities in the Cartesian directions [L/T] *g* = gravitational acceleration  $[L/T^2]$  *z<sub>s</sub>* = water surface elevation [L] *v<sub>t,xx</sub>* and *v<sub>t,yy</sub>* = horizontal eddy viscosity coefficients in *x* and *y* directions  $[L^2/T]$  *τ<sub>b,x</sub>* and *τ<sub>b,y</sub>* = bottom shear stresses in the *x* and *y* directions  $[M/L/T^2]$  *τ<sub>s,x</sub>* and *τ<sub>s,y</sub>* = surface wind stress directions  $[M/L/T^2]$  *R* = hydraulic radius [L] *h* = water depth [L]*f<sub>c</sub>* = Coriolis parameter

The left-hand side of the SWE represents the acceleration terms. The right-hand side contains the internal or external forces acting on the water. The following assumptions are made to allow the use of the SWE:

- Incompressible flow
- Uniform density
- Hydrostatic pressure
- · Reynolds averaging (to approximate turbulent motion with eddy viscosity)
- · Vertical length scale much smaller than horizontal length scales
- Small vertical velocity

In the 2D modelling approach the Diffusion Wave Equations (DWE), see 2.13 and 2.14, are used as a simplified approximation of the momentum equation (HEC-RAS Reference Manual, 2021).

$$\frac{\delta h}{\delta t} = \nabla * \left(\beta \nabla z_s\right) + q \tag{2.13}$$

$$\beta = \frac{R^{\frac{2}{3}}h}{n|\nabla z_s|^{\frac{1}{2}}}$$
(2.14)

Where:

h = water depth [L] t = time [T]  $z_s$  = water surface elevation [L] q = source/sink flux term R = hydraulic radius [L]

The unsteady 2D hydrodynamic calculations performed by numerical methods for which the domain must be subdivided into non-overlapping polygons to form a grid. The accuracy and stability of the numerical solution depend greatly on the geometrical characteristics, orientation, and size of the grid elements, so the grid choice is very important. In addition, a dual grid is necessary because of the second order derivative terms. The dual grid defines a correspondence between regular grid cells and dual nodes, and dual cells and regular grid nodes. The dual grid spans the entire domain, see Figure 2.4a.

Usually, the topographic data is too dense to directly use as grid for the numerical model. In order to combine the dense information of the topographic features and the much coarser computational grid, a bathymetric subgrid can be defined as can be seen in Figure 2.4b. Additional information such as hydraulic radius, volume and cross sectional area can be pre-computed from the fine bathymetry, so that even though the high resolution details are lost, the numerical method can account for it through mass conservation. For hydraulic modelling this method is often appropriate since the free water surface is much smoother than the bathymetry (HEC-RAS Reference Manual, 2021).

After defining a grid, HEC-RAS will be able to apply numerical methods to solve the SWE and DWE. These numerical methods will be elaborated upon in section 2.5.





(a) Regular grid & nodes (blue) and dual grid & nodes (red) (HEC-RAS Reference Manual, 2021)

(b) Fine (sub) grid is represented by the Cartesian grid in gray and the computational grid in blue (HEC-RAS Reference Manual. 2021).

Figure 2.4: Grid used in computation of 2D unsteady flow in HEC-RAS.

#### 2.5. Numerical Background HEC-RAS

To obtain output from the 2D unsteady flow model, either the Diffusion Wave Equations (DWE) or the Shallow Water Equations (SWE) must be solved. For the SWE, two solvers are available in HEC-RAS: the Eulerian-Lagrangian Method (ELM), and the Eularian Method (EM). The solvers are similar, but the ELM is more appropriate for studying sediment transport (which will not be considered in this report), and is more computationally expensive (Saidi et al., 2014). For that reason, only the EM will be considered. Both the DWE and EM-SWE solver are based on finite difference and finite volume methods on an unstructured polygonal mesh with subgrid bathymetry (HEC-RAS Reference Manual, 2021).

USACE (2016) recommends to first create a model based on the DWE, as it is a simplified approximation of the SWE. When the model is in good working order, it is recommended to make a second model based on the SWE. If there are significant differences the SWE solution should be assumed to be more accurate. Note that the exact derivations of all formulas fall outside the scope of this research. Therefore only the concepts and general strategy of the numerical model will be discussed here.

Both solvers use current velocities normal to the faces. These face tangential velocities are reconstructed using a double-C stencil, a graphical representation of the finite difference formulas, which can be seen in Figure 2.5. Using a least-squares approach, the tangential velocity is computed, since the face normal velocity is known. The final tangential velocity is the arithmetic average of the left and right cell tangential velocities (HEC-RAS Reference Manual, 2021).



Figure 2.5: Double-C stencil used for computing the face tangential velocity, where L, R,  $u_T$ ,  $u_N$  and f correspond to the left and right grid cells, the tangential and normal velocities, and the face respectively (HEC-RAS Reference Manual, 2021).

Both solvers also make use of a (discretized) grid size and the same boundary conditions. The grid cells in HEC-RAS can have up to eight sides, each representing detailed cross sections which get processed into elevation versus area, wetted perimeter, and roughness relationships. This way, larger cell sizes can be used while accurately representing the underlying terrain. Larger cells are computationally more efficient, but might not capture rapid changes in water surface elevation or velocity as these are computed for the cell centers. The flow boundary conditions must also be discretized and can either be given by the water surface elevation, the normal depth (based on friction slope) or the flow (based on local conveyance).

#### **Diffusion Wave Equations solver**

Since the Diffusion Wave Equation (DWE) solver is a discrete scheme, the equations must be discretized. To do this, first the continuity and momentum equations are discretized, and from these, the discrete DWE are obtained.

To discretize the continuity equation, a time step needs to be chosen. Since face areas are treated explicitly, the stability of the solver increases, but the accuracy for larger time steps reduces (HEC-RAS Reference Manual, 2021). However, the DWE solver can still handle larger time steps than the SWE solver, while getting numerically stable and accurate results (USACE, 2016).

The discrete DWE can be obtained by substituting the Diffusion Wave Approximation of the momentum equation into the continuity equation. With this, the final system of equations can be written in a compact vector form (HEC-RAS Reference Manual, 2021):

$$\Omega(\mathbf{Z}) + \Psi Z = \mathbf{b} \tag{2.15}$$

Where:

 $\Omega$  = the vector of all cell volumes

**Z** = the vector of all cell water surface elevations at time n + 1

 $\Psi$  = the coefficient matrix of the system

**b** is the right-hand-side vector

A Newton-like technique can be applied to solve the matrix equation, resulting in the iterative formula (HEC-RAS Reference Manual, 2021)

$$Z^{m+1} = Z^m - \left[\Psi + A^m\right]^{-1} \left(Q^m + \Psi Z^m - b\right)$$
(2.16)

Where:

m = the iteration index

 $\mathbf{A}(\mathbf{Z})$  = the Jacobian of  $\Omega$  with respect to  $\mathbf{Z}$ 

The  $\theta$ -method can be used to solve this set of equations. It is a weighted average of the forward and backward Euler scheme and is unconditionally stable for  $\frac{1}{2} \le \theta \le 1$ . For  $\theta = \frac{1}{2}$ , this equals the Crank-Nicolson scheme, and for  $\theta = 1$  it corresponds to the backward Euler scheme (Zijlema, 2021). Then, using the scheme, the system of equations 2.16 is solved iteratively using Newton-like iterations with the given boundary conditions to obtain a candidate solution for the water surface elevation at the next time step  $(z^{n+1})$ . If the difference between the current iteration  $(z^m)$  and the next one  $(z^{m+1})$  is sufficiently small, the candidate solution is accepted. If it is larger than the given tolerance and the maximum number of iterations has not been reached, a new candidate solution must be computed. The computed  $z^{m+1}$  is then accepted as  $z^{n+1}$  and the velocities  $\mathbf{V}^{n+1}$  are calculated with the discrete version of the momentum equation. Finally, n is incremented and the process is repeated until the final time step is reached (HEC-RAS Reference Manual, 2021).The scheme used is second order accurate in space on a regular Cartesian grid and first order accurate on distorted grids. In the time domain the scheme is first order accurate for the backward Euler scheme ( $\theta = 1$ ) and second order accurate for the Crank-Nicolson scheme ( $\theta = \frac{1}{2}$ ) (HEC-RAS Reference Manual, 2021).

#### Shallow Wave Equations solver

Similar to the DWE solver, the first step of the EM-SWE solver is to do a discretization. In this case, it concerns a momentum-conservative discretization of the acceleration terms. This means that local conservation of momentum is assumed about a control volume centered on a cell face, as seen in Figure 2.6 (HEC-RAS Reference Manual, 2021). Quantities for the left cell are denoted with L, and quantities for the right cell are indicated with R. The weights of both are calculated in Equation 2.17 and can be used to calculate the water depths as shown in Equation 2.18. The acceleration terms are computed before the start of the outer iteration, as this is an explicit method.



Figure 2.6: Advection variables Eulerian Method ( $\theta = \frac{1}{2}$ ) (HEC-RAS Reference Manual, 2021).

$$\alpha_f^L = \frac{\Delta x_f^L}{\Delta x_f^L + \Delta x_f^R}, \quad \alpha_f^R = 1 - \alpha_f^L$$
(2.17)

$$\bar{h}_f = \alpha_f^L h_L + \alpha_f^R h_R \tag{2.18}$$

When a semi-discrete equation of the Eulerian momentum equation is obtained, it can be rewritten to an expression for the velocity at n + 1. This equation should then be substituted into the discrete continuity equation. Now that a system of equations is obtained, it can be solved using identical Newton-iterations as explained in 2.5.

# 3

## Developing a Hydraulic Model in HEC-RAS

This chapter lies the foundation of the hydraulic models by answering three of the research questions: 'Which parameters must be taken into account when designing one- and two-dimensional numerical models?', 'How can bridges and culverts reliably be obtained and implemented in HEC-RAS?' and 'How does the flood model compare with previous models made by the government?'. To do so, first the design of both 1D and 2D HEC-RAS models is explained in section 3.1. Hereafter, section 3.2 explains how the bridges, pipeline and culvert are modelled using photogrammetry. The determination of the surface roughness in the Rambla de la Carrasquilla is discussed in section 3.3 and the hydrological input data is given in section 3.4. Finally, a comparison with the government model is made in section 3.5.

#### 3.1. Design HEC-RAS model

This section elaborates on the design of a HEC-RAS model. The necessary parameters are discussed and the model calibration is explained.

#### 3.1.1. Creating a Spatial River Model with QGIS and RiverGIS

In order to create a hydraulic model in HEC-RAS, the program needs spatial (river) model of the river. This model can be created in *QGIS*, which is an open source Geographic Information System (QGIS Development Team, n.d.). This model is needed for HEC-RAS to recognize where and how calculations should be carried out. It consists of the main channel of the river, which is characterized by the stream centerline and bound by bank lines. Additionally, the flow paths give an indication of the flow direction in the left floodplain, main channel and right floodplain. Based on the previously mentioned parameters, cross sections will be drawn for the 1D model. These cross sections are important, as HEC-RAS computes hydrodynamic calculations for solely the area that is covered by the cross sections. More details about these features can be found in subsection 3.1.2.

In QGIS, the river stream centerline, bank lines and flow paths are drawn based on a combination of the digital terrain model and an orthophoto of the area in which the Rambla de la Carrasquilla is situated. This river outline can then be processed by *RiverGIS*. RiverGIS is a plug-in that can be installed in QGIS, which uses spatial data and converts this to HEC-RAS flow model geometry. RiverGIS can compute the flow model geometry once connected to the *PostgreSQL* database, which must contain the spatial database extender *PostGIS* (RiverGIS, 2015).

After the database connection is set, RiverGIS needs a schema for every new model. The PostgreSQL database can be used by several models and the schema makes sure that each model has its own location to store data. When a schema for the new model is created, the necessary flow data can be put into RiverGIS. This concerns river networks, flow paths, bank lines, cross sections, land-use data and hydraulic structures like bridges, culverts and pipelines. The input (from QGIS) concerns basic

data, without any quantification. The river network is built by RiverGIS, which means that the program computes the topology and reach lengths. Moreover, RiverGIS calculates the position of cross sections, bank stations and computes point elevations from the digital terrain model and blocked obstructions layer. After computing all necessary information, it is possible to extract a 'RAS GIS Import File' which can then be used to import the geometry data in HEC-RAS.

#### 3.1.2. 1D Model Parameters

The geometry of the basic model set-up in QGIS and RiverGIS serves as a crucial foundation for the hydrodynamic calculations in HEC-RAS. This subsection will go into the how and why.

#### Bank station placement

Bank stations in HEC-RAS separate the river channel into three different conveyance zones: the left floodplain, the main channel and the right floodplain. This enables appropriate use of Manning's equation for the determination of energy loss (USACE, 2016). The purpose of bank stations is to capture the change in conveyance between the main channel and the floodplains. The main channel width must not change significantly from one cross-section to another (Goodell, 2017). It is important that the placement of bank stations is done according to location of change in conveyance. For example, vegetation changes along the width of the river can be used to determine where the separation between main channel and left or right floodplain should be.

The definition of a floodplain is: "an area of land adjacent to a river which stretches from the banks of its channel to the base of the enclosing valley walls, and which experiences flooding during periods of high discharge" (Goudie et al., 2004). In the case of the Rambla de la Carrasquilla, a flooding extends far beyond the theoretical floodplains of the river. For example, the whole town of Los Nietos was affected in the 2019 flood, even though the town is not a part of the theoretical definition of a flood plain. There is a higher area in between the river and the town (enclosing valley walls), which means Los Nietos is excluded from the floodplain. The flooding areas in a 1D HEC-RAS model can only be computed over the area that is spanned by the cross sections. In order to predict floodings in Los Nietos, the cross sections have to span across the town. Therefore, in this case the definition of respectively the *left floodplain* and the *right floodplain* refer to the areas spanned by the cross sections on the left and right side of the main channel. The *main channel* is bound by the banks of the Rambla de la Carrasquilla, which can be recognized by the bank stations.

#### **Cross sections**

When building a 1D model, it is not necessary to know the bathymetry at every point of the river. The bathymetry at any location can be represented by interpolation between adjacent cross sections. In determining the most accurate representation of the river, cross sections should be placed at locations where notable changes are observed. For example: changes in discharge, channel slope, shape, flow direction or roughness (HEC-RAS Reference Manual, 2021). Cross sections at these locations should have a close spacing, as too little information (i.e. more interpolation) will lead to inaccuracies in modelling the flow.

The cross sections that are placed along the river reach should have a width extending the river stream and its floodplains. In addition, the cross sections need to be perpendicular to the flow lines. This could mean that occasionally, from a top view perspective, the layout of the cross sections can be oddly shaped and be broken up, as seen in Figure 3.1. It is important that the cross sections accurately portray the streams alignment (HEC-RAS Reference Manual, 2021). An estimated sketch of the streamlines in the main channel and floodplains helps to draw the cross sections perpendicular to the flow. The flow path in the main channel is perceived as the centre of mass. The final cross sections provide a good basis for calculations of the 1D model. It should be noted that any 1D model will not be able to compute anything outside of the area that is covered by cross sections.

#### Bridges

Bridges affect the hydraulics of river flow in a multitude of ways, as the flow contracts and expands through control structures like bridges. A bridge is composed of piers, abutments and a bridge deck, all of which exert influence on the river flow. Therefore, all of these elements should be considered in the 1D and 2D models.

The implementation of bridges in a 1D model is closely related to the placement of cross sections. It



Figure 3.1: Top view of cross sectional layout (HEC-RAS Reference Manual, 2021).

starts with determining the locations with energy loss due to the bridge, as seen in Figure 3.2. Here four parameters are crucial for the determination of the different cross sectional reaches: the expansion reach length,  $L_e$ ; the contraction length,  $L_c$ ; the expansion coefficient,  $C_e$ ; and the contraction coefficient,  $C_c$  (Brunner and Hunt, 1995). A general rule of thumb is that, in top view, the expansion reach has a slope of 2:1 and the contraction reach a slope of 1:1.



Figure 3.2: Recirculation areas portraying ineffective flow in a modelled bridge (C.M. Carillo Sánchez, personal communication, November 18, 2021).

Cross section 4 is located upstream of the modelled bridge, right before the flow enters the contraction zone, see Figure 3.2. The temporal change of momentum at any location has to be neglected here. Generally, the contraction of the flow occurs over a shorter distance than the expansion. This contraction of the flow occurs in between cross section 4 and 3. Cross section 3 and 2 are located just upstream and downstream of the bridge, respectively. These flow sections utilize selective active flow, making use of the ineffective flow areas option in HEC-RAS. In other words, not all portions of the cross section contribute to the effective area of the section, which forces the water to flow through the bridge opening. The location of ineffective flow areas should be determined for the entire reach between cross section 4 and 1. Lastly, cross section 1 is located at the downstream reach where the structure no longer influences the flow. The reach over which the bridge area still exerts influence, better known as the expansion reach, varies based on the shape of bridge, flow magnitude and the flow velocity (HEC-RAS Reference Manual, 2021). In addition to the four cross sections described above, more cross sections should be added up- and downstream of the bridge for computational modelling of the water surface profile. This way, it will be ensured that user-defined boundary conditions will not affect the outcome of the results through the bridge.

#### **Parallel bridges**

In the Rambla de la Carrasquilla two parallel bridges, which means they are located relatively close to each other, with different dimensions are present. The hydraulic loss over these two structures changes since the bridges influence each other. This interaction should be taken into account when modelling these bridges. The range for hydraulic loss through two structures is in the order of one to two times the loss of one structure, as a hydraulic loss of two indicates that the bridges fall out of each others zone of influence (Apelt, 1983). With an indication of loss for two bridges ranging from 1.3 to 1.55 times the loss for one bridge with different bridge spacing's tested (HEC-RAS Reference Manual, 2021). However, the latter should only be assumed if the bridges are so closely spaced such that the flow isn't able to fully expand in between the two bridges, in such a way that the expansion zone of the bridge upstream doesn't interfere with the contraction zone of the downstream bridge. Yet, should the flow have enough length to fully expand and contract in between those section then both bridges should be implemented in the model separately.

#### Culverts

For modelling the culverts, roughly the same steps are carried out as for bridges. However, the contraction and expansion coefficients for the culvert are larger. This has to do with its different shape and size, which leads to a larger influence on contraction and expansion. Furthermore, the location of the second and third cross sections in Figure 3.2 are different. As the cross sections do not take the shape and structure of the embankment or culvert into account, but the shape of the channel where the river flows (USACE, 2016).

An additional difference is found in the fact that culverts pose a problem with accumulation of debris in the barrel. This accumulation of debris could result in a less efficient culvert, as the flow velocity is reduced. This means that the drainage of water is slowed down, leading to a higher flood risk upstream of a culvert.

#### **Blocked obstructions**

Finally, for the geometry it is necessary to provide the model with an indication of the buildings in the area over which the model is computed. The digital terrain model serves as a base layer for the river and floodplain bathymetry in the area, but as mentioned earlier it does not contain any buildings or obstructions. Therefore, the location and elevation of buildings and other obstructions should be added manually. This is done by adding a layer in QGIS, which can then be imported into HEC-RAS via RiverGIS.

#### Type of flow

Flow along a river can be classified into two types, subcritical and supercritical flow, depending on the Froude number, which gives a relation between the flow velocity and the depth of the water. For the 1D model it was first assumed that subcritical flow would dominate along the whole river. However, after a trial run it was found that this assumption is invalid. The Froude number appeared to be larger than one for three cross sections where relatively large jumps in the bed occur, see Figure 3.3. For all other cross sections the Froude number stays below one, which means that subcritical flow prevails there. As a result of the different flow areas, a mixed steady flow computation was carried out. The two boundary conditions discussed above are necessary in order to realize this computation.

#### 3.1.3. 2D Model Parameters

The parameters which form the basis of a 2D numerical HEC-RAS model are specified in this section.

#### Computational mesh

The first step for a 2D model is to draw a 2D flow area around the main channel and areas where flood is expected to occur. This area should be big enough so no water will reach the edges of this 2D flow area. For this 2D flow area, a suitable computational mesh has to be made. Meaning a computational mesh with cell sizes that are suitable for both simulating the terrain and the water flowing over it. Each cell is a complex elevation volume/area connection that shows the underlying terrain's characteristics, rather than a single elevation. Detailed cross sections of HEC-RAS cell faces are processed into elevation versus area, wetted perimeter, and roughness correlations. With HEC-RAS, the modeller can use bigger cell sizes while still accurately representing the underlying terrain using this method. The key to



Figure 3.3: Profile plot Rambla de la Carrasquilla.

creating a decent computational mesh in HEC-RAS is to make sure that the cell faces capture the high point of flow barriers. Changes in the water surface slope and velocity must also be taken into account. In the middle of each cell, a single water surface elevation is calculated. As a result, the higher the cell size, the further apart the computed water surface values between different computation points are, and therefore the water surface slope is averaged over longer distances in two dimensions. Choosing an appropriate mesh cell size is important to get accurate answers and good convergence with 2D flow areas (USACE, 2021d). It is also important to note that it is desirable to align the mesh with the flow, to reduce numerical diffusion and improve computational accuracy. Generating a mesh is an iterative process, which may needs several runs to find and solve problems (USACE, 2021b). For the Rambla de la Carrasquilla, an average cell size of 15 by 15 was chosen.



Figure 3.4: Terminology computational mesh.

Break lines can be added within the 2D flow area to indicate where the mesh should follow the present structures, such as the known flow lines or the river banks. The cell size can be set to a different, in general smaller, value around the breaklines. It is desirable to have more detail, thus a smaller mesh size, in the main channel since most of the water is flowing through here. In our case the mesh size in and near the main channel was set to 5 by 5 meter, this is the lowest mesh size possible since creating a mesh size with smaller cell sizes than the digital terrain model would not make any sense. For the city of Los Nietos a size of 10 by 10 meters was chosen, in order to capture the flooding through streets in more detail. For an overview of the mesh used for the 2D computations see Appendix G.

#### Time step

Getting accurate answers in a 2D flow model does not only rely on the grid size, but is also highly dependent on the assigned time step. HEC-RAS provides an adjustable time step option to further

enhance the computational results, in which the computational engine dynamically recomputes the required time step during the simulation based on the Courant number (USACE, 2021d). For stability and accuracy reasons the Courant number should be below 3 when the full momentum equation is used and below 5 when the applying the diffusion equation. However, in case of a dry channel and rapidly changing hydrograph, a Courant number equal or smaller than 1 is needed for the diffusion equation to become stable.

$$C = u * \frac{\Delta t}{\Delta x} \tag{3.1}$$

where:

*C* = Courant number *u* = Flood wave velocity

 $\Delta t$  = Computational time step

 $\Delta x$  = Average cell size

When the flow approximates a steady state, the software will speed up the computations through increased time steps, while still meeting the Courant criterion. Then, if sudden changes appear, it automatically decreases the time step to record this change (CivilGeo, 2021). This way significant savings on computational time can be achieved.

In HEC-RAS a starting time step must be chosen, with an upper- and lower boundary for the Courant number. This way the appropriate time step for each cell can be determined and the model will run more efficiently. Based on several testing runs and the fact that the research area is relatively small, the upper bound for the Courant number is chosen to be 0.7 and for the lower bound a value of 0.2. As a starting time step a value of 1 second was selected in order to produce an accurate and stable solution.

#### 3.1.4. Model Calibration

Calibration of the model can be done by selecting and adjusting certain parameters, which possibly have a large influence on the model outcome. Using standard techniques for model calibration, historical data are required (Boyd, 2001). Since these are not available, proper calibration is hard to carry out. However a comparison can be made with the study done by the government (see for further explanation on the government model section 4.3), this way some sort of calibration of the model can be done and the influence on each parameter can be analyzed.

#### Parameters for 1D model calibration

For the calibration of a 1D model, two parameters are investigated: the effect of cross sections and the effect of a bifurcation. As explained in subsection 3.1.2 cross sectional spacing is an important parameter to capture the different structures and meandering of the river. Also, the extent of the width of a cross section can impact the results quite significantly and therefore needs to be examined by varying this in several runs.

Another important aspect to check, is the addition of a bifurcation. Since a 1D model only describes the flow in one direction per cross section, a bifurcation can help to create a better representation of the flood because it allows the water to flow in a different direction. This should be checked with the previously mentioned government model.

#### Parameters for 2D model calibration

The 2D model is set up differently than the 1D model and therefore has also other parameters which can be used to calibrate it. For example, an important aspect of running a numerical model is choosing an appropriate time step. The software may become unstable if the changes in hydraulic properties at a particular cell vary significantly over time, as derivatives of the flow equations with respect to time and space are computed. In general, reducing the time step is the answer to this instability problem. However, a smaller time step will significantly increase the computational time, thus a balance must be found where the model converges to a stable solution while minimizing computational time (Brunner, 2014). However, a smaller time step does not always have to lead to a large increase of computational

time. For each solution HEC-RAS iterates several times, until the solution is acceptable. The iterative process is computational heavy and taking a smaller time step requires often less iterations. As a consequence, the additional run time for a smaller time step is sometimes less than expected (USACE, 2021c). According to USACE (2021d), picking a time step is highly case dependent and should be tested on convergence and stability of the solution, while keeping the computational time in mind.

The water surface elevation is dependent on the roughness of the riverbed and its floodplains, thus having an impact on the extent of a flood. Adjusting the roughness (determined by the Manning's n value) can give significantly different results. Even though its value can be estimated from fieldwork, it is useful to try different values and compare them with each other and the government model. At last the two different numerical methods can be used to solve the equations. One is more simplified than the other, so here it should be checked if the faster Diffusion Wave Equations solver gives more or less the same results as the more accurate, but computational slower, Shallow Wave Equations solver.

#### 3.2. Structure Modelling with Photogrammetry

An important aspect of the numerical models is the influence of different structures (e.g. bridges, culverts and a pipeline) on the flow. To correctly implement these structures in the numerical model, the dimensions of the structures must be known. Since this information is not publicly available, a method was developed to estimate these dimensions by creating 3D models with photogrammetry.

#### 3.2.1. Theoretical Basis of Photogrammetry

Note that, since every step of the process is a field of research by itself, this report focuses only on the methods which are implemented in the used softwares: *VisualSFM* and *Meshlab*. Details of the implementation of these softwares are discussed in subsection 3.2.2.

Photogrammetry is a technique aiming to creating 3D models by taking at least two images of the object and reconstructing the exact positions and orientations of the cameras. This means a 3D model must be created from 2D images. To make this transformation, a technique called *stereography* is used. Stereography combines a minimum of two 2D images, for which every image pixel can be transformed to a 3D point with (R. Lindenbergh, personal communication, February, 2019):

$$\begin{pmatrix} x \\ y \\ -f \end{pmatrix} = \lambda \mathbf{R} \begin{pmatrix} X \\ Y \\ Z \end{pmatrix} + \begin{pmatrix} t_x \\ t_y \\ t_z \end{pmatrix} = [\lambda \mathbf{R} | \mathbf{t} ]_{3x4}$$
(3.2)

Where

 $\lambda$  = the scale factor **R** = the rotation matrix **t** = the translation vector (*x*, *y*, -*f*) = the coordinates of the projection (*X*, *Y*, *Z*) = the coordinates of the 3D point

Since the focal length f of the camera is known, Equation 3.2 can be solved for image coordinates x and y. By applying this to both images, and then combining them, the degrees of freedom are reduced and the 3D points can be located in the reference system as visualized in Figure 3.5.



Figure 3.5: Principle of stereography, where P denotes the 3D point, O the position of the camera (lens), f the focal length of the camera and P' and P'' the negative and positive image projections respectively.

So, the orientation and location of the camera describe the *external* parameters of the camera and can mathematically be described by ( $\mathbf{R}$ ,  $\mathbf{t}$ ) respectively ( $\mathbf{R}$ . Lindenbergh, personal communication, February, 2019). In addition, the camera has internal parameters known as the *intrinsic* parameters. Both parameters have uncertainties contributing to the final error of the 3D model, this will be elaborated upon in subsection 3.2.3. The parameters can be described by the calibration matrix  $\mathbf{K}$ . It is not possible to estimate  $\mathbf{K}$  based on external measurement alone, but a series of measurements can estimate both  $\mathbf{K}$  and ( $\mathbf{R}$ ,  $\mathbf{t}$ ) ( $\mathbf{R}$ . Lindenbergh, personal communication, February, 2019).

Once the images are captured, two main steps need to be carried out. First, 'common object points' or 'features' must be detected in each image and matched with other images. After that, the 3D reconstruction can be made.

#### Feature detection and matching

The exterior orientation can be estimated if sufficiently common object points are recognized in different images. A common strategy to obtain this exterior orientation of the camera positions follows three steps (R. Lindenbergh, personal communication, February, 2019):

- 1. Feature detection
- 2. Feature description
- 3. Feature matching

Features that are in specific locations of the images are often called 'keypoint features' and are wellsuited to use as common object points. These keypoints are expressive in texture, are points at which the direction of the boundary of the object changes abruptly or are intersection points between two or more edge segments (Tyagi, 2020). Keypoints must have a well-defined position in image space, be stable under local and global perturbations in the image domain as illumination/brightness variations, and should provide efficient detection. The keypoints can then be identified based on either the brightness of the image or on boundary extraction (M. Kohde, personal communication, February 27, 2021).

Next, feature descriptors are used to describe image patches in a compact matter. These feature descriptors are invariant to changes in orientation and scale, so they will not be influenced by transformations of the images (R. Lindenbergh, personal communication, February, 2019). Descriptors can either be local, trying to shape and appearance in a local neighborhood, or global, describing the whole image. In this report, only local descriptors will be discussed, as global descriptors are generally not very robust. This is because a (small) change in part of the image may cause global descriptors to fail, as it will affect the whole resulting descriptor (Tyagi, 2020).

The final step is feature matching, where the algorithm searches for the most likely matching candidates (R. Lindenbergh, personal communication, February, 2019). The performance of feature matching based on keypoints depends on both the properties of the underlying keypoints and the feature descriptors (Tyagi, 2020).

Fortunately, multiple available algorithms are capable of carrying out this process. One of these is the *Scale Invariant Feature Transform* (SIFT). SIFT is invariant to image scale, rotation and partially invariant to change in illumination and 3D camera viewpoint. Therefore, it is well-suited for photogrammetry purposes (Lowe, 2004).

#### **3D reconstruction**

Once the images and keypoints in the images are matched, the 3D model can be constructed. This consists of creating sparse and dense point models (in *VisualSFM*) and finally reconstructing a continuous model of the surface (in *Meshlab*). Note that these are not the only steps of creating the model, but they are the steps influencing the model's accuracy.

After feature matching, a (sparse) reconstruction can be made. With a given set of measured image feature locations and correspondences, 'bundle adjustment' can be used to find 3D point positions and camera parameters that minimize the reprojection error (Triggs et al., 1999). To find the 'best' 3D model, the sum of all reprojection errors is minimized, which can be formulated as a non-linear least squares problem. The most common method to solve the problem is the Levenberg-Marquardt method (Wu et al., 2011b). When solving the equation, the sparse nature of the problem should be exploited: not all features can be observed from all camera locations. This gives only a small number of dependencies in the design matrix. Therefore often the computation time is not the main challenge of bundle adjustment, but errors in the data association are (Engels et al., 2006).

Next, using the in- and external camera parameters obtained from the bundle adjustment, images are clustered and certain image sets with the best content for reconstruction are clustered (Hosseininaveh et al., 2013). With these selected images, a dense 3D point cloud using an *Multi-View Stereo* (MVS) algorithm can be created. By assuming a stereo vision and pairing two images, the MVS algorithm can analyze depth and direction from these views. By computing this for numerous pairs, a 3D model can be constructed. *VisualSFM* works with *Clustering Multi-View Stereo* (CMVS), which takes the output of a structure-from-motion (*SfM*) software (*VisualSFM*) as input, then decomposes the input images into a set of image clusters of manageable size (Furukawa et al., 2010a). The image set is decomposed into clusters that have a small overlap, since a strict partition would lead to undersampled surfaces near cluster boundaries (Banerjee et al., 2005). An *MVS* software can then be used to process each cluster independently and in parallel, and the resulting reconstructions are merged into a single dense point-based model (Furukawa, 2010).

Finally, using the dense 3D point cloud, the surface can be reconstructed to fit the data and fill the surface holes. A commonly used algorithm which is implemented in Meshlab is the *Poisson Surface Reconstruction* (PSR). It is a popular choice for surface reconstruction, since it is a global solution that considers all the data at once by reducing its solution to a well-conditioned sparse linear system (Kazhdan et al., 2006). The concept of Poisson reconstruction can clearly be visualized in 2D (see Figure 3.6). When reconstructing the surface of a 3D model, a 3D *indicator function*  $\chi$  (defined as 1 at points inside the model, and 0 at points outside) is used. Then the reconstructed surface can be obtained by extracting an appropriate isosurface (Kazhdan et al., 2006).



Figure 3.6: Intuitive illustration of Poisson reconstruction in 2D (Kazhdan et al., 2006).

By taking the gradient of the indicator function  $(\nabla \chi)$ , a vector field is obtained that equals zero almost everywhere (since the indicator function is constant almost everywhere), except near the surface, where it equals an normal vector facing inward. So the indicator point samples  $\vec{V}$  simply are non-zero points of the gradient of the indicator function. Thus, the problem reduces to finding a scalar function  $\chi$  which minimizes the difference between a set of samples  $(\vec{V})$  and the gradient of the indicator function, i.e.  $\min_{\chi} ||\nabla \chi - \vec{V}||$ . By applying the divergence operator to the problem, it becomes a standard Poisson problem, which can be solved by a least-squares approximation (Kazhdan & Hoppe, 2013). This approach might appear straightforward, but unfortunately there are a number of difficulties in practice. One of these is the non-uniformity of the sampling points, which will occur when using a data set obtained with photogrammetry. Besides that, sample points and normals often are noisy due to the accumulated inaccuracies of the process. Moreover, intuitively one might first try to reconstruct the surface with a 3D grid. However, such a uniform structure cannot capture fine-details without quadrat-

ically increasing the number of surface triangles, which is not feasible for regular laptops. Fortunately, an accurate representation is only needed near the surface that is to be reconstructed. Therefore an adaptive octree (O) is implemented in the algorithm, to represent both the implicit function as well as solve the Poisson system. The maximum tree depth is denoted by D. The octree is defined such that at least one sample point falls into a leaf node at depth D.

#### 3.2.2. Structure Modelling with Photogrammetry

This section elaborates upon the methods used to create the virtual 3D models of the bridges and culverts. The pipeline of the process is visualized in Figure 3.7.



Figure 3.7: Pipeline of the photogrammetry process to create 3D models of structures.

#### In-field measurements

To obtain the input data for the 3D models, field work is necessary to acquire measurements of the structures dimensions and photos from all angles of the structures. At each structure first brightly colored post-it notes were put on some solid and straight parts of the structures, to create easily identifiable common tie points in each picture. Extra post-its were put on places with low reflectance, to avoid holes in the eventual model. The post-its where stuck on the structure in pairs, with no more than one meter between them. This was done in order to reduce measurement errors both when measuring the distance in real life, as well as measuring the distance in the 3D model. If the structure is symmetric, the variety of differently brightly colored post-its where used, stuck on in asymmetrically.

Then, of each structure between the 70 and 150 photos were taken from different angles, but with the same camera settings. This is crucial, since the focal length of the camera is used to compute the 3D coordinates from the images. A bridge model camera (Canon Powershot SX 60 hs) was used to capture the scenes in high details. By taking all pictures with the same camera, all images are the same size and have the same lens distortion, therefore reducing the number of pre-processing steps and increasing accuracy.

#### Creating the 3D model

Once the input data is obtained from the field work, the image matching, sparse reconstruction and dense reconstruction can be done in *VisualSFM* and the rescaling and dense reconstruction in *Meshlab*.

The steps performed in VisualSFM are rather straightforward. Each step in the pipeline that is carried out in VisualSFM corresponds to a tool in the software. In VisualSFM, pairwise matching is used for the feature detection and matching (SIFT), sparse reconstruction for the bundle adjustment and finally the dense reconstruction for the CMVS algorithm.

The disadvantage of this very simple workflow is that there are not many parameters that can altered and the available background information given by Wu et al. (2011a) is very limited.

After loading both the sparse as well as the dense model in Meshlab, the models can be 'cleaned' by manually removing noise. Then, the surface can be reconstructed following the approach of Kazhdan

et al. (2006). Three essential parameters for this Poisson reconstruction are the minimum number of samples, the interpolation weight and the octree depth. For all three parameters a trade-off between computational efficiency and accuracy must be made. By setting the minimum number of samples to small values [1.0 - 5.0], noise-free samples can be accurately created. Larger values in the range [15.0 - 20.0] may be needed to provide a smoother, noise-reduced, reconstruction. The interpolation weight and octree depth increase accuracy for larger values, and smoother, less-detailed models for small values. For the models in this report, the minimum number of samples was set to 1.5, the interpolation weight to 4 and the octree depth to 8.

When loading a dense point cloud in Meshlab, the model has unitless dimensions. In order to compute (average) distances, the model needs to be rescaled. For this purpose, the post-it notes which were stuck on the structure are used as benchmarks. In Meshlab, the measuring tool was used to make three measurements between at least three benchmarks for every structure to reduce the measurement error. Next, the scaling factor could be computed with the averaged Meshlab measurement and the real life measurements. Once the model was rescaled, the dimensions of the structures could be retrieved either with the measuring tool or exporting the point cloud to *CloudCompare*, which is able to compute average distances between point clouds. This is a useful feature when interested in the average height above a non-flat surface such as a riverbed. Also larger distances can be measured with higher precision.

#### 3.2.3. Error Analysis

The 3D modelling pipeline is long and each step comes with its own errors. Fortunately, for the purpose of hydraulic modelling, no sub-centimeter precision is required. Based on the sensitivity analysis of our model, we estimate that an accuracy of  $\pm 5$  % is sufficient. For the model of the large culvert, this corresponds to  $\pm 20$  cm in height. However, since it would be interesting to see if photogrammetry could be relevant for other studies (possibly using 3D hydraulic models), we will roughly estimate the error.

However, before estimating the error, it is useful to briefly discuss the main challenges of the first step of the pipeline: the image acquisition. When acquiring the images of the structures along the river, there are both physical and non-physical obstructions. Physical obstructions include vegetation which is blocking the view, water in the river due to which not all angles can be covered, and the large geometry of some structures, which prevents us from capturing images of the top of the structures and retaining detail if images must be taken from far away. Other problems are often caused by non-constant lighting. Ideally, one would have a light of constant brightness, illuminating all angles of the structure. This is hard to achieve when using natural light. Shadowy parts are difficult to match, but changing the camera settings is also not recommended. Also other internal parameters such as focal length should not be altered, since this is used to estimate the 3D point. Besides areas with low reflectance, non-Lambertian, textureless, very highly reflective or transparent areas and repetitive patterns are also problematic. These challenges are difficult to quantify and flaws in the model can only be seen in the reconstruction steps further in the pipeline. Therefore it is of great importance to take a large amount of pictures.

Once these images are acquired, the errors of the 3D modelling can be computed. Errors to consider include:

- · error due to the radial distortion of the camera
- real life measurement error between the common object points (post-it notes), which will be used for scaling
- false SIFT feature matches
- errors in the bundle adjustment (reprojection error)
- · reconstruction error due to Multi-view Stereo algorithm
- · Poisson surface reconstruction error
- · error magnification of measurement errors due to rescaling of the model

The place where in the pipeline these errors come into play is visualized in Figure 3.8.



Figure 3.8: Photogrammetry pipeline where the error sources for each step correspond to the double-lined boxes.

Since the errors are not correlated, we can simply find the total actual error using

$$e_{actual,total} = \sum_{i=1}^{N-1} e_i \tag{3.3}$$

Where:

 $e_i$  are the errors of each step

*N* the number of error sources, in this case N = 6

Because the errors are squared, we might see that some of the errors are negligible compared to others. Note that the error of the final step (N = 6) is not taken into account in Equation 3.3, since this is the rescaling error. This means we cannot simply add this error, but we need to use the relative error due to the multiplication.

The relative and absolute error for each model can be found in Appendix D, together with a description of all contributing errors. The maximum relative (and absolute) error is associated with the model of the large culvert. Which has a mean relative error of 0.043 %, or 2.3 cm. Since deviations at this scale do not alter the outcomes of the numerical models significantly, these errors are negligible for this study.

Note that the measurements of the larger structures (bridges), which were not modelled with photogrammetry, will have larger real life measurement errors due to the tape which is not strictly levelled. This made it much more difficult to keep under enough tension over large distances. Also the height of the structures had to be estimated using images, giving a larger uncertainty. We estimate the measurement error with the tape to be  $\pm 2$  cm, and the measurements obtained from the images approximately 10 cm.

#### 3.2.4. 3D Photogrammetry Model Results

Of all bridges and culverts along the river, three were modelled with photogrammetry. The 3D model of the large culvert (after 2021) can be seen in Figure 3.9.



Figure 3.9: Front view photogrammetry result large culvert with measurements in Meshlab.

Figure 3.9 does not only illustrate the result of the 3D model, but also the measurements made in Meshlab. The smaller yellow lines on the sides of the culverts are the measurements between the benchmarks (post-it notes), which are used to scale the model. Then line from the bottom to the top of the culvert is one example of a measurement for the structure dimensions (height in this case). All dimensions of this culvert and the other structures can be found in Appendix A. The results of the other photogrammetry models can be seen in Appendix C.
## 3.3. Surface Roughness Determination

To estimate the energy losses that are present in the model, the roughness of different bed types and flood plains that are present along the river must be known. For this a land cover map based on available data was made, but also fieldwork research was done. The surface roughness is described by the Manning' n value, a higher value means more surface roughness. For areas with higher surface roughness, the risk of flooding is higher as the water flow gets more obstructed in its path.

#### 3.3.1. Determination Manning's n Value Main Channel

At the delta of the river, the land cover can be described as a muddy channel bed with presence of low vegetation at the river banks and in the floodplain. Later on, the river is less deep and the land cover is dominated by grasses and medium-low vegetation, representing a more swampy area. While at places further upstream the river, it can be dry and rocky, with some trees only in the floodplains. Places where the river bed is fully overgrown with very high reeds and trees are also present. In order to find the appropriate Manning's n for all these different undergrounds, the river bed was divided into different sections, each representing a different cover.

The type and size of the materials that make up the bed and the shape of the channel, influence the selection of the n values the most. The value of n is computed as follows Arcement and Schneider (1989):

$$n = (n_b + n_1 + n_2 + n_3 + n_4) * m$$
(3.4)

Where:

 $n_b$  = a base value for n

- $n_1$  = a correction value for surface irregularities
- $n_2$  = a value for variations in shape and size of the channel cross section
- $n_3$  = a value for obstruction
- $n_4$  = a value for vegetation
- m = a correction factor for meandering

#### Base value n

First of all, a distinction is made for the general underground being either classified as a stable channel or as a sand channel. A stable channel is defined as a bed with firm soil, cobbles, gravel or bedrock and the channel remains relatively unchanged during different flow scenarios. Based on Table 1 from Arcement and Schneider (1989) a base value n is chosen for each river section. However, these values are based on straight and uniform channels and still need to be corrected for irregularities and obstructions which are present on the Rambla de la Carrasquilla. The value of n should be adjusted, by adding the different factors influencing the roughness of the channel (see Table 2 Arcement and Schneider (1989)). The types of base that are encountered in the Rambla de la Carrasquilla are firm soil and gravel.

#### Bed surface irregularity

Eroded and scalloped banks can increase the roughness in a situation where the width is relatively small compared to the depth. Also exposed tree roots along the river banks should be accounted for by relative large correction values.

#### Variation in channel cross section

Large changes in the cross sections do not influence the value of n significantly if the variation is gradual and uniform. Alternation between large and small cross sections, sharp bends and constrictions are related to an increase in roughness. The number and magnitude of these alternations determine the magnitude of the correction factor that is needed.

#### Obstructions

Objects in the river such as logs, boulders or bridge piers disturb the flow pattern and will increase the roughness of the channel. Depending on the shape and size of the obstruction, the amount of roughness increases. Also the number, spacing and arrangement of obstructions have an influence on the roughness.

#### Vegetation

Several factors determine the extent to which the vegetation influences the value of n. The depth of the flow, percentage of river bed that is covered by vegetation and the alignment of the vegetation relative to the flow are factors that affect the in- or decrease of the roughness. For example, vegetation perpendicular to the flow has a greater impact than that parallel to the flow. Also the type and height of vegetation determines the magnitude of the correction factor.

#### Meandering

For a river which meanders, more energy will be lost relative to a more straight river. The meandering ratio is defined by the total length of the river divided by the straight length of the river. In the case of the Rambla de la Carrasquilla, this ratio is only between 1.0 and 1.2 along all parts of the river, and therefore the effect of meandering can be neglected.

Based on site investigation, see Appendix E, the different n value for each section of the main channel was determined by estimating the base value and the appropriate correction factors as described by the criteria above. The values for n in the different sections range from 0.026 to 0.11.

#### 3.3.2. Determination Manning's n Value Floodplain

For the land surface classification of areas outside of the main channel, the Corine Land Cover data produced by Copernicus is used (Papaioannou et al., 2018). The different classes present near the Rambla de la Carrasquilla are stated in Table 3.1 and are shown in Figure 3.10.



## Corine Land Covers of ROI

Figure 3.10: Different Corine land covers.

Corine Land Cover	Manning's n
Continuous urban fabric	0.013
Discontinuous urban fabric	0.013
Sport and leisure facility	0.025
Non-irrigated agricultural land	0.03
Permanently irrigated land	0.03
Fruit trees and berry plantations	0.08
Coniferous forest	0.1
Natural grassland	0.04
Sclerophyllous vegetation	0.05
Coastal lagoons	0.07

Table 3.1: Corine land cover types in the region of interest, with their corresponding Manning's n.

#### 3.4. Hydrological Boundary Conditions

A hydraulic HEC-RAS model needs at least two boundary conditions in order to be able to carry out calculations. A discharge or flow hydrograph must be specified at the upstream boundary of the model. For the downstream boundary, it is necessary to establish for example a fixed water level, normal depth or stage hydrograph.

#### 3.4.1. Upstream Boundary Condition

For the upstream boundary condition, hydrological data on discharges or flow hydrographs is usually obtained by estimations from previous floods. However, no measurements are available of previous floods in the Rambla de la Carrasquilla. Therefore, an estimation of the discharge in the river should be made based on historical precipitation data in combination with neighbouring dams and their capacity. As a result of a hydrological study by CEDEX (2009), peak discharges for different return periods have been obtained. These discharges are calibrated based on information from 6 previous floodings in the area. The peak discharges are based on return periods of 10, 100 and 500 years, referred to as Q10, Q100 and Q500 respectively. An overview can be observed in Table 3.2. These different discharges will be used as hydrodynamic boundary conditions in the HEC-RAS models. For the 1D model a single value for the discharge can be used as input. The 2D model needs a flow hydrograph as boundary condition, which consists of the constant peak discharge value.

The hydrological data is obtained for a certain catchment area. This catchment area does not begin at the most upstream end of the Rambla de la Carrasquilla, but it includes a certain upstream part of the river. As a result, the model will start somewhat downstream from the actual beginning of the Rambla de la Carrasquilla. In this manner, the obtained discharge from the hydrological data can be used as a direct boundary condition at the chosen 'upstream' point in the river.

Return period [years]	Discharge [m <sup>3</sup> /s]
10	22
100	68
500	116

Table 3.2: Peak discharges for the three different return periods (de España, 2020).

#### 3.4.2. Boundary Conditions for the 1D Models

The downstream boundary of the Rambla de la Carrasquilla is the lagoon 'Mar Menor'. The lagoon is modelled to have a constant water surface elevation of 0.2 m. For the upstream boundary condition, a normal depth was assumed with a slope of 0.00118. A normal depth boundary condition calculates an energy slope, the input can be given by measuring the slope of the river at the place of the boundary condition using the Digital Terrain Model Table 3.3.

	Boundary condition	Value	Unit
Upstream	Normal depth (slope)	0.00188	[-]
Downstream	Water surface elevation	0.2	[m]

Table 3.3: Boundary conditions 1D Mixed Steady Flow Analysis.

#### 3.4.3. Downstream Boundary Conditions for the 2D Models

Along the perimeter of the 2D area, various boundary and initial conditions can be applied. Four types of external boundary conditions can be linked directly to the boundary of 2D flow areas, these are:

- Flow hydrograph
- Stage hydrograph
- Normal depth
- · Rating curve

The normal fepth and rating curve boundary conditions can only be applied when flow leaves a 2D flow area. For putting flow into or taking flow out of a 2D flow area, the flow and stage hydrograph boundary conditions can be used. Positive flow values in a flow hydrograph will send flow into a 2D flow area, while negative flow values will take flow out of a 2D flow area. In a 2D flow area, stages higher than the ground or water surface will send flow in, and stages lower than the water surface will send flow out, according to the stage hydrograph. Flow will not transfer if a cell is dry and the stage boundary condition is lower than the 2D flow area cell minimum elevation (USACE, 2021a).

Since the river spills into the lagoon Mar Menor, it is chosen to set a stage hydrograph boundary at the end of the river, with a constant value of 0.2 m. Upstream of the 2D flow area, a flow hydrograph is used to compute the incoming flow, with a constant value of 116 m<sup>3</sup>/s, representing the peak flow of the scenario with a return period of 500 year.

#### 3.5. Comparison with Government Model

The Spanish government set up a hydraulic model using HEC-RAS software in combination with Geo-RAS for ArcGIS. This study was done on a national level and therefore requires more detailed local studies to improve the accuracy (Sánchez & Lastra, 2011). The approach that was used by the government is similar to the approach chosen for this research. This section highlights the similarities and differences between the government's approach and our approach, so that a comparison can be made between the results. All information and assumption on the approach the government uses is based on Sánchez and Lastra (2011).

The most important differences between both approaches as well as the different datasets are mentioned in Table 3.4.

The flood maps produced by the government that are used to make a comparison were made using a 2D HEC-RAS model. As input for this model a Digital Terrain Model (DTM) of 2x2 meter was used with an accuracy of  $\pm$  15 cm in height. This digital terrain model does not contain any structures like the bridges, culvert or pipeline as these were detected on orthophotos of the same area and then removed from the DTM manually. Vegetation and similar object were removed from the DTM, while buildings are vectorized and then added to the DTM. Also the bridges and culverts were added later on manually for investigation of their influence. The dimensions on these structures were based on the orthophotos and available sketches.

To construct the final hydraulic model, the DTM is divided into two parts. In the study these are described as the 'current' and 'natural' DTM. The current DTM corresponds to the 'real' DTM and includes embankments, channels and some buildings, while vegetation and bridges are removed. This current DTM is used to model the flood zones. The natural DTM excludes the buildings as well and is used for modelling of the natural flow regime. The natural DTM is also supported by historical orthophotos, in order to estimate how the water used to flow when there were less buildings, roads and structures obstructing the flow.

	Government model	Our model
Software	HEC-RAS HEC-geoRAS for ArcGIS	HEC-RAS RiverGIS for QGIS
Digital Terrain Model	2x2 mesh; split in natural and real	5x5 mesh
Orthophotos	Historical photos for natural flow Current photos for structure identification	Determining route for field work
Structure measurements	Sketches Use of DTM and orthophotos	Field work measurements: •Structure dimensions •Photogrammetry
Surface roughness	SIOSE CORINE Land Cover Orthophotos	CORINE Land Cover Field work documentation
Historical study	Orthophotos Historical flood series	Satellite images 2019 Aerial photos 2019
Geomorphological study	Preferred flow zone	None
2D mesh	Mesh based on height contour lines Constructed with ArcGIS	Mesh following the center flow line of the river Constructed with RASmapper
Calibration	Compare flow with and without structures	Compare flow with and without structures

Table 3.4: Overview comparison government model & our model

The hydrological study that was used to determine the in flowing water volume at the upstream boundary is the same as used in this report and is based on three scenarios with a return period of 10, 100 and 500 years. The corresponding values are 22, 68 and 116 m<sup>3</sup>/s correspondingly. This study was carried out by the Center for Studies and Experimentation of Public Works called CEDEX (CEDEX, 2009).

The surface roughness in this government study, used to determine the Manning's n values (see section 3.3 about more explanation on this concept), was determined using land cover data that is available on national or international level. The data used for this are those of SIOSE (a land occupation information system in Spain) and the CORINE Land Cover map of 2000, a project by Copernicus (a European earth observation program). Additionally, orthophotos produced by the national plan for aerial photography were used for the calibrations of the land cover classification.

Historical analysis was used in order to verify the flood maps that were created in this study that show the flooded area of different regions in the three different peak scenarios. This historical analysis consists of two parts:

- 1. Evolutionary study of the river environment through historical aerial photographs.
- 2. Reconstruction of historical flood series.

The main advantage of the historical method is that it has shown a high reliability to define the categories of greatest danger, precisely those of greatest interest for flood risk management. Different flood areas are defined per return period.

# 4

# **Results and Analysis**

In this chapter, the results are presented and analyzed. In section 4.1 the results of the 1D HEC-RAS model can be found, after which the 2D HEC-RAS model results are found in section 4.2. Lastly, the obtained floodmaps are compared to the floodmaps of the government in section 4.3.

#### 4.1. 1D Model Results

The 1D model was computed for different geometries. Results were obtained for an 1D model with one reach, as well as for an 1D model with a bifurcation near the downstream end. These different approaches are discussed in the following subsections.

#### 4.1.1. 1D Models with One Reach Excluding Structures

#### First 1D model: one reach

The first 1D HEC-RAS computations were done for models that do not contain the bridges, pipeline and culvert that are present in the Rambla de la Carrasquilla. This was done to get a first impression of the flow in the river and potential flooding of the area. The structures will be added in subsection 4.1.3 to investigate the effects they have on the flow and floods.

The geometry of the first 1D model that was made can be observed in Figure 4.1a. The river is represented by one single reach. On the downstream end, the cross sections extend across about half the town of Los Nietos. This was done because from the 2019 flood event, it is known that this area flooded. The result of the mixed steady flow calculations for a return period of 500 years can be observed in Figure 4.1b. The results for all three return periods can be observed in Appendix F.

Figure 4.1b shows that a relatively small area is flooded in the most upstream part of the Rambla de la Carrasquilla for a rainfall event with a return period of 500 years. It can be observed from the legend that this flood only concerns a small water depth. When moving along with the flow, no additional overbank area experiences significant flooding until approximately the location of the culvert is reached. Figure 4.1b and 4.2a show that the left overbank area just upstream of the location of the culvert is flooded. Downstream of the culvert, the terrain changes and a higher area arises on the left side of the main channel which acts as a natural levee. The right floodplain is flooded here, since the ground on this side is lower.

In the last part of the river, the whole area covered by the cross sections seems to flood. This has to do with the terrain over which the cross sections are drawn. In the downstream part, cross sections extend across Los Nietos, where the ground level is lower than the bottom of the main channel of the Rambla de la Carrasquilla. This can be seen in Figure 4.2, where the red arrow indicates the lowest level of the main channel. It can be observed that for the upstream cross section (4.2a), this is indeed the lowest level present in the cross section. When looking at the downstream cross section (4.2b), it can be observed that the left part is situated lower than the lowest level in the main channel. For HEC-RAS to be able to compute accurate 1D modelling results, the main channel always has to be the

lowest point in the cross section. If this is not the case, the program will assume the lowest part of the floodplain to work as a main channel. Therefore, HEC-RAS assumes flooding over the whole cross section which leads to flood maps as shown in Figure 4.1b.



(a) Geometry of the 1D model with one reach.



(b) Floodmap for a return period of 500 years.

Figure 4.1: 1D HEC-RAS model where structures are excluded and the river is represented by one single reach.



(b) Cross section downstream of the culvert.

Figure 4.2: Cross sectional view of the 1D HEC-RAS model with one reach excluding structures.

#### Second 1D model: one reach with narrow downstream cross sections

An additional option for an 1D HEC-RAS model with one single reach was researched. In this second case, the 1D model contains narrower cross sections downstream than in the first model. The geometry of this second 1D model can be found in Figure 4.3a. The floodmap for a return period of 500 years is shown in Figure 4.3b. The results for all three return periods can be observed in Appendix F.

As expected, the downstream area seems to flood completely. Even though the channel is the lowest point in this case, the cross sections cover a relatively small area which contains the main channel and a part of the overbank area. Both of these flood with peak flow, resulting in a floodmap as shown. The upstream results are quite well comparable with the first 1D model.



(a) Geometry of the second 1D model with one reach.



(b) Floodmap for a return period of 500 years.

Figure 4.3: 1D HEC-RAS model where structures are excluded and the river is represented by one single reach with narrow cross sections downstream.

#### 4.1.2. 1D Model with Bifurcation Excluding Structures

In addition to the 1D model with one single reach, it was decided to try another approach to see if this would lead to different results. In this third 1D model, a bifurcation is introduced at the location of the culvert. This is done because the flow split into two directions at this location during the 2019 flood. The new "Los Nietos" reach introduces a second channel downstream, which flows through the town of Los Nietos. This may avoid the earlier mentioned problems where the floodplain is situated lower than the main channel. The "Rambla" reach represents the main channel of the Rambla de la Carrasquilla. The geometry of this new model can be observed in Figure 4.4a. No changes were implemented upstream, but downstream new bank lines, flow paths and cross sections were drawn. The resulting floodmap for a return period of 500 years can be seen in Figure 4.4b. The results for all three return periods can be observed in Appendix F.

When looking at the newly obtained floodmap in Figure 4.4b, there is no significant overflow in the most upstream part of the Rambla de la Carrasquilla, as was the case for the first 1D models. When moving from the upstream end towards the location of the culvert, no floods are encountered and the water depth can be observed to be lower than for the first model. When arriving at the location of the culvert, hence the bifurcation, a flood starts spreading out over the downstream area. The floodmap in Figure 4.4b shows that flooding starts directly at the location of the bifurcation. Certainly this was expected to happen, as the Los Nietos channel is getting some amount of discharge as input. However, compared to Figure 4.1b it looks like the total flooded area is larger.

2.5315

3.1641 3.7967

4.3807

5.7519



(a) Geometry of the 1D model with bifurcation.

(b) Floodmap for a return period of 500 years.

Figure 4.4: 1D HEC-RAS model where structures are excluded and the river is represented by a single reach upstream and a bifurcation downstream.



<sup>(</sup>b) Cross section in the downstream Rambla reach.

Figure 4.5: Cross sectional view of the 1D HEC-RAS model with bifurcation excluding structures.

Furthermore, the problem with the lower ground level in a floodplain is still present. Figure 4.5 shows a downstream cross section for both the Los Nietos and Rambla reach. It can be observed that in both scenarios the main channel is situated higher than the lowest point in the cross section, leading to fully flooded cross sections.

#### 4.1.3. 1D Model Including Structures

The previous analyses were done for 1D models excluding structures (bridges, pipeline and culvert). Now, the influence of these structures on the river flow needs to be investigated. Specifically interesting is the 2019 culvert compared to the 2021 culvert. The structures are embedded in the first 1D HEC-RAS model with one reach. The dimensions and layout of the implemented structures can be observed in Appendix B.

The implementation of the structures results in floodmaps as seen in Figure 4.6. It can be observed that there is no significant difference in flooded area between the models with 2019 and 2021 culvert. The model with the 2019 culvert leads to a flood that covers a few more pixels at the location of the culvert than the model with the 2021 culvert. When comparing the model including structures with the model excluding structures, it can be observed that there are no significant differences in flooded area and water depths as well. Equivalent results were found for the two other 1D models.



(a) Floodmap for a return period of 500 years including the bridges, pipeline and 2019 culvert.



(b) Floodmap for a return period of 500 years including the bridges, pipeline and 2021 culvert.

Figure 4.6: 1D HEC-RAS model where structures are included and the river is represented by one single reach.

#### 4.2. 2D Model Results

In order to come to these results a model was build using a 2D mesh over the river and areas that are in a risk of flooding, including the town of Los Nietos and many agricultural fields. In order to accurately modelling the water flow, while maintaining low computation times, the general mesh was kept rather large, namely 25 by 25 m, and the river and part of the town where given a smaller mesh by using break lines. An example of the mesh in and next to the river can be seen in Figure 4.7a, as well as an example of the mesh in the town in Figure 4.7b.



(a) Finer mesh generated for accurately modelling the water flow inside the river.



(b) Finer mesh in the town of Los Nietos for accurately modelling flood.

Figure 4.7: Two examples inside the mesh that was generated for the 2D model

#### 4.2.1. Time Step Analysis

Several runs have been carried out to compare the effect of the time step on the 2D HEC-RAS model outcomes and to check whether or not the solution is stable. Simulations based on the Diffusion Wave Equations (DWE) for a return period of 500 years with time steps of 10 seconds, 5 seconds and 1 second have been performed (excluding the bridges, pipeline and culvert). Figure 4.8 shows the three corresponding floodmaps. The difference in flooded area and maximum water depth per time step can be observed in Table 4.1.

The first simulation was done for a time step of  $\Delta t = 10s$ . From Figure 4.8a, it can clearly be seen that the solution becomes unstable. The flooding area is significant and extends until the boundary of the mesh. The overbank areas of the Rambla de la Carrasquilla flood almost everywhere, even in the most upstream part of the river. Moreover, significant water depths in the main channel can be observed. An attempt has been made to obtain a more stable solution by halving the time step to 5 seconds, see Figure 4.8b. It can be observed that the flooding area present. In addition, the water depths inside the main channel show to be fairly large flooding area present. In addition, the water depths inside the main channel show to be fairly large again. These factors indicate another unstable result. For this reason, the time step has been decreased further to find a stable solution. A time step of 1 second leads to a floodmap as seen in Figure 4.8c. Around this value of 1 second, decreasing the time step to lower values hardly changes the results of the model. This indicates that the solution is stable and converges to the exact solution around this time step of 1 second.

∆t [s]	Flooded area [m <sup>2</sup> ]	Max. water depth [m]
10	1648330	6.67927
5	1312300	6.67298
1	637225	4.64475

Table 4.1: Comparison of flooded area and maximum water depth between different time steps.



(a) Time step dt = 10 seconds

(b) Time step dt = 5 seconds.

(c) Time step dt = 1 second.

Figure 4.8: The effect of a different time step. Floodmaps for a return period of 500 years computed by 2D HEC-RAS models excluding structures.

#### 4.2.2. Comparison of Different Surface Roughness Values

The surface roughness of the modelled area is implemented through the use of Manning's n values. The influence of changes in this surface roughness are studied by comparing a constant Manning's n value with a land cover map containing different Manning's n values for different parts in the river and different land uses outside the river. For the constant Manning's map, this means no distinction was made between the different underlying terrains, a value of n = 0.03 is considered everywhere. For the map with variable values, Manning's n is generally higher inside the main channel of the river than in its floodplains, because of the vegetation in the main channel. The resulting floodmaps are computed for a return period of 500 years and can be observed in Figure 4.9.

From the floodmaps in Figure 4.9 and the data in Table 4.2 it can be seen that the 2D model with a constant value for Manning's n leads to a considerably smaller flooded area than the 2D model with variable values for Manning's n. Meanwhile, the difference in maximum water depth is relatively small: in the order of centimeters. The 2D model with a constant value for Manning's n has a lower maximum water depth than the model with variable values for Manning's n.

	Flooded area [m <sup>2</sup> ]	Max. water depth [m]
Variable values for Manning's n	700650	4.0814
A constant value for Manning's n = 0.03	546425	3.9402

Table 4.2: Comparison of flooded area and maximum water depth between variable values for Manning's n and a constant value for Manning's n = 0.03.

#### 4.2.3. Comparison of Numerical Methods

As explained in section 2.5, there are three numerical methods that can be used in HEC-RAS. The results of the Diffusion Wave Equation (DWE) can be observed in Figure 4.10a. The Shallow Water Equations (SWE) in combination with the Eulerian-Lagrangian Method (ELM) and the Eulerian Method (EM) are shown in respectively Figure 4.10b and 4.10c. It should be noted that a constant value for Manning's n = 0.03 was used in these models. For the numerical computations to be stable, different time steps and boundary conditions had to be used, as seen in Table 4.3. This table also contains information about the output of the models. It can be observed that the DWE results in the smallest



(a) Variable values for Manning's n, according to section 3.3, were used.



(b) A constant value for Manning's n = 0.03 was used.

Figure 4.9: The effect of surface roughness of the riverbed and floodplain. Floodmaps for a return period of 500 years computed by 2D HEC-RAS models excluding structures.

flooded area and lowest maximum water depth. The intermediate values for flooded area and maximum water depth belong to the SWE-ELM. Finally, using the SWE-EM gives the largest flooded area and maximum water depth.

	∆t [s]	Stage hydrograph [m]	Flooded area [m <sup>2</sup> ]	Max. water depth [m]
DWE	1	0.2	700650	4.0814
SWE-ELM	1	0.5	806325	4.4383
SWE-EM	0.5	0.5	714325	4.3668

Table 4.3: Difference in parameters and outcomes of the comparison between the DWE, SWE-ELM and SWE-EM.



(a) Obtained by using the DWE.

(b) Obtained by using the SWE-ELM.

(c) Obtained by using the SWE-EM.

Figure 4.10: The effect of different numerical methods. Floodmaps for a return period of 500 years computed by 2D HEC-RAS models where structures are excluded.

#### 4.2.4. 2D Models for Different Return Periods Excluding Structures

Similar to the 1D model, the 2D model was computed for three different return periods: 10, 100 and 500 years representing peak discharges of respectively 22, 68 and 116 m<sup>3</sup>/s. The resulting floodmaps can be observed in Figure 4.11. The floodmaps show that for a rainfall event with return period of 10 years (Figure 4.11a), the water stays within the river banks until approximately the location of the culvert is reached. From this moment onwards, a relatively small amount of the water flows to sea over the left floodplain through the town of Los Nietos.

For the return periods of 100 (Figure 4.11b) and 500 (Figure 4.11c) years, the most upstream part of the river experiences some overflow from the main channel. Hereafter, the flow stays within the river banks, but larger water depths are encountered. A significant amount of water starts spreading out from approximately the location of the culvert until the downstream end of the Rambla de la Carrasquilla. Table 4.4 shows numerical data on the flooded area and maximum water depth. The flooded area increases along with the increase in return period. However, the maximum water depth does not necessarily increase for a rainfall event with larger return period.

Return period [years]	Flooded area [m <sup>2</sup> ]	Max. water depth [m]
10	359275	3.6890
100	557850	4.1479
500	700650	4.0814

Table 4.4: Comparison of flooded area and maximum water depth between different return periods.



(a) A return period of 10 years was used. (b) A return period of 100 years was used. (c) A return period of 500 years was used.

Figure 4.11: The effect of different return periods. Floodmaps computed by 2D HEC-RAS models excluding structures.

#### 4.2.5. 2D Models Including Structures

The previous subsections analyzed models that were constructed using only the Digital Terrain Model for information on the river bathymetry. In this subsection the two bridges, pipeline and the culvert were added manually to the 2D model. This was done in order to investigate the influence of these structures on the results. The comparison between 2D modelled floodmaps excluding and including structures can be seen in Figure 4.12. Figure 4.12a shows the floodmap of the 2D model excluding structures. In Figure 4.12b the structures are included. It is seen that the flood area downstream of the culvert has increased. This additional flooding occurs mainly in the town of Los Nietos and on the right side of the downstream part of the river. Table 4.5 shows a significant increase in the total flooded area. The maximum water depths are in the same order of magnitude.

The implementation of the 2021 culvert led to a large volume error and unstable model when the same boundary conditions as in previous 2D models were used. The use of a different boundary condition led to results that can be recognized in Figure 4.13. These floodmaps are obtained using a normal depth boundary condition downstream. The 2D model excluding structures and including the 2019 and 2021 structures were all computed with this new boundary condition, so that these results can be compared.

In general, the 2D models with this new downstream boundary condition have a larger flooding area downstream. This can be seen when comparing Figure 4.12a and Figure 4.13a. Additionally, the floodings on the most upstream part of the Rambla de la Carrasquilla are somewhat more intense compared to the previous results. In between the most upstream part of the river and the location of the culvert, the 2D model results with new boundary condition are comparable to the 2D models with the initial boundary condition.

The influence of the new culvert which was placed in 2021 was also investigated. The floodmap result of the 2019 culvert and the 2021 culvert can be seen in respectively Figure 4.13b and Figure 4.13c. Numerical outcomes are shown in Table 4.6. The difference in flooded area between the two scenarios is rather small with 0.029 km<sup>2</sup> more for the 2019 culvert. The maximum water depth excluding structures is nearly equal to the maximum water depth for the 2D model including the 2019 and 2021 culvert, which have the same value for maximum water depth.



(a) Excluding bridges, pipeline and culvert.



(b) Including bridges, pipeline and 2019 culvert.

Figure 4.12: The effect of including structures in the model. Floodmaps for a return period of 500 years computed by 2D HEC-RAS models.

	Flooded area [m <sup>2</sup> ]	Max. water depth [m]
Excl. bridges, pipeline and culvert	700650	4.0814
Incl. bridges, pipeline and 2019 culvert	777875	4.4227

Table 4.5: Comparison of flooded area and maximum water depth between 2D models excluding and including structures.

Flooded area [m <sup>2</sup> ]	Max. water depth [m]
842550	4.4071
874175	4.4092
843150	4.4092
	Flooded area [m <sup>2</sup> ] 842550 874175 843150

Table 4.6: Comparison of flooded area and maximum water depth between 2D models excluding and including structures, with a normal depth = 0.05 [m] boundary condition downstream.



Figure 4.13: The effect of including structures in the 2D HEC-RAS model with a normal depth = 0.05 [m] boundary condition downstream. Floodmaps for a return period of 500 years.

### 4.3. Government Model Comparison

A comparison is made between the floodmaps obtained by the 2D HEC-RAS models in this research and the earlier obtained floodmaps by the Spanish government (as discussed in section 3.5). The floodmaps for three return periods can be observed in Figure 4.14. The floodmaps of the government are orange, whereas the floodmaps obtained with the 2D HEC-RAS models can be recognized by the variable blue colour (representing different water depths). The total area that is covered by the different floodmaps is shown in Table 4.7. It can be seen that the flooded area obtained with the government model is smaller than the flooded area obtained with the 2D HEC-RAS models in this research. This holds for every return period.

For a return period of 10 years, the floodmaps are very similar from the upstream end until approximately the location of the culvert. In the government floodmap, the river flow stays within the main channel over the whole catchment area. Only very close to the sea some part of the delta is flooded. In contrary, the floodmap obtained with the 2D HEC-RAS model shows a small amount of flooding in Los Nietos. From the legend it can be seen that this only concerns very small water depth (in the order of millimeters).

For a return period of 100 and 500 years, the floodmaps of the government versus this research show relatively much resemblance over the whole catchment area. The same flow paths can be observed. A difference is found in the fact that more flooded areas are observed for the floodmaps obtained with the 2D HEC-RAS models. It should be noted that these additional areas concern very small water depths (in the order of millimeters).

Return period [years]	Government: Flooded area [m <sup>2</sup> ]	2D HEC-RAS model: Flooded area [m <sup>2</sup> ]
10	87461	359275
100	351878	557850
500	515905	700650

Table 4.7: Comparison between the area covered by the floodmaps for the government models and the 2D HEC-RAS models.



(a) A return period of 10 years was used. (b) A return period of 100 years was used. (c) A return period of 500 years was used.

Figure 4.14: Floodmaps computed by 2D HEC-RAS models exluding structures versus floodmaps computed by 2D government models.

# Discussion

In this chapter, the assumptions that have been made and the results of the analyzed models will be discussed. In section 5.1 the hydrological boundary conditions are discussed, followed by section 5.2 which describes the limitations and assumptions in the use of the Digital Terrain Model. In respectively section 5.4 and section 5.5 the limitations and assumptions in 1D and 2D modelling are addressed.

### 5.1. Hydrological Boundary Conditions

For the boundary conditions in the HEC-RAS models of the Rambla de la Carrasquilla, estimations based on a study of hydrological data from precipitation and the capacity of neighbouring dams were used. This study calibrated the hydrological data by 6 previous floods in the area. The examination and validation of the catchment area and precipitation data used in the study was outside the scope of this research. Therefore, a degree of uncertainty is present in the used boundary conditions.

## 5.2. Digital Terrain Model

The terrain of the Rambla de la Carrasquilla is represented by a Digital Terrain Model (DTM). Differences in land rendering can lead to distinct results for the same hydraulic conditions, because objects such as roads or flood plains are not preserved well on the map. This can have a big impact on the maximum water level in the river or flooded area, as can be seen in Figure 5.1. The flooding in a lower resolution DTM covers a bigger area and has a larger maximum water depth compared to the DTM with a higher resolution. The latter preserves specific features, such as roads, better and is therefore able to model a more detailed flooding.

Increasing spatial resolution and accuracy frequently translates to better modelling results. It can be stated that the higher the resolution, the greater the demands on processing time and data storage. Whereas a lower resolution means lower accuracy but faster run time. A coarser spatial resolution will suffice for national-scale flood maps, such that different flooding areas across countries can be compared, but also indicate which areas need more detailed assessment using a higher resolution DTM. Whereas for smaller areas of interest a DTM with a much higher resolution might be required to get accurate results.

Figure 5.1 illustrates the influence of the DTM on the resulting flood map. In this study, a DTM of 5x5 m was used to model the river and its catchment. The constant DTM is apparent when looking at the flood maps: although the resolution seems sufficient for the upstream river section, much details are lost when the flowing patterns become more complex. This happens for example in Los Nietos, where different surfaces, buildings and narrow streets are difficult to capture with a 5x5 m DTM. The effect of using a higher resolution DTM has not been investigated in this research.



(a) 10 m resolution DTM



(b) 1.5 m resolution DTM

Figure 5.1: Flooding map with the lower resolution on the left (10 m) and higher resolution on the right (1.5 m) (Nofal, 2020).

## 5.3. Photogrammetry

As was shown in subsection 3.2.4, the 3D structure models appear accurate, but have their imperfections such as holes and missing sides due to shadows and vegetation. This would have severe implications when the structures would be implemented in a 3D hydraulic model, but for the 2D model of this study the negative implications are negligible. The goal of 3D models was to obtain the (average) dimensions of structures, such that these could be implemented in the 2D hydraulic model. When varying the dimensions of the structures with the uncertainty computed in subsection 3.2.3, the output of the hydraulic model is not notably affected.

## 5.4. 1D Model

The geometry is an important aspect of the 1D Model as it forms the basis of the results. Details about the 1D model geometry set-up are discussed in subsection 3.1.2. The cross sections must be drawn perpendicular to the flow paths of the main channel and floodplains. The flow paths are not always situated parallel to each other, for example in river bends. This makes it more difficult to consistently draw cross sections exactly perpendicular to the flow paths. Additionally, no information on the exact flow paths in the main channel and floodplains of the Rambla de la Carrasquilla was available. For that reason, assumptions were made for the course of the flow paths. It may be possible that the modelled flow paths deviate from the actual flow paths in case of a flood. The width of the cross sections contain assumptions as well. This has been analyzed in section 4.1. All together, a certain inaccuracy is present in the geometry of the 1D models used in this research.

Further complications involving the 1D model are seen in the downstream reaches of the Rambla de la Carrasquilla. As discussed in section 4.1, the lowest point of the cross section is not always found in the main channel. This leads to the fact that HEC-RAS assumes flooding over the whole cross section. It should be carefully noted that for a 1D HEC-RAS model to accurately model a flooding, the river's main channel has to be the lowest point in the cross section.

A 1D model including a bifurcation was used to simulate an additional main channel, to see if this would solve the previously mentioned problem with a lower situated floodplain. This is an assumption based on the prior knowledge that the river overflowed its banks in the flood of 2019 at the location of the culvert. An obstacle that arises with this approach is found in the discharge distribution at the bifurcation. The 1D HEC-RAS model is not able to automatically compute the discharge distribution at a bifurcation. Therefore, the distribution over the two downstream reaches had to be chosen manually. This was done by checking the energy head in both reaches, as this cannot be a mismatch at a bifurcation. In reality, the river will adjust the discharge distribution in such a manner that this will not occur. For this research, the discharge distribution was chosen such that the energy head difference was minimized. However, it was not possible to exactly match the energy head of the two reaches at the bifurcation. This will

Discharge [m <sup>3</sup> /s]			
Return period [years]	Los Nietos	Rambia	lotal
10	10	12	22
100	30	38	68
500	16	100	116

have led to an inaccurate result, and the problem with the lower situated floodplain was not solved.

Table 5.1: Discharge distribution between the two different reaches in the 1D model with bifurcation.

### 5.5. 2D Model

There are several implications that follow along with using a 2D HEC-RAS model. The computation time of a 2D model compared to a 1D model is significantly higher. In order to run the same 8 hour simulation, the 2D model takes roughly 2.5 hours to run whereas the 1D model only needs 3 seconds. section 3.1 explains how adjusting the numerical time step in a 2D model relates to the time needed to run a model. When a large number of models has to be computed in a relatively short time span, 2D models could prove to be too time consuming.

Subsection 4.2.2 analyzed the difference in the flooded area when varying the values for Manning's n. To analyze this, two situations were modelled, one in which the river and floodplains have different values based on their land cover and amount of vegetation, and one situation in which the Manning's n value was assumed to be constant with n = 0.03. From this, it followed that a larger Manning's n value leads to a greater flooded area. This was as expected, as a higher Manning's n value represents a rougher surface and thus the water flow gets obstructed more. The different Manning's values were obtained from national land cover data and rough estimates based on fieldwork. As the impact of these values on the amount of flooded area is quite large, more precise estimates of the Manning's n values for the river bed could further improve the accuracy of the model.

The results obtained for the different numerical methods in HEC-RAS are shown in subsection 4.2.3. The default numerical method for 2D modelling in HEC-RAS is the Diffusion Wave Equations (DWE) solver, as it is relatively fast and inherently stable for most flood models. However, the use of the Shallow Water Equations (SWE) may lead to more accurate results for some applications. The SWE-ELM is the original version and somewhat faster than the SWE-EM. The latter uses a more conservative momentum equation, which makes the solution generally more stable and leads to less numerical diffusion. It can be very useful in determining changes in water surface elevation around structures. The 2D model with a downstream boundary condition proved to be unstable, due to large volume errors. Therefore, a stage hydrograph with a constant surface elevation of 0.5 [m] (Table 4.3) was chosen. For the SWE-EM run, a time step of  $\Delta t = 1$  [s] led to unstable results as well. The 2D model proved to be stable for a smaller time step of  $\Delta t = 0.5$  [s].

Instability in a 2D model can occur when flow through structures, such as the barrels of the culvert, are not properly aligned with the 2D mesh cells. The inlet and outlet of the culvert should be connected to a single mesh cell. If this is not the case, the model is unable to compute the flow from one cell to another, resulting in large volume errors. For all 2D model runs in this research the volume error was under 2%.

The choice for the downstream boundary condition has large influence on the stability of the model. If the chosen boundary condition for the downstream surface elevation is lower than the downstream surface elevation of the river, the water is not able to flow out of the system. For the implementation of the 2021 culvert, this proved to be a problem. To solve this problem, the stage hydrograph boundary condition, was changed to a normal depth boundary condition with a value of 0.05 m. The assumption is made that the downstream flow has reached equilibrium conditions, meaning that the slope of the bed and the water are equal. Using this assumption, it was possible to obtain a floodmap result for the 2021 culvert. In order to be able to compare the effect of the 2021 culvert with respect to the 2019 culvert and a system without structures, the normal depth boundary condition was used for all scenarios described in subsection 4.2.5.

# $\bigcirc$

# **Conclusion and Recommendations**

#### 6.1. Conclusion

The question this report aims to answer is: 'How can the flood areas and peak water levels of the Rambla de la Carrasquilla and its catchment be obtained by simulating three different scenarios based on the peak discharges using HEC-RAS models?'

Based on numerous model runs, it must be concluded that the 1D model is not very suitable for modelling of the Rambla de la Carrasquilla. Although the model gives good results for the upstream part of the reach, where the river has a rather constant geometry and the risk of flooding is low. The downstream 1D model results cannot be used to draw any valid conclusions about the flooded areas in the town of Los Nietos. The 1D model does not compute continuous floodmaps, see for example the top left of Figure 4.6a, which means the floodmap does not simulate a real life flood event. An attempt was made to obtain more accurate results in the town of Los Nietos, by adding a bifurcation in the downstream part of the 1D model. However, these results appeared not to be reliable and cannot be used to draw conclusions, as this 1D model assumes that a certain amount of the water will always flow through Los Nietos. In reality, this will only happen when the main channel of the Rambla de la Carrasquilla is overflowing. Furthermore, structures that were implemented in the 1D model did not lead to significant changes in the flooded area. The improvement of the culvert capacity (the difference between 2019 and 2021 culvert) was hardly visible in the results.

In contrast to the 1D model, the 2D model lead to much more accurate results. These results give a better and more realistic representation of what areas are in risk of flooding in the case of extreme rainfall events. When comparing the floodmap for a return period of 500 years with the floodmap obtained by the government for the same return period, the difference between the two floodmaps is rather small. This indicates that both models are seem to be reliable. For floodmaps of a smaller return period the results have larger differences, and thus the uncertainty on what areas are in risk of flooding seems to be higher. However, as discusses before the model should be validated in order to make valid conclusions.

It can be stated that it is necessary to use a 2D model when more precise and accurate predictions of flooding in cities are required. The 1D model can be very useful for computing rivers with a relatively uniform bathymetry and no flooding in a matter of seconds. However, since water flowing in only one direction cannot accurately describe water flows on the floodplains, a model using at least two dimensions is necessary. In general, 2D modelling has become a very powerful tool that can give a lot of insight in river flows, if the right data is available.

The effect of including bridges, the pipeline and culvert is clearly visible in the 2D results. These structures lead to an obstruction of the flow, forcing the water to follow a different path than the main channel and thus leading to a larger flooded area. Also the maximum depth of the flood map increases when structures are added. The replacement of the 2019 culvert with the 2021 culvert does not have a large effect. It leads to a slightly smaller flooded area but the maximum water depth remains constant.

The effect is small because the culvert is positioned at a critical location in the Rambla de la Carrasquilla, where the river would flood even if it was not obstructed by a culvert at all.

The structures were manually added to the model after their dimensions were constructed using photogrammetry. Photogrammetry is a cheap and easy alternative for laser scanning. The absolute error for the measured structures is around 2 cm, which is acceptable since the structures are relatively large.

For constructing the 1D model, the most important aspects are the placement of the bank stations and cross sectional spacing. The bank stations determine the conveyance between the main channel and the floodplains, this influences the rate at which flooding occurs. Cross sections need to be placed in a manner such that structures and river bends can be captured correctly by the model. The chosen mesh size and time step for the 2D model have the biggest impact on the results. A finer mesh will give more accurate results than a coarser mesh, but it will also lead to a larger computation time. To get a stable solution for the 2D model, an appropriate time step must be chosen such that the flow through each cell is described properly. In this report a time step of 1 second was chosen, as the model reached a stable result around this time step. Decreasing the time step further strongly increased computation times while the accuracy did not improve significantly. An overall mesh size of 25 x 25 m appears to be sufficient for areas where the risk of flooding is low, but needs to be decreased to reach higher accuracy in areas with large water flows such as the main river channel and the town of Los Nietos. Again, a trade-off between computation and time and accuracy has to be made. A mesh size of 5 x 5 m inside the river and 10 x 10 m in the urban area (Los Nietos) appeared to give results with sufficient accuracy.

To investigate the effect of the Manning's, two different models were compared. The first model containing the different Manning's n values for different parts in the river and the different land covers outside of the river. Large part of the river and surrounding area have a Manning's n value of 0.06 or higher, except of urban area close to the coast, which has a Manning's n value of 0.03 or lower. The flooded area was smaller for the model containing a constant value. It can be concluded that a smaller value for Manning's n leads to a smaller flood area.

The process of determining whether a model properly depicts the system's behaviour is known as model validation. Stated otherwise, the model results are in agreement with observed data Kerr and Goethel (2014). However, it should be noted that no definitive conclusions can be drawn from the models, since there is no data available to validate the models.

#### 6.2. Recommendations

As this research does have shortcomings, recommendations to overcome these limitations in future work will be discussed in this section. Also, some suggestions are given for future studies that can build on this research.

One major limitation of this research is the lack of accurate and precise hydrological input data. As explained in section 3.4, the peak discharges are an estimation based on historical precipitation data and may not be that precise. Although, some changes to the river system have already been made to prevent a flooding event like the one in 2019, it is advisable to document future precipitation with the accompanied water depth in the river. Also it can be useful to install some sort of tapeline at critical points in the flooding areas, to be able to map a flood better. Furthermore, a finer definition of the catchment area can help to specify the runoff into the river and thus the discharge.

As explained in chapter 5, a lower resolution Digital Terrain Model (DTM) impacts the outcome of the model results. It is recommended Alexandre (n.d.) to choose a higher resolution DTM when looking at smaller scale floodings. Since the DTM of the government model has a resolution of 2 x 2 m and the constructed 2D model only has a DTM with a 5 x 5 m resolution, the comparison is not completely correct. Also the used mesh size if of influence on the accuracy of the results, this can be further investigated to find an optimum.

The 1D HEC-RAS model did not prove to be a valid approach for simulating floodings in the Rambla de la Carrasquilla, due to the lower situated floodplains (see Figure 4.5). Therefore, in this research it was not possible to derive a reliable approach for the simulation of flood areas and peak water levels using a 1D HEC-RAS model. For future research, it is recommended to try and model a river system

with floodplains that are situated higher than the main channel. In this way, the 1D HEC-RAS modelling result will most likely not be distorted and lead to more accurate results. The comparison between this new river system and the Rambla de la Carrasquilla can be used to asses the applicability of a 1D model for obtaining floodmaps and peak water levels.

In order to construct mitigation techniques for the area around the Rambla de la Carrasquilla, 2D models with morphological adaptations in the Digital Terrain Model can be made. For example, to see if widening of the river can reduce the flooded area, the river can be widened in the Digital Terrain Model and then the results can be compared to the original situation. In this way this can be checked for different locations in the river and for different mitigation techniques to choose the most effective solution before making any physical changes to the area. Besides to widening the river, the river banks can be (temporary) heightened in critical places.

Although the uncertainty of the modelled structures already falls into our desired accuracy for the models, it would be interesting to further investigate the possibilities of photogrammetry for this purpose. Not only are the 3D models useful for obtaining measurements and visualizing the structures, while being more cost and time efficient than laser scanning, but they could possibly be directly used in in 3D numerical hydraulic models when higher accuracy is desired. To optimize this time efficiency even further, the possibility of using video (or a combination of photos and videos) could be researched.

Lastly, changing the downstream boundary condition impacts the results for both the 1D and 2D models. Even though, the selected water surface elevation is a reasonable estimation for the boundary condition. Measuring the real-time water surface elevation gives the true boundary condition and may change the outcome of the model as the water surface elevation will be rising due to drainage of water into the lagoon.

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# A

# Overview of Structures in the Rambla de la Carrasquilla

The location of the structures (bridges, pipeline and culvert) in the Rambla de la Carrasquilla can be observed in Figure A.1. Figure A.2 to Figure A.14 show the photos and 3D models with their dimensions.

# A.1. Location Structures



Figure A.1: Locations of all the structures in the Rambla de la Carrasquilla.



# A.2. Structures Present in the Rambla de la Carrasquilla

Figure A.2: Photo of the large bridge.



Figure A.3: 3D model of the large bridge.



Figure A.4: Side view of the 3D model of the small bridge.



Figure A.5: Front view of the 3D model of the small bridge.



Figure A.6: Photo of the small bridge.



Figure A.7: 3D model of the small bridge.


Figure A.8: Side view of the 3D model for the small bridge.



Figure A.9: Photo of the pipeline.





Figure A.11: Side view of the 3D model of the pipeline.



Figure A.12: Photo of the 2021 culvert.



Figure A.13: 3D model of the 2021 culvert.



Figure A.14: Side view of the 3D model of the 2021 culvert.

# B

# Structures in 1D HEC-RAS Model

The structures as modelled in the 1D HEC-RAS software can be observed in Figure B.1 to Figure B.6 in this Appendix.

Note: The smaller bridge was modelled as a culvert in HEC-RAS instead of a bridge in order to easily implement the horizontal underground and the arch shape.



Figure B.1: 1D HEC-RAS model of the large bridge.



Figure B.2: 1D HEC-RAS model of the small bridge.



Figure B.3: 1D HEC-RAS model of the pipeline.



Figure B.4: 1D HEC-RAS model of the 2019 culvert.



Figure B.5: 1D HEC-RAS model of the 2019 culvert including partially blocked openings (due to debris).



Figure B.6: 1D HEC-RAS model of the 2021 culvert.

# $\bigcirc$

# **Photogrammetry Results**

With the use of VisualSFM and Meshlab 3D model representations of the encountered structures. In this appendix, the structures that could be reconstructed with photogrammetry and measurements done with the program can be found here. The results of photogrammetry can be foun in Figure C.1 till Figure C.5.



Figure C.1: Measurements on the front of the large bridge.



Figure C.2: Measurements underneath the large bridge.



Figure C.3: Measurements of the small bridge.



Figure C.4: Front view photogrammetry result of the pipeline.



Figure C.5: Side view photogrammetry result of the culvert.

# Uncertainty Analysis Photogrammetry Structure Models

For this research, three different structures have been modelled using photogrammetry: a large culvert, small culvert and a pipe that resembles a bridge (which will be referred to as 'Pipe bridge' for convenience). For every 3D model, the relative and absolute average errors are computed using the error propagation formulas, described in section D.1. in a simple Python script which can be found in section D.2.

### **D.1. Error Computations**

The resulting errors can be seen in Table D.1.

Average error	Large culvert	Small culvert	Pipe bridge	
Relative [%]	0.043	0.029	0.025	
Absolute [cm]	2.3	1.8	1.8	

Table D.1: Average relative and absolute uncertainty of modelled structures.

The errors for each step in the pipeline are:

- 1. **Radial distortion** As mentioned, for this project the *Canon Powershot SX60 HS* was used, which corrects for radial distortion of images (Canon, 2021), so  $e_1 = 0$ .
- 2. False SIFT matches Theoretically this can be computed by counting the true and false matches between keypoints to calculate the accuracy metrics. Unfortunately, VisualSFM does not contain accuracy assessment tools in contrast to commercial softwares (Niederheiser et al., 2016). Therefore we cannot take this into account in our analysis. Note that it is also difficult to find studies in which these metrics are computed, since many studies on this topic only estimate the overall accuracy using a laser scanned model as ground truth.
- 3. **Bundle adjustment** Can be computed in VisualSFM, using the matched images. The approximated reprojection error is the average of all projected points. For the error estimation, the dataset of the small culvert was used. This yielded an average error of 3.352 pixels  $\approx 0.001 = 1 \cdot 10^{-3}$  m.
- 4. CMVS In the research on which the algorithm is based, Furukawa et al. (2010b) does describe the uncertainty analysis, but using parameters which (to our knowledge) cannot be easily extracted from VisualSFM. Therefore we choose to apply the relative error computed for a similar dataset to our models. An overview of the application on different data sets is described in (Scharstein & Szeliski, 2006). Here the relative error of a highly textured data set created from 47 images is given by 0.4 %. For our small culvert this corresponds to 2.5 · 10<sup>-3</sup> m.

- 5. **Surface reconstruction** since we are only interested in a rough approximation of the error, we can use the relative error computed for a similar data set and apply it to our model to estimate the actual error. Fortunately, screened Poisson surface reconstruction has been extensively studied. One study by Kazhdan and Hoppe (2013) describes the relative error for different data sets as function of octree depth. Here we choose to compare our model to that of the Stanford bunny, since both clouds approximately contain 0.3 million points. For an octree depths of eight to ten (which were used for our models), the relative error is 0.08%. Applying this to our small culvert, this gives  $e_5 \approx 5.1 \cdot 10^{-4}$  m.
- 6. Scaling factor This factor is the result of averaging over three different scaling factors of control points. So in each scaling factor, the error must account for the real life measurement error and for the measurement error in Meshlab. Since the real life measurement were done using a simple measurement tape, we only looked at the centimeter notations. The actual error for each of these is therefore  $\pm 0.5$  cm =  $5 \cdot 10^{-3}$  m. The measurement tool in Meshlab shows the measurement to five digits. However, it is quite difficult to draw straight lines between the points of interest. Also, there is no need for us to obtain this precision. For this reason we have decided to draw three lines under slightly different angles and look at the deviation from the mean of these measurements. The average relative error is computed from these three measurements and the relative error of the scaling factor can be computed by adding the relative errors.

#### **D.2. Python Script Error Analysis**

timport libraries import numpy as np import pandas as pd	
F %%	
<pre># general errors (all units [m]) #1 = 0</pre>	ic but negligible compared to other errors gible compared to other errors gible compared to other errors eps 1 through 5
7 %%	
def actual_error(m, m_mesh):	
Function to compute the actual errors of the structures	
Input: m = matrix of measurements between common object points (post-its : rows are measurements of between one pair of control points (1 columns are measurement nr 1 to 3 m_mesh = array of measurements in Meshlab [-]> compute mean and devia	in our case) [m] pair of post-its) ation to estimate error! min-max/mu
Output: mre = mean relative error [%] e_av = actual error for structure [m] """ n = len(m_mesh[0,:]) N = len(m_mesh[:,0]) scale_one = np.zeros((N)) re_mesh_one = np.zeros((N))	
<pre>for i in range(N):     scale_one[i] = m[i]/np.mean(m_mesh[i,:])     re_mesh_one[i] = 0.5*(np.max(m_mesh[i,:])-np.min(m_mesh[i,:]))/(np.sqrt</pre>	<pre>(n)*np.mean(m_mesh[i,:])) # RE one measurement Meshlab</pre>
MRE_mesh = np.mean(re_mesh_one) re_real = e_real/m re_scale = MRE_mesh*np.ones(len(m)) + re_real	# Mean relative error # relative error real life measurement # relative error of scaling
<pre>re_15 = e_15/m re = re_scale + re_15 mre = np.sum(re)/len(m) e_act = re * m e_av = np.mean(e_act) print('The average relative error is {:.3f} %'.format(mre),</pre>	<pre># relative error for each measurement step 1 to 5 # error per object points # average error , e_av*100))</pre>

#### D.2. Python Script Error Analysis

#%%	
<pre># structure measurements # large culvert m_lc, m_mesh_lc = np.array([0.35, 0.61, 0.61]), np.array([[0.419007, 0.397001,</pre>	0.393746], 73], 92]])
<pre># Small Culvert m_sc, m_mesh_sc = np.array([0.64, 0.64]), np.array([[2.48176, 2.44857, 2.51895] [2.52948, 2.41984, 2.63938]]</pre>	5
<pre># pipe bridge m_pb, m_mesh_pb = np.array([0.65, 0.70, 0.81]), np.array([[1.34366, 1.30098, 1.</pre>	38738], 9395], 7948]])
<pre>#% # print results print('Large culvert') mre_lc, e_av_lc = actual_error(m_lc, m_mesh_lc) print('Small culvert') mre_sc, e_av_sc = actual_error(m_sc, m_mesh_sc) print('Pipe bridge') mre_pb, e_av_pb = actual_error(m_pb, m_mesh_pb)</pre>	
# 9.95	

# export results to csv file
df = pd.DataFrame(data={'Average error': ['Relative', 'Absolute'], 'Large culvert': [mre\_lc, e\_av\_lc],
 'Small culvert':[mre\_sc, e\_av\_sc], 'Pipe bridge':[mre\_pb, e\_av\_pb]})
df.to\_csv('photogrammetry\_structure\_errors.csv', index=False)

# \_\_\_\_

### Determination of the Manning's n Values

This Appendix shows how the determination of Manning's n in the Rambla de la Carrasquilla was carried out. The composition of Manning's n per section can be seen in Table E.1. During this survey, photos were taken showing the different types of vegetation and undergrounds present in the river bed. Based on this investigation, an estimation of the riverbed friction was be made, which is needed to compute the energy loss in the model. An overview of the locations of the different sections in the river can be observed in Figure E.1. The different riverbed types of the Rambla de la Carrasquilla are seen in Figure E.2.

			irregularities	shape variation	obstructions	vegetation	meandering	∑(n)*m
ID	description	n <sub>base</sub>	n <sub>1</sub>	n <sub>2</sub>	n <sub>3</sub>	n <sub>4</sub>	m	n <sub>total</sub>
1	river delta	0,02850	0,00600	0,00000	0,00000	0,02500	1,00000	0,05950
2	vegetated	0,02850	0,00800	0,00000	0,00000	0,03000	1,00000	0,06650
3	grass bed	0,02850	0,00800	0,00000	0,00000	0,02300	1,00000	0,05950
4	rocky with vegetation	0,02850	0,01000	0,00100	0,00000		1,00000	0,03950
5	culvert							0,03600
6	rocky, trees on higher levels	0,03100	0,01000	0,00500	0,00000	0,02700	1,00000	0,07300
7	reed and many trees	0,02850	0,00600	0,00100	0,00000	0,08000	1,00000	0,11550
8	street/asphalt							0,016
9	creek through grassland	0,02850	0,00100	0,00300	0,00000	0,01000	1,00000	0,04250
10	grass bed with some trees	0,02850	0,00800	0,00100	0,00000	0,02000	1,00000	0,05750
11	bridge, concrete bed							0,02600
12	swamp, lot of grass	0,0285	0,00800	0,00000	0,00000	0,06000	1,00000	0,09650
13	muddy with lot of plants	0,02850	0,00800	0,00200	0,00000	0,06000	1,00000	0,09850
14	muddy with lot of plants	0,02850	0,00800	0,00200	0,00000	0,06000	1,00000	0,09850

Table E.1: Determination of Manning's n in the Rambla de la Carrasquilla channel based on Arcement and Schneider (1989).



Figure E.1: Overview of sections in the Rambla de la Carrasquilla with different values for Manning's n.



Figure E.2: Different riverbed types of the Rambla de la Carrasquilla

# Additional Results 1D Model

This appendix shows the results of the three different 1D model calculations for the three different return periods: 10, 100 and 500 years in Figure F.1 until Figure F.3.



(a) Floodmap for a return period of 10 (b) Floodmap for a return period of 100 (c) Floodmap for a return period of 500 years. years.





(a) Floodmap for a return period of 10 years.

(b) Floodmap for a return period of 100 years.

(c) Floodmap for a return period of 500 years.

Figure F.2: 1D HEC-RAS model where structures are excluded and the river is represented by one single reach with narrowcross sections downstream.



(a) Floodmap for a return period of 10 years.

(b) Floodmap for a return period of 100 years.

(c) Floodmap for a return period of 500 years.

Figure F.3: 1D HEC-RAS model where structures are excluded and the river is represented by a single reach upstream and a bifurcation downstream.

# $\mathbb{G}$

### 2D Mesh

The full mesh for generating results in the 2D model is shown in Figure G.1. The red lines represent break lines which were used to define areas where a finer mesh was used. The blue lines represent the boundary conditions that were implemented with upstream a flow graph boundary condition, and at the coast a stage hydrograph boundary condition. A few cross sections can be seen perpendicular on the river flow, these are the locations of the different structures.



Figure G.1: The 2D mesh used for the 2D model, including break lines, boundary conditions and the locations of structures.