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# A Bidimensional Model of the Tagliamento River

Marco Petti <sup>1</sup>, Silvia Bosa <sup>1</sup>, Sara Pascolo <sup>1</sup>

<sup>1</sup> Dipartimento Politecnico di Ingegneria e Architettura, Università degli Studi di Udine, via Cottonificio 114, 33100 Udine, Italy

silvia.bosa@uniud.it

**Abstract.** The propagation of a flood wave is a very challenging topic, crucial in managing the flood risk. In the literature, several numerical models have been proposed to deal with this issue; most of them need the roughness coefficients to be assigned by the operator. The bottom roughness calibration of floodplains and channels represents a key point for flood studies, because it can heavily influence the results of any kind of numerical simulation. In this study, a numerical model is applied to the Tagliamento River, in North-East Italy. One of the main characteristics of this river is its natural environment, which changes from a very wide braided channel in the middle course to a narrow meandering river moving towards the sea. This makes the bed roughness extremely variable along the river, with different kind of vegetation, braiding, different grain size, meandering, etc. In this regard, particular care should be devoted to the roughness coefficient attribution and calibration. In the present paper, we present the detailed step of calibration and validation of a bidimensional numerical model on the Tagliamento River. A novel method to assign and calibrate roughness coefficient is introduced. Finally, the model is validated against two main flood events occurred in 1966 and 1996.

## 1. Introduction

Flooding is worldwide recognized to be one among the most threatening and frequent natural disasters, which caused for example over 1100 deaths in Europe alone between 1998 and 2009 and huge economic losses, with over 200 major events [1,2]. In order to reduce the economic and social impact of these phenomena on the society, the European Community issued in 2007 the Directive 2007/60/EC to assess and manage the flood risk all over the continent. In particular, the Directive requires flood hazard maps, together with an indication of the water depths and the flow velocity or the relevant water discharge. One way to achieve this result is to study first the flood wave propagation along the river and then to identify areas, which could be subject to bank overflow or breach, depending on the hydraulic characteristics of flow. In this context, numerical hydrodynamic models represent a key feature in improving the knowledge of a stream, as they are able to estimate and predict water depths and velocities induced by a flood wave.

Often, the study of a flood wave propagation requires to deal with a wide domain, extending from the mountains to the river mouth, that can be in the range of 100 km. On the one hand this entails the use of a large number of computational cells, and on the other hand this arguably implies that a variety of different bottom materials may be involved in the simulation. In such situations, a technique to simplify the assignment of Manning coefficients would be very useful and time-saving.

In this study, a numerical model is applied to the Tagliamento river, in North-East Italy. In recent



centuries the river flooded several times the surrounding territories causing extensive damages and also a few victims [3]. The most dramatic events date back to September 11th 1965 and November 4th 1966, when the river overrun its banks which also collapsed in a few locations as it happened in many areas in Central-North Italy. However, floods which require monitoring and alertness occur relatively often. Since 1966, a few physical and numerical models were proposed to study the river and to prevent the flooding. Due to the length of the main course (178 km) up to now physical models have been realized on very limited areas, with all the problems that this concerns regarding boundary conditions and the understanding of phenomena, which evolve at basin scale.

Moreover, the Tagliamento river is well known for its natural environment, which changes from a very wide braided channel in the middle course to a narrow meandering river moving towards the sea, with active floodplain width ranging between 130 and 1000 m [4]. This causes the bed roughness to be extremely variable, with different kind of vegetation, braiding, different grain size, meandering, etc. A proper evaluation of bed roughness is the base for a correct evaluation of bottom shear stress, which is usually expressed by means of Manning equation, under the assumption of fully developed turbulent flow, with the Manning coefficient dependent, from a theoretical point of view, on the bed roughness only. Actually, it is well known in the literature that it is not possible to assign a proper value to this parameter, only on the base of the bottom material and in this way, all hydrodynamic models need an accurate calibration of the Manning coefficient, regardless of they are one- or two-dimensional, hyperbolic or parabolic models [5,6]. This is crucial, in order to set up a reliable model, because the uncertainty of the roughness coefficient strongly affects the hydraulic results. This is usually an iterative procedure consisting in varying the Manning coefficient, in order to fit as well as possible numerical results and observed data (e.g. [7-12]).

In the light of these observations, the present paper presents the detailed setup of a bi-dimensional flood wave propagation numerical model of the Tagliamento River. Moreover, as pointed out above, in such a large domain it is easy to deal with local variations of bed roughness due to the presence of changing bed materials (gravel and sand for example) and density vegetation in the floodplains. Therefore, particular attention is paid to the assignment and calibration of Manning roughness coefficient. Finally, the model is validated against two main flood events occurred in 1966 and 1996.

The numerical model is briefly described in section 2, while the study site is presented in section 3. In sections 4 and 5 the calibration and validation procedures are presented and discussed.

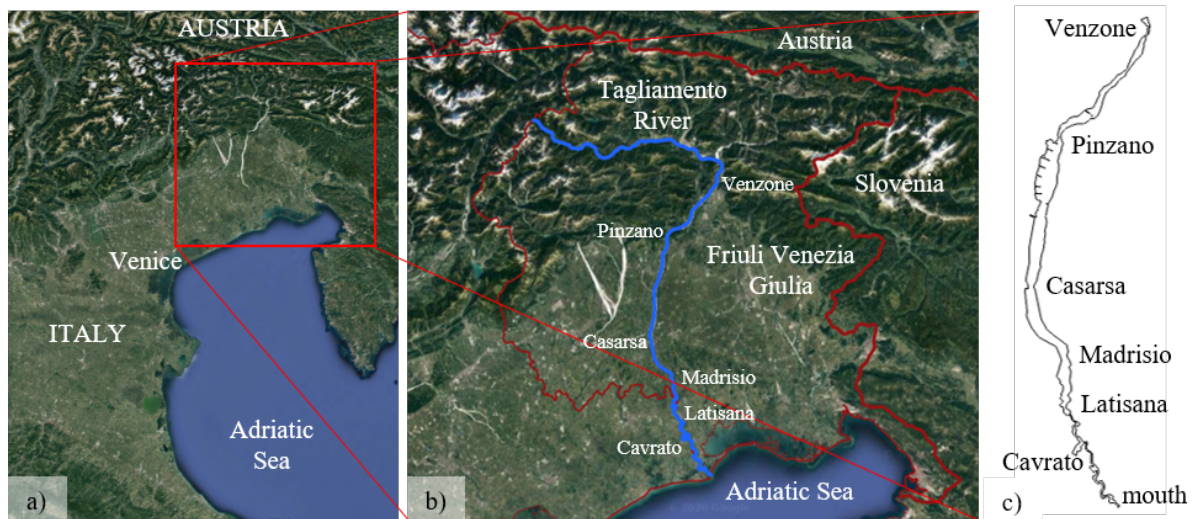
## 2. Numerical model

The adopted numerical model is based on the well-known bi-dimensional depth-averaged shallow water equations, which have been integrated over time by means of a Strang – splitting approach. A finite volume scheme has been adopted in space, which is based on HLLC Riemann solver and a hydrostatic variable reconstruction [13,14]. The same computational structure can be properly modified to study morphodynamic evolution of the bottom, also in presence of wave motion [15,16], making this model suitable to be applied in very different contexts [17-21]. The shock-capturing characteristics of the implemented model are essential to give a realistic representation of possible local changes from super-critical to sub-critical flow and vice versa as well as to assure the scheme to be well balanced also in wet and dry transition, which usually occurs in the alluvial areas during the floods. As usual, flow resistance is evaluated through Manning equation, where the value of Manning coefficient is supposed to be the same in both uniform or unsteady flow, assuming a fully developed turbulent flow.

## 3. The Tagliamento River and the field site

The Tagliamento River is located in North – East Italy, it has a 2433 km<sup>2</sup> wide mountain catchment area in the Alps and a difference in altitude of 1195 m from the source to the mouth in the Adriatic Sea (figure 1b). Its total length is 178 km, with an average slope of about 0.67%. From the source to Madrisio, in

its middle course, the bed is fully braided with gravel material. Moving towards the river mouth the bed slope decreases to 0.1‰, the sediments nature changes to sand, and the course turn to a single meandering channel. The mean annual discharge is about 70 m<sup>3</sup>/s, with two or three flood waves in a year on average. After the devastating event of 1966, there were several human interventions such as the raising of the riverbanks, to prevent further damages caused by flooding. Moreover, the 1966 occurrence is considered a 100-year return flood, which has not been exceeded in the recent past.



**Figure 1.** a) The study site in Italian context. b) The Tagliamento River. c) border of the domain considered as the computational model. The marked names indicate the locations of the water level gauges.

The modelled area includes the river and the floodplains up to the main external banks, from the upstream section of Venzone to the river mouth with a total length of about 100 km (figure 1b,c). Venzone has been chosen as the inflow boundary of the computational domain, because this is the only section where a reliable rating curve is available [22]. As a first approach, it has been chosen to neglect any possible bank overflow, which should be studied at a later time on a more detailed domain. Thus, a total reflective boundary condition has been applied to the longitudinal border of the mesh, with the only exception of the Cavarato floodway, where a cinematic outflow boundary condition has been assigned. At the river mouth, the tidal oscillation has been imposed.

The mesh has been built using the regional numerical cartography and orthophotos; the final result consists of about 90000 quadrangular irregular elements, having side dimension ranging between 5 and 150 m, where the smallest cells give a better representation of riverbanks and riverbed, and the largest are used on the floodplains (figure 2). Due to the impressive width of the middle course of the river, a large portion of the domain is usually dry almost all over the year and this made it possible to assign the bottom height on the base of laser data, whereas the submerged areas have been dimensioned through traditional topographical surveys of the river bed, performing a linear interpolation between consecutive cross sections.

#### 4. Calibration

A proper assignment of the Manning roughness coefficient is a key point in the arrangement of a hydraulic model. To a better understanding of this feature, we can observe that in case of a steady flow in a wide rectangular section, the momentum equation reduces to:

$$Q = \frac{b}{n} h^{5/3} \sqrt{S_0} \quad (1)$$



where  $Q$  is the flow rate,  $b$  the channel width,  $h$  the water depth,  $S_0$  the bed slope and  $n$  the Manning coefficient.

From equation (1), it can be easily shown that the relative error  $\Delta h/h$  on the water depth is closely related to that on the Manning coefficient,  $\Delta n/n$  as:

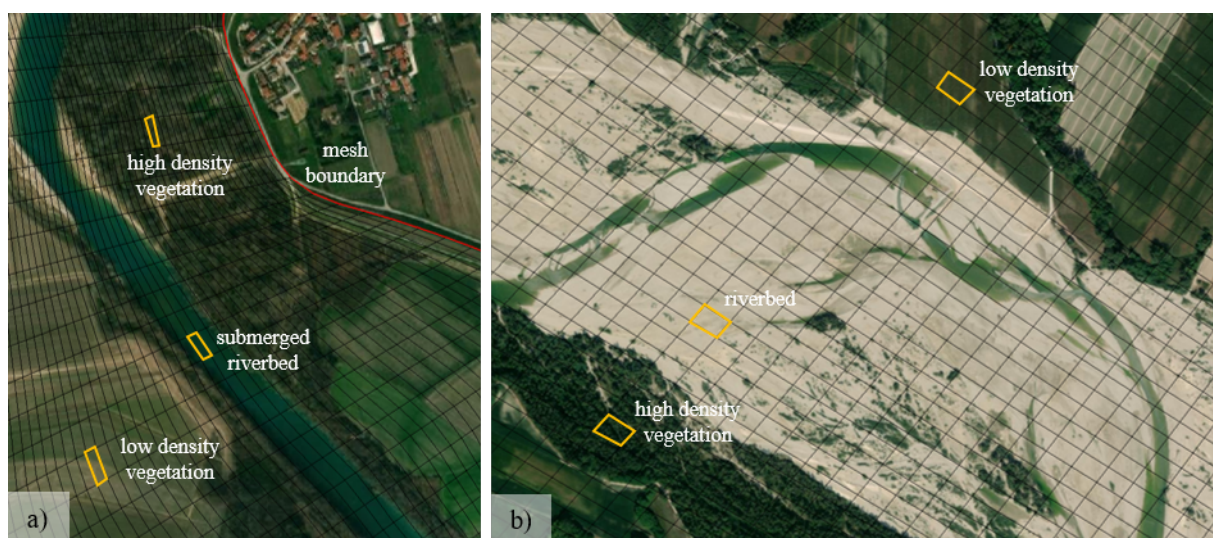
$$\Delta h/h = 0.6\Delta n/n. \quad (2)$$

For instance, if the roughness coefficient would be assigned only comparing values proposed in the literature, an error of as much as 50% could be expected on the Manning coefficient, and this alone would result in an error of about 30% on the water depth. Therefore, particular care is required in the choice of the Manning coefficient to be attributed to each computational cell.

#### 4.1 Roughness Classes

Due to the fully developed turbulent flow hypothesis, Manning coefficient does not depend on the Reynolds number, and hence, in the literature, it is often related to the sediment diameter only (for instance, [23]). If this could be an attractive choice when dealing with clear channels, it cannot be applied in natural environment like the Tagliamento river, where the flood usually inundates also vegetated areas, making the dimension of the grains insignificant in evaluating the bed shear stress. The same also applies to other environments like for example lagoons, where seagrasses influence shear stress much more than grain size [24].

Due to the extension of the considered domain, several features are encountered; in particular, four different roughness classes have been recognized: riverbed, and high-, medium- and low-density vegetation (figure 2). Moreover, considering that the size of river sediments decreases from coarse gravel to fine sand moving from the inflow section towards the sea, the riverbed roughness class has been further split into four subclasses, depending on the position along the river: Venzone – Pinzano, Pinzano – Casarsa, Casarsa – Madrisio, and Madrisio – river mouth.



**Figure 2.** Details of mesh elements and roughness classification: a) the lower meandering course; b) the wide gravelled middle course.

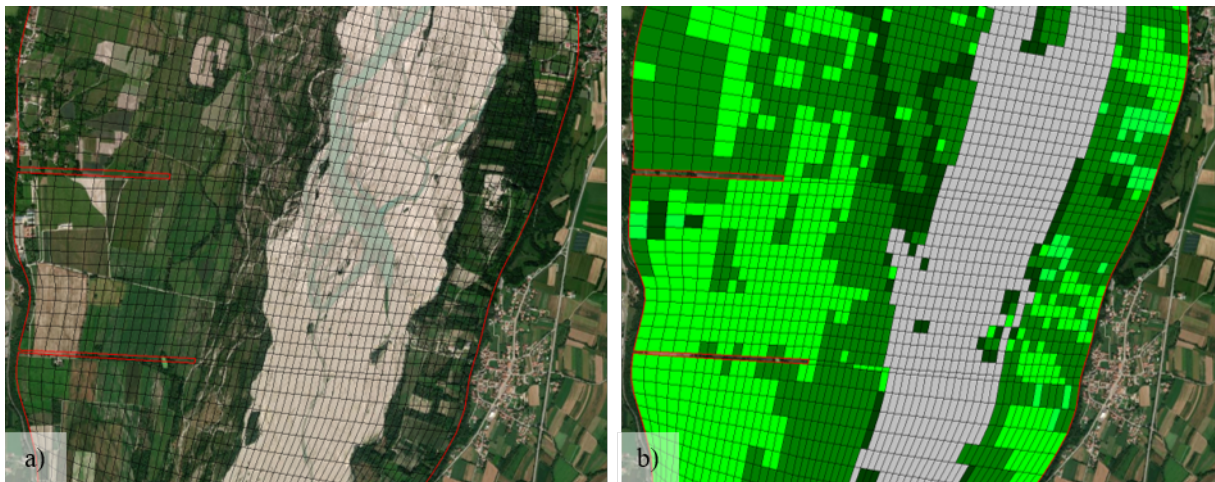
Particular attention has been paid to the method used to divide the computational grid into these roughness classes, specifically when dealing with a surfaced cell. In this case, an automatic procedure

has been identified, which considers the colours of the orthophotos as representative of particular kind of land cover.

Grey and white for instance are typical of gravel and sand of the unsubmerged riverbed, dark green is distinctive for tree, light green for bushes and brown for tilled field. With this in mind, a simple numerical code has been implemented to link the colours of orthophotos to the respective land cover. First, the orthophotos have been converted in grayscale images; then the grayscale value of each pixel has been associated to its barycentre, creating a set of scatter points, having coordinates  $(x_i, y_i, c_i)$ , being  $(x_i, y_i)$  the planar coordinates of the  $i$ -th point and  $c_i$  its grayscale code value. After that, at each mesh element a grayscale code has been assigned, as the mean value of the codes of the scatter points falling inside the cell itself:

$$\tilde{c}_k = \frac{\sum_{i=1}^{N_k} c_i}{N_k} \quad (3)$$

being  $\tilde{c}_k$  the colour assigned to the  $k$ -th mesh element and  $N_k$  the number of scatter points located inside the  $k$ -th cell. Finally, the cell has been assigned a roughness class relating to that colour. Afterwards, a further roughness class has been added to describe the cells characterized by a transition between vegetation and riverbed. The result is the roughness map shown in figure 3.



**Figure 3.** Detail of roughness map (b) obtained from orthophoto (a). The different colours mean different roughness classes.

#### 4.2 Roughness calibration

To each roughness class a Manning coefficient has been assigned, as obtained through an iterative procedure, starting from literature values according to Cowan formula [25].

To calibrate Manning coefficient, a few relative low magnitude flood events have been considered, for which ultrasonic gauges located in different survey stations along the river provided measurements of water level. In particular, table 1 summarizes some flood events, which show interesting features: for instance, events 1 and 2 have approximately the same water depth measured at the inlet section of Venzone, but very different water depths in Latisana, which is located relatively close to the river mouth. Moreover, event 3 has a lower water depth in Venzone than event 2, but the water depths measured in Latisana are reversed.

This may be due to two different aspects of the floods that take place in Tagliamento river: the shape of the hydrograph and the last part of a possible previous flood event. In particular, in Latisana the cross

section suddenly shrinks and the bed slope decreases to 0.1‰, creating a sort of natural water basin just upstream of the town. As a consequence, a large volume of water is accumulated during a flood event, and it requires at least one week to return to a steady flow condition. If in the meantime a second flood occurs, its effects are inevitably amplified.

**Table 1.** Flood events used to calibrate the roughness.

	Date	Maximum measured water depth (m)	
		Venzone	Latisana
<b>Event 1</b>	7-8 November 2000	3.68	8.92
<b>Event 2</b>	30-31 August 2003	3.60	4.84
<b>Event 3</b>	1-2 November 2003	2.84	6.20
<b>Event 4</b>	1-2 November 2004	4.32	8.28

During the calibration procedure, several runs have been carried out, iteratively adjusting the Manning coefficients to fit as well as possible the simulated and measured water level at the gauge stations. The final values of Manning roughness coefficients are summarized in table 2.

**Table 2.** Manning coefficients for each roughness class.

Roughness class	Manning coef. ( $\text{m}^{-1/3}\text{s}$ )	Roughness class	Manning coef. ( $\text{m}^{-1/3}\text{s}$ )
<b>Riverbed Venzone - Pinzano</b>	0.0294	<b>High density vegetation</b>	0.1000
<b>Riverbed Pinzano - Casarsa</b>	0.0263	<b>Medium density vegetation</b>	0.0556
<b>Riverbed Casarsa - Madrisio</b>	0.0238	<b>Low density vegetation</b>	0.0400
<b>Riverbed Madrisio - mouth</b>	0.0217	<b>Transition riverbed-vegetation</b>	0.0303

## 5. Validation

To validate the model, several major events have been simulated. In this paper we refer about the floods of November 4<sup>th</sup> 1966 and November 14<sup>th</sup> 1996.

Compared to the first flood event, which happened more than 50 years ago, the present riverbed has obviously changed, and the river banks have no more the same height. Nevertheless, in the authors opinion the modifications occurring to the riverbed have no major influence on the hydrograph in Latisana, which is actually mainly influenced by the narrowing of the cross section.

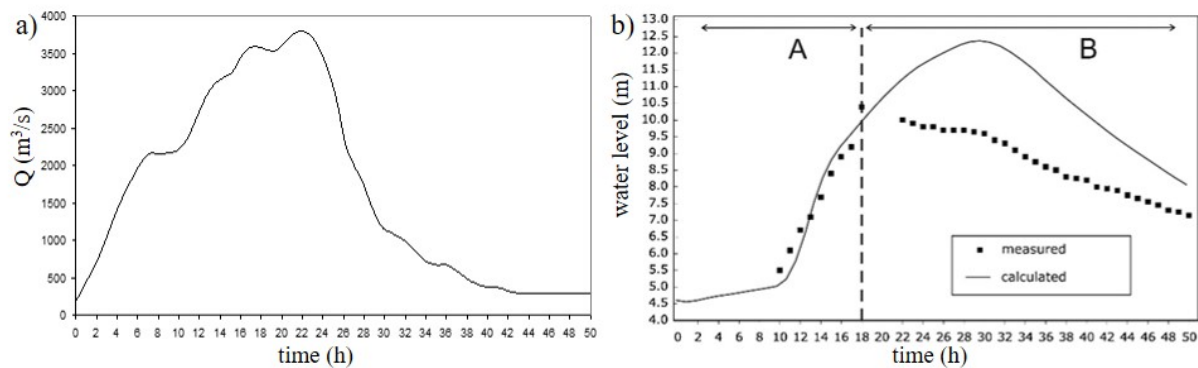
### 5.1 The flood event of November 4<sup>th</sup> 1966

The flood event of November 4<sup>th</sup> 1966 actually began the day before, in fact at midnight of the 4<sup>th</sup> almost 100 mm of rain had already fallen in a 15-hours long precipitation and the discharge in Venzone has been estimated as 200  $\text{m}^3/\text{s}$ . Thus, as the initial condition for the simulation, a steady flow of 200  $\text{m}^3/\text{s}$  has been assumed.

As the inflow boundary condition in Venzone, the discharge hydrograph has been assumed, which has been obtained from the measured water level hydrograph, with the application of the stage - discharge curve proposed by Maione and Machne [22], as depicted in figure 4a. At the river mouth, a tide of 2 m was imposed, as it was measured during the flood.

The flood lasted for about 48 hours, with a peak occurring after 21 hours estimated in Venzone as 3848  $\text{m}^3/\text{s}$ . During this event, between Casarsa and the sea, several embankments broke and overflowed, with large inundations. In particular, figure 4b shows a comparison between measured and simulated water level at the gauge located in Latisana, just after the sharp narrowing of the river. It can be observed that the measured data increase till the 18th hour (range A, 0-18 hour), when no embankment overflow

or collapse had already occurred. Later, the river broke or overflowed the banks in several sites, with a consequent decrease of the water level at Latisana (range B, after the 20th hour). As already stated, in the present simulation a total reflective boundary condition has been applied to the banks, that act as if they were infinitely high and hence, the overflowing cannot be described.



**Figure 4.** Flood event of November 4<sup>th</sup> 1966: a) inflow boundary condition in Venzone; b) measured and simulated water level in Latisana before (A) and after (B) the embankment overflow and collapse. Levels refer to the zero point of the gauge.

Nevertheless, until the overflow or collapse occurred, the numerical results seem to fit well the observed data, confirming that the numerical model is able to describe the effects of the flood in Latisana. This good agreement might be surprising considering the uncertainties in the bottom height, which could be slightly different to the actual one. However, water level in Latisana is probably mainly influenced by the strong narrowing of the river and it is not really affected by bed resistance or height.

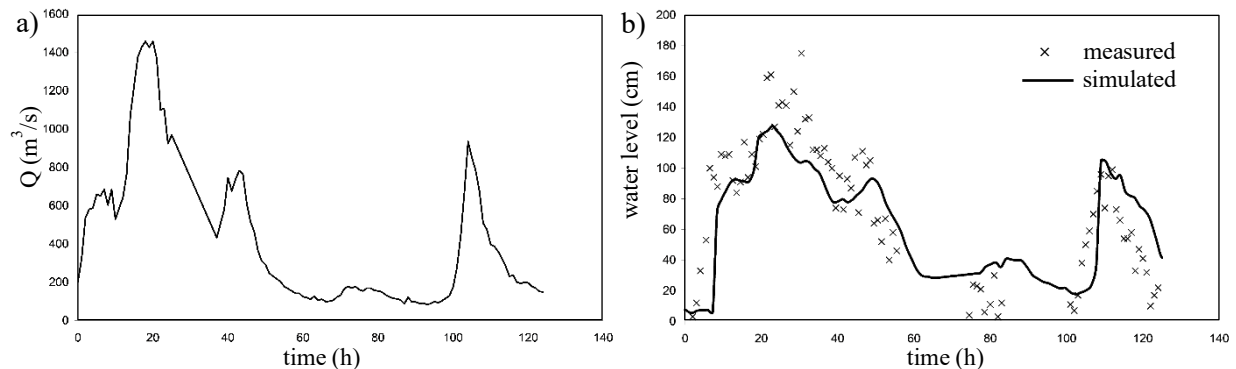
### 5.2 The flood event of November 14<sup>th</sup> 1996

The flood event of November 1996 was considerably less intense than the one of 1966 and it did not cause any damage; however, it is interesting from a hydraulic point of view, because its hydrograph shows more than one peak, as it can be seen in figure 5a. Moreover, for this event the data were available, recorded by an automatic gauge station, which was not yet working in 1966.

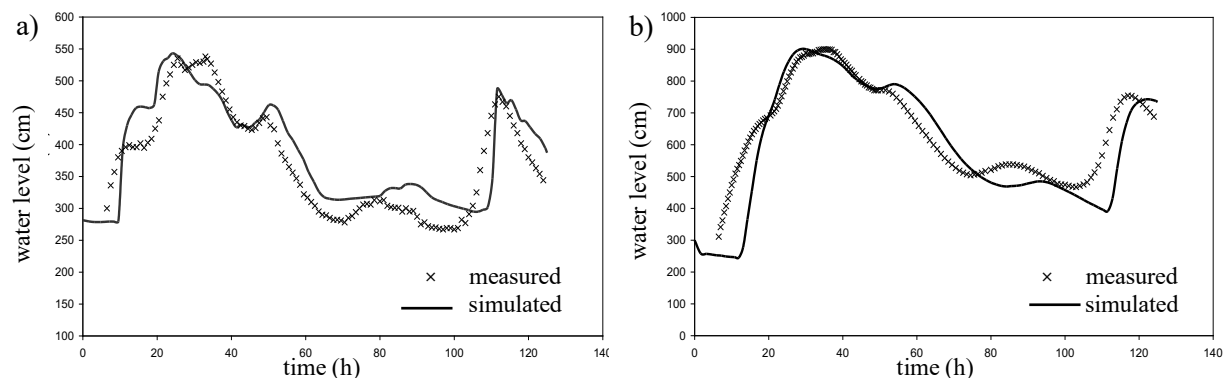
The record of the flood event in Venzone began on November 14<sup>th</sup> at 4:00 a.m., when the discharge was estimated as 200 m³/s and the measured level was 1.60 m. A first significant peak of about 1450 m³/s occurred after 20 hours, followed by a second one of 780 m³/s after about 40 hours and the last one of about 935 m³/s after 105 hours, being the overall flood duration just over 5 days (figure 5a). This discharge hydrograph has been imposed as the inflow boundary condition in Venzone, while at the river mouth a mean tidal level of 1 m has been applied, as it was measured during the flood.

The numerical results have been compared to data measured by ultrasonic gauges located along the river (in figures 5b, 6 the gauges of Casarsa, Madrisio and Latisana are shown). The overall agreement between computational results and observed data is good, with the largest deviation in the gauge of Casarsa (figure 5b). This might be mostly ascribed to the form of the cross section, which is particularly wide in this part of the course, exceeding 1000 m, causing particular shallow water depths even during floods, and making the flow field locally very irregular and turbulent. Moreover, the gravel riverbed undergoes continuous changes in the shape of the bottom, also close to the instrument measuring point.





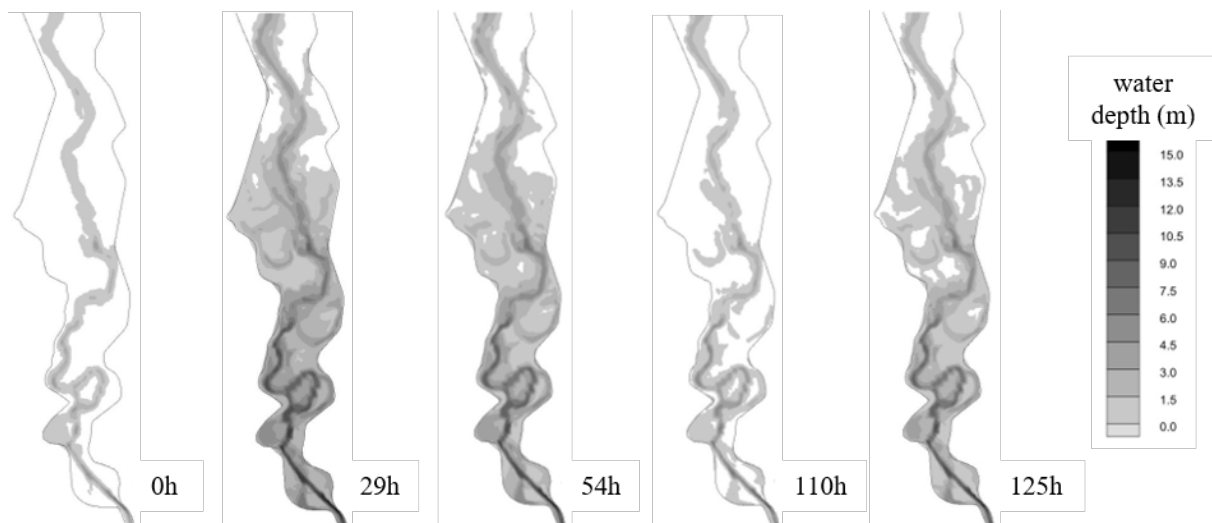
**Figure 5.** Flood event of November 14<sup>th</sup> 1996: a) inflow boundary condition in Venzone; b) measured and simulated water level in Casarsa. Levels refer to the zero point of the gauge.



**Figure 6.** Flood event of November 14<sup>th</sup> 1996: measured and simulated water level in Madrisio (a) and Latisana (b). Levels refer to the zero point of the gauges.

The fitting between simulated and measured water level considerably improves moving towards the river mouth, showing an overall good agreement at the gauge of Madrisio, where all peaks are well described by the model (figure 6a). Also at the gauge of Latisana (figure 6b) numerical and observed levels show a good correspondence, with a slight advance in the first numerical peak and a delay of about 4 hours in the last flood peak.

In all simulated flood events, the most critical reach of the Tagliamento River is where the town of Latisana is located. This is mainly due to the simultaneous sudden change in the width of the cross section, that shrinks to 175 m in Latisana, and the decrease of bed slope, which reaches 0.1%. In these conditions, during a flood the flow slows down after Latisana, and the water accumulates causing the filling of the flood plain upstream of the narrowing. In the receding limb of a flow event, after the peak is reached, the stored water slowly begins to drain, towards the sea. Preliminary simulations showed that some days are required to completely dry the flood plain and to restore the steady flow water levels. Thus, if a second flood peak occurs before the first one is run out, the danger of flooding significantly increases. This circumstance is well represented in figure 7, where it is evident that the first flood wave is not yet finished when, at time 110 h, the second one arrives.



**Figure 7.** Flood event of November 14<sup>th</sup> 1996: water depth between Madrisio and Latisana at different points in time.

## 6. Conclusions

A bi-dimensional numerical model of the middle and lower reach of the Tagliamento River between Venzone and the mouth has been presented. The work focused on the calibration and validation procedures, which are a key point in the set-up of a river flow model. In particular, the peculiar methodology for assigning the bed roughness Manning coefficient to the computational cells has been presented and discussed. Roughness coefficients have been calibrated with respect to water level in a number of relative low magnitude flood events, for which water level registrations were available in several gauge stations along the river. Furthermore, the model has been validated against some large magnitude floods, including the 1966 and 1996 events, which are briefly described. The former was the historical flood of 1966, which caused several bank failures and overflows. The latter was a more recent and less destructive event, occurred in 1996, which consists in the superposition of two floods, within a very limited time interval. The temporal proximity of the events caused the first flood not to be completely run out, when the second one came up, causing an emphasized increase in water depth during the second flood, and providing hence a possible explanation of why higher water depth can be reached, even in presence of lower peak flows. This fact is of particular importance close to the river mouth, where the slope generally decreases, and possibly near strong narrowing of the riverbed, as it happens in Latisana.

## Acknowledgments

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