Flood hazard analysis for river systems

Application to the territory of Comunità Montana Valtellina di Tirano

Ilaria Poretti

Università degli Studi di Milano-Bicocca
Facoltà di Scienze Matematiche, Fisiche e Naturali
Dipartimento di Scienze dell'Ambiente e del Territorio

Dottorato di Ricerca in Scienze Ambientali
Ciclo XXIII (2008-2010)

ANALISI DI PERICOLOSITÀ PER FENOMENI DI ESONDAZIONE FLUVIALE

FLOOD HAZARD ANALYSIS FOR RIVER SYSTEMS

Ilaria Poretti
Matr N. 033261

Tutor:
Dott. Mattia De Amicis

Coordinatore:
Prof. Marco Vighi

A Corrado
e ai miei genitori

Abstract

Flood hazard assessment and mapping is a necessary step to define flood risk reduction strategies and to develop risk management plans. Anyway, in Italy, and in particular in Lombardy Region, legislation provides only vague indications on how to assess flood hazard, therefore the definition of risk is lacking in a scientific basis, and wide space is left to subjectivity and to approximate analyses. This PhD research aims to improve the topic presenting an approach for flood hazard analysis and mapping that fits the Lombardy Region legislative framework, but introduces a level of experimental modelling. The approach has been applied on an area located in the medium Valtellina (Alps, northern Italy) – 26 km^2 wide – and makes use of advanced flood modelling tools, in order to support the development of Emergency Plans and to provide suggestions to deepen the analyses required for Urban Planning.

Hydrologic and hydraulic conditions of the site are quite complex, and data availability is not optimal. Therefore, several modelling strategies (1D, 2D and combined 1D2D approaches) and three software packages (SOBEK, FLO-2D and FloodArea) were tested and results were compared and discussed. Lots of efforts have been spent in trying to define an accurate topographical description: a TIN was constructed from available 3D cartography and cross sections profiles, then converted into a DEM. Institutional values of peak discharges for the return times of 20, 100 and 200 years were used to construct input hydrographs. Roughness coefficients were set according to literature tables and available local studies, and their influence on models behaviour was tested through sensitivity analyses. Difficulties related to some of the models and/ or verification of inappropriate results led to exclude two software packages and to select SOBEK 1D2D as the most suitable tool for flood modelling in the study area. Results were converted into hazard maps useful for both the purposes of Civil Protection and Urban Planning, basing on an innovative method, including an expression of uncertainty. Most of the complexities of the issue are analysed and discussed, referring to a wide literature background, which the research will contribute to enrich.

Table of contents

List of figures and tables

CHAPTER 1
Introduction

"*Scientific studies are providing evidence that extreme flood events are becoming increasingly common and severe, and more frequent and more intense phenomena are to be expected. Such extreme events are bound to affect the economy and the lives of European citizens. We have to act jointly, on the European, national, regional and local levels, to prevent and mitigate future flood damage. We must learn to live with floods, and thus must think and act more preventively in order to mitigate their consequences. More research is necessary to enhance our flood management and early warning capabilities.*"

Philippe Busquin, European Research Commissioner, 13 October 2003

1.1 Context

A riverine flood is a temporary overflowing of water onto land that is normally dry, due to water flows which exceed the river channel capacity.
Floods are one of Europe's most widespread disasters. Major flooding has occurred nearly every year somewhere in our continent during the previous decades. An interesting research was presented by the European Commission in 2003, with the aim of looking into how better to prevent, predict, mitigate, adapt and manage floods. For the period 1980-2002, the greatest number of floods occurred in France (22%), Italy (17%) and the UK (12%). The highest number of fatalities occurred in Italy (38%), followed by Spain (20%) and France (17%). The greatest economic losses occurred in Germany and Italy (both 11 billion of euro), followed by Spain and the UK (both around 6 billion of euro). It emerges, therefore, a quite unpleasant position of Italy in the ranking of European countries mostly affected by floods and their negative consequences.
Italian legislative framework ascribes river basin authorities the task to assess general hydrologic and hydraulic conditions of pertinence basins, and to define intervention measures to reduce risks to people, infrastructure and economic activities, which could be structural, i.e.

physical interventions on the river system or non-structural, i.e. land use planning and prevision/prevention activities, supported by Civil Protection. All these measures, established at a regional, provincial, and municipal level, are based on the estimate of the intensity and temporal probability of occurrence of the expected phenomena, which is generally referred to as the "hazard". When the aim is to define non-structural measures, hazard assessment not always makes use of best available technologies, and it is usually considered an ordinary analysis to be treated with engineering approaches or merely geomorphological observations, both of which often operate at spatial scales which are not appropriate. The unavoidable theme of uncertainty, moreover, is generally ignored. It seems, however, that the issue is so complex and important to deserve a more in-depth and interdisciplinary treatment.
Hazard is generally assessed making use of numerical models, which are being developed at always higher levels of complexity. Anyway, the use of advanced approaches is often limited to research environments, while practical applications for local government plans or civil protection activities are few. A main reason for this trend could be that flood modelling is quite complex, and usually models with a huge background of experience are preferred to newer and more composite ones which could be more appropriate, but still need to be tested on various contexts and data availability.

1.2 Research objectives

The main objective of the PhD research project is to test some advanced approaches for flood modelling on an area located in the territory of the Mountain Consortium of Valtellina di Tirano, in northern Italy. This area experienced floods in the past, and due to typical conditions of alpine valleys similar events could reoccur in the future. For the development of its Civil Protection Plan, the Consortium established a convention with the CNR-IDPA of Milan and the Department of Environmental Sciences (DISAT) of the University of Milano-Bicocca, which includes the hazard assessment of natural risks. This allowed to establish a sound and profitable cooperation with territory managers from the various Municipalities, and to make use of available local data and information.
Flood modelling results should be useful for the development of flood event scenarios for civil protection purposes, and should also provide suggestions to improve hazard maps for urban planning. For these reasons, the research had to observe the national and regional legislative framework for the assessment of hydrogeological hazards.
A second objective of the research is to compare the performances of three flood modelling software packages, in order to establish their

appropriateness to treat a territorial context similar to the one represented by the study area.

1.3 Outline of the report

This PhD report is organized in nine Chapters. After the Introduction (Chapter 1), a summary of the national and regional legislative framework for urban and civil protection planning is presented, and main deficiencies and possibilities to improvement are highlighted (Chapter 2). Then, numerical approaches for flood modelling are described and discussed, with their positive aspects, limitations, and proper application contexts; moreover, necessary data are listed, and problems related to the calibration phase of models are analysed (Chapter 3). Chapter 4 presents and describes the three software packages used during the research activity, which are SOBEK, FLO-2D and FloodArea.
Then, the study area is characterised, with a particular reference to the complex situation of the river system (Chapter 5). The methodology applied to perform flood modelling is then presented (Chapter 6), and results from various models are compared and discussed (Chapter 7). Basing on the most reliable results, several maps are produced both to support urban planning and the construction of event scenarios for civil protection purposes (Chapter 8), which include main uncertainties. In the Conclusions (Chapter 9), a summary of the activity and main research outcomes are presented and discussed, and suggestions for future developments are provided.

CHAPTER 2
Flood risk legislation in Italy and Lombardy Region

2.1 Introduction

The Italian legislative framework is facing the novelties introduced by the D.Lgs. 23 February 2010 n. 49, which acknowledge the European Directive 2007/60 about the assessment and management of flood risk, so some new regulations are expected, but at the moment the topic of flood risk assessment is governed by the application of Hydrogeological River Basin Plans (L. 183/89, art. 17) at local scale. These Plans define the areas where floods could be expected for major rivers (these areas are called *Fasce Fluviali*, in Italian) on the base of statistical expected occurrence and simple hydraulic modelling, and provide land use regulations. Duties for Provinces and Municipalities are defined independently by each Region on the base of the portion of river basins they comprise. Regions provide regulations also for Civil Protection and Emergency Planning, basing on general operational principles defined by the State Minister. Procedures for a detailed flood hazard and risk assessment, therefore, have to be sought generally at a regional legislative level.

This Chapter contains a summarised analysis of legislation regarding flood hazard, mainly at a regional level, with references to the specific context of the study area.

2.2 European Directive and Italian acknowledgement

The Directive 2007/60 is the first legislative measure for flood risk assessment and management at the European level. Its objective is to establish a framework which will reduce the adverse consequences of floods on human health, the environment, cultural heritage and economic activities. The Directive recognises that “floods are natural phenomena which cannot be prevented; however, some human activities […]

contribute to an increase in the likelihood and adverse impacts", and that "concerted and coordinated action at Community level would bring considerable added value and improve the overall level of flood protection". It also reaffirms some wise concepts, such as that "flood risk management plans should focus on prevention, protection and preparedness. With a view to giving rivers more space, they should consider where possible the maintenance and/or restoration of floodplains, as well as measures to prevent and reduce damage to human health, the environment, cultural heritage and economic activity", and that "Member States should base their assessments, maps and plans on appropriate «best practice» and «best available technologies» not entailing excessive costs". In this context, "flood risk" means "the combination of the probability of a flood event and potential adverse consequences for human health, the environment, cultural heritage and economic activity associated with a flood event".
State Members have to define competent authorities and units of managements (river basin districts), and subsequently accomplish three phases, whose outputs have to be updated every six years:

<u>phase 1</u>: **PRELIMINARY FLOOD RISK ASSESSMENT** (by 22 December 2011). This step should be undertaken basing on "available or readily derivable information, such as records and studies on long term developments, in particular impacts of climate change on the occurrence of floods", with the aim to "provide an assessment of potential risks" and to define critical areas where vulnerable elements are present.

<u>phase 2</u>: **PRODUCTION OF FLOOD HAZARD AND RISK MAPS** (by 22 December 2013). Hazard maps should "cover the geographical areas which could be flooded according to the following scenarios: (a) floods with a low probability, or extreme event scenarios; (b) floods with a medium probability (likely return period ≥ 100 years); (c) floods with a high probability". For each scenario, flood extent, water depth and flow velocity and/or relevant flow directions, where appropriate, should be shown. Risk maps "shall show the potential adverse consequences associated with flood scenarios" and should be expressed mainly in terms of "indicative number of inhabitants potentially affected, type of economic activities of the area potentially affected, and installations which might cause accidental pollution in case of flooding".

<u>phase 3</u>: **DEFINITION OF FLOOD RISK MANAGEMENT PLANS** (by 22 December 2015). On the base of previous maps, Plans should be established which include measures aiming at "the reduction of potential adverse consequences of flooding for human health, the environment, cultural heritage and economic activity, and, if considered appropriate, on non-structural initiatives and/or on the reduction of the likelihood of flooding". These Plans "shall address all aspects of flood risk

management focusing on prevention, protection, preparedness, including flood forecasts and early warning systems and taking into account the characteristics of the particular river basin or sub-basin. Flood risk management plans may also include the promotion of sustainable land use practices, improvement of water retention as well as the controlled flooding of certain areas in the case of a flood event". When units of management fall entirely within a Member State's territory, "one single flood risk management plan, or a set of flood risk management plans coordinated at the level of the river basin district" could be produced.

The European Directive has been acknowledged in Italy by the D.Lgs. 23 February 2010 n.49. It fixes an advance of three months for the conclusion of each phase comparing to the Directive, and identifies competent authorities as the River Basin Authorities defined by L. 183/89 art. 12, and units of management as the river basin districts defined by D.Lgs. 3 April 2006 n. 52, art. 63. Regions, in cooperation with the National Dept. of Civil Protection, are responsible for the meteorological warning system. Hazard maps should be produced at an appropriate scale, not lower than 1:25000, with an optimal choice in 1:10000, and low/medium/high probability should refer to return times of > 500 years, 100-200 years, and 20-50 years respectively. Risk maps should adopt the classification proposed in the D.P.C.M. 29 September 1998, which relates to expected damages. As stated by the Directive, Flood Risk Management Plans could be produced for sub-basins instead of the whole district, when appropriate, and should be in compliance with already defined territorial plans. At the moment, provisions regarding which methodologies to apply to perform hazard analysis are not provided, but following the Directive indications it seems that best available approaches should be applied, whenever possible.

2.3 Hydrogeological Po River Plan

In the present national legislative framework, river basins are managed by River Basin Authorities (L. 183/89), who are organisations where Regions and Local Authorities cooperate to define planning strategies and interventions for the use and safeguard of natural resources and the protections against hazards.
The Hydrogeological Basin Plan (called PAI, 2001) identifies high hazard areas and provides regulations and limitations for land use and development. Its aim is to reduce hydrogeological risk within the basin by directly involving Municipalities, through the compliance of their urban planning provisions. Regarding floods, PAI analyses principal rivers within the pertaining basin and delimits areas where overflow could be expected (these areas are called *Fasce Fluviali* A, B, C in Italian, from the highest

to the lowest degree of hazard, see Figure 2.1) for high discharge events with different return times, supposing interventions proposed for the reduction of risk are realised. In particular:

- *fascia* A is the area occupied by ordinary floods;
- *fascia* B is the area that could be inundated by the "project flood" (generally, a 200 years return time flood, except in some cases when it is 100 years return time);
- *fascia* C is the area possibly flooded for higher return time flood events (500 years or the worse flood ever recorded).

For the Po Basin, *Fasce Fluviali* are defined by one-dimensional numerical modelling and geomorphological considerations, basing on topographical knowledge available at the time. Within these areas precise regulations have been established to ensure a satisfying level of safety for human beings and holdings, a dynamic equilibrium condition for the river system and a proper space to let the water overflow without causing negative effects in case of flood events. Flood hazard for minor (i.e. torrential) rivers is also identified related to the Municipality it belongs to, even if it is not always bounded, due to the frequent lack of historical evidences.

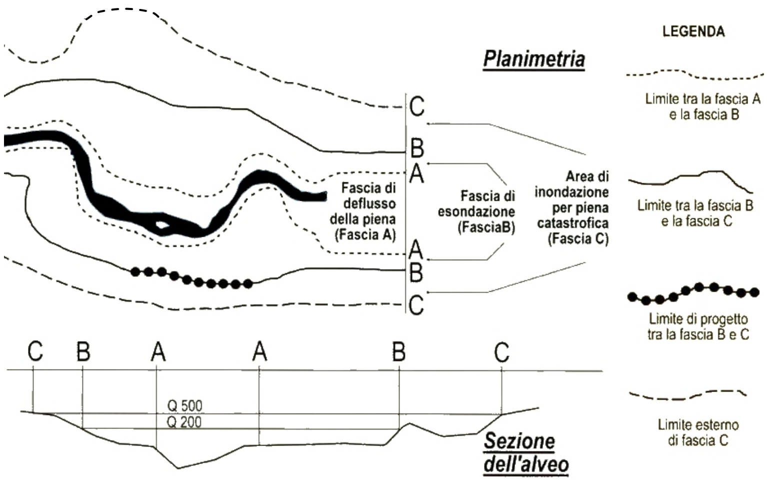


Figure 2.1 – Fasce Fluviali (PAI, 2001).

Within the Regional Legislative framework, PAI regulations are binding and must be incorporated by Municipalities in their Urban Plan (called *Piano di Governo del Territorio*, PGT, defined by the Lombardy Region with L.R. 11/03/2005, n. 12) through a process called "compatibility verification". Municipalities whose Urban Plans are not in compliance with PAI identification and delimitation of hazards, should update the Plans with new geological reports and provide new "hydrogeological disruption" maps (D.G.R. 22/12/2005, n. 8/1566).

2.4 Regional and Provincial Territorial Plans

The task of Regional and Territorial Plans is to deepen the knowledge of hydrogeological processes acting on the territory under their jurisdiction, and to define intervention measures to protect from potential hazards, in compliance with river basin regulations. These Plans are called *Piano Territoriale Regionale* (PTR) and *Piano Territoriale di Coordinamento Provinciale* (PTCP) for the regional and provincial level, respectively.

2.5 Flood risk analysis for Urban Plans

At the Municipal scale, flood hazard and risk analysis has to be performed in order to support the Urban Plan (PGT) which contains regulations for land use development, consistently with Provincial and Regional analogous Plans. Hydrogeological analysis should be carried out following the prescriptions of D.G.R. 22 December 2005 n. 8/1566 (updated by D.G.R. 28 May 2008 n. 8/7374). For the particular case of floods, it requires to accomplish the following phases:

ANALYSIS: it consists in historical and bibliographical survey and production of overview maps at 1:10000 scale. It is recognised that historical data are particularly necessary since they are used to calibrate flood models. Maps should indicate both for principal and minor rivers the fluvial pertinence areas identified by means of data relating to past events, geomorphological considerations or calculated making use of methods indicated in Enclosure 4 (see below), referring to the return time of 100 years. Critical areas subjected to erosion, and locations where hydraulic works and gauging stations are present should also be identified. A deeper analysis (rules are contained in Enclosure 4) is optional, except for five cases, in which it is mandatory:

- when there is a willingness to re-delimitate areas bound by PAI or provincial/regional Plans;
- when there is a willingness to reduce the level of risk defined for urban development;
- when urban areas are lying in *Fascia Fluviale* A or B;
- when *Fascia Fluviale* B is "*di progetto*", i.e. when its delimitation depends on the realization of physical protection measures;
- when analysing risk conditions for minor rivers.

It is suggested to make use of hydro-morphological data provided by the SIBCA (Informative system for basins and rivers) regional SIT, but they are based on hydrological raster analysis performed automatically on a 20 m grid and its use is quite complicated. Moreover, due to the way in which they are derived, some doubts can arise about the correspondence to truth of these data.

SYNTHESIS / EVALUATION: on the base of results from the previous phase, a synthesis map is produced containing a “hazard” description for the main hydrogeological processes acting on the area. The term is in quotation marks since it does not correspond to the scientific concept of hazard, even if it gets quite close, since it only requires to delimitate possible flooded areas basing on return times.
In detail, it is necessary to define:

- areas repeatedly flooded in the past or which could be flooded frequently (return time less than 20-50 years), and/or with significant water depths and velocities and/or considerable solid transport;
- areas flooded in case of extreme meteorological events or which could be flooded less frequently (return time of more than 100 years), and/or with limited water depths and velocities which should not produce damages to people, buildings, infrastructure and economic activities;
- areas that could be flooded basing on geomorphological considerations, considering critical points within the river due to erosional processes, possible bank failures, overtopping, presence of obstructing material, inadequately dimensioned cross sections, etc.;
- areas flooded in the past on the base of historical evidences;
- areas subjected to erosional processes and not adequately protected.

This analysis could produce a useful and efficient description of hazard, but since no methodological indications are provided in order to define the required areas, a subjective approach is usually applied, and this results in maps with different legends and delimitations, which are highlighted at Municipality boundaries. Moreover, in some cases, not all the areas are represented (since probably not enough data are available) or they are grouped together.

PROPOSAL: a final proposal map is produced, which contains regulations and limitations for land use and development basing on the results of the previous phase, considering also bonds defined by PAI or Provincial and Regional Plans. In theory, there should be a clear correspondence among areas defined in the synthesis map and proposal classes, but in practice there could be differences arising from the different contents of the synthesis maps (see, as an example, par. 5.3 and Figure 5.13). Another problem affecting the definition of the synthesis map is that there is a low degree of freedom to interpret and treat locally *Fasce Fluviali*, which have to be traced exactly: only limited modifications are allowed when it is possible to refer to topographical details not comprised in the PAI analysis which could limit flood propagation, but not in the case of new hydrologic or hydraulic studies. The only exception to this rule, so further studies are allowed, is when *Fascia* B is "*di progetto*" or urban areas falls within *Fascia* A or B.

As stated before, Enclosure 4 of the D.G.R. contains rules to conduct in-depth flood hazard analysis. In absence of other indications, they should be applied every time this analysis is required by the three phases described above and not enough data or previous studies are available for the purpose. This rules are complementary to PAI Directives "Criteria to evaluate the hydraulic compatibility of public infrastructure within *Fasce* A and B" and "Directive about the project flood to be used for hydraulic compatibility verifications". The final aim is to obtain water depths and velocities for expected floods characterised by different probabilities of occurrence (return times).
First, the expert should collect all the available documentation: PAI peak discharges and *Fasce Fluviali* should be adopted without modifications, but the expert can decide to perform new studies, when the available description of hazard is considered inadequate, focusing on the specific aim of the study.
Referring only to prescriptions for hydraulic (not hydrologic) analysis, a detailed topographic survey should be carried out and a calculation approach should be chosen being as complex as required by river conditions: simulations could thus run in steady-state (constant discharge but topographical variations along the channel are allowed) or unsteady (the discharge can vary over time). As an input, both a constant discharge or an hydrograph could be supplied. Flooded areas are bounded following:

- a simplified approach, i.e. performing a 1D analysis and then comparing water levels with floodplain elevations and morphology, integrating with knowledge on past events;
- an intermediate approach, i.e. 1D modelling with possibilities of bank overflows and expansion of flow volumes;
- a detailed approach, i.e. 2D modelling, in case of particularly complex situations.

This is the only mention of possible modelling approaches to be applied, but it is up to the expert to decide which one to choose, and this is not an easy task since many issues and problems should be considered.

2.6 Civil Protection regulations

In Lombardy Region, L.R. 22 May 2004 n. 16 "*Testo unico delle disposizioni regionali in materia di protezione civile*" and "*Direttiva regionale per la pianificazione di emergenza degli enti locali*" (approved by D.G.R. 16 May 2007, n. 8/4732) are currently in force for the topic of civil protection organization and planning. The first law defines responsibilities of Local Authorities: in particular, it establishes that Regions should provide a Plan for Prevision and Prevention about natural risks, which should be deepened by a similar provincial Plan, together with a provincial emergency Plan. For Lombardy in general, and for the Province of Sondrio (in which the study area lies). In particular, these Plans actually represent areas at risk of flooding by tracing out PAI *Fasce Fluviali*, even if a more detailed hazard analysis, in theory, is allowed (L.R. 11 March 2005 n. 12, art. 56). Sondrio Province emergency Plan is not available yet, so Municipalities and Mountain Consortiums should prepare their own civil protection Plans following the prescriptions of the *Direttiva* 16 May 2007 cited above. Also in this case, lots of indications are provided for the operational part of the Plan (procedures and activities to perform in the emergency phase), but no rules are defined to assess preventively flood hazard, except for the indication that in order to activate the emergency phase, in complex or particularly wide areas, numerical models for flood propagation or for rainfall-runoff analyses could be applied. Apart from that, a list of already available studies and documents to refer to in order to retrieve data is provided.

2.7 Conclusions

The legislative analyses carried out allowed to understand how the “flood hazard” topic is treated by regional laws related to territorial and civil protection planning. Hazard assessment and the development of hazard maps, even if it is not always clearly specified, represents the base for all planning purposes, but a clear definition of what methodologies to apply, except for the hydrogeological analyses requested by the PGT, is not provided. This seems to be a severe gap since there is no encouragement to apply best available technologies. Usual studies, when implemented, refer to 1D flood modelling which has generally accepted limits, or other approaches are applied only at research level. It is hoped that approaches experimented in research contexts will converge into national documents containing guidelines on how to apply more advanced modelling tools at various scales, basing on available data and resources, in order to improve flood hazard assessment performed by Local Authorities. These guidelines are provided, instead, for countries like Great Britain (Néelz and Pender, 2009(1); Asselman, 2009; Hagemeier-Klose and Wagner, 2009) and Germany (LAWA, 2006) and partially originate by the results of European Projects (e.g. FLOODsite 2004-2009, www.floodsite.net). The work presented in this thesis aims to give a contribution for this progress and communication of knowledge related to flood modelling tools applied at a local scale.

CHAPTER 3
Flood modelling

3.1 Introduction

In order to produce flood hazard and risk maps, and especially where few events occurred in the past, not allowing to define critical areas on the base of historical knowledge, inundation models are indispensable (Asselman, 2009).
As a general definition, a model is a simplification of reality for the purpose of making it more comprehensible. It consists of a user's best estimate of the processes that are perceived to be relevant to the particular application. These processes are typically a small subset of the known physical mechanisms. The key step in selecting an appropriate numerical modelling framework for flood event analysis is therefore to identify relevant processes and to decide how these can be discretized and parameterised in the most computationally efficient manner.
Since several modelling approaches exist, and many software packages and tools are available both in the research and commercial environment, is it often difficult to choose the most appropriate one, and to apply it in the proper way. This is even more true when the context in which a model is applied is far from the ideal one, for lacking or low quality of data.

Asselman (2009) provides a good summary of flow processes occurring in natural channels.
Areas subject to floods normally consist of a main channel and adjacent floodplains, where water flows when bank full height is exceeded (Figure 3.1). During a flood, floodplain may either act as storage or an additional means of conveyance. In the language of fluid dynamics a flood is a long, low amplitude wave (a kinematic wave with a diffusion component) passing through a compound channel with complex geometry (Bates and de Roo, 2000). Flood waves are translated downstream and attenuated by frictional losses such that in downstream sections the hydrograph is flattened out (Figure 3.1).
In-channel processes include: the formation of shear layers at the junction between the main flow and slower moving dead zones;

secondary circulations at the scale of cross sections; turbulent eddies ranging from heterogeneous structures at the scale of roughness elements; obstructions on the bed.

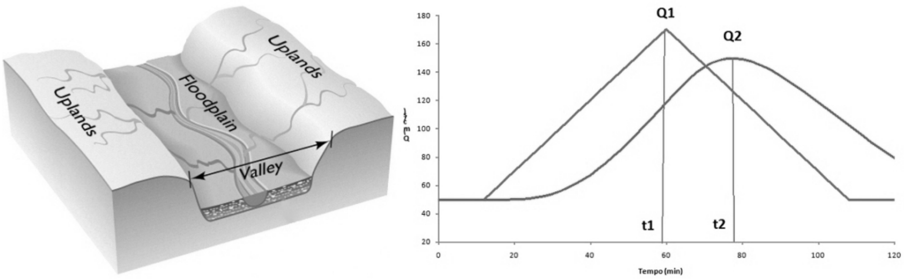


Figure 3.1 – Definition of river channel and floodplain (on the left); translation and attenuation of a flood wave (on the right).

In case of overflow, in addition to the above processes new physical mechanisms occur: momentum exchange between the fast moving channel and slower floodplain flow; interaction between meandering channel flows and flow on the floodplain. It is proved that failure to account for the momentum exchange can lead to errors of up to ±25% in the discharge calculated using uniform flow formulae such as the Manning and Chézy equations. Further vigorous momentum exchange occurs during out-of-bank flow in meandering compound channels; here, water spills from the downstream apex of channel bends and flows over meander loops before interacting with channel flow in the next meander. These three-dimensional interactions modify secondary circulations within the channel and represent an additional energy loss in the near channel area. Floodplain flows beyond the meander belt will not be subject to such energy losses and this region may provide a route for more rapid flow conveyance. The impact of these additional energy losses will be at a maximum at some shallow overbank stage, when the interaction between main channel and floodplain is at its greatest, before slowly decreasing as depth increases and the whole floodplain and valley floor begins to behave as a single channel unit.

Away from the near channel zone, water movement on the floodplain may be more accurately described as a typical shallow water flow (i.e. one where the width/depth ratio exceeds 10:1) as the horizontal extent may be large (up to several kilometres) compared to the depth (usually less than 10 m). Such shallow water flows over low-lying topography are characterised by rapid extension and retreat of the inundation front over considerable distances, potentially with distinct processes occurring during the wetting and drying phases. Correct treatment of this moving boundary problem is therefore important both to capture adequately the

shallow water energy losses (which may be high due to large relative roughness) and to correctly define flood extent.
Flow interactions with micro-topography, vegetation and structures may all be important, thereby giving a complex modelling problem. In particular, where the floodplain acts as a route for flow conveyance rather than just as storage, energy losses are typically dominated by vegetative resistance, even if these processes are relatively poorly understood at present. Moreover, many numerical models of floodplain flow assume that the channel bed is fixed over the course of the event, and for very large floods this may not be the case as embankment failure or geomorphic change may considerably affect the flow field.

When the interest is more focused on the prediction of water levels at particular points of interest, the modeller is primarily concerned with the downstream routing of flow through a compound cross-section, and may be less concerned to represent floodplain flow and storage accurately. Here, the flow processes of interest are one-dimensional in the down-valley direction and one-dimensional models may therefore be used to represent such flows; actually, the interest is in the one-dimensional outcome of a three-dimensional process. This approach can be justified by assuming that the additional approximations involved in continuing to treat out-of-bank flow as if it were one-dimensional are small compared to other uncertainties. Alternatively, one can attempt to correct one-dimensional flow routing methods to account for the additional energy losses and/or mass transfers or develop hybrid schemes that combine one dimensional modelling for channel flows with a two-dimensional treatment of the floodplain.

Lastly, whilst typical hydraulic models do not consider water exchanges with the surrounding catchment, for whole catchment modelling or flood inundation simulation over long river reaches such exchanges (e.g. direct precipitation or runoff to the floodplain surface, evapotranspiration losses, interactions with alluvial groundwater, and along preferential flow paths, such as relict channel gravels, within the floodplain alluvium) may, in some cases and/or at particular times, become important.

Methods for modelling flood inundation should be reliable, practicable in terms of computational expense and input data, and capable of generating the required hydraulic information in an appropriate format and level of detail. These predicted quantities should, however, be recognised as uncertain, and therefore the potential need to evaluate model and data uncertainties may also influence the type of modelling approach selected (Hunter et al., 2007).

3.2 Dimensional approaches

Hydraulic models can be classified according to the number of dimensions in which they represent the spatial domain and flow processes. Both 1D and higher order models are based on the solution of the basic Navier-Stokes equations for real fluids, which originate from mass, momentum and energy conservation physical laws. These equations are very complex for the presence of non-linear terms, therefore the solution requires the application of one of the following approaches: finite differences, finite volumes or finite elements, which refer to the physical discretization of the computational domain. In the first case, the spatial domain is divided into regular steps (rectangular grid cells), so equations are converted into incremental ratios, which are easier to solve. The second approach uses a regular spatial discretization based on finite volumes on which boundary conditions are applied. The third approach adopts a spatially variable discretization, i.e. makes use of an unstructured mesh (triangles and quadrilaterals for 2D domains). The most common and easy approach is the first one. Another possibility is to reduce Navier-Stokes equations to simpler ones, making assumptions or approximations.

In order to define accurately in-channel processes, the spatial discretization should be theoretically very detailed. Commonly, anyway, applications do not require such a level of accuracy, since mean properties provide enough information.
When a variable discharge is applied to the system (i.e. a hydrograph), whatever the discretization and dimensional approach chosen are, unsteady equations should be applied (Morvan et al., 2008).

3.2.1 1D modelling

The basic one-dimensional unsteady open channel flow equations are commonly called Saint-Venant equations, which are based on the following assumptions:

- the flow is one-dimensional, i.e. the velocity is uniform in a cross section and the transverse free-surface profile is horizontal;
- the streamline curvature is very small and the vertical fluid accelerations are negligible; as a result, the pressure distributions are hydrostatic;
- the flow resistance and turbulent losses are the same as for a steady uniform flow for the same depth and velocity, regardless of trends of the depth;
- the bed slope is small enough to satisfy the following approximations: $\cos\theta \sim 1$ and $\sin\theta \sim \tan\theta$;

- the water density is a constant, and no sediment motion is considered.

With these basic hypothesis, the flow can be described at any point and any time by two variables, e.g. velocity and water depth, or discharge and water depth. Flow properties are described by two equations: the continuity equation (conservation of mass) and momentum equation or dynamic wave equation (conservation of momentum). It has to be noted that equations of conservation of momentum and conservation of energy are equivalent if the two relevant variables (e.g. velocity and water depth) are continuous functions; this does not happens at a discontinuity (e.g. a hydraulic jump). The advantage of momentum equation is that it also applies to discontinuous flow situations. Complete equations are:

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

Continuity equation

$$\frac{1}{g}\left(\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x}\right) + \frac{\partial d}{\partial x} + S_f - S_0 = 0$$

Momentum equations (or dynamic wave equation)

where

Q	is the flow discharge
A	is the cross section surface area
V	is the cross section averaged velocity
$\partial V / \partial t$	is called “local acceleration”
$V \partial V 7 \partial x$	is called “convective acceleration”
$\partial d / \partial x$	is called “pressure term”
S_f	is called “friction slope or resistance term”
S_0	is called “bed slope or gravity term”

The dynamic wave equation may be simplified when one or more terms become negligible, resulting in diffusive (eliminating acceleration terms) or kinematic (eliminating acceleration and pressure term, thus assuming that the friction and gravity forces balance) wave equation.

The 1D approach is best applied to in-channel flows where a clear downstream direction can be indentified. Anyway, also the floodplain flow could be treated as one-dimensional in the same river direction, even if this is a limitation, since floodplain flow is truly two-dimensional. Adopting this approach, both the river and the floodplain are considered a unique entity (i.e. they share the same water level and they both store and convey water), and they are described by a sequence of topographical cross sections, transversal to the main (1D) direction of flow.

To obtain a flood extent map, the following approach is usually applied. Initially, the longitudinal profile of maximum levels is projected on a 2D plane. On the assumption of constant water levels along each cross section, water levels are allocated to geo-referenced cross sections elevation points; these are then interpolated to ensure a contiguous surface which is then compared with the DEM of the floodplain and only depths greater that zero are retained. To ensure low lying areas (e.g. behind embankments) are not mistakenly considered flooded, all inundated cells unconnected to the main channel are removed from the final flood extent map (Werner, 2004; Werner, 2001). Anyway, this approach is considered inappropriate because floods are not planar surfaces but, rather, waves where the shape of the wave (or hydrograph as it would appear to a stationary observer) will control the rate of floodplain wetting and drying (Bates and de Roo, 2000).

In general, 1D models are not appropriate in the following cases:

- when floodplains are large, i.e. when their width is more that three times the width of the main river channel;
- when the floodplain is separated from the main channel by embankments, levees or any raised ground, since in this case floodplain effectively behaves as a single channel (Néelz and Pender, 2009).

Finally, some situations require special attention when modelled in 1D:

- river confluences, where water from one river can flow over the floodplain into the other river;
- flood plains that locally are characterised by storage of water rather than flow;
- rapidly varying cross section widths, which requires a large number of cross sections at short intervals;
- rivers with a multiple channel system where the connectivity between the different channels is complex.

1D models are often selected because they seem less complex than 2D models. However, in areas with irregular topographies and complex or varying flow patterns, the application of a 1D model is much more difficult than the application of a 2D model (Asselman, 2009).

3.2.2 2D modelling

Complete equations for 2D models are termed the 2D Saint-Venant equations (or shallow water equations). They are derived from depth-integrating the Navier–Stokes equations, in the case where the horizontal length scale is much greater than the vertical length scale (so vertical

velocity is removed from the equations), thus are most often applied to flows that have a large areal extent compared to their depth and where there are large lateral variations in the velocity field, e.g. in case of overbank flood flows in compound channels, tides, tsunamis or even dam breaks.
Their form is:

$$\frac{\partial H}{\partial t}+\frac{\partial}{\partial x}(Hu)+\frac{\partial}{\partial y}(Hv)=0$$

Continuity equation

$$\frac{\partial}{\partial t}(Hu)+\frac{\partial}{\partial x}\left(Hu^2\right)+\frac{\partial}{\partial y}(Huv)+gH\frac{\partial H}{\partial x}=Friction\_terms$$

Momentum equation in the x-direction

$$\frac{\partial}{\partial t}(Hv)+\frac{\partial}{\partial x}(Huv)+\frac{\partial}{\partial y}\left(Hv^2\right)+gH\frac{\partial H}{\partial y}=Friction\_terms$$

Momentum equation in the y-direction

where
H is the depth of water
u is the mean velocity in x-direction
v is the mean velocity in y-direction
ρ is the water density.

Friction terms are not better defined here since several expressions of the equations describe them in a different way. They depend, in fact, on the adopted formulation of friction/ roughness.
Also in this case it is possible, in order to reduce computational effort, to simplify the complete flow equations, e.g. removing some friction terms. Equations are then applied in the two directions (x and y) of the space. A mesh (which could be structured or unstructured) is defined for the computational domain, and for each element neighbours are set. Flow equations are then applied to exchange flow among these neighbours. Structured meshes can be generated more easily and the results can easily be processed in GIS-packages. Unstructured grids are very suitable for areas with irregular topography and with obstacles with varying orientations so that the cell boundaries can follow the lining of the objects. Commonly, meshes are structured and makes use of raster DEMs covering both the river channel and the floodplain.

Within the channel, the main advantage of 2D models is that local variations of velocity and water levels and local changes in flow direction can be represented. In the near channel region of a compound channel, they are able to capture some important aspects of the acting processes. Finally, they can also represent easily moving boundary effects and are therefore useful for simulating problems where inundation extent changes dynamically through time (Asselman, 2009). Disadvantages are that these models cannot reproduce accurately hydraulic structures and even some in-channel processes, and are not generally suitable for medium scale analyses, but more for local studies, because required details imply very long computational times (Frank et al., 2001).

3.2.3 Combined 1D2D modelling

Whilst one-dimensional codes are computationally efficient, they do suffer from a number of drawbacks when applied to floodplain flows. These include the inability to simulate lateral spreading of the flood wave, the lack of a continuous treatment for topography and the subjectivity of cross-section location. Whilst all of these constraints can be overcome with higher order codes, the computational cost of running a two or three dimensional simulation may be high. Consequently, recent research has begun to examine hybrid one-dimensional/two-dimensional codes that seek to combine the best of each model class (Asselman, 2009).
The simplest 1D2D approach (called 1D+) is to model river dynamics using 1D Saint-Venant equations and to discretize the floodplain into a series of regions which can exchange flow (storage cell concept). Recent developments in topographic data capture have, however, allowed to produce high resolution Digital Elevation Models of floodplain areas. This has allowed storage cells to be discretized as a high resolution grid. In basic models, only mass transfer is accounted for between channel and floodplain, while in more complex ones also the momentum exchange is considered.
Combined 1D2D approach proved its efficiency in several cases (Frank et al, 2001; Verwey, 2001), and particularly for medium-scale analyses (Apel et al., 2009).

A visual comparison of described approaches is provided in Figure 3.2.

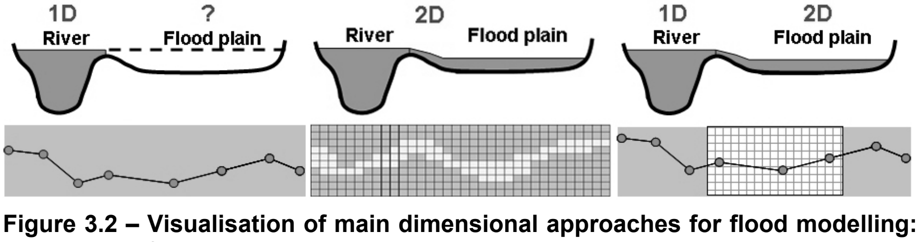


Figure 3.2 – Visualisation of main dimensional approaches for flood modelling: 1D (on the left), 2D (in the centre); 1D2D (on the right).

3.2.4 3D modelling

From a theoretical point of view, flow is a process acting in the three dimensions of space, so a 3D model would be the most appropriate choice. Moreover, in order to be accurately described, several processes (e.g. sediment transport and flow-vegetation interaction) need this kind of representation. Anyway, 3D models require too much data, and are too complex and computationally expensive at the moment to be used for common applications.

For these reasons, dynamically varying flows in compound channels have, to date, been treated predominantly with 1D and 2D models (Hunter et al., 2007).

3.2.5 0D modelling or non-modelling approach

In certain situations one may not even need a model at all to predict inundation extent. Given gauged water surface elevations along a reach, or water surface elevations predicted on the basis of flood frequency analysis, it is possible to approximate the flood wave as a plane (or series of planes) which are intersected with the DEM to give extent and depth predictions. Clearly, the planar approximation will work well for reaches that are short compared to the wavelength of the flood and where there is good gauged data to constrain the position of the plane. Even in these circumstances, however, lack of mass conservation will mean that areas are predicted as flooded that are not hydraulically connected to the channel. Nevertheless, this may be a useful method under some circumstances, and provides a benchmark level of performance that all hydraulic models should exceed to be considered skilful (Asselman, 2009).

3.3 Data required for flood modelling

Table 3.1 shows that the choice of a model could be made according to the scale of the problem, the available computational resources and the needs of the user. However, availability and quality of data also plays an important role. Data required for flood modelling relate mainly to: boundary conditions, topography, friction (roughness), and real hydraulic measures for model validation.

Dimensional approach	Description	Application	Inputs	Outputs	Indicative computational time
0D	No physical laws included in the simulations.	Broad scale assessment of flood extents and flood depths.	DEM. Upstream water level. Downstream water level.	Inundation extent and water depth by intersecting the planar water surface with the DEM.	Seconds
1D	Solution of the 1D Saint-Venant equations.	Design scale modelling which can be of the order of 10s to 100s of km depending on catchment size.	Surveyed cross sections of channel and floodplain. Upstream discharge hydrographs. Downstream stage hydrographs.	Water depth and average velocity at each cross section. Inundation extent by intersecting predicted water depths with DEM. Downstream out-flow hydrograph.	Minutes
1D+	1D plus a storage cell approach for the simulation of flood plain flow.	Design scale modelling which can be of the order of 10s to 100s of km depending on catchment size, also potential for broad scale application if used with sparse cross-section data.	As for 1D models.	As for 1D models.	Minutes to hours
2D	Solution of the two-dimensional Shallow Water Equations	Design scale modelling of the order of 10s km. May have the potential for use in broad scale modelling if applied with very coarse grids. Possibility to model accurately transcritical flows, dam break and fast transient flows.	DEM. Upstream discharge hydrographs. Downstream stage hydrographs.	Inundation extent. Water depths. Depth-averaged velocities. Downstream outflow hydrograph.	Hours to days

Table 3.1 – Approaches to flood modelling (from Asselman, 2009).

3.3.1 Boundary conditions

Boundary conditions define the water flowing into and out to the model domain. The precise data required depends on the model and the reach hydraulics. Anyway, when there is no interest in the modelling of rainfall, it is common practice to define a boundary input point within the river where a hydrograph is assigned. The type of output condition depends on the state of the current. In case of supercritical flow, i.e. when there is no possibility that flow could move upstream, there is no need to define a hydraulic condition, except for the case that the model requires it for computational or stability reasons, while for subcritical flows, i.e. when possible changes of flow directions along the channel can occur (i.e. from downstream to upstream), it is necessary to define a value of discharge, water level, or rating curve, being the last a relation among discharge and water level in correspondence of a cross section.

Some models require a “spin-up period”, i.e. a period previous to t=0 in which water should fill up the river system and produce an equilibrium in hydraulic properties. This is usually done by letting water flow until ordinary discharges or water levels are reached in channels, expanding the input hydrograph before the flood event for a sufficient lag of time. Another possible approach is to perform a steady simulation before the dynamic one to obtain the equilibrium.

3.3.2 Topography

Topography is considered the key data set for flow routing and inundation modelling since it affects the propagation of a flood (Reese and Smart, 2009; Alkema, 2007; Haile and Rientjes, 2005, Hardy et al, 1999; Horrit and Bates, 2001; Cook and Merwade, 2009; Néelz and Pender, 2009; Büchele et al., 2006). Local topographic details may cause obstructions or may concentrate or accelerate the flow of water.
Traditionally, topography has been represented by means of ground surveyed cross sections perpendicular to the channel, spacing between 100 and 1000 m. Such data could be very accurate for the specific cross section and integrate well with one-dimensional hydraulic models; however, their collection is expensive and time consuming, and the spatial resolution is relatively low. Moreover, these data need to be interpolated, with the disadvantage of losing important small-scale features that affect flood propagation, especially in floodplain areas, and they are not generally suitable for 2D models. High resolution data from remote sensing, e.g. LIDAR, can improve the topographical representation (Murphy et al.,. 2008; Néelz et al., 2006), but their use has the main drawback of extremely high calculation times.

3.3.2.1 Spatial resolution and quality of representation

The space discretization of a flood model depends on the resolution of available terrain data, the length scale of terrain features and of relevant flow processes. With the development over the last decade of high resolution mapping technologies, terrain data are usually available at scales much finer than it is computationally possible to model over wide areas. However, deciding which terrain and flow length scales need to be incorporated in a model is a much more subjective choice (Hardy et al., 1999).
Clearly, as spatial resolution is lowered particular terrain and flow features will no longer be adequately represented since all the features that have an area smaller than the cell grid size are ignored, or, better, averaged up, and the impact of these sub-grid scale effects on the model predictions will need to be parameterized (Asselman, 2009). All the important features that can exert an influence to the flow (e.g. levees, road tracks and walls) should anyway be included in the spatial representation, as least with their correct elevation and then, possibly, area and length (Haile and Rientjes, 2005). Main problems arise when representing very small entities, such as riffles and pools.
Geometrical properties of topography (slope gradients, slope aspect and drainage density) may represent an obstruction but they could also conduct or accelerate the flow of water. Since hydraulic variables can be highly variable over small spatial scales and are thus extremely sensitive to terrain parameterisation, small errors in bed elevations may have a large impact on the predicted variables and flood extent (Hunter et al., 2007). When resolution is modified, flow directions could be different, and this could be the main reason for differences in the output variables (Haile and Rientjes, 2005).
The quality of a DEM can be assessed by quantitative methods such as RMSE, which analyses the correctness of single points values compared to reference ones (which were not used for the construction of the DEM), even if more fruitful approaches should be based on looking at pattern of values (Wise, 2000) and analysing DEM derivatives, e.g. hillshade, slope, curvature, aspect and flow direction (fonte). Anyway, that the assessment of DEM quality should be done in the context of a particular type of analysis. A DEM may appear to be very poor, but if it produces the correct results, then its quality is clearly adequate for that particular task (Wise, 2000).

Asselman (2009) states that:

- grid resolutions in rural areas with a gentle and/or regular topography can be coarser than grids developed for areas with a more complex topography or urban areas;

- in general, less resolution is required if only water level is to be predicted, finer resolution if the velocity field is also required for flood characterisation.

Hardy et al. (1999) developed some useful researches on the effect of changing mesh resolution on 2D models, and the main results are that:

- there is usually an optimum mesh resolution beyond which result may not significantly vary. It would thus be useful to indentify this optimum resolution, also because of the fact that generally, when the spatial resolution increases, also the computation tine increases, and so a compromise should be found among topographical detail and time necessary to obtain the required output data;
- as the resolution increases, generally the extent of inundation decreases, because when mesh resolution is lower, the channel is represented as wider (this could be somehow dependent on mesh filtering processes, but it is also confirmed by Cook and Merwade, 2009);
- spatial resolution has a greater effect than the typical calibration parameters, i.e. friction, in altering the hydraulic simulations (this is also confirmed by Horrit and Bates, 2001).

A good approach would thus be to test different spatial resolutions within any modelling project, and to compare the results.

Another important issue to consider is that an unsatisfactory spatial representation could be caused by:

- problems in the DEM itself (poor quality input data or low resolution), e.g. random, systematic errors or blunders, which usually manifest themselves as artefacts in the DEM, which could be best indentified by visual inspection;
- problems in the algorithm used to construct the DEM (this will be discusses in par. 6.3.1.2).

3.3.3 Roughness

Hydraulic resistance (or roughness) is a lumped term that represents the sum of a number of effects: skin friction, form drag and the impact of acceleration and deceleration of the flow. The precise effects represented by the friction coefficient for a particular model depend on model dimensionality, as the parameterization compensates for energy losses due to unrepresented processes, and the grid resolution. Complex questions of scaling and dimensionality hence arise which may be somewhat difficult to disentangle. The coefficients are also strongly dependent on water depth (Morvan et al., 2008).

Roughness affects flow pattern as the flow chooses that pathway with the steepest slope, but also with the minimal resistance. It also influences flood wave celerity and computed water depths and flow velocities (Asselman, 2009).

When performing 1D and 2D modelling, roughness parameterisation mainly relates to bottom friction coefficients (Néelz and Pender, 2009(1)).
Applications of 1D models benefit from decades of hydrometric data collection, user experience in model calibration and validation (Cunge, 2003), and flood wave propagation (at least in the case of in-bank floods) is now predicted by 1D models with an accuracy that can be considered excellent for many engineering applications. Nevertheless the issue as to whether models should be parameterised using engineering judgement informed by experience, or simply by calibration, or even by a combination of both is still debated in the literature (Beven, 2000 and Cunge, 2003).
The parameterisation of friction in 2D models benefits to some extent from the knowledge and experience available in 1D modelling, although the formulation of friction is different in 2D models, because a) bed friction only concerns the interaction of the flow with the river bottom while in 1D models it concerns the entire wetted perimeter, and b) viscosity is explicitly represented in the 2D shallow water equations whereas it is effectively taken into account as part of the friction parameterisation in 1D models. Theoretically this should result in lower values (assuming that lower values are used for less rough beds, as is the case with Manning's n) of friction in 2D models compared with 1D models (Morvan et al., 2008).
The essential point is that friction parameters are scale-dependent effective values that compensate for varying conceptual errors in the model. Implications are that inundation extent and floodplain water level measurements alone cannot usually be used to calibrate 2D floodplain models in the same way as river levels are used to calibrate 1D river models (Hunter et al., 2005, Werner et al., 2005, Néelz et al., 2006; Horrit and Bates, 2002).

3.3.3.1 Definition of coefficients

Roughness is usually defined by referring to tabulated values (Chow, 1959), formulas (Cowan, 1956) or even by comparison with reference photographs (Arcement and Schneider, 1989), and is further adjusted in the calibration phase (Vidal et al., 2007).

When referring to **tabulated values**, a range (minimum-medium-maximum) is provided according to the physical characteristics of the

channel (bed material granulometry, lining materials, level of artificiality, presence of vegetation).
A commonly used expression of roughness is the Manning coefficient, n, which derives from the Manning formula

$$V = \frac{1}{n} R^{2/3} S_e^{\,1/2}$$

where V is the mean velocity of flow (m/s), R is the hydraulic radius i.e. the cross sectional area of flow divided by the wetted perimeter (m), S_e is the slope of energy gradient line (m/m) and n is the Manning coefficient (s/m$^{1/3}$). The term V could be replaced by Q=VA. Typical n values are reported in Table. 3.2.
This formula applies to uniform flow, i.e. bottom slope, cross sectional size, shape and roughness characteristics of natural channels must be at least approximately constant. River channel characteristics are usually not that uniform over an extended length, but this assumptions could be reasonably accepted for a particular section (called “reach”) of the river (Bengtson, 2010). In some cases, a similar formula is applied, which refers to channel conveyance:

$$K = \frac{1}{n} A R^{2/3}$$

were K is the channel conveyance (m^3/s), A is the cross sectional area of the channel (m^2), and R is the hydraulic radius (m).

Table 3.2 – Typical Manning n values for channels and floodplains.

	Minimum	Medium	Maximum
Natural minor streams (top width at floodstage < 30m)			
Plain streams, channel			
clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
same as above, but more stones and weeds	0.030	0.035	0.040
clean, winding, some pools and shoals	0.033	0.040	0.045
same as above, but some weeds and stones	0.035	0.045	0.050
same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
same as two above, with more stones	0.045	0.050	0.060
sluggish reaches, weedy, deep pools	0.050	0.070	0.080
very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages, bottom with			
gravels, cobbles, and few boulders	0.030	0.040	0.050
cobbles with large boulders	0.040	0.050	0.070

Natural major streams (top width at floodstage > 30m)			
regular cross sections, no boulders or brush	0.025	-	0.060
irregular cross sections	0.035	-	0.100
Flood Plains			
Pasture, no brush			
short grass	0.025	0.030	0.035
high grass	0.030	0.035	0.050
Cultivated areas			
no crop	0.020	0.030	0.040
mature row crops	0.025	0.035	0.045
mature field crops	0.030	0.040	0.050
Brush			
scattered brush, heavy weeds	0.035	0.050	0.070
light brush and trees, in winter	0.035	0.050	0.060
light brush and trees, in summer	0.040	0.060	0.080
medium to dense brush, in winter	0.045	0.070	0.110
medium to dense brush, in summer	0.070	0.100	0.160
Trees			
dense willows, summer, straight	0.110	0.150	0.200
cleared land with tree stumps, no sprouts	0.030	0.040	0.050
same as above, but with heavy growth of sprouts	0.050	0.060	0.080
heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
same as above, with flood stage reaching branches	0.100	0.120	0.160
Excavated or Dredged Channels			
Earth, straight, and uniform			
clean, recently completed	0.016	0.018	0.020
clean, after weathering	0.018	0.022	0.025
gravel, uniform section, clean	0.022	0.025	0.030
with short grass, few weeds	0.022	0.027	0.033
Earth winding and sluggish			
no vegetation	0.023	0.025	0.030
grass, some weeds	0.025	0.030	0.033
dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
earth bottom and rubble sides	0.028	0.030	0.035
stony bottom and weedy banks	0.025	0.035	0.040
cobble bottom and clean sides	0.030	0.040	0.050
Dragline-excavated or dredged			
no vegetation	0.025	0.028	0.033
light brush on banks	0.035	0.050	0.060

Rock cuts			
smooth and uniform	0.025	0.035	0.040
jagged and irregular	0.035	0.040	0.050
Channels not maintained, weeds and brush uncut			
clean bottom, brush on sides	0.040	0.050	0.080
same as above, highest stage of flow	0.045	0.070	0.110
dense weeds, high as flow depth	0.050	0.080	0.120
dense brush, high stage	0.080	0.100	0.140
Lined or Constructed Channels			
Cement			
neat surface	0.010	0.011	0.013
mortar	0.011	0.013	0.015
Wood			
planed, untreated	0.010	0.012	0.014
planed, creosoted	0.011	0.012	0.015
unplaned	0.011	0.013	0.015
plank with battens	0.012	0.015	0.018
lined with roofing paper	0.010	0.014	0.017
Concrete			
trowel finish	0.011	0.013	0.015
float finish	0.013	0.015	0.016
finished, with gravel on bottom	0.015	0.017	0.020
unfinished	0.014	0.017	0.020
gunite, good section	0.016	0.019	0.023
gunite, wavy section	0.018	0.022	0.025
on good excavated rock	0.017	0.020	-
on irregular excavated rock	0.022	0.027	-
Concrete bottom float finish with sides of:			
dressed stone in mortar	0.015	0.017	0.020
random stone in mortar	0.017	0.020	0.024
cement rubble masonry, plastered	0.016	0.020	0.024
cement rubble masonry	0.020	0.025	0.030
dry rubble or riprap	0.020	0.030	0.035
Gravel bottom with sides of:			
formed concrete	0.017	0.020	0.025
random stone mortar	0.020	0.023	0.026
dry rubble or riprap	0.023	0.033	0.036
Brick			
glazed	0.011	0.013	0.015
in cement mortar	0.012	0.015	0.018
Masonry			
cemented rubble	0.017	0.025	0.030

dry rubble	0.023	0.032	0.035
Metal			
smooth steel surfaces	0.011	0.012	0.014
corrugated metal	0.021	0.025	0.030
Asphalt			
smooth	0.013	0.013	-
rough	0.016	0.016	-
Vegetal lining	0.030	-	0.500

In general, a reasonably accurate value of n can be defined for most man-made open channels, but obtaining good values of the Manning coefficient for a natural channel is a bit more of a challenge, because of the great variability in both the bottom and side surfaces (Bengtson, 2010).

When referring to **formulas**, the main reference is a procedure established by Cowan (1956) which allows one to estimate the effects of several factors to determine the value of n for a channel (Arcement and Schneider, 1989; Bengtson, 2010). The value of n may be computed by:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m$$

where

n_b is a base value of n for a straight, uniform, smooth channel in natural materials

n_1 is a correction factor for the effect of surface irregularities

n_2 is a value for variations in shape and size of the channel cross sections

n_3 is a value for obstructions

n_4 is a value for vegetation and flow conditions

m is a correction factor of meandering of the channel.

The idea is that channel irregularities, alignment, obstructions, vegetation, and meandering increase the roughness of a channel. The value for n must therefore be adjusted accordingly by adding increments of roughness to the base value, n_b.

Depth of flow must be also considered when selecting n values for channels. If the depth of flow is shallow in relation to the size of the roughness elements, the n value can be large. The n value decreases with increasing depth, except where the channel banks are much rougher than the bed or where dense brush overhangs the low-water channel (Arcement and Schneider, 1989).

The same approach is then applied for floodplain n values:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m$$

where

n_b is a base value of n for the floodplain natural bare soil surface

n_1 is a correction factor for the effect of surface irregularities on the flood plain

n_2 is a value for variations in shape and size of the flood-plain cross section, assumed to equal 0.0

n_3 is a value for obstructions on the flood plain

n_4 is a value for vegetation on the flood plain

m is a correction factor for sinuosity of the flood plain, equal to 1.0.

Detailed correction factors values are not provided here but are available in Arcement and Schneider (1989).
Both the approaches presented refers to coefficient to be used for 1D simulations, and are usefully applied when main cross section are divided in sub-sections according to the roughness characteristics. For 2D models, some adaptations are required, as already stated.

Land use maps are used to determine a first guess of the friction coefficients to be used in the floodplain model (Todini, 1999), but their utility is dependent of the level of detail represented. Moreover, it has to be considered that obstruction posed by vegetation is dependent by its density, and so by the season of the year.

3.3.4 Calibration and validation

The calibration task could be defined as "the procedure of adjustment of parameters values of a model to reproduce the response of reality within the range of accuracy specified in the performance criteria", where the performance criteria is the "level of acceptable agreement between model and reality" (Vidal et al., 2007). Validation is, then, the test of the predictive power of the calibrated model (Horrit, 2006); it is the process of demonstrating that a given site-specific model is capable of making accurate predictions, defined with respect to the application, for periods outside a calibration period. A model is said to be validated if its accuracy and predictive capability in the validation period have been proven to lie within acceptable limits or errors for a particular practical purpose (Hunter et al., 2007). Validation, anyway, should not be just a check that computed values are not very far from observed ones: it is a study of the reasons why there is a difference between the two (Cunge, 2003).

A calibration is generally required to successfully apply a floodplain flow model to a particular reach for a given flood event (Werner, 2004). This step is undertaken in order to identify appropriate values for parameters such that the model is able to reproduce observed data, e.g. measures at river gauging station, water levels in the floodplain, and/or flood extent.

Regarding these data, the usefulness of making use of more then one type of measure consists in the fact that single data type will be likely to have complex errors, and will only test some, but not all, aspects of model performance. Typically, roughness coefficients assigned to the main channel and floodplain are considered the main calibration parameters; they should thus be recognised as being effective values that may not have a physical interpretation outside of the model structure within which they were calibrated (Hunter et al., 2007). In addition, the process of estimating effective parameter values through calibration is further convoluted by a number of error sources inherent in the inundation modelling process, that the coefficients try to compensate. Principally, these errors relate to the inadequacies of data used to represent heterogeneous river reaches but also extends to the observations with which the model is compared during calibration and the numerical approximations associated with the solution of the controlling flow equations.
Regarding the use of real measured data, Hunter et al. (2007) states that maximum water level data do not test the ability of a model to simulate dynamic flooding. Internal gauging stations, actually, produce data that could be highly resolved in time but not in space; post-event trash and sediment deposit surveys can be misinterpreted and ‘soft’ data based on human recollection can become confused because of the stressful experience of being involved in a flood event. All calibration and validation data are thus in some way limited in terms of their spatial and temporal coverage and are inherently uncertain. An obvious solution here may be to formalise the statistical rigor of inferences made using such sparse and uncertain data using a Bayesian framework, but some authors have argued that model and data errors may be uncertain in ways which may be difficult to quantify through a formal error model as required by strict Bayesian methods. This has led to an interest to more generalised methods for assessing simulation likelihood (e.g. the GLUE methodology, Beven and Binley, 1992; Hunter et al., 2005; Werner et al., 2005), which relax certain statistical assumptions of the Bayesian approach at the expense of being able to make formal probabilistic statements about particular predictions.
With calibration it is also likely that many different types of model may fit available calibration data equally well (yet give different results in prediction). This issue is called “equifinality” (Beven, 2006). In this case it becomes even more difficult to conclusively discriminate between model types and determine precisely the correct model type for a particular application.

Lack of calibration and validation data is the biggest constraint on future model development and to make progress in this area concerted flood

measurement campaigns using all available sensor technologies are desirable (Hunter et al., 2007).
In “good practice”, lacking of adequate data should require the use of the most advanced and reliable modelling tools, but this arises other issues. Cunge (2003) provides a very interesting treatment of the subject. He recognises that calibration for data-driven and physically based models should be performed in a different way. In the first case, parameters usually do not have a physical meaning, and the model should be applied accomplishing these stages:

1. Instantiation or set-up or “construction”. This consists in defining such features and parameters as discretisation, computational grid, limits and boundary conditions; an introduction of topography, soil occupation, structures, initially assessed values of roughness coefficients, etc.
2. Calibration, which consists in executing a number of simulations of past observed events and in varying the parameters of the model until an acceptable (to the modeller) coincidence between observations and computations is obtained.
3. Validation, which consists in executing with a calibrated model a number of simulations of past observed events (different from those used for calibration) and checking to see if the simulated results are sufficiently close to observation.
4. Exploitation runs (studies) with the model recognised as a validated tool.

When considering, however, deterministic modelling (which is based on physical laws describing simulated processes and their interactions), this four-stage paradigm is not only illusory as a way of increasing accuracy but it may also lead to dubious and unreliable results.
Some examples could be useful to understand this statement.

One-dimensional models of rivers have a typical resolution of computational grids between 100–1,000 m, with distances between gauging stations where the water stages are recorded being of the order of 10 km. Thus along a 50 km channel there might be four calibration sections (boundary conditions excluded). In open channel flow engineers can evaluate values of roughness and head-loss coefficients by inspection, within a narrow range of error. If a visual inspection of the river stretch suggests a Manning coefficient of 0.03, it is easy to accept that the actual value of the coefficient may vary between, say, 0.025 and 0.035. If, however, the calibration of roughness (coincidence between computed and observed water stages at gauges at a distance of some 10 km) leads for this reach to values such as 0.04 or 0.05, this is unacceptable. Indeed, such a river bed would be, according to the

Strickler formula, covered with equivalent roughness elements of diameters 0.78 or 3 m high! The only possible conclusion in such a case is that the model does not reproduce reality and that the calibration is meaningless. The reason, instead, is that, when instantiating the model for this case, something has been forgotten: a bridge, a singular head loss, river shape-induced head losses, the appropriate representation of an inundated plain, etc. Another possibility is that the river geometric characteristics are not correct in the model and calibration gives absurd values because it compensates for narrows or for sills that influence more the surface elevations than does the roughness. Or, worst of all, the model is based on equations that do not describe adequately the physical process, such as fixed-bed equations applied to alluvial bed rivers, or a diffusive wave equation model applied to downstream-influenced or inertia-dominated flows. At any rate, from the point of view of predictivity and future exploitation, the calibration effort is futile and useless.

Another example of meaningless "calibration" of parameters until a coincidence between computed and observed free-surface elevations is reached is two-dimensional modelling of inundated plains. In this case, the only past-observed data concerning the unsteady evolution of water stages that can be found on inundated plains are those rare marks of the highest elevations attained during historical floods. The only one known to the author – and a never repeated historical case – where the records were adequate for calibration purposes of such a situation was in the case of the Mekong Delta Model (Cunge 1975). There were 350 computational points and three consecutive floods (1963, 1964 and 1965) were recorded at 300 gauges located over the modelled area. The cost of the modelling and measurement campaigns was over US$ 1 million (at 1963 values: this would be ten times more in 2002). This number alone shows that this approach would not be repeated today. Moreover, the calibration of large areas through fitting computed and observed results may well be meaningless because the calibration criteria for large domains are really dependent upon the local effects of features located near stations.

To take a very crude intellectual shortcut, one may attempt to say that the calibration is still a common practice because it makes both sides happy: the modeller (who may estimate his or her intellectual effort as finished when the model is 'calibrated') and the end-user/client who feels that his or her duty of control and supervising has been done. Neither realises that their satisfaction is so often related to a formal coincidence and not to any understanding of the physical problems, with this last criterion as the most important point for projects and future developments, and the very reason for commissioning the model at all. This is to say that the technology in such a case is not directed to an understanding of the underlying phenomena, but only to persuading an end-user or client that something of value has been done. It thus corresponds to the technologies of persuasion in their most negative sense.

The previous four-stages process should therefore be abandoned and a modified paradigm is to be applied. More precisely, the calibration stage should be eliminated from the paradigm while the validation stage, as compared to current practice, should be carried out in a different way and in a different spirit:

1. Instantiation or set-up or 'construction' of the model: definition of the methodology necessary to define the range of uncertainty in the results of the computations.
2. Validation, which consists of executing a number of simulations of past-observed events with the model, computing or otherwise finding the range of uncertainty for the results and analysing and finding physically logical reasons for differences between the simulated and observed results. After this, analysing the impact of the differences as well as of the uncertainties upon the exploitation results.
3. Exploitation runs (studies): supplying the results and impacts and their range of uncertainty to the end-user or client in a comprehensible form.

It is claimed that a deterministic model, with values of parameters defined by inspection on the basis of engineering practice, should simulate reality correctly and its results should be close to past observed results without calibration in its irrational sense. If the differences between the computed and the observed lie within an acceptable interval of uncertainty, or can be explained by physical reasons, and if the consequences of differences upon exploiting the model as it is are analysed and acceptable, then there is no reason to go any further with the modification of parameters. If, instead, differences are greater than the uncertainty interval, then they must be explained. The reasons must be found and analysed, taking into account, once more, the consequence of using the model as it is or amending it. Most often the findings lead to modifications of originally erroneous data, such as topography, hydraulics characteristics or boundary conditions, and have not much to do with parameters. Sometimes there are factually important errors in values of parameters assessed during a visual inspection. But, sometimes, one may find that the modelling tool is not adequate: such often occurs when using 1D models where only 2D can simulate the real flows.

3.4 Some final considerations

For any given situation there is a variety of modelling tools that could be used to compute floodplain inundation and a variety of spatial resolutions at which these codes could be applied. All codes make simplifying assumptions and only consider a reduced set of the processes known to occur during a flood event. Hence, all models are subject to a degree of structural error that is typically compensated for by calibration of the friction parameters. Calibrated parameter values are not physically realistic, as in estimating them we also make allowance for a number of distinctly non-physical effects such as model structural error and any energy losses or flow processes which occur at sub-grid scales. Thus, whilst we may denote the resistance coefficient in a wide variety of hydraulic models as “Manning’s n”, in reality the precise meaning of this resistance term changes as we change the model physical basis, grid resolution and time step. In general, as the dimensionality increases and grid scale is reduced we require the resistance term to compensate for fewer unrepresented processes and: (i) the model sensitivity to parameter variation reduces; and (ii) the calibrated value of the resistance term should converge towards the appropriate skin friction value.

Table 3.3 – Suggestions for the choice of a flood model (from Asselman, 2009)

Area characteristics	Data	Applicable models
Wide, relatively flat areas with natural or agricultural land use	Detailed data available (laser altimetry terrain data, channel bathymetry information, land use data, accurate boundary conditions). Data for model validation.	2D models. Storage-cell approach also usable if limited discharge through the floodplains (mainly storage).
	Detailed topographical data missing	1D model with approximate storage cells
Steep sloping rivers with large floodplains	Detailed data available	2D models coping with transcritical flows
	Detailed topographical data missing, cross sections available	1D models coping with transcritical flows and preferably shock-capturing
Steep sloping rivers with narrow floodplains		1D or 2D models coping with transcritical flows. If available, 1D model with mass and momentum exchanges between subsections.
Urban areas	Detailed data available (laser altimetry terrain data, digital map data, accurate boundary conditions). Data for model validation.	2D models, with full shallow water models where local inertial effects are important. 2D storage cell models currently give reasonable results but at high computational cost.

Some general guidelines are provided by Asselman (2009) in order to choose the most appropriate model for a given context, as reported in Table 3.3. Usually, anyway, before making a choice, some tests of different models should be performed (Bates and de Roo, 2000).

Another important issue, not accounted for in most of the models, is the solid transport associated to the flood flow. In some cases, this transport may constitute a worse hazard than the water wave itself. In turn the solid transport may affect seriously the topography and worsen the flood in terms of water depth and local velocities (Asselman, 2009).

To conclude this chapter remarking the complexity of the issue, a hypothetical debate originating from two statements from Cunge et al. (1980) and Pappenberger et al. (2005) seems appropriate.

"*The modeller must resist the temptation to go back to one-dimensional schematization because of lack of data otherwise necessary for an accurate two-dimensional model calibration. If the flow pattern is truly two-dimensional, a one-dimensional schematization will be useless as a predictive tool […]. It is better to have a two-dimensional model partially calibrated in such situations than a one-dimensional one which is unable to predict unobserved events. Indeed, the latter is of very little use while the former is an approximation which may always be improved by complementary survey*"

(Cunge et al., 1980)

"*If the possibility exists to use distributed data, then of course one might ask why the 2D flow pattern of the floodplains should be approximated by a 1D model. The answer is simple and straightforward: The more complex model will have similar uncertainty problems to the simpler one, but on a larger scale, because it will normally require more parameter values, which will be again effective parameters at the model element scale compensating for the remaining model errors. Experience suggests that there will still be significant uncertainty in reproducing both pattern information and discharge hydrographs with higher dimensional models, particularly when predictions for design or warning purposes outside of the discharge range of the available calibration data are required*"

(Pappenberger et al., 2005)

CHAPTER 4
Hydraulic software packages

To model the process of riverine flood basing on numerical equations, calculation codes are needed. When coupled to GIS (Geographical Information Systems), these codes allow to convert modelling results into spatial maps.
Several commercial tools are available, with their own useful components and limitations. During the development of the research project, it was possible to make use of three commercial software packages for flood modelling: SOBEK, FLO-2D and FloodArea. All of them apply physically-based models, with different levels of complexity. Before testing and comparing their performance, a description of each of them is provided.

4.1 SOBEK

SOBEK is a model developed by Deltares | Delft Hydraulics (formerly known as WL | Delft Hydraulics, the Netherlands). In particular, the application SOBEK-Rural has been used, which allows to model natural streams in lowlands and hilly areas. SOBEK consists of several modules: the ones for inundation modelling are the Overland Flow module (SOBEK 2D) and the Channel Flow module (SOBEK 1D).
SOBEK Channel Flow is based upon the solution of the full de Saint-Venant equations, presented in a slightly adapted form as follows:

$$\frac{\partial A_t}{\partial t} + \frac{\partial Q}{\partial x} = q_{lat}$$

$$\underbrace{\frac{\partial Q}{\partial t}}_{\text{INERTIA}} + \underbrace{\frac{\partial}{\partial x}\left(\frac{Q^2}{A_f}\right)}_{\text{CONVECTION}} + \underbrace{gA_f \frac{\partial h}{\partial x}}_{\text{WATER LEVEL GRADIENT}} + \underbrace{\frac{gQ|Q|}{C^2 RA_f}}_{\text{BED FRICTION}} - \underbrace{W_f \frac{\tau_{wi}}{\rho_w}}_{\text{WIND FRICTION}} = 0$$

where

A_f is the wetted area [m^2]

q_{lat} is the lateral discharge per unit length [m^2/s]

Q is the discharge [m^3/s]

t is time [s]

x is the distance [m]

g is the gravity acceleration [m/s^2] (=9.81)

h is the water level [m] (with respect to the reference level)

C is the Chézy coefficient [$m^{½}$/s]

R is the hydraulic radius [m]

W_f is the flow width [m]

τ_{wi} is the wind shear stress [N/m^2]

ρ_w is the water density [kg/m^3] (normally 1000).

Forces caused by bed friction and earth gravity usually determines flow conditions primarily: other forces are far less important. The Chézy coefficient C during computation may be determined in a number of ways referring to various formulations, e.g. Chézy, Manning, Strickler and White-Colebrook. In particular, if a Manning n value is provided, Chézy coefficient is calculated as: $C = R^{0.125} / n$

SOBEK Overland Flow consists of a two-dimensional modelling system based on a subset of the Shallow Water Equations:

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

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \zeta}{\partial x} + g\frac{u|V|}{C^2 h} + au|u| = 0$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial \zeta}{\partial x} + g\frac{v|V|}{C^2 h} + av|v| = 0$$

where

u is the velocity in x-direction [m/s]

v is the velocity in y-direction [m/s]

V is the velocity calculated as $V = \sqrt{u^2 + v^2}$

Z is the water level above the plain of reference [m]

C is the Chézy coefficient [$m^{½}$/s]

d is the depth below the reference plane [m]

h is the total water depth: ζ+d [m]

a is the wall friction coefficient [1/m]

As opposed to the shallow water equations, the described equations do not incorporate the turbulent stress terms, because they are relatively

unimportant for flood flow computations, in order to save computational effort. The wall friction coefficient has been introduced to account for the added resistance that is caused by vertical obstacles, like houses or trees.
All equations are solved through a fully implicit finite difference formulation. They allow the computation of sub- and supercritical flows, and so the behaviour of standing and moving hydraulic jumps.

In combination with the 2D modelling system, SOBEK is able to handle 1D elements such as small water courses and hydraulic structures. This 1D2D modelling approach allows to simulate inundations for river channels which in normal conditions are modelled in 1D (Frank et al., 2001). The 2D layer describes, on the basis of a rectangular computational grid (made of squared cells), flow over DTM-defined topographies adjusted for objects blocking flow in floodplains, such as dikes and natural levees. All sub-grid conveyance objects, such as channels, local depressions and hydraulic structures are described on the 1D schematisation layer. Subsequently, 1D and 2D schematisations are be linked to each other via water level compatibility at selected computational nodes. In particular, the 1D2D coupling occurs every time a 1D channel lies on a 2D cell and a calculation point is located on that cell, being the cell dimension comparable to the channel width (see also par. 7.2.1). For the momentum balance the 1D and the 2D systems remain strictly separated. This means that velocities or discharges belong either to the 1D part or to the 2D part, even if the exchange of momentum is accounted for. For the conservation of mass, being a scalar quantity, the appropriate 1D and 2D volumes are combined so that they share the same water level.

The model is constructed through an interface similar to a GIS (Geographical Information System) which is called Netter (Figure 4.1), where channel, floodplain and input inflows could be defined and described.

The Overland Flow and the Channel Flow modules of SOBEK are based upon the same numerical principles and both allow for stable and robust computations. Firstly, this is based upon the properties of the numerical schemes applied. Secondly, a number of checks are made at every step in the computation to prevent physically unrealistic results, such as negative water depths. If such a constraint is not satisfied, the time step will be reduced. Such a procedure is also applied in the flooding and drying of cells in the Overland Flow module. Every time only one neighbouring computational cell can be wetted or dried, otherwise the time step will be reduced to satisfy this criterion.

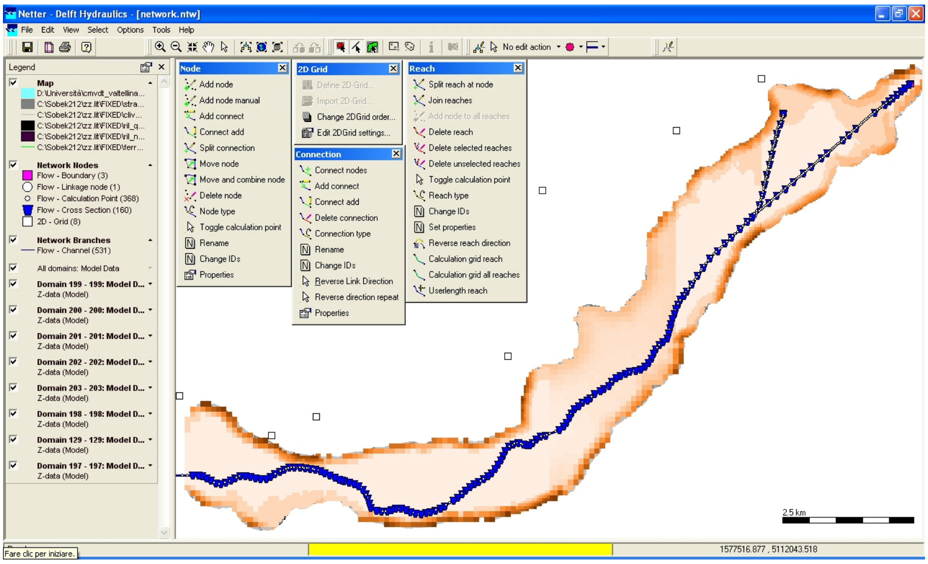


Figure 4.1 – SOBEK Netter interface for model construction.

Remarks

SOBEK is a very complex software. It allows many internal checks which are always not very clear to the user, who feels not to have a complete control of the processes running, when he is not an expert modeller. Anyway, when an essential model is constructed, the software seems really robust and reliable. Help documentation for the user is quite scarce, and this is a main lack, but e-mail assistance is efficient. Other aspects such as, in particular, the way in which 1D and 2D modules are coupled, will be discussed later.

4.2 FLO-2D

FLO-2D (www.flo2d.com) is a simple volume conservation model that numerically routes a flood hydrograph over a system of square grid elements using the 1D Saint-Venant Equations (solved by means of a finite difference numerical scheme), allowing flood hazard simulations and the design of flood mitigation measures. The key to model applicability is volume conservation that tracks the floodwave progression over an unconfined surface.

Flood hazard delineation can be enhanced by including details such as rainfall and infiltration or bridge, culvert and levee components. The effects of buildings or flow obstructions can also be simulated by means of Area Reduction Factors (ARF) or Width Reduction Factors (WRF). All the physical processes simulated are represented in Figure 4.2.

The software is user-friendly and it is based on a pre-processing and operating module, called Grid Developing System (GDS), for input data editing (which is similar to the SOBEK Netter) and a post-processing module, called Mapper for model results visualization and maps production. The main FLO-2D code is an executable file which is able to run in every folder where appropriate input files (.DAT) are located. FLO-2D is a Federal Emergency Management Agency (FEMA) of the Unites States approved model for both river studies and unconfined alluvial fans. It has been used extensively by national authorities all over the world (Nardi et al., 2009).

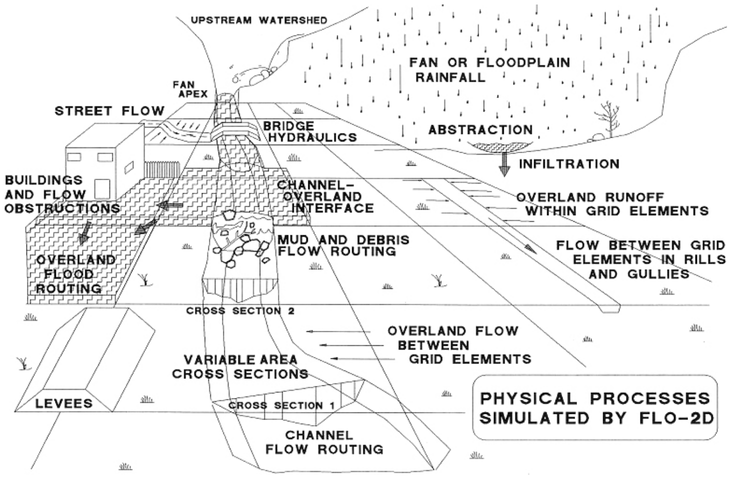


Figure 4.2 – Physical processes simulated by FLO-2D.

Channel flow is one-dimensional with the channel geometry represented either by natural, rectangular or trapezoidal cross sections. Overland flow is modelled in 2D as either sheet flow or flow in multiple channels (rills and gullies). Channel overbank flow is computed when the channel capacity is exceeded. An interface routine calculates the channel to floodplain flow exchange including return flow to the channel. Similarly, the interface routine also calculates flow exchange between the streets and overland areas within a grid element (Figure 4.3). Once the flow overtops the channel, it will disperse to other overland grid elements based on topography, roughness and obstructions.

1D flow equations are applied both for the channel and the floodplain. For the floodplain, the average flow velocity across a grid element boundary is computed one direction at time. There are eight potential flow directions, the four compass directions (north, east, south and west) and the four diagonal directions (northeast, southeast, southwest and northwest). Each velocity computation is essentially one-dimensional in nature and is solved independently of the other seven directions. The stability of this explicit numerical scheme is based on strict criteria to control the size of the variable computational timestep. The relationship between the channel cross section flow area, bed slope and roughness controls the floodwave routing, attenuation and numerical stability. Flow area has the most important affect on channel routing stability.

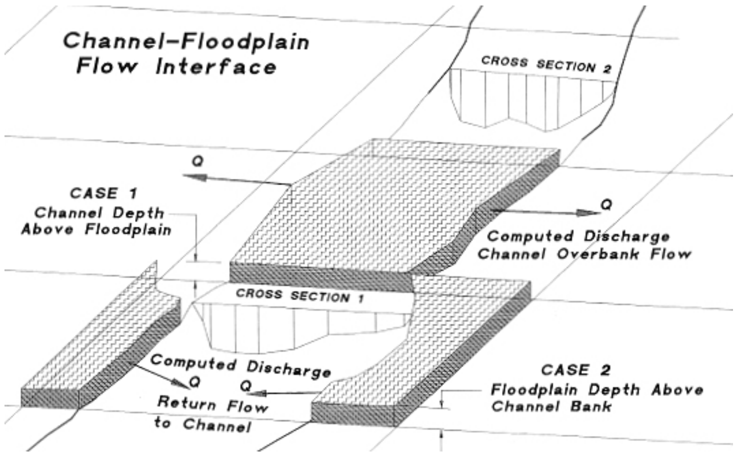


Figure 4.3 – Channel-floodplain flow interface within FLO-2D.

The relationship among 1D and 2D system is interesting, and apparently smarter than the one use by SOBEK. Channel width can be larger than the grid element and may encompass several elements (Figure 4.4). If the channel width is greater than the grid element width, the model extends the channel into neighbouring grid elements. The model also makes sure that there is sufficient floodplain surface area after the extension.

The channel interacts with the right and left bank floodplain elements to share discharge. Each bank can have a unique elevation. If the two bank elevations are different, the model automatically splits the channel into two elements even if the channel would fit into one grid element.

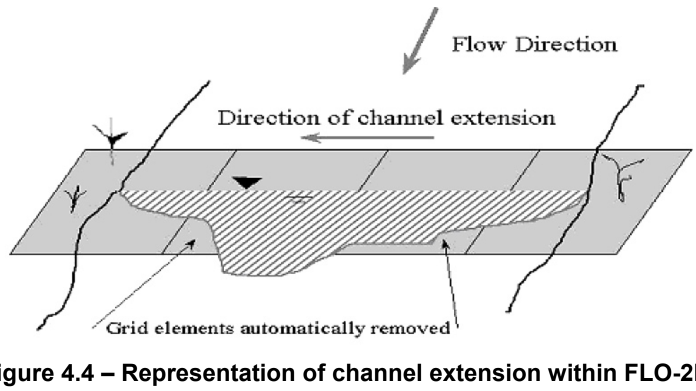


Figure 4.4 – Representation of channel extension within FLO-2D.

As for SOBEK, stability criteria have to be satisfied to ensure the continuation of calculation. If the stability criteria continue to be exceeded, the timestep is decreased until a minimum timestep is reached. If the minimum timestep is not small enough to conserve volume or maintain numerical stability, then the minimum timestep can be reduced, the numerical stability coefficients can be adjusted or the input data can be modified. The timesteps are a function of the discharge flux for a given grid element and its size. Small grid elements with a steep rising hydrograph and large peak discharge require small timesteps. Accuracy is not compromised if small timesteps are used, but the computational time can be long if the grid system is large.
All the inflow volume, outflow volume, change in storage or loss from the grid system area are summed at the end of each time step and the volume conservation is computed. Results are written to the output files or to the screen at user specified output time intervals.

Remarks

FLO-2D was developed to simulate large flood events on unconfined surfaces. To ensure reasonable computational times, the discretization of the floodplain topography into a system of square grid elements should be defined in such a way that this relation is observed: 0.03 < peak discharge (m^3/s) / cell area (m^2) < 0.3. When simulating high intensity events, floodplain representation could thus be very coarse, obscuring some topographic features such as levees and depressions. This is justified by FLO-2D developers by the fact that topographic variability will not affect the water surface when the entire valley is flooded (as states also by Nèelz and Pender, 2009(2)), but it represent a main drawback for the model applicability.
Moreover, the model does not have the ability to simulate shock waves, rapidly varying flow or hydraulic jumps, and these discontinuities in the flow profile are smoothed out in the model calculations. Subcritical and supercritical flow transitions are assimilated into the average hydraulic conditions (flow depth and velocity) between two grid elements.
Despite this limitations, FLO-2D has a main advantage to be delivered with detailed and complete manuals to support the user. They clarify the amount of data which should be provided to the model as .DAT files, and make the whole modelling process more intelligible to the user. E-mail support, moreover, can be provided in Italian.
Other issues about the software are discussed later

4.3 FloodArea

FloodArea is a joint product of Geomer GmbH, Heidelberg, Germany, and Ingenieurgemeinschaft Ruiz Rodriguez + Zeisler, Wiesbaden (Germany).

It is an ArcGIS extension completely integrated in the graphical user interface of ArcGIS desktop, utilizing Spatial Analyst functionality (Figure 4.5).

The main purpose of FloodArea is the delineation of areas inundated by a flood.
Calculations are based upon one of these input data:

- a drainage network grid with water levels assigned to it. Though the water levels can vary spatially (e.g. along a river stretch) they remain constant during the simulation process. The water levels can be changed, however, by modifying them between single model runs;
- one or more hydrographs at user definable coordinates;
- a rainstorm simulation over a wider area, specified by a weighted grid.

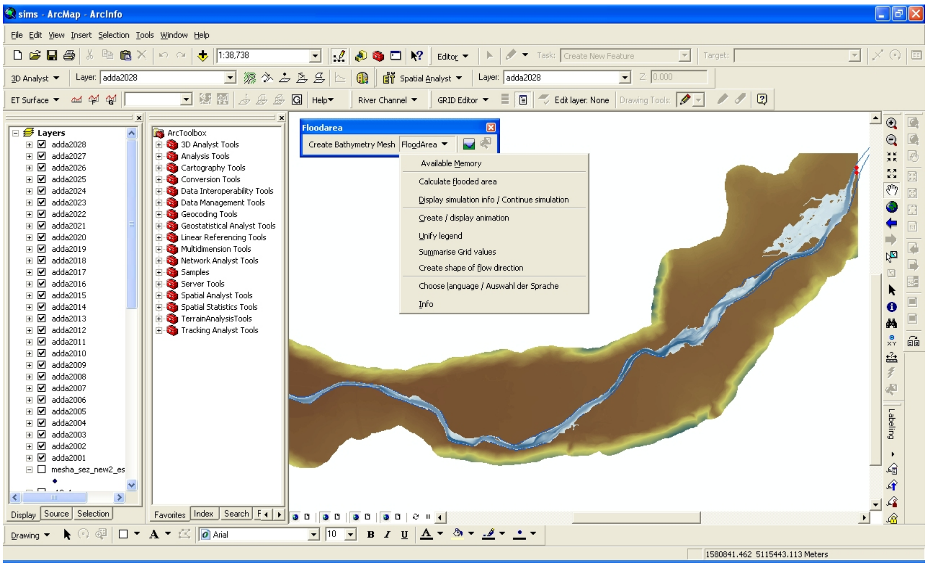


Figure 4.5 – FloodArea extension in ArcMap 9.2

Model results are stored as ESRI Grids at user defined intervals, providing for the possibility to reproduce the temporal aspect of the flooding process. If needed, the flow direction vectors can be output for each individual grid.
Additional parameters which are not represented by the elevation model can be specified for a simulation run, e.g. flow barriers such as road embankments, and locations of dam failures, making dike break scenarios possible.

The calculation of inundation areas is based upon a hydrodynamic approach.
All eight neighbours of a raster cell are considered. The discharge volume to the neighbouring cells is calculated using the Manning-Strickler formula.

$$V = k_{St} r_{hy}^{\;2/3} I^{1/2}$$

with r_{hy} being the hydraulic radius and I the gradient.
In order to define appropriate k_{St} (roughness) values, as in the case of Manning n coefficients, reference tables can be used. The quality of simulation results depends very much on using appropriate roughness values since flow velocity is linearly related to roughness.
The inclination and the direction of the water table is re-calculated in every iteration step and the steepest slope used as the inclination in the Manning-Strickler formula.

$$slope = \sqrt{\left(\frac{\partial z}{\partial x}\right)^2 + \left(\frac{\partial z}{\partial y}\right)^2} \qquad aspect = 270 - \frac{360}{2\pi}\alpha \tan 2\left[\frac{\partial z}{\partial y}, \frac{\partial z}{\partial x}\right]$$

In cases of linear elements with a width of just one raster cell, this method will fail, because the steepest slope may be perpendicular to the actual direction of flow. This is the case when the inclination of the river bed (e.g. in a small ditch) is lower than the surrounding topography. To avoid such errors, slope calculations are internally tested for their plausibility by comparing the elevation of the central raster cell to the elevation cell of the slope direction (aspect). If the difference is exceeding a certain threshold, inclination is re-calculated by comparing it with the lowest neighbouring cell.
Flow velocity as derived by the formula is multiplied by the flow cross section and the iteration time step in order to get the exchanged water volume between cells for the current iteration.
The Manning-Strickler formula is usually valid only for normal discharge, where loss due to friction equals the gain in potential energy. In other cases calculated velocities values may be too high. To control this, the velocity values are checked for the threshold criterion: $V = \sqrt{gh}$.
Together with the volume, also the velocity vectors are passed for the next iteration. Mean flow velocity is defined as the arithmetic mean of the current velocity calculation and the vector addition. By this, sudden changes in flow behaviour will be minimized and inertia effects rendered in a simplified way.
The smallest iteration time step is adjusted dynamically. An important control criterion for this adjustment is the amount of water available. If the discharge rates become too large compared with the available volume,

the iteration time step will be reduced. Only water level changes exceeding 1 mm are considered by that control mechanism. If the volumes exchanged between cells are very small, the algorithm will increase the iteration time step. This permanent optimization keeps processing time at a minimum.

<u>Remarks</u>

FloodArea is primarily intended to calculate areas affected by a flood, and not to model other physical processes. It has been used in few literature cases (Mueller et al., 2009).
Essentially it is a simplified two-dimensional hydraulic model, integrated in a GIS. The simplifications mainly affect open channel hydraulics, which can be described only roughly with the available parameters (resolution of the elevation model in the channel, no cross sections). Furthermore the algorithms do not contain the impulse transfer, therefore some phenomenon such as the sloping of a water level in a river bend is not described correctly (FloodArea User Manual, 2006).
Other issues are discussed later.

CHAPTER 5
Study area

5.1 General description

The research has been applied on a portion of the territory of the Mountain Consortium of Municipalities of Valtellina di Tirano (*Comunità Montana* Valtellina di Tirano, in Italian), and particularly on the floodplain of the municipalities of Tirano, Villa di Tirano, Bianzone and Teglio, for an area of 26 km^2 (Figure 5.1).

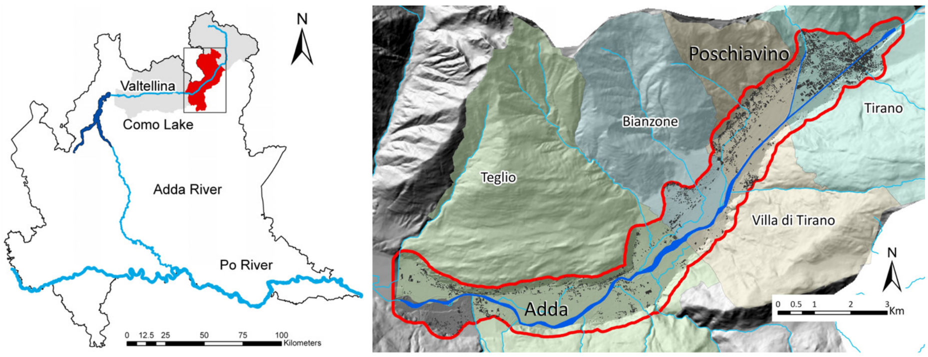


Figure 5.1 – Geographical location of Valtellina di Tirano (on the left) and delimitation of the study area (on the right).

The territory represents a typical alpine valley, where steep flanks delimit a quite narrow valley (extending from 500 m to 2 km), which is covered by quaternary deposits produced by glacial and subsequent fluvial activity, crossed by the Adda river. Several minor rivers join Adda, flowing from tributary valleys which close in correspondence of alluvial fans. The main affluent is the Poschiavino torrent, which originates in Swiss territory and joins Adda at Tirano (Figure 5.4a).

Land use in the area (Figure 5.2) is strongly controlled by climate and relief: human settlements, agriculture and light industry are concentrated on the bottom of the valley and on alluvial fans; lower parts of south facing (Rhaetic) slopes are covered by vineyards and apple orchards;

north facing (Orobic) slopes are mainly occupied by forests. Grass covers the main part of the floodplain (Figure 5.3).

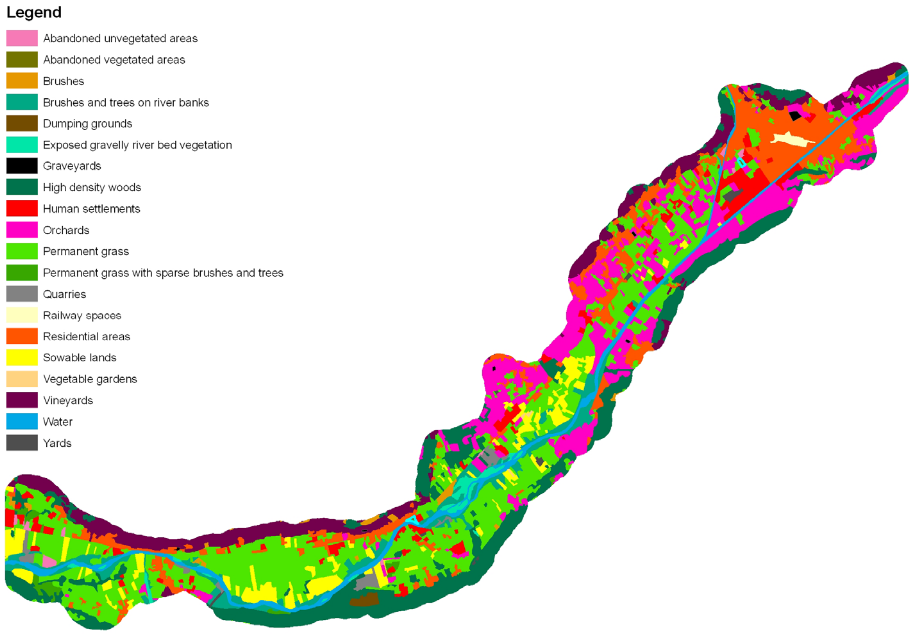


Figure 5.2 – Land use of the study area (DUSAF 2.1, 2007).

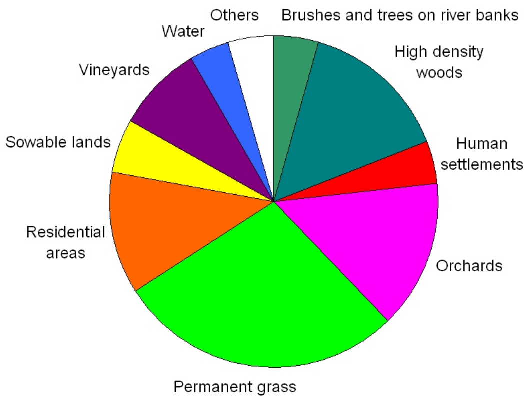


Figure 5.3 – Distribution of land use types.

5.2 Hydrological and hydraulic aspects

The Adda is the main river responsible for floods in the study area. Its bed elevation ranges from 453 to 347 m a.s.l., with a slope of around 1% in the upper part and 0.2% in the lower part of the reach, being 17 km the total length of Adda reach within the area. Its morphology is particularly

heterogeneous. Where slope is steeper, i.e. in Tirano and Villa di Tirano, Adda is almost completely artificial (Figure 5.4b): the shape of cross sections is trapezoidal with a clear definition of banks and no presence of vegetation, the flow direction is rectilinear and the width ranges from 25 to 35 m. When it leaves urban areas and runs in the fields through orchards and lawns slope decreases, wide bends appear, and river width reaches 200 m (Figure 5.4c). When meanders approach the main road (National Road – SS 38), the river bed reduces its width and adopts yet another morphology (Figure 5.4d).
Besides Poschiavino, other minor affluents (which will not be considered in the modelling phase) are, from NE to SO: Val Maggiore, Rivalone, Bianzone, Boalzo, Caronella, Bondone, Margatta and Malgina. Erosional processes act upon various portions of the Adda river reach, and are accentuated by the often considerable solid transport conveyed by affluents in case of severe rainfall events.

Figure 5.4 – Photos representing the heterogeneity of Adda riverbed in the study area.

The Adda river basin above the Lake of Como comprises an area of almost 4,000 km^2. It includes highly variable features and landforms as it includes a range of elevations of more than 3,000 m. The presence of both a dense river network and artificial basins exploited for hydro-electrical purposes by AEM/A2A with a total capacity of approximately

400 million m^3 (Figure 5.6) makes the hydrological analysis of the basin quite complex (this is probably the reason why similar studies have not been performed in recent years). Only three stations are available which provide historical data of river discharges (near Bormio, at Tirano, and at the river closure section at Fuentes – PAI, 2001), which is a number clearly inadequate to the characteristics of the basin. These stations, moreover, have data whose timespan is limited, covering often periods not wide enough to perform reliable statistical analyses. That is the reason for which, when comparing discharges calculated by different studies for different return times, values could be considerably different (AIPO, 2008). Moreover, due to all the changes that occurred after the catastrophic 1987 event, the basin characteristics are quite changed, and relying to old data may introduce notable errors. Until an extended and complete hydrological basin analysis will be performed, e.g. making use of physically based models, the wisest solution for flood hazard analysis approaches which apply to local and/or regional planning is to refer to institutional discharges, provided by PAI (2001).

Within the study area there are four gauging stations currently operating (Figure 5.5). From upstream to downstream they are:

- Madonna di Tirano, owned by the *Centro Monitoraggio Geologico di Sondrio*, monitoring the Poschiavino torrent;
- Tirano, owned by the AEM/A2A hydroelectric company;
- Stazzona, owned by the Adda Consortium;
- S. Giacomo, owned by the Consortium of Valtellina di Tirano.

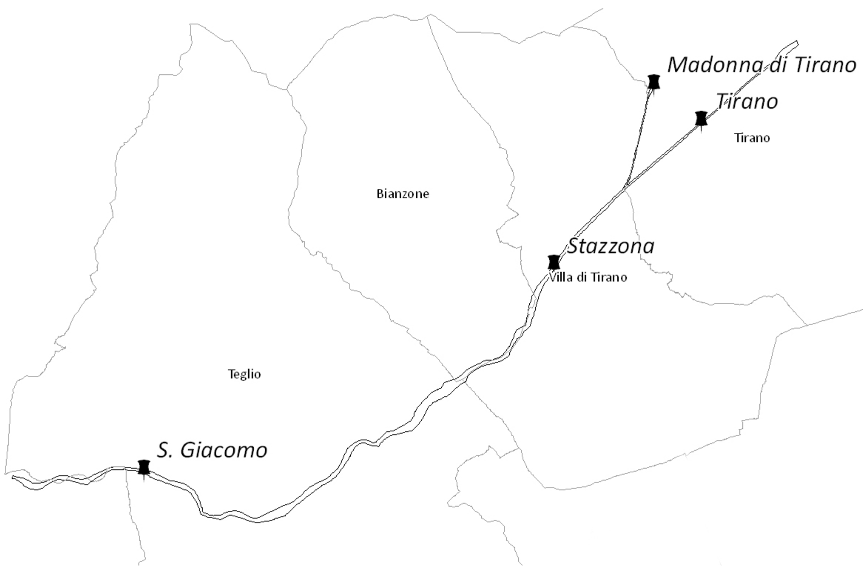


Figure 5.5 – Location of river gauging stations currently operating in the study area.

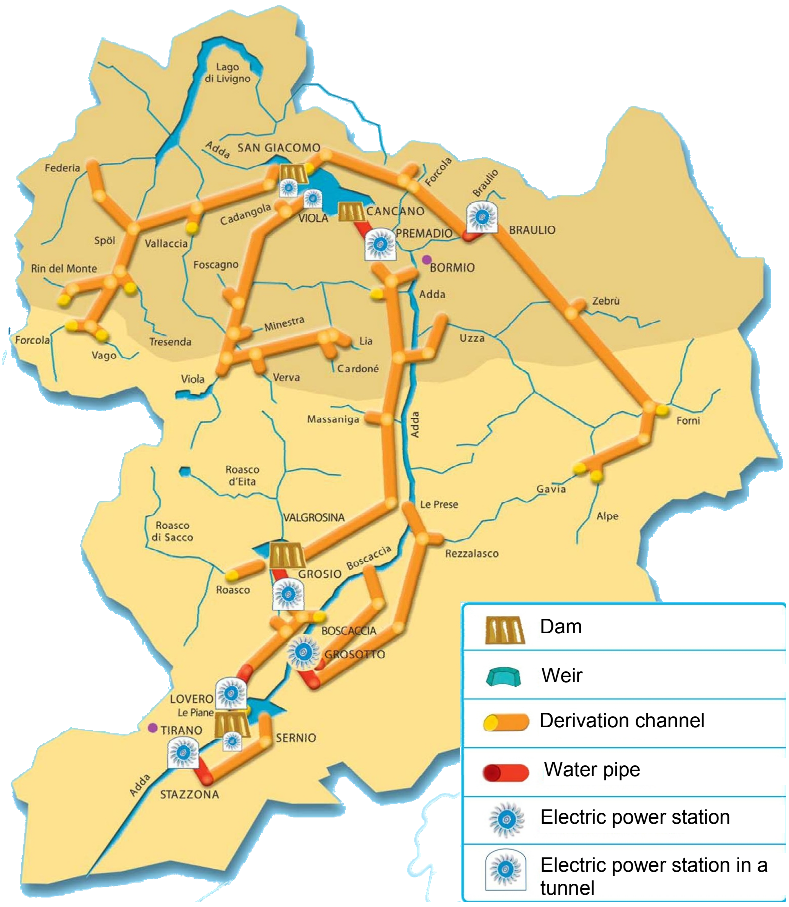


Figure 5.6 – Hydroelectric system of AEM/A2A in Valtellina.

Besides the common problems associated to river gauges which often experience problems and lacking of data (see also Hunter et al., 2007), these stations are managed by different entities, and each of them has its operational standards and willingness to deliver data. Their combined use is therefore limited, but it was possible to get enough data to define a yearly discharge trend (referred to the year 2009) for the Adda river basing on Stazzona and S. Giacomo gauging stations. This reach is also interesting because the A2A water pipe releases water coming from the Sernio basin immediately after the Stazzona gauging station, so the contribution of the upper part of the Adda basin is evident here.

As it can be seen from graphs Figure 5.9, Figure 5.10 and Figure 5.11, Adda discharge varies significantly along the reach during both the year (which is reasonable due the different water availability within the months) and the day. The graphs refer to the same tree days (8, 9 and 10) of each

month (the whole monthly trend would have been too long to be shown). Data of both discharge and rainfall are available at 30 minutes intervals.
Ordinary discharge when water is not released for low demand of hydroelectric energy, which is usually during the night, is comprised between 3 and 10 m^3/s, while it increases until 30 to 100 m^3/s when water is released during the day. The increase registered at the Stazzona station is related to the contribution of Poschiavino, which is also subject to releases distributed during the day.
Discharge is naturally higher in summer (months of June and July).

Due to the artificiality of the discharge trend, it is not possible to establish a relation between values at Stazzona and S. Giacomo (Figure 5.7). Minimum values of discharge are similar (see Figure 5.8) due to their natural origin and the fact that lateral inflows from minor affluents in the reach is limited. Mean and maximum values, instead, are highly variable.

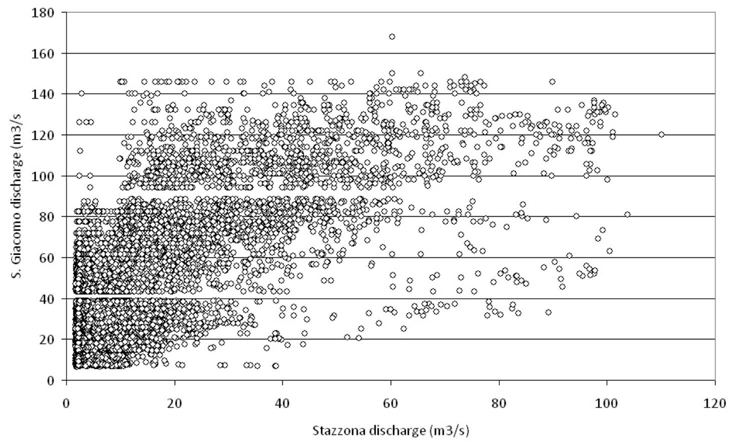


Figure 5.7 – Relation among Adda discharges at Stazzona and S. Giacomo.

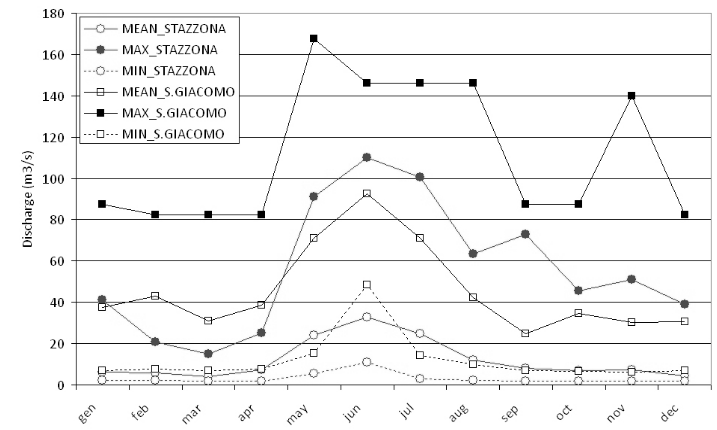


Figure 5.8 – Monthly mean, maximum and minimum Adda discharges.

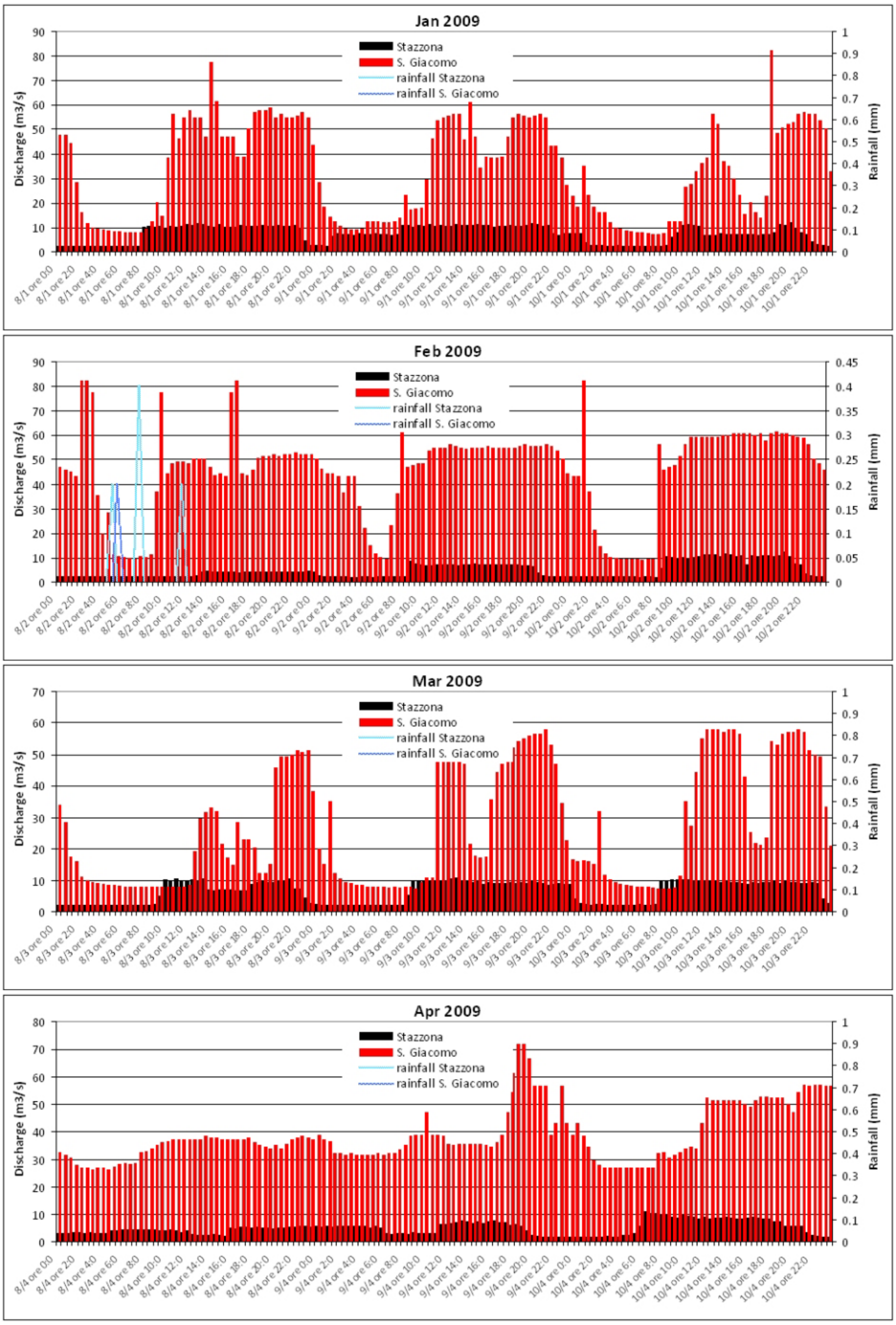


Figure 5.9 – Adda discharge trend for the year 2009, related to three days (January to April).

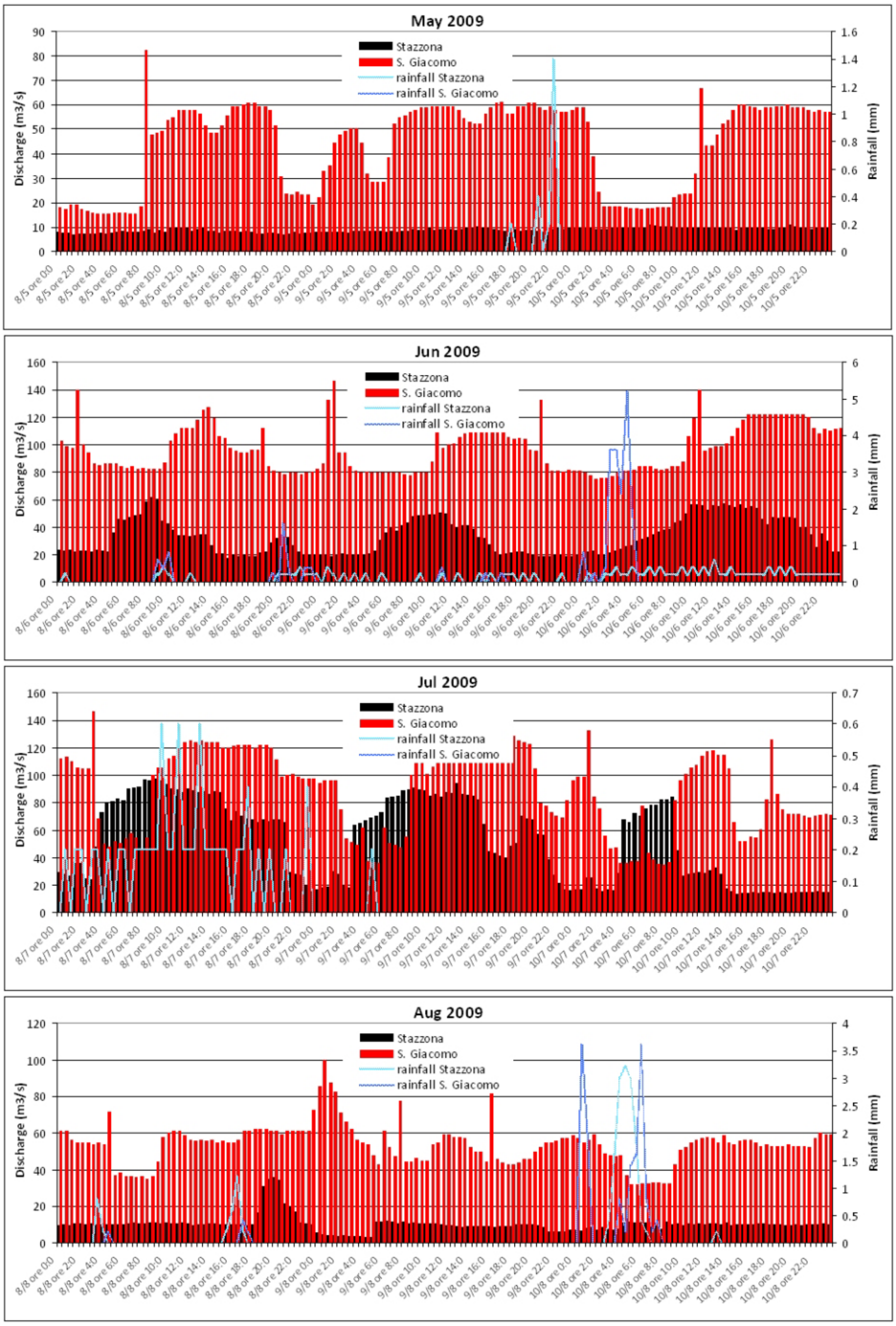


Figure 5.10 – Adda discharge trend for the year 2009, related to three days (May to August).

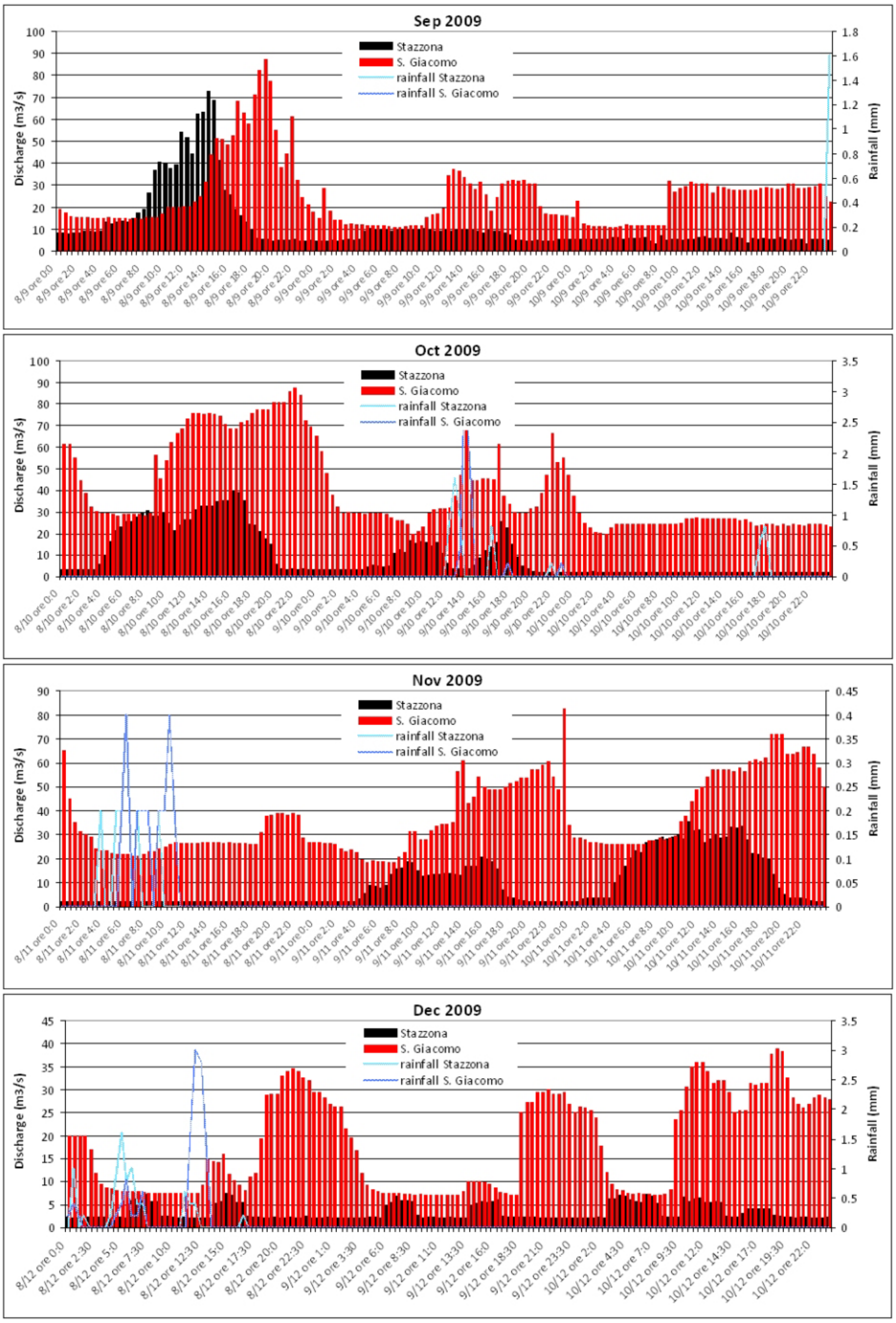


Figure 5.11 – Adda discharge trend for the year 2009, related to three days (September to December).

5.3 Available flood risk studies

Several recent and reliable sources were consulted and critically analysed relating to flood hazard and risk assessment in the study area.

Hydrogeological Plan for the Po river basin (PAI, 2001)

The description of PAI content is provided in par. 2.3. Within the study area, PAI states that the most hazardous sectors are located at the confluence of the Adda and Poschiavino streams and from the bridge of Stazzona in Villa di Tirano to S. Giacomo at Teglio, basing on historical evidence, and that there is a general instability situation due to minor river processes. It also provides the delimitation of *Fasce Fluviali* (Figure 5.12) for Adda. As can be seen from the picture, large areas could be affected by floods, and two portions in particular still require bank reconstruction works to ensure an adequate level of safety. These portions are delimitated by *Fascia* B "*di progetto*" and require in-depth hydraulic studies (which are partially provided by two of the four documents analysed further on).

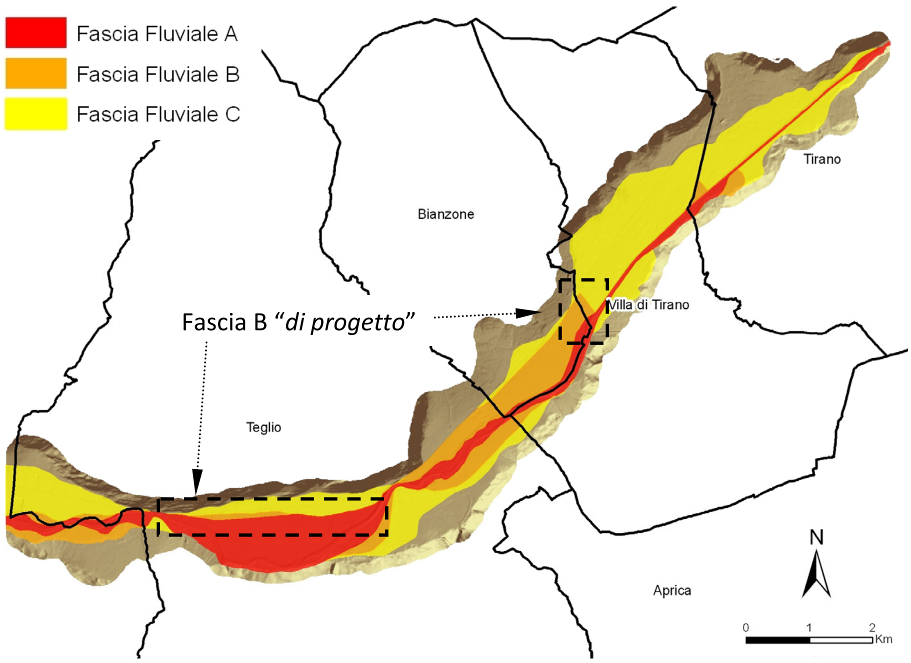


Figure 5.12 – Delimitation of *Fasce Fluviali* in the study area.

Moreover, PAI provides Adda discharges in various sections and for the 20, 100, 200 and 500 years return times (the last is considered unreliable, this is why it will not be used in the modelling phase).

Three of the four Municipalities where not in compliance with PAI provisions, so they had to conduct new geologic studies, which were completed in 2003. Bianzone was the only compliant Municipality; however, it provided a geologic update in 2006.

Geological studies for Municipal PGTs

Synthesis maps treat the presence of flood-prone areas as complementary geological information that could be employed to define land use limitations.
Among the maps produced by the four Municipalities, several differences exist, testifying the level of subjectivity included in these mapping approaches. In particular, Tirano strove for a classification in compliance with DGR 8/1566 regulation (Figure 5.13a), even if a clear description of the methodology applied is not provided. Similar considerations refer to the territory of Villa Di Tirano, although less effort was spent here in trying to add more local information to the PAI (Figure 5.13b); Bianzone did not provide any additional information to *Fasce Fluviali* (Figure 5.13c), and the same holds for Teglio. However, for this last Municipality, another study related to flood hazard assessment improved this delimitation.

Hydraulic study for engineering evaluations in Tirano (ANAS S.p.a. and Lombardy Region, 2002)

This study is a part of the EIA (Environmental Impact Assessment) procedures for the design of a new ring road in Tirano, and it aims to assess whether any negative interaction could be produced on existing local flood hazard conditions. Its usefulness for the research regards mainly the analyses of the flow contribution in case of flood of Poschiavino and Rivalone torrent, since for Adda, as all the other available studies, it makes use of PAI peak discharges. For Poschiavino, a concentration time (i.e. the time necessary to reach the peak discharge in the flood hydrograph) of 6 hours has been derived, a peak discharge equal to 166 m^3/s has been calculated for the 200 years RT, and Manning coefficients have been defined for the reach ranging from 0.03 to 0.05; results of the analysis showed that overflows can occur close to the Adda confluence. It seems, instead, that the Rivalone contribution in case of flood it not significant. It is confirmed again that possible Adda overflows are expected at the Poschiavino confluence and immediately downstream on the hydrographical right.

Hydraulic study for river banks design in Tirano and Villa di Tirano (AIPO, 2008)

This study was conducted with the aim of providing a hydraulic base for the design of protection measures for the Adda reach comprised between the industrial area of Tirano (before the Poschiavino confluence) and the Stazzona bridge in Villa di Tirano. These measures should conclude the necessary works requested by Po basin Authority in order to protect these areas from Adda floods, which were started in Tirano in 2000 and partially concluded in 2004, establishing the actual artificial riverbed morphology along the civic crossing.

Hydraulic simulations were performed making use of the 1D HEC-RAS software package (commonly used for engineering applications), assuming a hydrograph obtained by a quite coarse geomorphological approach, with a peak discharge for the 200 years return time flood derived erroneously by the PAI discharge value provided for the Tirano station (because of an error in PAI maps, there has been a misunderstanding of whether a gauging station was located before or after the Poschiavino confluence). Together with 1D simulations, necessary to define engineering details, a 2D analysis was also performed in order to define risk conditions supposing the new works are concluded (to solve the "*di progetto*" bond on *Fascia* B). Due to the wrong discharge value adopted, results are not reliable for the specified return time; it seems anyway that these works should ensure a higher protection level until a few hundreds of meters after the Stazzona bridge.

Useful quantitative data for the research are roughness coefficients defined for 1D modelling along the reach.

Hydraulic study for the evaluation of flood risk in Teglio and Bianzone (Merizzi and Baldini, 2007)

This study was the most in-depth and appropriate since its aim is flood risk assessment. Three causes are indentified for floods occurring in the area: (1) decreasing of flow velocity, (2) raising of water level and (3) deposition of sediments carried by affluents. Risk conditions are defined basing on extent of historical floods (Figure 5.13b-e), even if sources of information are not cited, and expected water depths and flow velocities are provided, which are obtained by HEC-RAS 1D simulations and geomorphological flood line tracing (this basic approach is suggested by DGR 8/1566). A flood extent for the 200 years return time flood is thus provided, which will be referred to in the comparison phase of modelling results (see par. 7.1).

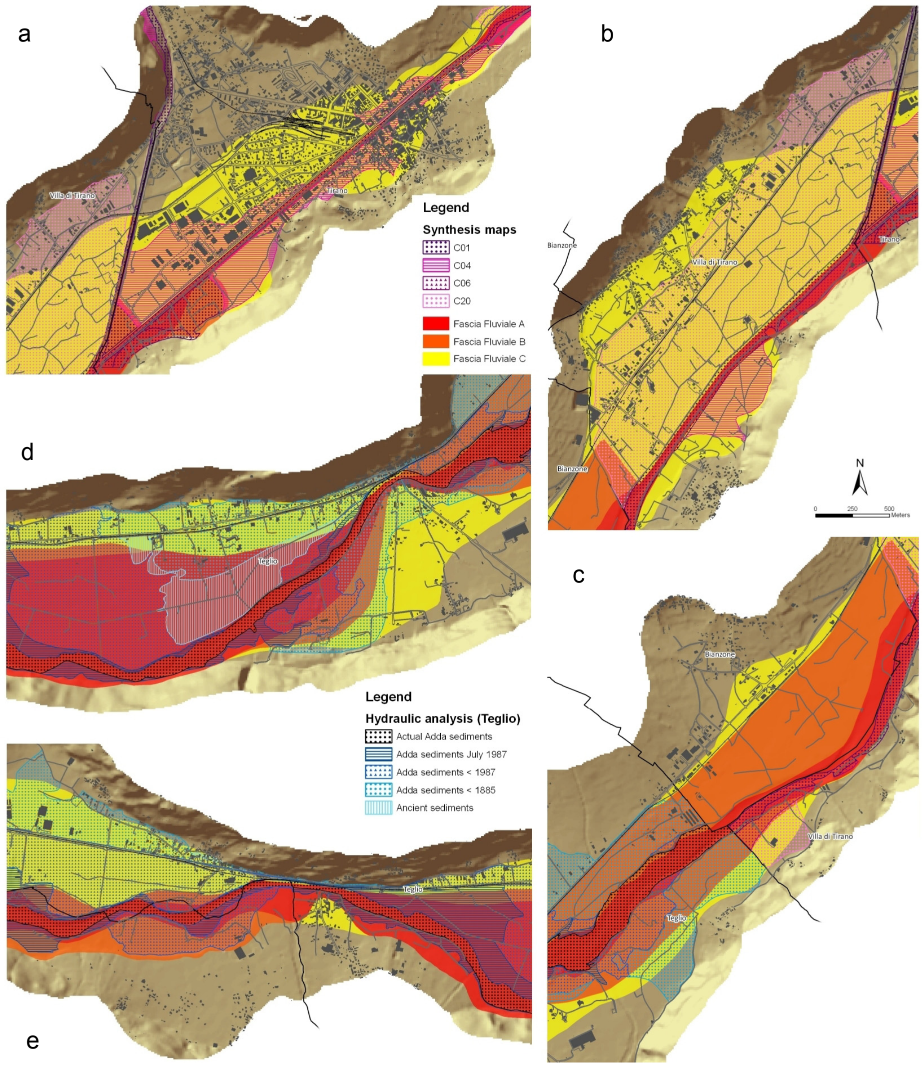


Figure 5.13 – Cartographies of studies related to flood hazard in the study area (from NE to SW the order of maps is a-b-c-d-e). In the legend: C01= frequently flooded areas (TR<50 years); C04 = less frequently flooded areas (TR<100 years); C06 = areas potentially flooded due to current conditions (narrow cross sections, erosional processes, possible banks breaks, etc.); C20 = areas with an even lower probability to be flooded.

5.4 Conclusions

From the previous paragraphs, a very complex situations for the study area emerges. A flood hazard is present, and it threatens agricultural lands, urban areas and infrastructure. The two main rivers responsible for floods are Adda and its affluent, the Poschiavino torrent, but their actual hazard conditions have not been analysed and mapped in an efficient way: several local studies are available, but they apply different methodologies (some of them are quite crude) on small and unconnected portions of the channel, lacking on validation data, since historical data on floods is scarce (see also par. 6.1). PAI, on the other hand, operates on a scale which is not detailed enough to describe local hazard conditions. A more appropriate study is thus necessary, but it is unfortunately hampered by the difficulty of developing a basic hydrological study which relates rainfalls to expected discharges, because of the high artificiality of river network discharge trend.

CHAPTER 6
Methodology

In order to be able to define flood hazard conditions for the study area, the following steps were taken:

1. historical analysis of past flood events;
2. collection of data and available documentation/reports/studies;
3. preparation of modelling input data;
4. experimental modelling;
5. application of models on the entire study area.

Each step will be discussed in the following paragraphs.

6.1 Historical analysis of flood events

Historical knowledge is a fundamental basis for natural hazards assessment, especially when the studied phenomena can recur in the future similarly than in the past (Ferrier and Haque, 2003).
Information of historical events for the entire territory of Valtellina di Tirano was collected and organised in a database, comprising both landslides and floods.
The following sources were consulted to retrieve information:

- national AVI database: A Bibliographical and Archive Inventory of Landslides and Floods in Italy (CNR-GNDCI: Guzzetti et al., 1994);
- PAI: Hydrogeological Plan of the Po River Basin (PAI, 2001);
- the book "Bibliographical Research for a Catalogue on Landslides and Floods in Valtellina" compiled by Govi and Turitto from the CNR-IRPI of Turin (1994);
- other books from the National Research Council – CNR (Guida et al., 1979; CNR-GNDCI, 1983; Govi et al., 1996; Cardinali et al., 1998; Tropeano et al., 1999; Tropeano et al., 2006)
- municipal geological reports;
- interviews with Mountain Consortium representatives dealing with territorial and civil protection management.

There are 86 records within the database referring to floods. They are distributed all over the territory of the Mountain Consortium (see Figure

6.1), but especially in Teglio and Tirano, covering the period from 1616 to 1987. Even if the characterization is not always clear, they can be divided in flood plain inundations (about 60%) and events in which only a rapid increase of discharge was registered (about 40%). In some cases, in fact, “flood” does not mean inundation, but simply the state of a river that is at an abnormally high level. The events for which information about the month is available occurred mainly in the period between July and November. Quantitative data are available only for the records related to high discharges and only for the Adda River, while for inundations a general description of affected areas and occurred damage is usually provided.
Adda River is responsible for almost half of the events, and in nine cases it acted in synergy with two of its main tributaries, the Poschiavino and Roasco torrents. The other events were caused by tributary rivers/torrents, which include debris transport. Regarding the location, 30% of floods occurred on fan areas. In five cases, inundations were caused by dike breaks; they happened in Bianzone, Teglio and Tirano in the second half of the 19th century.

Geo-referencing inundations posed several problems, since most events are represented only by points, without any further specification of areal extent. Aerial photos could help in defining flooded areas, but unfortunately we couldn’t obtain complete covers for main events; not even many ground photos have been found.

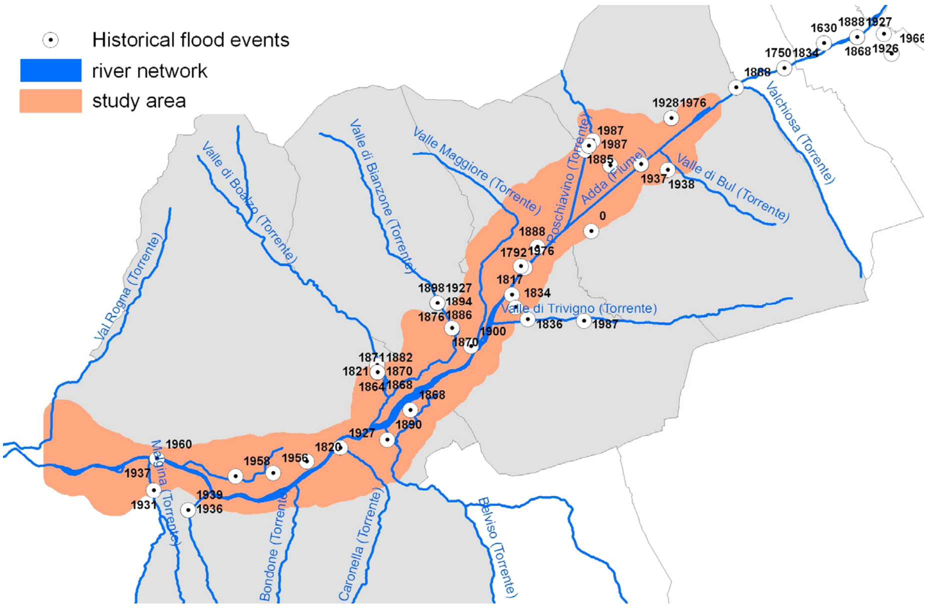


Figure 6.1 – Location and year of occurrence of historical floods in the study area.

Analysing collected data, it was noticed that only a few data are available regarding discharges in the case of past flood events, which were, to sum up, quite frequent in the past (seven large events occurred from 1900 to 2000) but which have decreased in recent decades (there have been no noteworthy events from 2000 till the time of writing). This could have different meteorological and hydrological reasons, but the main one should be that after the severe event of 1987 (Luino, 2005) protection measures were taken and river morphology was redesigned in some areas to ensure a higher level of safety.

From available documents, it seems that the highest discharge ever reached by Adda at Tirano was around 600 m3/s in 1960, while the second should be reached in the 1987 flood. During this last event, the main inundation was caused by the Poschiavino at Madonna di Tirano, while secondary overflows were registered immediately upstream of the S. Giacomo bridge in Teglio (Merizzi and Baldini, 2007); the bridge itself was severely damaged and reconstructed in 1991. It is generally thought that worse damages were avoided because the Adda riverbed was particularly low in those years. Information about this event, however, are not very useful for the research purposes for three main reasons:

- the value of peak discharge is uncertain (ranging from 500 to 900 m^3/s), since Tirano gauge station was damaged during the event; therefore, it is not possible to associate a return time to the event, or to perform a back analysis;
- the lack of aerial photos does not allow one to define the flood extent;
- even if these photos were hypothetically available, the morphological setting of the Adda riverbed changed profoundly during the subsequent years, so a direct comparison of actual overflow conditions is not possible.

Merizzi and Baldini (2007) provide a very interesting historical delimitation of flooded areas, but unfortunately it is limited to the territory of Teglio.

6.2 Collection of data and documentation

All the available data were collected in various stages directly from the administrative offices of both the Consortium and single Municipalities, since this is the level at which the most detailed and updated data should be accessible. Hydrogeological reports and a 3D territorial cartography at 1:2000 scale were thus obtained. Further researches involved an in-depth analysis of PAI contents, regional and provincial studies and databases, and internet surveys.

6.3 Input data preparation

As stated in par. 3.3, the necessary data to perform flood modelling are related to topography, friction setting and boundary conditions. They were all produced with the aim of performing both 2D and coupled 1D2D simulations, and considering a river network composed by Adda and its main affluent, the Poschiavino stream, since the other affluents seem to have only a minor influence in case of a flood, i.e. a very low value of liquid peak discharge (while their contribution in debris transport could be high). Spatial data were processed making use of ArcGIS 9.2.

6.3.1 Definition of topography

6.3.1.1 1D cross sections

The representation of riverbed morphology in 1D2D models requires a series of cross sections transverse to the main flow direction, i.e. the river axis (or thalweg).

Three sources providing this kind of data were found (Figure 6.2), and it was decided that the spatial distribution was sufficiently dense for not to proceed with a new (costly and time-consuming) topographical survey.

Due to the different origin and time of acquisition, there are some differences when comparing almost overlapping cross sections; therefore, they were slightly modified in order to produce a quite homogeneous set of data, based on the assumption that the more recent data are the most accurate.

A tool was then applied (Merwade et al., 2006; Merwade et al., 2008) which interpolates cross sections to produce a 3D mesh. It requires that cross sections are represented as a PolylineZ, i.e. an ESRI feature that has a Z elevation attribute. Part of the mesh can be seen in 3D view in Figure 6.2.

The clear advantage of this tool is that the interpolation is accomplished in the coordinate system fitted to the channel, thus producing a realistic bathymetry. Interpolation, in fact, is a very important step since it determines shape, capacity and banks elevations of the channel. When it is performed by software packages, sometimes, the procedure adopted is not very clear. To apply the tool, it is necessary to define boundaries of the channel. Since the definition of banks is another key issue for flood modelling (Horritt and Bates, 2001; Merwade et al., 2008), in this phase only a buffer of 100 m was allowed both to the left and right of the channel as it is defined by Consortium cartography. A better definition of river banks is provided hereafter.

The resulting mesh was visually inspected. Some local details or sharp irregularities had to be manually corrected. They were caused by the often high heterogeneity of cross sections shape, especially where compound reaches are present. Elevations at known locations were verified to be consistent with input data.

Cross sections at a constant 100 m distance along the channel were then extracted from the mesh and their shape was analysed in order to define banks, i.e. the cross sections limits which define the riverbed only, considering also the location of road tracks running along the channel.

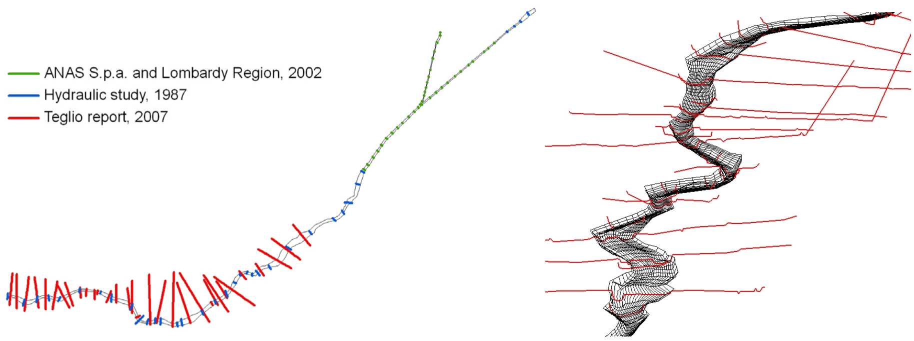


Figure 6.2 – On the left: distribution of original cross sections from available studies. On the right: constructed 3D mesh representing river bathymetry, with original cross sections in red colour.

This step was done combining four different approaches within ArcMap 9.2: aerial photos visualisation, consideration of the cartographic channel delimitation, analysis of flow direction and slope/aspect maps derived by a 2 m DEM obtained by the 3D fluvial mesh, and requiring that the following conditions are verified:

- flow direction and aspect to be as concordant as possible, perpendicular to flow direction towards the inner of the channel (Figure 6.3a)
- slope locally maximum or diminishing in the direction towards the out of the channel (Figure 6.3b)
- minimum difference, when reasonable, among the bank lines defined in this way and the channel area defined by cartography and aerial photos (Figure 6.3c).

Once defined bank lines, new correct cross sections were extracted from the mesh and simplified in their Y representation, being Y the transversal direction, since main of the YZ points were redundant. In particular cases, e.g. when the cross section coincided to a gauging station or a bridge crossing, the reconstructed cross section was substituted with the original one, in order not to introduce possible errors due to the interpolation of complex sections.

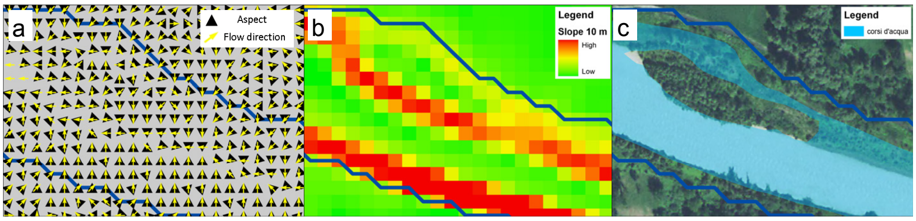


Figure 6.3 – Conditions applied to define river banks: (a) aspect and flow direction; (b) slope; (c) aerial photos and cartographical delimitation.

6.3.1.2 <u>2D Digital Elevation Models</u>

Floodplain elevation data contained in the 3D Consortium cartography included elevation points, contours, railway and road tracks, buildings and other infrastructure. The main useful information is related to ground surface, i.e. ground elevation points, contours, railway, road tracks and levees (Figure 6.4a), but not to buildings, since their representation is complex and moreover it could introduce even more uncertainties in the simulation results (Haile and Rienties, 2005; David et al., 2009; Néelz and Pender, 2009(2); Néelz and Pender, 2007). Similar difficulties yields in the reproduction of hydraulic works and bridges: attributing a different roughness value could be sufficient instead of modelling the engineering functioning of the object, but due to high uncertainties in its definition also these features were not included in the model. It seems, moreover, that their hydraulic influence is limited to their close neighbourhood and does not affect flood extent significantly (Pappenberger et al., 2006).

Analysing data, it was found out that many elevation errors were present (see Figure 6.4b), so much time was spent in correcting them; in particular, completely wrong elevations where substituted with more likely ones according to neighbour information, and sharp irregularities, especially in road tracks elevations, were smoothed.

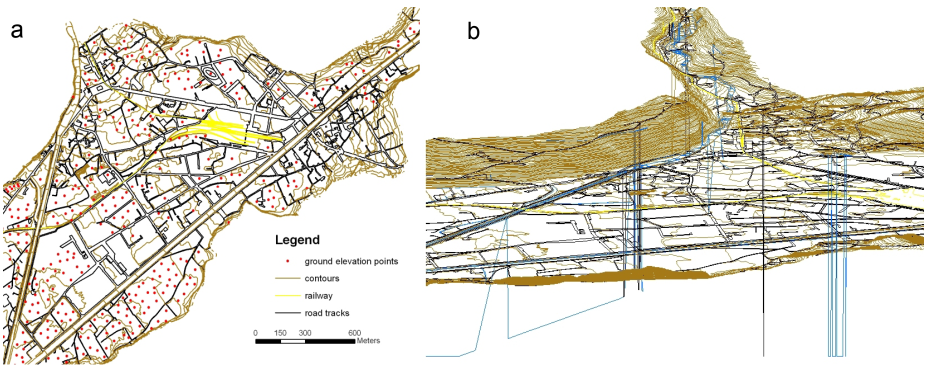


Figure 6.4 – Input 3D data (a) and elevation errors (b).

Thereafter, some raster interpolation methods available in ArcMap 9.2 were applied for deriving DEMs from these input data, including also the corrected 3D mesh. The main difficulty of all the approaches is that they require point elevation data, so polygons and lines had to be converted to points. Results, anyway, were not appropriate, since these methods fail in reproduce linear elements such as roads, banks and levees (Figure 6.5), whose representation is fundamental in floodplain modelling (Werner, 2004).

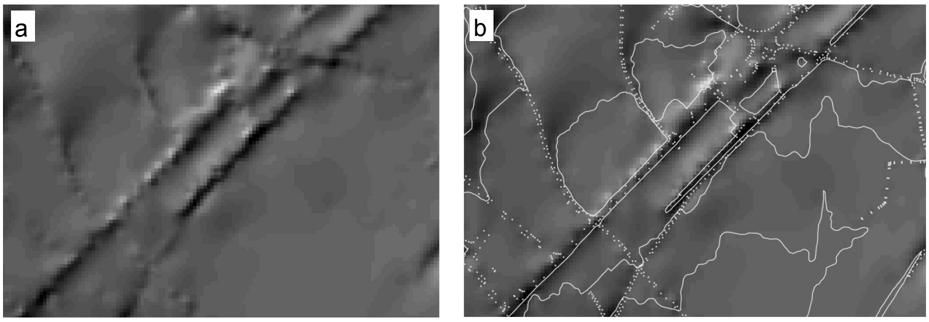


Figure 6.5 – Errors produced by raster based interpolation methods in the reproduction of linear features: (a) non continuity of a road track; (b) elevation data used.

A method which allows the use of lines is Topo to Raster (Wise, 2000), which is based on the ANUDEM algorithm developed by Hutchinson in 1988, but it seems more suitable to very natural and smooth landscapes than to the heterogeneous morphology of the study area. It also fails, anyhow, in the reproduction of linear features.

So, another methodology was tried. A basic DEM was constructed with Topo to Raster (TtR) making use of only ground elevation points and contours. A second DEM was constructed with the Nearest Neighbour (NN) technique basing on roads and levees elevation points. The two DEMs were then superimposed, as suggested also by Alkema (2007). Also this solutions proved not to be appropriate, since in some portions the sharp discontinuity among ground and road/levee level is not realistic (see Figure 6.6).

Finally, TIN (Triangulated Irregular Network) was utilised and it turned out to be the best possible approach. Differently from the other raster based methods, it has a vector data structure. It partitions geographic space using a set of irregularly spaced data points, each of which has x-, y- and z-values; these points are connected by edges that form contiguous, not overlapping triangles and create a continuous surface that represents the terrain. Its usefulness consist in the fact that it allows to represent a surface according to the local density of information available, and that it preserves linear features by the use of both soft or hard edges of triangles (Wise, 2000). The main problem in the TIN construction phase is that it requires input features not to be overlapping, since possible elevation differences will produce erroneous discontinuities. It was thus necessary to erase any part of overlapping polylines ensuring the right priority level, and to check coherence with elevation point data. This adjustment process was very time consuming.

The final representation (Figure 6.7 and Figure 6.8) was satisfying, since all the important features were represented in a correct way; the surface resulted to be quite sharp, but this is a consequence of the presence of many man-made features on the ground. A smoothing was performed afterwards, when TIN was converted into a DEM making use of the Nearest Neighbour technique. This step was necessary since SOBEK, FLO-2D and FloodArea require structured (raster) 2D grids. The use of DEMs is also wiser than the use of TIN when this last does not guarantee the level of detail of its representation, because of low quality of input data. A DEM with 5 m resolution or even more allows to consider a higher level of uncertainty.

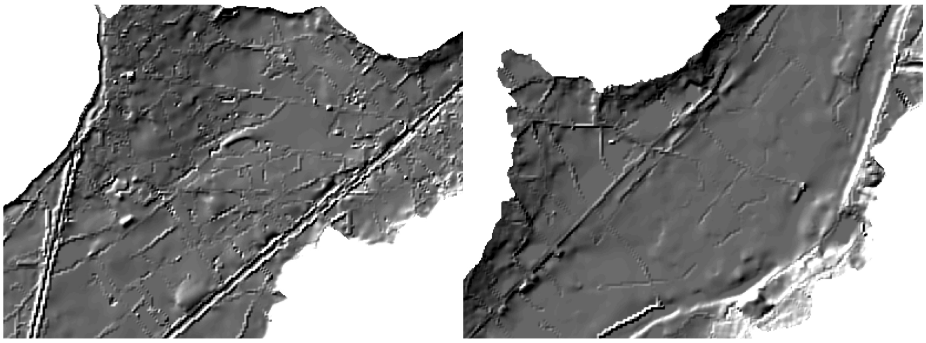


Figure 6.6 – Results of the DEM construction approach which superimposes roads and levees layers (NN technique) to ground layer (TtR technique) in two portions of the study area.

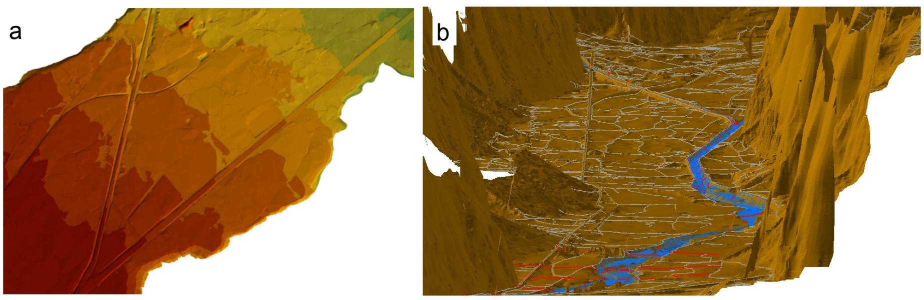


Figure 6.7 – Final TIN representation in the portion of Tirano: (a) 2D and (b) 3D view.

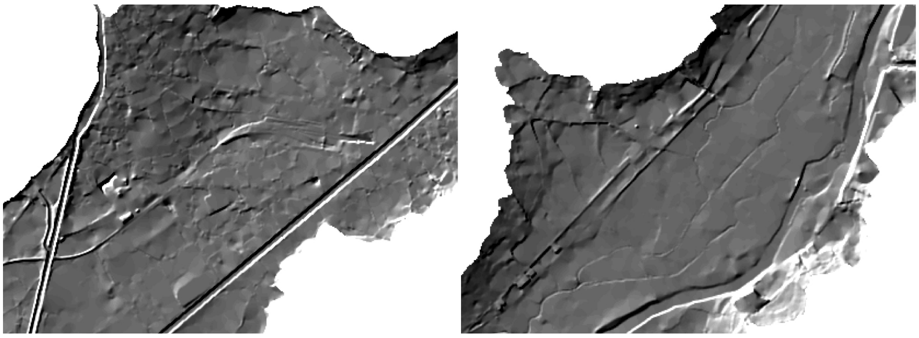


Figure 6.8 – Results of TIN interpolation which allows the comparison with NN-TtR previous approach.

Two types of DEMs were generated:

- complete topography DEMs, comprising also riverbed description, for 2D simulations (2D DEMs);
- DEMs where the riverbed description is limited to banks elevation, required by 1D2D models, as for river morphology they use the 1D module which is connected to 2D one by banks level (1D2D DEMs).

Due to the quality level of input data, it was established that DEMs could not have a resolution higher than 5 m.

An issue would be to establish whether fill technique, i.e. a procedure that automatically fills DEM depression, have to be applied at the DEMs to ensure water flow is not blocked by artificial obstructions. It was decided not to apply it, since it was assumed that input ground elevation data were reliable enough and moreover the TIN technique, differently from other raster based interpolation methods, does not produce many of these artificial depressions. In particular, the absence of significant depressions within the river bed is ensured by the interpolation tool used to create the 3D mesh. The greater utility of smoothing compared to filling technique is proved also by Wise (2000).

6.3.2 Friction setting

Since the Manning expression of friction, or its derivatives, is adopted by all the models, these coefficients were established both for the 1D (riverbed) and the 2D (floodplain) part. Uncertainty analysis on these coefficients will be presented later (see par. 7.2.2).

6.3.2.1 <u>1D roughness</u>

For the riverbed, values adopted by previous studies and ranging from 0.025 to 0.05 for Adda and Poschiavino were defined, trying to reproduce the characteristics of different types of reaches, especially in the area of Tirano and Villa di Tirano. These values were found to be consistent with physical characteristics of the river and so the were not questioned.

6.3.2.2 <u>2D roughness</u>

For the floodplain, n values related to land use (DUSAF 2.1 project) were adopted, according to Table 6.1 (Chow, 1959; Arcement and Schneider, 1989). Detailed definition of coefficients is not possible due to landuse scale (1:10000) and the general definition of classes. Time constraints did not allow to perform a more detailed survey, because the study area is very wide. Moreover, these values have been defined for 1D studies, so an adaptation to the 2D system could be required (see their sensitivity analysis in par. 7.2.2).

Table 6.1 – 2D floodplain Manning's n roughness values adopted in the simulations

Abandoned and not vegetated areas	0.05
Uncultivated green areas	0.05
Broad-leaf forests	0.04
Quarries and yards	0.02
Brushes	0.07
Cultivated lands	0.03 – 0.09
Banks vegetation	0.035
Orchards	0.035
Asphalted surfaces	0.02
Lawns	0.13
Lawns with some brushes and trees	0.2

6.3.3 Boundary conditions

To reconstruct a likely hydrograph for the Adda upstream conditions in Tirano, two discharge trends were analysed: the first one is a reconstruction of the 1987 event (Magistrato per il Po, 1997) and the second is the result of a geomorphological analysis performed by AIPO (2008). Adda concentration (or lag) time varies among 20 and 40 hours, so a medium value was adopted (30 hours). Considering also the falling limb of the hydrograph, a total of 70 hours were computed to restate discharges to their original ordinary conditions (equal to 30 m^3/s - even if the real value is highly variable - for Adda and 10 m^3/s for Poschiavino).

The peak discharges provided by PAI for the 20, 100 and 200 years return time, are respectively 530, 750 and 830 m^3/s.
Another hydrological study calculates for Poschiavino a concentration time of 6 hours (Lombardy Region and ANAS S.p.a., 2002). Peak discharges were derived by PAI, corresponding to 106, 133 and 145 m^3/s for the same Adda return times.
In the absence of sound indications to define a shape, Adda and Poschiavino hydrographs were supposed to be simply triangular (Figure 6.9); this is also a precautionary assumption, as input volumes will be slightly superior than in reality.

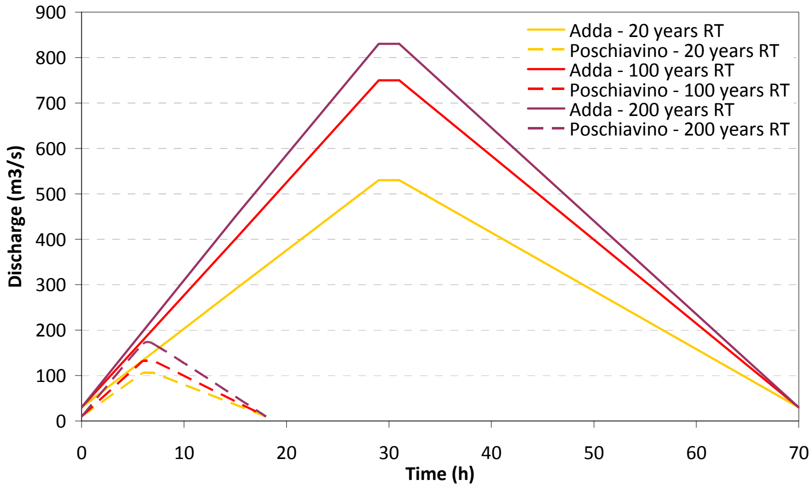


Figure 6.9 – Input hydrographs for Adda and Poschiavino.

6.4 Experimental modelling

In order to compare the performances of the three available flood models, some experiments were primarily conducted.

2D simulations were initially tried in the area of Tirano, but even at the highest resolution allowed by input data, i.e. 5 m, overflows occurred even for the lowest return time, which is evidently wrong. This is reasonable due to the fact that 2D modelling has been designed to deal with high resolution data. SOBEK developers suggested that within riverbed width there should be at least ten grid cells in order to be able to reproduce reasonably flow dynamics, capacity and conveyance processes (this is confirmed also by Verwey, 2005 and Werner, 2004). Adda and Poschiavino riverbeds in the area of Tirano and until the Stazzona bridge in Villa di Tirano are from 20 to 40 m wide, and therefore even using the 5 m DEM there are only four to eight cells within it, which is not enough. It was thus verified that 2D modelling is not a suitable

approach for the whole territory of the study area, but in order to compare it with the other approaches some tests were performed on a smaller area, comprised between the Stazzona bridge in Villa di Tirano and the S. Giacomo bridge in Teglio, where Adda riverbed is at least 50 m wide.
In this test area, several experiments were conducted, first with the aim to analyse in-channel results (input defined as a constant discharge equal to 30, 60 and 100 m^3/s flowing for 2 hours), in a second phase supposing overflows and expansion of water in the floodplain (input defined as a fictitious hydrograph extending for 9 hours with a peak discharge equal to 550 m^3/s).

During these experiments, some important outputs were discovered.

- 2D simulations took really a lot of time to run and so it was decided that this approach is definitely not suitable for flood modelling in the study area, because it does not allow multiple runs which would be necessary to assess model requirements and to perform even a simple calibration. Anyway, for in-channel analysis these results were compared with the ones from 1D modelling.
- The use of an appropriate 2D resolution did not solve the problem of roughness definition. When applying for 2D simulations the same roughness coefficients used for 1D2D simulations (which are set as the mean literature tabulated values for the specific physical conditions of the channel), water did not run for the same distance, but for a shorter one; it was therefore necessary to reduce the roughness (this problem is highlighted also by Werner, 2004 and Morvan et al., 2008). The amount of reduction was even different between SOBEK and FloodArea, the latter requiring a higher reduction.
- 2D flood simulations with FLO-2D were not possible since at the resolution of 5 m the model hardly started to run within several hours.
- FLO-2D at the resolution of 50 m resulted in problems which could not be solved within the available research time, so only the 100 m resolution model was preserved.

The experimental simulations which were finally run are summarised in Table 6.2. Their comparison, based on water level predictions, is reported in the following paragraphs.

Table 6.2 – Experimental simulations input data
nr = not requested

Software package	Numerical approach	Adda 1D roughness setting	2D resolution (m) – floodplain roughness setting
IN-CHANNEL ANALYSIS			
SOBEK	1D2D	n = 0.03	*nr*
SOBEK	2D	n = 0.015	5 – *nr*
FLO-2D	1D2D	n = 0.03	100 – *nr*
FLO-2D	1D2D	n = 0.03	50 – *nr*
FloodArea	2D	k_{St} = 174	5 – *nr*
FLOODPLAIN ANALYSIS			
SOBEK	1D2D	n = 0.03	variable according to riverbed width – n = 0.04
FLO-2D	1D2D	n = 0.03	100 – n = 0.04

6.4.1 In-channel SOBEK results

From Figure 6.10 it can be seen that SOBEK 1D and 2D results in term of water levels are quite concordant, except for the three points highlighted, which are however particularly critical since they represent two compound channel reaches and a bridge cross section, conditions that the 2D model could have reproduced with difficulty.
Some variance occur in the flow arrival time, but this should be mainly due to the different dimensional approach adopted and by the differences in topographical representation among cross sections and 5 m DEM. The setting of a lower roughness coefficient seems not to have the same effect for every value of discharge; the best agreement among arrival times is for the highest discharge (100 m^3/s).

6.4.2 In-channel FLO-2D results

From Figure 6.11 it can be seen that FLO-2D results at the two resolutions of 100 and 50 m are even more similar than SOBEK ones. It seems therefore that grid resolution has a low influence on water levels predictions within the channel.

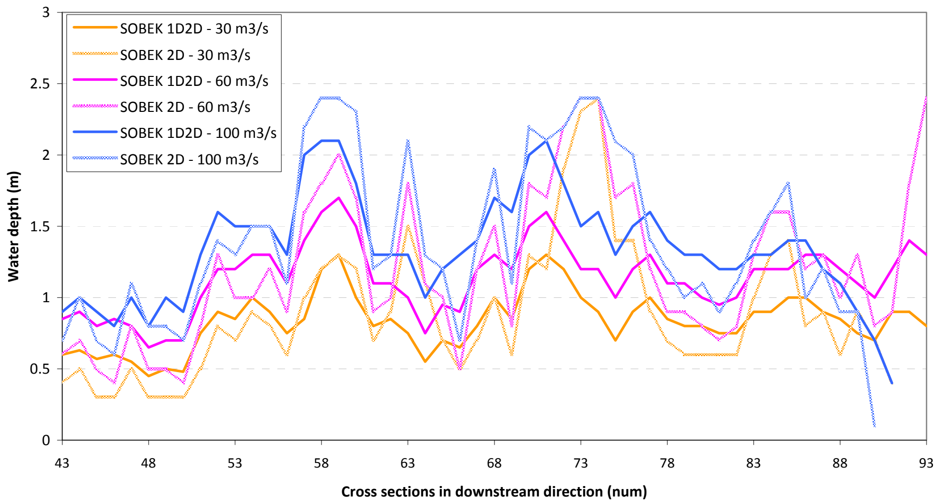


Figure 6.10 – In-channel SOBEK experimental results.

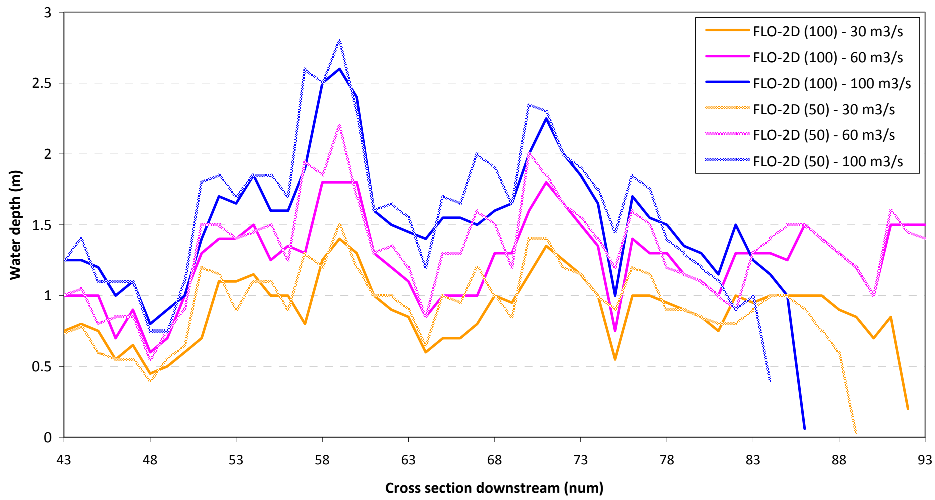


Figure 6.11 – In-channel FLO-2D experimental results.

6.4.3 In-channel all models results comparison

All modelling approaches from all the software packages were finally compared, and results are reported in Figure 6.12, Figure 6.13 and Figure 6.14, for the three flowing discharges. In the graphs, as a secondary Y axis the riverbed width is also shown, since it is important to highlight that every difference in water depth refers to cross sections which are from 50 to 200 m wide.

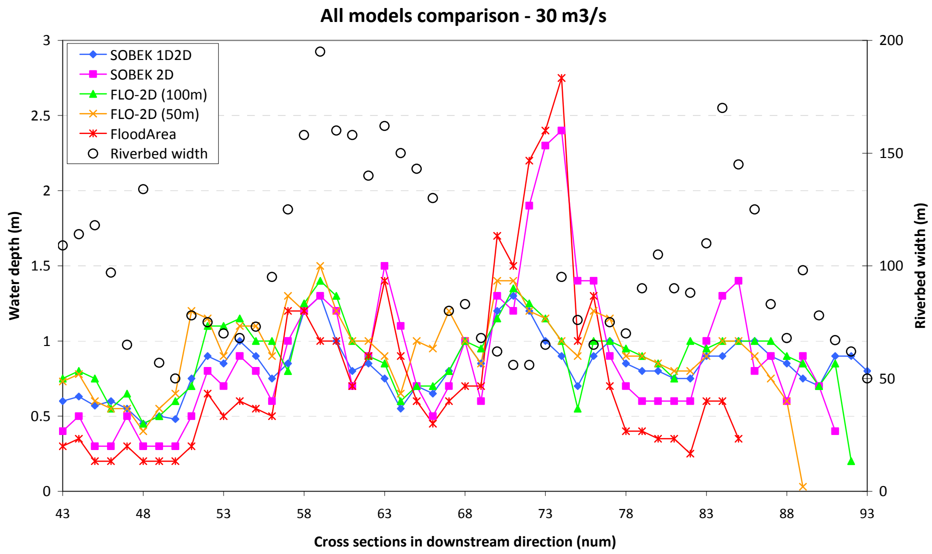


Figure 6.12 - In-channel all models experimental results comparison (Q=30 m^3/s).

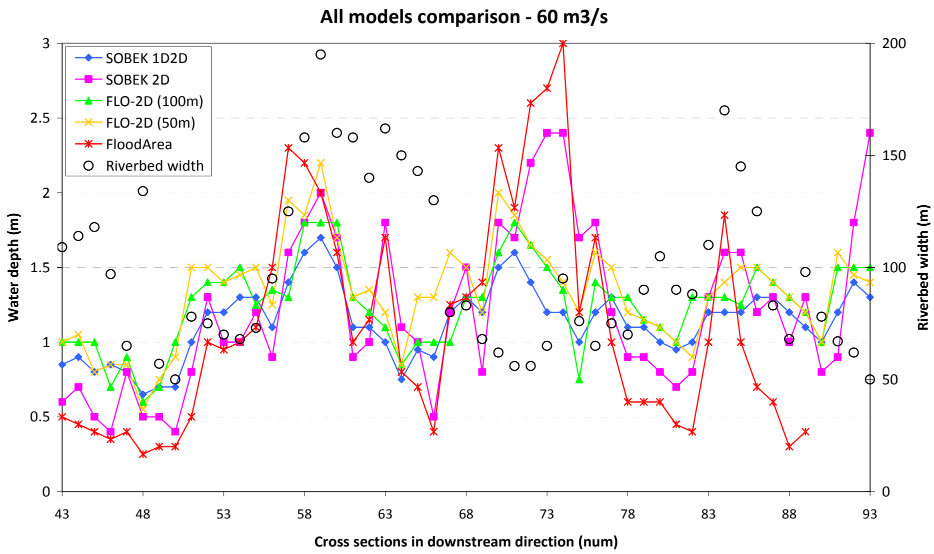


Figure 6.13 - In-channel all models experimental results comparison (Q=60 m^3/s).

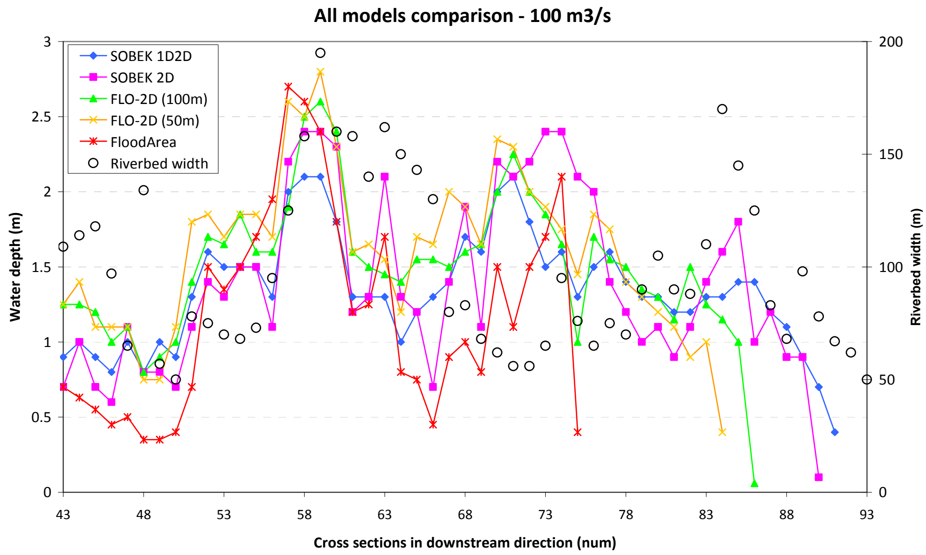


Figure 6.14 – In-channel all models experimental results comparison (Q= 100 m^3/s).

The following observations could be made:

- 2D models (SOBEK 2D and FloodArea) seem to perform in a similar way except for some minor details, but this becomes less true when the discharge increases (this general difference in model response to the calibration process for low and high flows has been observed also by Hunter et al., 2005);
- FloodArea results in much shorter travelling times compared to all the other approaches;
- water level range among models becomes higher for increasing discharge;
- SOBEK 1D2D and FLO-2D show the best agreement.

The most critical highlighted aspect is that friction setting cannot efficiently reproduce all the flowing conditions. Roughness parameters which ensured the best agreement among arrival times were set for low discharge simulations, but, as results show, this agreement is dependent on the discharge value (as observed also by Hunter et al., 2005).

In order to assess whether friction is the only parameter responsible for such differences, topographical representation of 1D and 2D systems was compared calculating the difference of elevation at every cross section YZ point. Elevation differences are only limited (Figure 6.15), and impossible to avoid completely. The shape of cross section was also compared from a sample of the whole set: except from few cases, cross sectional area does not change significantly, so it seems that friction is indeed the main responsible for predicted water depth differences.

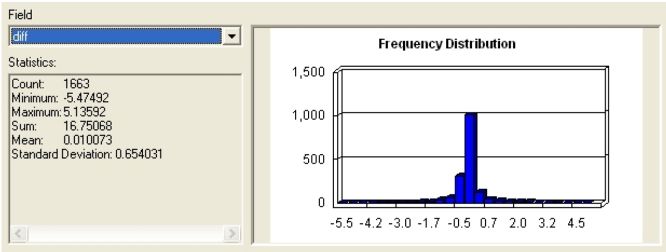


Figure 6.15 – Frequency distribution of elevation discrepancies among 1D and 2D channel representation.

6.4.4 Floodplain SOBEK 1D2D – FLO-2D comparison

The comparison among SOBEK and FLO-2D at 100 m resolution floodplain inundation results is satisfying (Figure 6.16). Applying the experimental hydrograph, both the models produce an overflow, with very similar flood extent, and only slightly different water depths. This is interesting because the two models adopt a different topographical representation, even if originating from the same input data. SOBEK 1D representation is exactly the one defined by the user by inserting the sequence of cross sections, and 2D grids have different resolutions according to the mean riverbed width of the various reaches (see par. 6.5), ranging from 50 to 100 m. FLO-2D, instead, adapts the extent of cross sections to the grid resolution, which is uniform and equal to 100 m, so banks location could be slightly different from original one. It seems therefore that both the models, even if in a different way, are able to capture the main processes that produce the overflow and to expand water on the floodplain in a similar way.

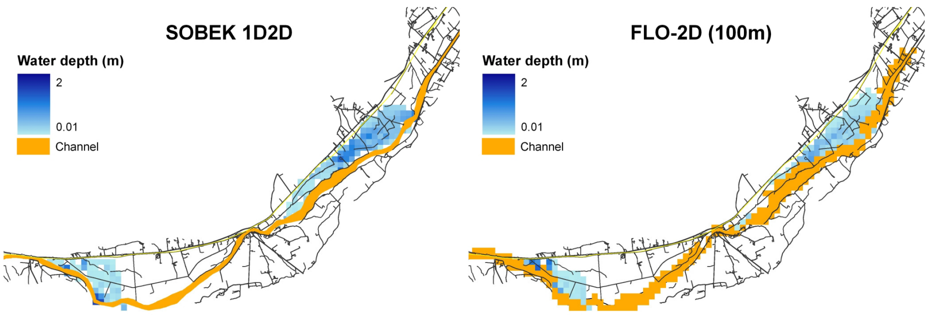


Figure 6.16 – Floodplain SOBEK 1D2D – FLO2D results comparison.

6.4.5 Verification with real data

In order to assess whether obtained results are realistic, they were compared with available data from cross sections. Since quantitative data on floodplain stages are not available, only in-channel water depth was verified.

As stated above, there are four gauging stations within the study area.
Except from the station of Madonna di Tirano which operates on Poschiavino, two of the other three stations have problems: Tirano, since it is owned by a private company, made available only data relating to hourly medium water levels (which have no utility if discharge values are not provided jointly), while Stazzona experiences some problems in the definition of Z zero value, so surveyed water depths have a degree of uncertainty, and the same holds for the cross section rating curve, i.e. the relation among discharge and water levels from experimental observation, which allows one to define one quantity if the other is known.
So, the only useful gauging station is S. Giacomo, for which a rating curve considered quite reliable (a degree of uncertainty is unavoidable, see Pappengerger et al., 2006) is available. According to the curve, when a 30 m^3/s discharge is flowing through this cross section, water level should be equal to 2.5 m. Model results provide levels comprised between 1.5 and 2.3 m. Anyway, it was verified that if the roughness is slightly increased, e.g. from n = 0.03 to 0.045, water levels according to the rating curve could be obtained.
All the experiments performed confirm the reasons for which channel roughness is usually considered the main calibration parameter in river flow modelling.

6.5 Flood modelling on the entire study area

Basing on the results of experimental modelling, SOBEK 1D2D was chosen as the most suitable software package and modelling approach to perform a flood analysis on the whole study area. This choice will be discussed in the following chapter, but the main reasons relate to the possibility of including different resolution grids and the easier and clearer modelling approach compared for example to FLO-2D.

As **input conditions**, the Adda and Poschiavino hydrographs described in par. 6.3.3 were used, resulting in three main scenarios for the 20, 100 and 200 years return time. No downstream boundary condition was applied, since the idea is to simply let the water flow out of the domain. The alternative for a river reach, since subcritical flow cannot be a-priori excluded, is to introduce a rating curve at the S. Giacomo station.

Anyway, this produces even more uncertainties (Hunter et al., 2007) and a longer simulation time. It was proved (Alemseged and Rientjes, 2007), moreover, that downstream condition has an influence on hydraulic quantities only in its surroundings (few hundred meters), even if it could depend on the type of condition defined. Since the distance among S. Giacomo, which is one of the most critical and thus interesting areas for the analysis, and the end of the study area is 2 km, it seems that defining a simple “letting flow out” downstream condition is reasonable and will not introduce errors in modelling results.

For the **1D module**, cross sections at a distance of 200-300 m were introduced for both Adda and Poschiavino in the area of Tirano and Villa di Tirano, since river morphology does not vary significantly along the reach here, while from Stazzona to the end of the study area Adda cross sections are defined at 100 m distance, with the exception of very complex compound channels, where the distance is higher. In these type of reaches, in fact, the true morphology of cross sections is uncertain and it was therefore decided to let SOBEK interpolate intermediate cross sections in the most computationally efficient way. Manning n values were attributed to the various reaches ranging from 0.025 to 0.05, according to previous studies.

For the **2D module**, as anticipated before, several grids with different resolutions were introduced instead of a single one. The reason for this choice is that SOBEK requires for 1D2D modelling that 2D cell size is not smaller than riverbed width. It means that the definition of 2D resolution cannot be a-priori defined, but it depends on 1D dimension. Since Adda channel is highly variable considering its width (from 25 to 200 m) and Poschiavino has a channel around 25 m wide, the use of a single resolution grid would require it to be around 100-200 m. This choice would cause a great loss of detail in floodplain representation, that it would be better to avoid. In order not to renounce the highest possible resolution for the 2D module, i.e. the maximum possible detail, eight grids of resolution ranging from 25 to 100 m (reaches 200 m wide are few, and this resolution would be really inadequate to describe the study area) were introduced, according to medium width of river reaches (Figure 6.17). The accordance could not be perfect due to the high heterogeneity of river width within the modelled channel, but it is reasonable (Figure 6.18). 2D Manning coefficients were set according to Table 6.1. The disadvantage of this approach is that floodplain detail is lost when the grid resolution is increased and so the representation of inundation process is quite coarse, but an important advantage is that channel volumes are modelled correctly in the 1D system.

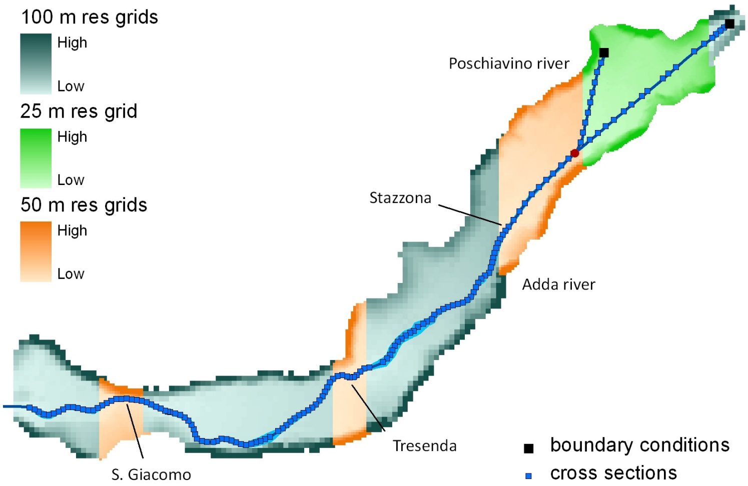


Figure 6.17 – Location of different resolution grids for SOBEK 1D2D simulations.

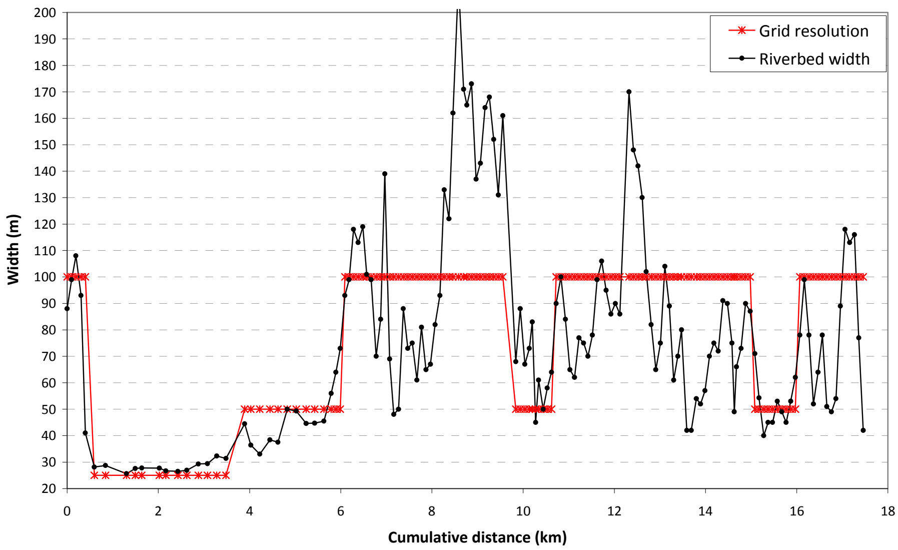


Figure 6.18 – Relation among grid resolution and riverbed width.

Excluding topography and simulation routine and options, there are two main sources of uncertainty within the model. The possibility of concomitant floods of the Adda and Poschiavino, i.e. peak discharges are reached at the same time (quite improbable but not to be a-priori excluded due to the degree of artificiality of the two river basins), and roughness setting.

In order to consider these uncertainties and to include them in the final results, four different **sub-scenarios** were simulated for each return time:

a1: n_{max} = 0.03 for Adda and 0.04 for Poschiavino – non concomitant floods
a2: n_{max} = 0.03 for Adda and 0.04 for Poschiavino – concomitant floods
b1: n_{max} = 0.04 for Adda and 0.05 for Poschiavino – non concomitant floods
b2: n_{max} = 0.04 for Adda and 0.05 for Poschiavino – concomitant floods.

If "severity" is defined as an expression of conditions which increasingly favour overflows and consequent inundations, it increases from the first (a1) to the fourth (b2) sub-scenario. In this way, not a single event is simulated for each return time, but four, and this results in a more in-depth and realistic description of hazard, since modelling uncertainties cannot be avoided and should be described in some way (see par. 8.1.2). Taking as an example the 200 years return time, results expressed in terms of expected flood extent and water depths for the four sub-scenarios are represented in Figure 6.19 and Figure 6.20. As it can be seen, increasing severity yields always wider flood extent, and this occurs for every return time.

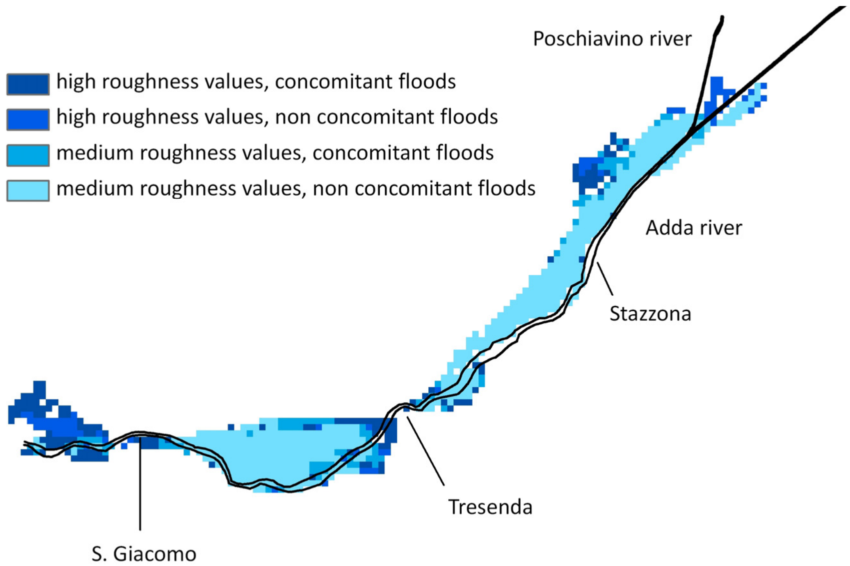


Figure 6.19 – Flood extent for the four sub-scenarios of the 200 years RT.

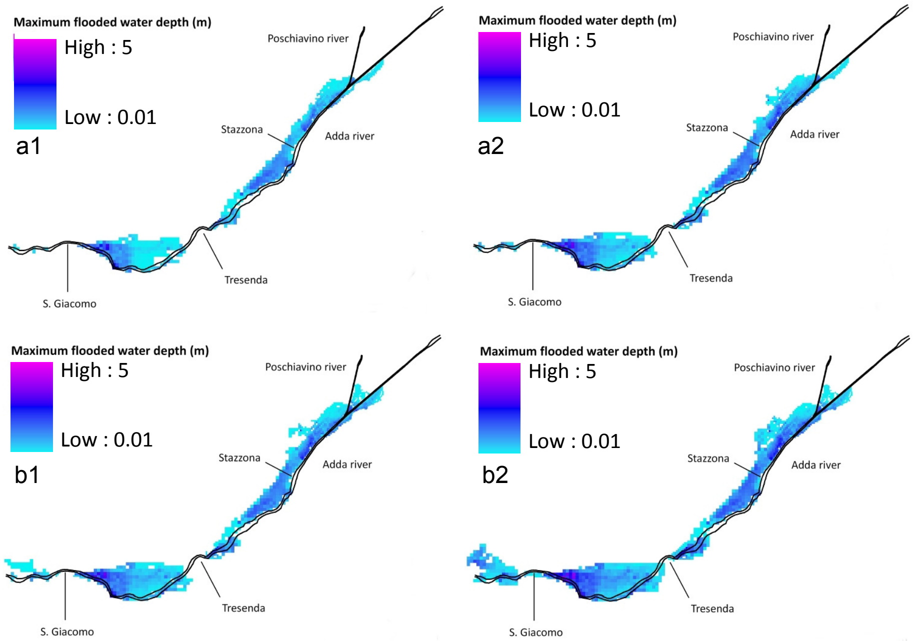


Figure 6.20 – Water depths for the four sub-scenarios of the 200 years RT.

CHAPTER 7
Discussion on modelling results

During experimental modelling, many options and approaches were tested, both to try to understand their functioning and to compare them in order to identify the most suitable approach for the research purpose and for the study area. Several difficulties were encountered, mainly due to long calculation times, unexpected and unintelligible errors communicated by software packages, failed runs and lack of calibration data to verify model correctness. All these issues led to the choice of SOBEK 1D2D as the main model for flood modelling in the study area, and are described and discussed hereafter.

7.1 General comparison among software packages and approaches

While comparing software packages and available approaches, the following issues were discovered.

Calculation times are extremely variable. While SOBEK 1D2D and FLO-2D took nearly 5 minutes for the floodplain flood experiment, SOBEK 2D hardly started and FloodArea almost stopped quite at intermediate time (that is why their results are not reported). Reasons could be that the number of active 2D cells, i.e. cells with an elevation attribute, is too high (70865), or the raising limb of the fictitious hydrograph is too steep in terms of calculation. In the first case, the bound is related to study area extension and resolution adopted, and they cannot be modified. In the second case, both a high constant discharge and the extension of the hydrograph worsen the problem even more. So these difficulties could not be solved. Maybe some other problems occurred which were not understandable.
A test on a smaller area would not have been meaningful since the space for flood propagation would have been too limited. This issue originates from the problem that for 2D modelling it was not possible to apply a simple "let flow out" boundary condition: FloodArea does not allow it, and SOBEK in theory should allow, but trials resulted in unintelligible errors.

When the modelling area is too limited and there is not a downstream boundary condition, water accumulates and so flood predictions are unreliable.

FloodArea seemed promising at the beginning due to its lower computational time compared to SOBEK 2D. It would have been very useful if simple equations could provide the same results of complete ones, but this did not happen for the study case. Travelling times were very different from the other models and a good agreement was not possible with a simple tuning of friction coefficients. Perhaps a more detailed approach is needed, but this falls within the field of engineering research and outside the scope of the present research. Problems with simple raster-based methods were also highlighted by Bates and de Roo (2000).

FLO-2D includes very useful features, such as the ability to include area and width reduction factors, i.e. coefficients which allows to reduce the flood volume storage on grid elements due to buildings or topography, and to partially or completely obstruct flow paths in the cell direction simulating floodwalls, respectively; they increase the detail of floodplain representation, when the resolution is coarse. Another advantage of FLO-2D is that the 1D system is adapted to the 2D representation (for SOBEK it is the opposite), so theoretically the user can choose the resolution that he considers the most appropriate according to the output level of detail he wishes for.
Beside these good features, some drawbacks are present:

- the software package requires much user's control on model settings and interpretation of results; the licensed version, moreover, contained many bugs which could not be completely solved, so several errors were repeatedly signalled and full potentialities of the package could not be exploited;
- the choice of 2D resolution is not so free, as it is subject to a constraint: the ratio peak discharge / grid cell size should be in the range 0.03-0.3 m^3/s / m^3; if this is not, calculation times could be very long, and this was tested to be true. This constraint is justified by the fact that for high discharge events modelling results are not so dependent on floodplain resolution (FLO-2D Reference Manual, 2009), but this seems not to be valid under any circumstance;
- the results of the test simulation at 50 m resolution were not completely intelligible and are characterised by a flood extents much wider than the one at 100 m resolution, so it seems that the representation at 50 m resolution is not suited to the study area, or alternatively unintelligible errors have been committed. Simulation at

100 m step length, anyway, has the disadvantage of a very coarse representation of 1D channel when its width is low, i.e. at Tirano. This problem is expressed by Figure 7.1, which compares SOBEK 1D2D and FLO-2D (100 m) results for the same input (200 years return time hydrographs) and roughness condition on the whole study area; as it can be seen, FLO-2D overflow at Tirano is more extended that the SOBEK one, and it is quite improbable;

- the coarse representation of the riverbed induces some difficulties in assessing hydraulic quantities at the boundary between channel and floodplain, as it is not very clear how that portion is represented and where it is located.

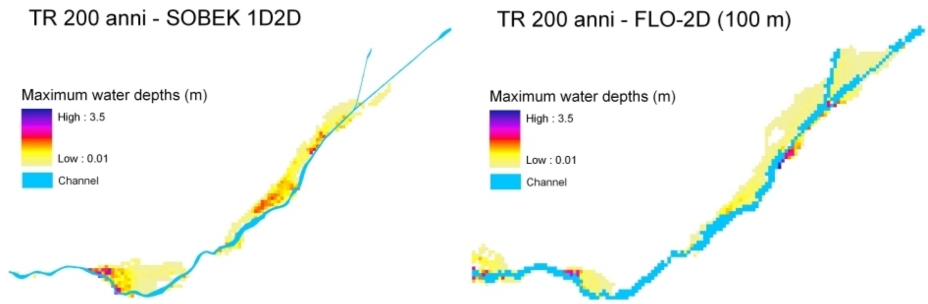


Figure 7.1 – Comparison between SOBEK 1D2D and FLO-2D (100 m) results on the entire study area, for the 200 years RT.

2D modelling was definitely not applicable for the whole study area. Some trials were performed in the area of Tirano and completely erroneous predictions resulted, e.g. overflow even at the lower return times. Difficulties encountered even in the test area led to the exclusion of this approach. As stated by Werner (2004), in some cases 2D cannot be proficiently applied, because of morphological conditions or resolution constraints.

SOBEK 1D2D was quick in simulation time and provided quite coherent and reliable results. A comparison is shown in Figure 7.2 among modelling results for the 200 years return time (comprising the four sub-scenarios) and available studies which provide a flood extent delimitation for the same return time (PAI, 2001 and Merizzi and Baldini, 2007).
In the area of Tirano and Villa di Tirano, there are strong differences when comparing to PAI *Fascia* B, but this is due to the fact that *Fascia* B is “*di progetto*” here, and thus it supposes the presence of protection measures which are still not implemented, so current conditions remain hazardous.

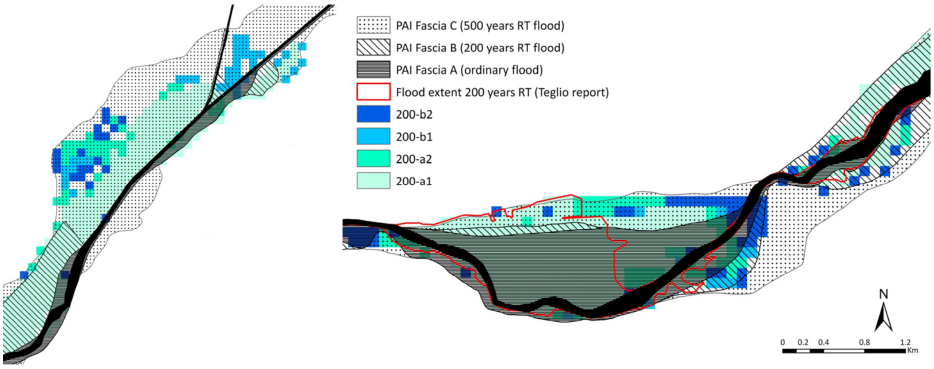


Figure 7.2 – Comparison between SOBEK 1D2D results for the 200 years RT and flood extent delimitation from available previous studies.

In the upper area of Teglio, differences should relate mainly to the fact that the areal extent is different, and in SOBEK simulations water is flowing into the floodplain also, from upstream.
In the area comprising Tresenda and S. Giacomo, SOBEK flood extent is higher probably because a hydrograph was applied instead of a constant discharge (as in the study of Merizzi and Baldini), and this again produced a flow in the floodplain.
Anyway, results in general are not so different, and this proved the reliability of SOBEK 1D2D modelling package.

7.2 Sensitivity analysis on SOBEK 1D2D model

Two kind of sensitivity analysis were conducted referring to SOBEK 1D2D modelling: the first is about technical settings; the second is related to the definition of roughness coefficients.

7.2.1 Technical setting

In this phase, effects of choice of grid resolution and bank level options were investigated. SOBEK, in facts, requires the user to make a choice regarding the way in which to connect the 1D system to the 2D one.

- Assume no embankments (NE). In this case the elevation of the grid cell is not modified. The 1D cross-sectional profile above the elevation of the underlying 2D grid cell is omitted (hence excluded from computation).
  * In case the water level is below 2D grid elevation, the flow is fully 1D only.
  * In case water level is above the 2D grid cell elevation; there is a 1D flow as well as 2D flow. In the 1D cross-sectional profile below the

2D grid cell elevation, 1D flow is computed in accordance with the underlying 1D profile and 1D hydraulic roughness. Above the 2D grid cell elevation, there is only 2D flow and this is over the full 2D grid cell size, both in x- and y- direction.

- Assume highest level of embankments (HE). In this case SOBEK raises the elevation of the underlying 2D grid cell to the highest embankment level. No part of the 1D cross-sectional profile is omitted. Thereafter the computation is identical to the option “assume no embankments”, with the notation that not the original 2D grid cell elevation is used but the raised grid cell elevation.
- Assume lowest level of embankment (LE). This is the same as the option above, but with notation that the part of the 1D cross-sectional profile above the lowest embankment level (i.e. minimum of left and right 1D embankments) is omitted and that the elevation of the underlying 2D grid cell is raised to the elevation of the lowest embankment level.

This choice is necessary since SOBEK (as FLO-2D, anyway) is not able to distinguish among left and right banks to define an overflow condition, which requires a single elevation value instead.
Hence, nine simulation were run according to Table 7.1, applying the same boundary conditions. Results are reported in Figure 7.3 as visualised by SOBEK Netter interface: in orange colour, the delimitation of the study area is represented, while in blue colour the water flood extent and depths are represented.

Table 7.1 – Simulation runs for sensitivity analysis on SOBEK technical settings.

	Grid resolution	Banks option
SIM1	25	NE
SIM2	25	LE
SIM3	25	HE
SIM4	50	NE
SIM5	50	LE
SIM6	50	HE
SIM7	100	NE
SIM8	100	LE
SIM9	100	HE

The general effect of changing from NE to LE to HE is to increase the flood extent, as a higher level of embankments is always imposed. This is true at least for the case under study, as artificial banks are often higher than the adjacent land outside the channel. In this case, it was assumed that LE is the best applicable approach, with the additional checking that elevations in the direction of highest bank is coherent with that elevation, at least for the immediate adjacent grid cell, in order to prevent unrealistic overflows. This is the approach applied in all the final SOBEK 1D2D simulations.

Moving from 25 to 50 m resolution DEM, the results are not particularly different, but the contrast increases when turning to 100 m, especially in the areas where the riverbed is more narrow. This is probably due to the fact that when grid cell is significantly larger than the channel, the definition of banks is really coarsely represented and generally smoothed out, and so overflow can occur more easily; consequently, flood extent is larger (this is observed also by Werner, 2004 and Haile and Rientjes, 2005).

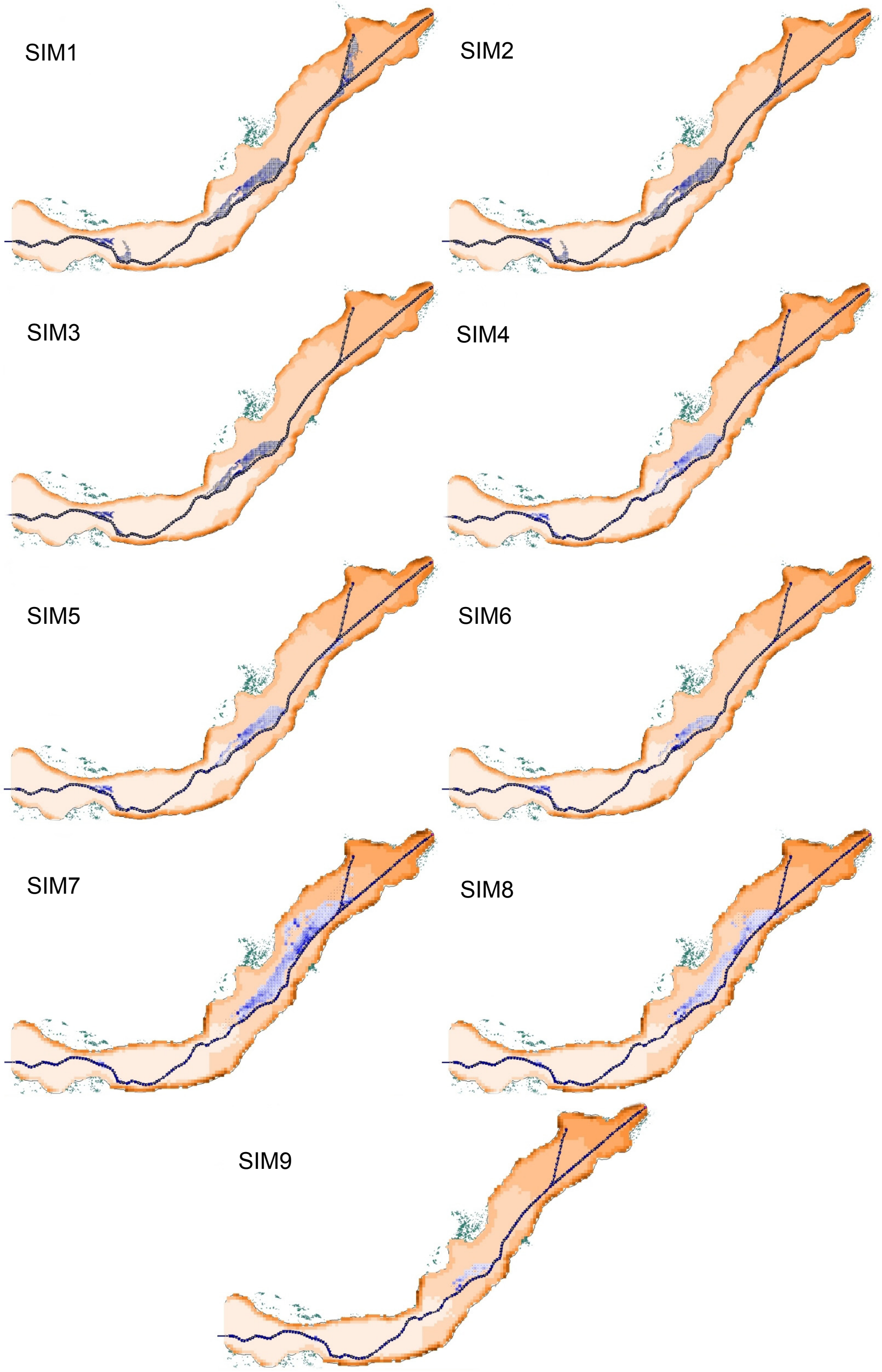


Figure 7.3 – Sensitivity analysis for SOBEK 1D2D technical settings.

Figure 7.4 shows the situation from a transversal to the channel point of view. It represents how overflow can occur for a simple cross section which is 50 m wide, when the different banks options are applied on different resolution grids. This could be the case of Adda cross sections before Stazzona.

For the <u>LE option</u>, it is evident that when resolution decreases a greater portion of land is flooded which should be out of the channel in the direction of higher bank. The propagation of water in that direction is dependent on the elevation of the neighbouring cell, which will be likely very different from the bank one (both for interpolation reasons and the user's choice – if the land in that direction is lower than bank elevation, it is unrealistic to define a higher elevation for a wide cell, since floodplain capacity will be severely reduced). The propagation of flow in the lowest bank elevation direction could be also wider (see Figure 7.3 - SIM 8) since obstructing features could be smoothed out in the low resolution DEM. For this option, it seems also convenient that cross section width is a little bit smaller than grid cell, if banks elevations are correctly represented by neighbouring 2D cells. Anyway, this could be true only for simple shape cross sections, since as the cross section shape becomes more complex, e.g. in compound channels, other problems in banks representation can take place.

For the <u>HE option</u>, as it can bee seen from Figure 7.3 (SIM 9), the effect of low resolution grid is a decrease in flood extent. This could be due mainly to the fact that 1D capacity on the 1D2D coupling cell is higher, and so overflow occurs when a higher volume of water is flowing in the channel, compared to the other resolutions.

The increasing in flood extent for the <u>NE option</u> can be explained by the fact that a minor volume is available for 1D flow, while a higher is treated as 2D flow (associated to the cell 2D roughness coefficient) which, as experimental modelling proved, tends to be slower than 1D one.

These observations, however, are dependent on cross sections width and shape. Probably, a banks option variable along the reach would be the optimum setting, but unfortunately this is not available within SOBEK at the moment.

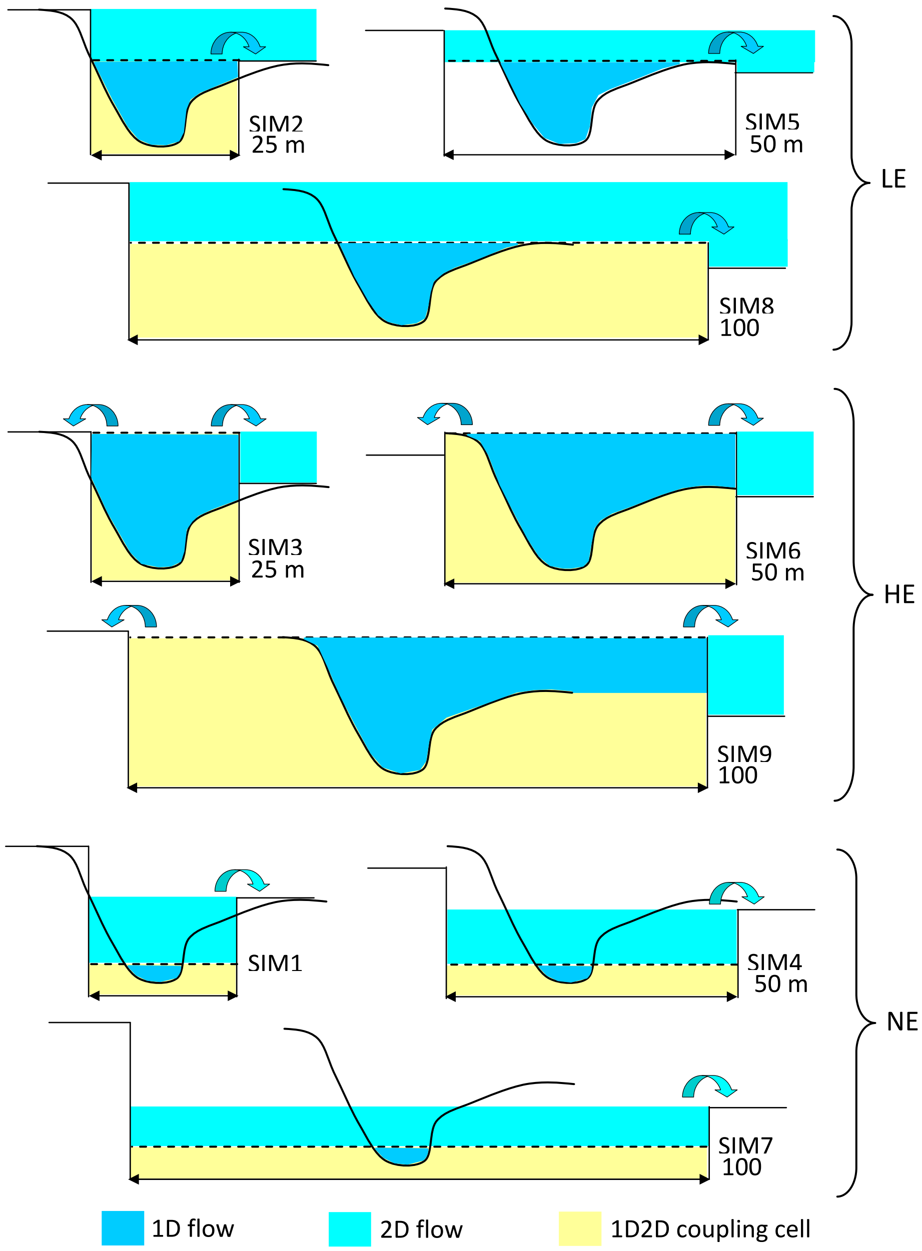


Figure 7.4 – Visual comparison of banks level option for different grid resolutions.

7.2.2 Roughness setting

Three different coefficients were tested both for the 1D and 2D module. In particular, for the river n = 0.015, 0.03, 0.07 and for the floodplain n = 0.02, 0.04 and 0.1 were applied. Differences in the outputs expressed as water depths are shown in Figure 7.5. The model revealed to be much more sensitive to changes in 1D roughness values, and this is also confirmed by literature (Apel et al., 2009; Alemseged and Rientjes, 2007; Werner et al., 2005; Horritt and Bates, 2001). It is thus more important to define accurately 1D coefficients than 2D ones, and this is an helpful indication since 1D roughness has a sound background of knowledge, while 2D one is still debated in literature.

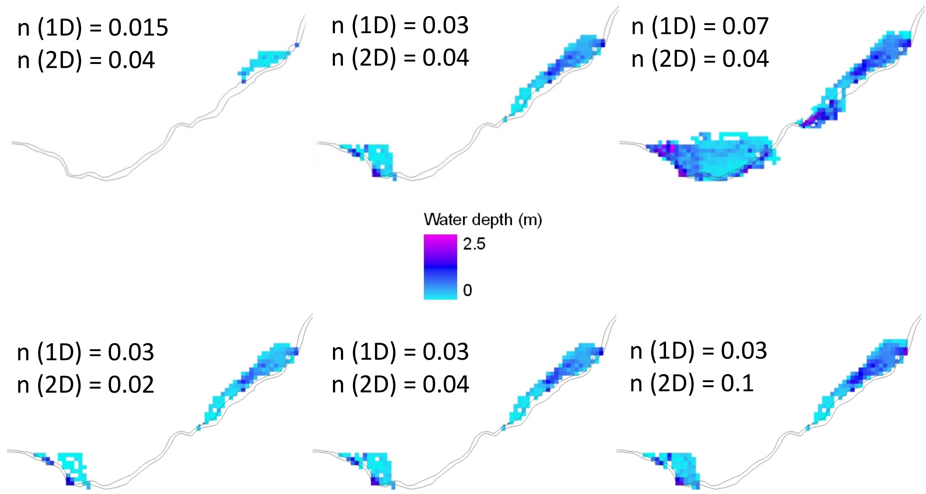


Figure 7.5 – Sensitivity analysis on SOBEK 1D2D friction settings in the channel (upper part) and in the floodplain (lower part).

7.3 Conclusions

This Chapter discussed all the difficulties encountered in the modelling phase. The comparison between models was complicated by the errors often encountered in the running phase, which did not allow to explore the complete capabilities of the software packages. Anyway, a model is chosen both for its suitability for the purpose of the study and data availability, and for its easy use and understanding; the lacking of one of these factors could make the user prefer another one. SOBEK 1D2D was selected as the more convenient modelling software and approach. Its sensitivity to both main technical and roughness setting proved that it was applied to the study case in a proper way, taking into consideration its unavoidable limits.

CHAPTER 8
Flood hazard mapping

8.1 Introduction

8.1.1 General contents and purposes

The analysis and management of flood risk requires the development of flood hazard maps. Basing on the hazard concept shared by scientific community, these maps should include the probability that a certain area could be affected by a flood of a certain intensity, within a specific period of time (Frank et al., 2001; Apel, 2009; Büchele et al., 2006; Directive 2007/60/EC). Intensity generally relates to water depths, especially in flat areas, but it could also include flow velocities and the quantification of energy or impact pressure, while the temporal component usually refers to the statistical probability of occurrence, i.e. the return time.

The production of hazard maps is not a trivial task. Information provided and the detail of representation should be appropriate to the scale of analysis, which could be national (entire river basins) or local (river reaches of few kilometres comprising municipal territories), and to specific needs of the end-users. Information should at minimum include flood extend and expected water depths, and this could be sufficient for urban planning to estimate possible damage; further information, i.e. flow velocities and time to arrive/ retire of water, is useful for disaster and emergency planning. Even if purposes are different, and final mapping could thus be slightly different, hazard assessment should be based on the same approach.

In some contexts, maps are distinguished among “floodplain maps”, i.e. geographical areas which could be flooded according to temporal probabilities, and “flood hazard maps”, i.e. detailed floodplain maps describing the type of flood, flood extent, water depths and flow velocity or the relevant water flow direction (Prinos et al, 2008). A similar approach has been adopted in the analysis here presented, since it seems that representing all the required information within a single map is quite impossible, and moreover not useful. Information overload, in fact, can undermine the practicability of the map (LAWA, 2006).

8.1.2 Consideration of uncertainty

Uncertainty, in contrast to error, assumes that no prior knowledge of the accuracy of the data exist (Zerger, 2002). Goodchild et al. (1992) note that there are three options to deal with the existence of uncertainty in models and spatial data: (1) omit all reference to it; (2) attach some form of description to the output; (3) show samples from the range of possible maps or outputs. The first option is unacceptable for risk management and evacuation planning, and the second may not adequately communicate this complex concept to end-users. The third option is preferable because it appears to have the greatest potential benefit in both communicating uncertainty and educating the user community to the significance of the issue (Hunter et al., 1994). It could be not the optimal solution for event management, since immediate decisions should be taken and ambiguities are not desired, but it is for sure a valuable approach for flood risk planning, preparedness and the assessment of mitigation options. A main advantage of cartographic representation is that it can present uncertainty without explicitly identifying it as such (Zerger, 2002).

Uncertainties arising from the creation of flood hazard maps can be divided into three types related to data, model and parameters (Prinos et al., 2008). Referring to predicted flood extent and water depths, uncertainties are mainly related to: model selection (1D or 2D approach), equations applied (complete or approximated), channel roughness and geometry, and the consideration of all the relevant acting processes (sediment transport and/or banks breaches). These uncertainties should be included in the map representation in order to show end-users how they affect the delimitation of hazard, as suggested by the option (3) above. This approach is also supported by Jones et al. (1998): making use of GIS tools, uncertainty areas, i.e. areas where we have less confidence that flooding will occur, should be indicated. An even more complex approach could apply the GLUE methodology to predict uncertainty flood extent bounds making use of quantiles (Pappenberger et al., 2005).

8.2 Hazard maps for urban planning

The main purpose of urban planning is to direct land use and urban development ensuring a compatibility with flood hazard conditions. It was then decided that relevant information should include the distribution of

flooded areas and expected water depths for different scenarios based on temporal and spatial probability.
As explained in the previous Chapter, SOBEK 1D2D was used as a tool to simulate floods for the entire study area, supposing twelve scenarios, i.e. four sub-scenarios (a1, a2, b1, b2) for three return times (20, 100 and 200 years). Results were used to produce two complementary hazard maps, as suggested by Büchele et al. (2006).

8.2.1 Hazard Map 1

The first map is a polygon map, and it was obtained through the following steps.

- Areas affected by inundation have been delimitated for the four sub-scenarios of each return time, producing a series of polygons.
- A probability index (sevPI) of 1, 0.75, 0.5 and 0.25 was attributed to these polygons according to the sub-scenarios severity, i.e. 1 is assigned to sub-scenario a1, 0.75 to sub-scenario a2, 0.5 to sub-scenario b1 and 0.25 to sub-scenario b2. These values were chosen since it was observed that from a1 to b2 the flood extent was constantly growing: so, the flooded area for a2 is the flooded area for a1 plus an extra area, and the same for the sequence b1 and b2. This means that, among the four sub-scenarios, area flooded for a1 is flooded also for the other 3 sub-scenarios (→ 1=always flooded), area flooded for a2 is flooded for other 2 sub-scenarios (→ 0.75=flooded 3/4), area flooded for b1 is flooded for another sub-scenario (→ 0.5=flooded 2/4), and area flooded for b2 is not shared by other sub-scenarios (→ 0.25= flooded 1/4). This allows to take into account the spatial probability.
- A probability index (PI) is then assigned to each polygon as:
  $PI_{RT} = sevPI \cdot RT^{-1}$
  in order to take into account also the temporal probability.
- The maximum value of PI among the three return times was assumed as the final probability index (FPI_1):
  $FPI_1 = \max(PI_{20}, PI_{100}, PI_{200})$.
- These values are then attributed to the areas resulting from the intersection of polygons for the twelve scenarios.

The resulting map is reported in Figure 8.1. FPI_1 values can be used to define hazard levels (e.g. low, medium, high): this classification step should be performed together with end-users. The map aims to show where water is expected to arrive in case of floods characterised by different return times, considering modelling uncertainties expressed by the four sub-scenarios.

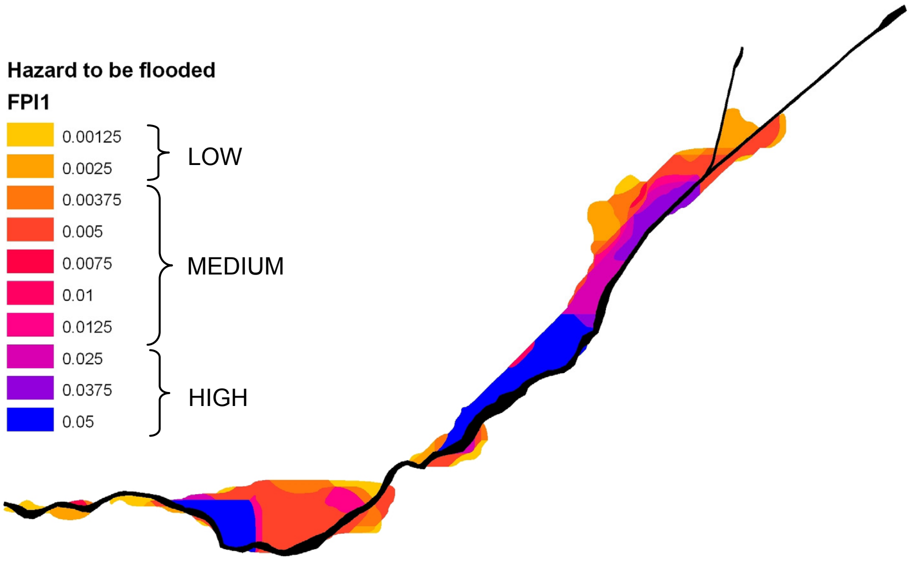


Figure 8.1 – Hazard map 1, with a possible classification.

8.2.2 Hazard Map 2

The second map has a raster basis. It makes use of both the presence of inundation and the expected water depth in each cell of the grids derived by SOBEK simulations, converted into a common 100 m resolution.

- For each cell, a value has been assigned which represents a water depth class:
  CLASS(1): water depths less than 1 m;
  CLASS(2): water depths ranging from 1 and 2 m;
  CLASS(3): water depths ranging from 2 and 3 m;
  CLASS(4): water depths more than 3 m.

- A probability index (PI) has been assigned to each class in each sub-scenario, as:

  $$PI_{CLASS(n),sub-scenario} = 0.25 \cdot RT^{-1}.$$

  In this case the multiplicative coefficient is constant since the information used refers to a certain class of water level present in one of four different situations (the four sub-scenarios).

- PI values are summed for each return time, and finally summed up for each class. The final hazard probability index (FPI_2) for each class is thus obtained by:

$$FPI_{2,CLASS(n)} = \sum_{RT=20}^{RT=200} \sum_{sub-scenario(a1)}^{sub-scenario(b2)} PI_{CLASS(n),sub-scenario}$$

In this case the sum was performed instead of considering the maximum value since it allowed to better distinguish the various levels of intensity.

Classes defined above are used also for the final hazard map, since the aim is to provide an index of probability that a certain class of intensity (expressed as water depth) is expected within a single cell, considering the temporal probability (return time) and the spatial probability (presence of water in the four sub-scenarios). The map is reported in Figure 8.2.

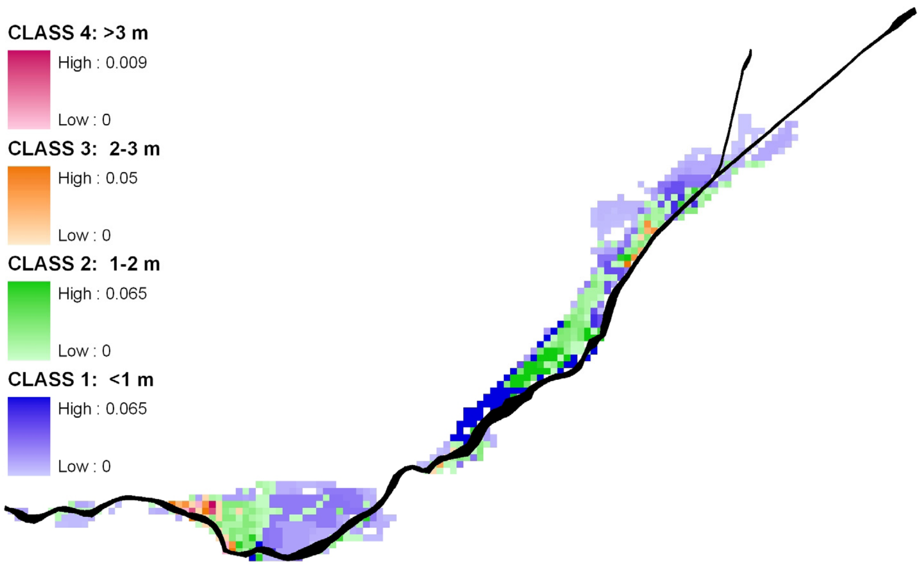


Figure 8.2 – Hazard map 2, representing four water depths classes.

8.2.3 Comments on the maps

The approach presented is not the only one that could be potentially applied for the purpose, e.g. expressions to calculate probability indexes could be slightly different, but the final ranking of hazard should be similar.

The first map is useful to visually compare different areas according to their degree of probability to be affected by flooding, whose intensity is not explicitly taken into account, but which can be related to some extent to the return time. The second map represents an enrichment since it includes also the intensity parameter related to water depth, but in fact the two maps are complementary.

It can be seen from details in Figure 8.3 that a high spatial probability for an area in Map 1 does not mean that the expected intensity would be higher than in other areas, since floodplain water depths, as an indicator of intensity, are determined both by discharges within the river system and by topographical local characteristics. Similarly, a low degree of intensity expected from Map 2 could be associated to a high spatial probability index in Map 1. The uncertainty is represented by means of the four sub-scenarios, and it is included both in the definition of probability indexes and in the visual delimitation of flooded areas.

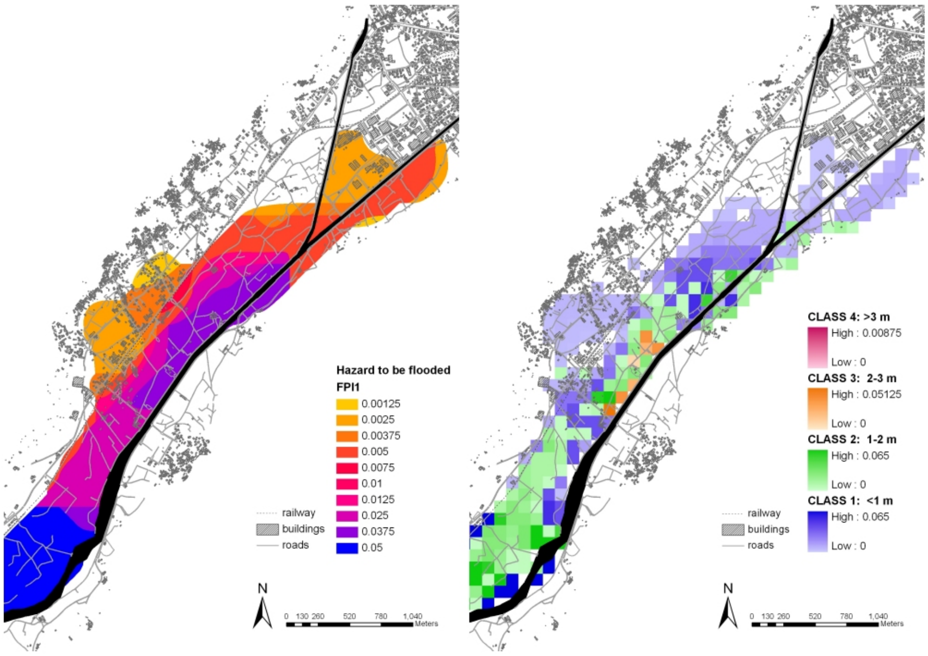
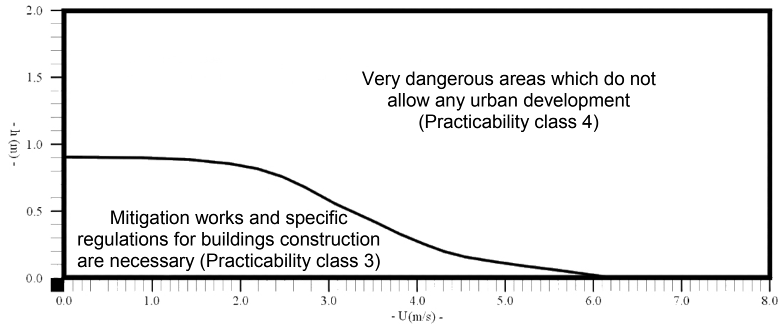


Figure 8.3 – Detail of hazard map 1 (on the left) and 2 (on the right), where also road tracks and buildings are represented.

8.2.4 Practicability map (D.G.R. 8/1566)

As an additional example, modelling results have been used to produce a map related to flood risk (called "practicability" – *fattibilità*, in Italian – map), as requested by the regional D.G.R. 8/1566. This map is based on expected water levels and velocities for the 200 years return time flood, and areas with a different level of suitability to urban development (i.e. practicability) can be distinguished basing on the graph shown below (D.G.R. Enclosure 4).



First, practicability maps have been obtained for each sub-scenario (Figure 8.4) on a raster basis; then, they had to be combined in a single map for the 200 years return time.

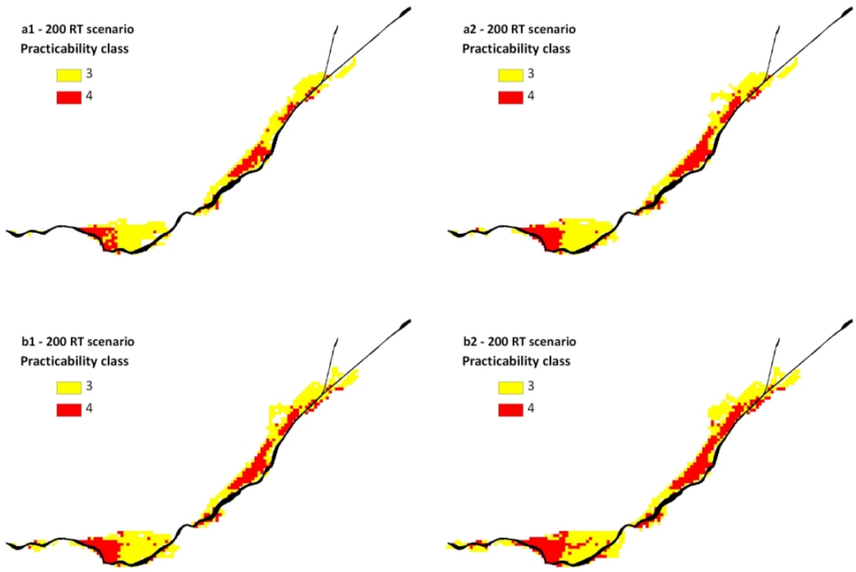


Figure 8.4 – Practicability raster maps for the four sub-scenarios related to the 200 years RT.

In order to do that, a value (x) equal to 1 was assigned to each cell in case of practicability defined as 3 or 4 and equal to zero where it was not, for each sub-scenario. Then, a probability (P) value both for practicability equal to 3 and to 4 was calculated by means of the following formula:

$$P(3,4) = \frac{x_{a1} * 1 + x_{a2} * 0.75 + x_{b1} * 0.5 + x_{b2} * 0.25}{\max(x)}$$

When calculating probability for practicability equal to 3, all x values refer to that value, while when it is equal to 4, they refer to 4. This probability values are used to represent the distribution of practicability classes within the study area (Figure 8.5). By means of this approach, uncertainty is included again in the representation. When converting the raster map into a vector one (Figure 8.6), some simplifications are necessary, but map interpretation becomes more immediate.

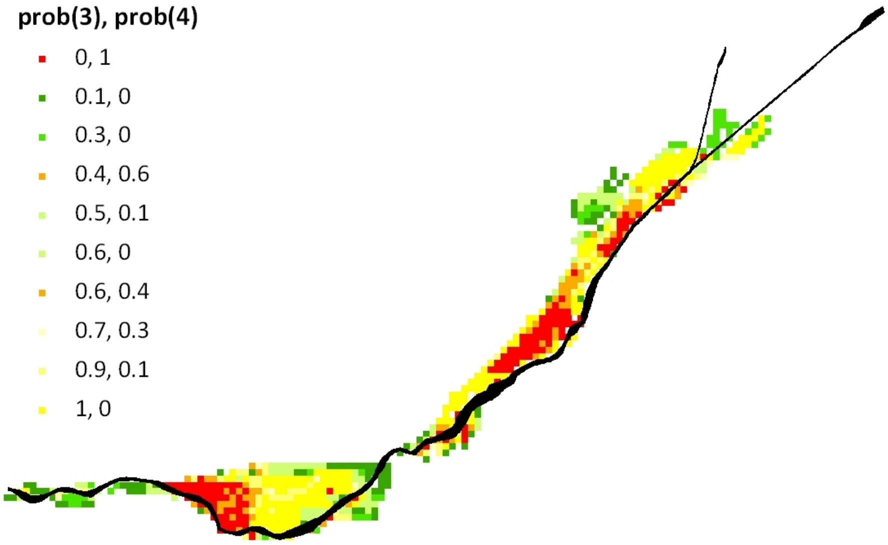


Figure 8.5 – Raster practicability map showing the different probabilities to have a class equal to 3 or to 4.

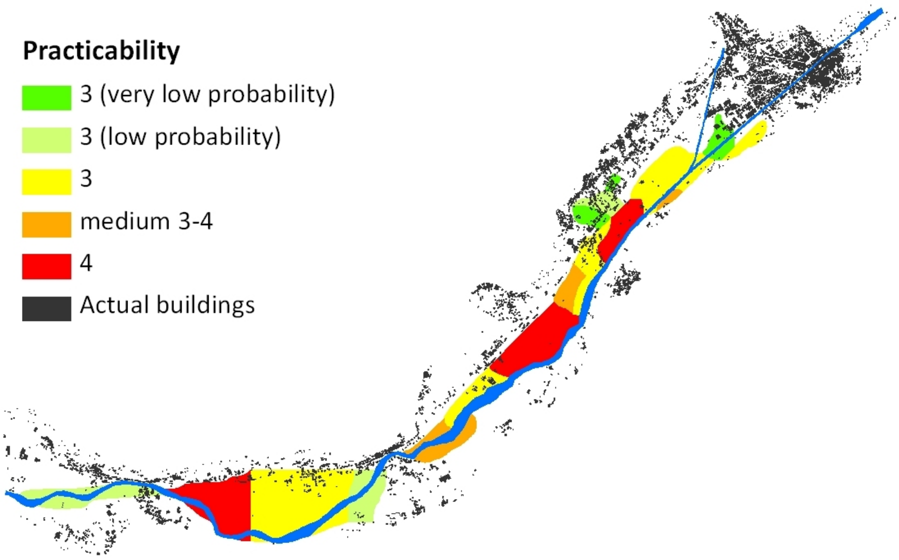


Figure 8.6 – Vector practicability map for the study area, expressing uncertainties (where there are) on the attribution of a certain class.

8.2.5 Additional remarks

Maps 1 and 2 represent a first level of scientific hazard mapping; if necessary, they could be further classified for specific needs. A single flood hazard map could not be sufficient to describe all the information that is necessary to communicate, and a double representation could improve the map understanding. This approach could be usefully exported to similar contexts and for similar purposes, extending to comprise more scenarios and including a more rigorous probability estimation process, i.e. Monte Carlo-based techniques, whenever possible.

Their advantages over PAI *Fasce Fluviali* are several: hydraulic modelling approach has been chosen among many as the most appropriate for the study area characteristics; bathymetric detail is higher, since 1D cross sections are more dense and morphologically correct (especially in the definition of river banks) than the ones used by PAI; a degree of uncertainty, which is unavoidable in flood modelling, is expressed and visually comprehensible; flood extent is the expression of an input hydrograph, which allows to represent the flood propagation in a more realistic way than in case of applying a constant discharge.
A connection with PAI contents is maintained, however, since input peak discharges are the same.

The analysis performed follows partially regional regulations for urban planning, e.g.: basic historical knowledge and consultation of available studies; a more detailed topographical representation than PAI one; the application of a 1D modelling approach for the river system), but introduces some new elements which are believed to improve the overall representation of hazard (2D approach for the floodplain, two maps - instead of one - with a complementary and useful meaning, and the inclusion of the expression of uncertainty). Even if this approach could not be directly included in urban planning, at the moment, since it needs adaptations and additional analyses (e.g.: a more in-depth consideration of hydraulic structures and possible banks breaches; the consideration of minor river network and solid transport contribution; a wider understanding of basin hydrology; the application made by a hydraulic engineer) it contains very useful suggestions for a more extended treatment of the flood hazard issue than the one usually applied at regional and local level. Its application to emergency planning and management (see next paragraph), instead, is very appropriate, since there are no regulations to perform hydraulic analyses in this field, so the proposed approach could be directly valid and, for the particular case of study, it can be adopted for the drafting of flood event scenarios for the Civil Protection Plan of the Mountain Consortium.

8.3 Hazard maps for civil protection purposes

Flood management comprises several issues: preparedness for floods; providing flood information; communicating the risk of flooding to raise public awareness; detecting and forecasting floods; communicating flood warning to the public and to professional partners; promoting effective responses, emergency exercises and planning; co-operation between emergency services; media management, and effective aftercare (McCarthy et al., 2007).

Emergency management deals with the actions which aim at reducing flooding impacts, at times when a warning is issued. Its short-term purposes are then quite different from the ones of flood risk urban management, where strategies are planned for long-term periods. It is particularly important, in this context, to understand the end-users needs and the data they require, which for the hazard component are mainly (David et al., 2009):

- frequency of flooding;
- flood extents, depths and velocities (useful to define possible damages);
- since often a preventive evacuation is preferred to an emergency evacuation, the time of the evacuation call is crucial (Mens et al., 2009), so the temporal evolution of the expected flood (time to flood and duration of submersion) should also be provided and described as accurately as possible.

When mapping hazard for emergency management, the representation should be accurate and specific to the intervention (Romang and Wilhelm, 2009); moreover, the content should be clear so that users can obtain all the relevant information easily, as operational decisions are taken in short time. The expression of uncertainty in this case is not a main issue and could be even misleading; it has to be considered also that risk managers commonly operate assuming a worst-case scenario (Zerger, 2002; Ferrier and Haque, 2003). Hazard maps to be used in emergency should have a dynamic more than a static use (Romang and Wilhelm, 2009). A good idea is to propose several scenarios representing different probabilities of occurrence, i.e. from a rather frequent to a rare and often more serious event, but with a single representation of hazard conditions for that temporal probability (the worst situation expected). It would be also useful to hypothesize not only “routine” events but also “unexpected” events such as breaches of flood defences or dam breaks (Lumbroso et al, 2009), but this depends on local characteristics of the river system. A possible temporal evolution of expected events should also be described (Romang and Wilhelm, 2009).

8.3.1 Event scenarios representation

In this case, a map cannot express uncertainty, and so a choice has to be made in cooperation with emergency managers on what a "scenario" should represent.
A small scale representation (i.e. the synoptic approach) is useful to define general hazard conditions, e.g. in order to define critical points and prearrange road blocks, while a large scale map (1:10,000 or higher) allows to identify possible affected elements.
In Figure 8.7 and Figure 8.8 both scale representations are provided for the study area: they express hazard conditions in term of maximum expected water depths, maximum flow velocities, and time to wet (time necessary for the various areas to be invaded by water) for the "worst case" scenario (case b2, 200 years return time).
A vector representation, for this purpose, is more appropriate than a raster one, since protection measures have to be taken for homogeneous areas and not for single cells, which for practical actions do not have a meaning.
It has to be remembered, nonetheless, that uncertainty does not appear in the maps but it is not absent. A flood event, moreover, will never be predictable in complete detail (Romang and Wilhelm, 2009). Uncertainty has to be communicated to emergency managers who will consider it in the definition of operational strategies.

8.4 Conclusions

Flood hazard mapping is a complex task which should be based on the most in-depth knowledge of the acting process, available data and consideration of end-users needs. Maps for different purposes, e.g. urban risk management or emergency planning should therefore be different, both on the contents and representation of uncertainty, but based on the same scientific approach for hazard assessment. These maps have been produced for the study area, which improve available local representations of flood hazard and include literature suggestions. Their partial respect of urban planning regulations does not allow a direct inclusion in this kind of plan, even if the adopted approach represents a possible improvement of usually applied tools, while their use is very appropriate for civil protection purposes, where there is not a methodological legislative framework for hazard assessment. In both cases, the maps are in compliance with the European Directive 2007/60 requirements, since they provide the required spatially distributed information on flood extent, flow depths and velocities for different return times.

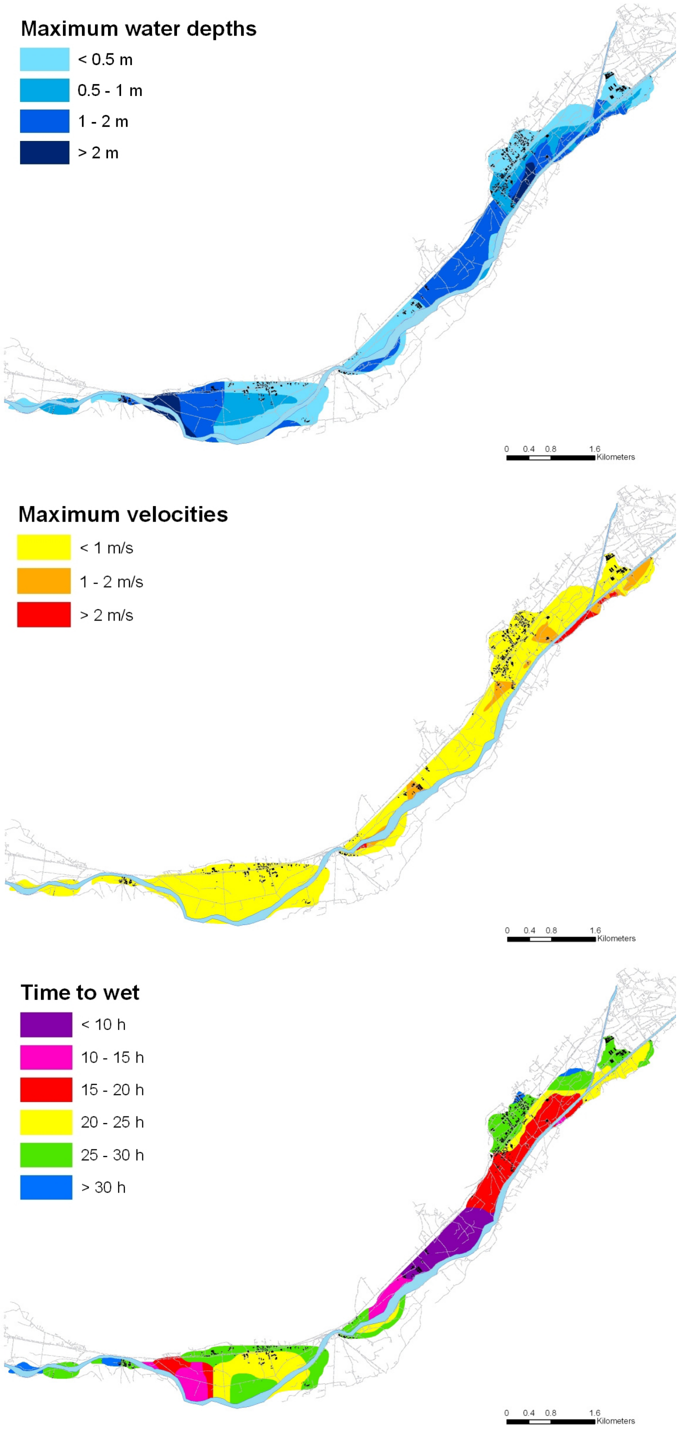


Figure 8.7 – Event scenarios maps for emergency planning

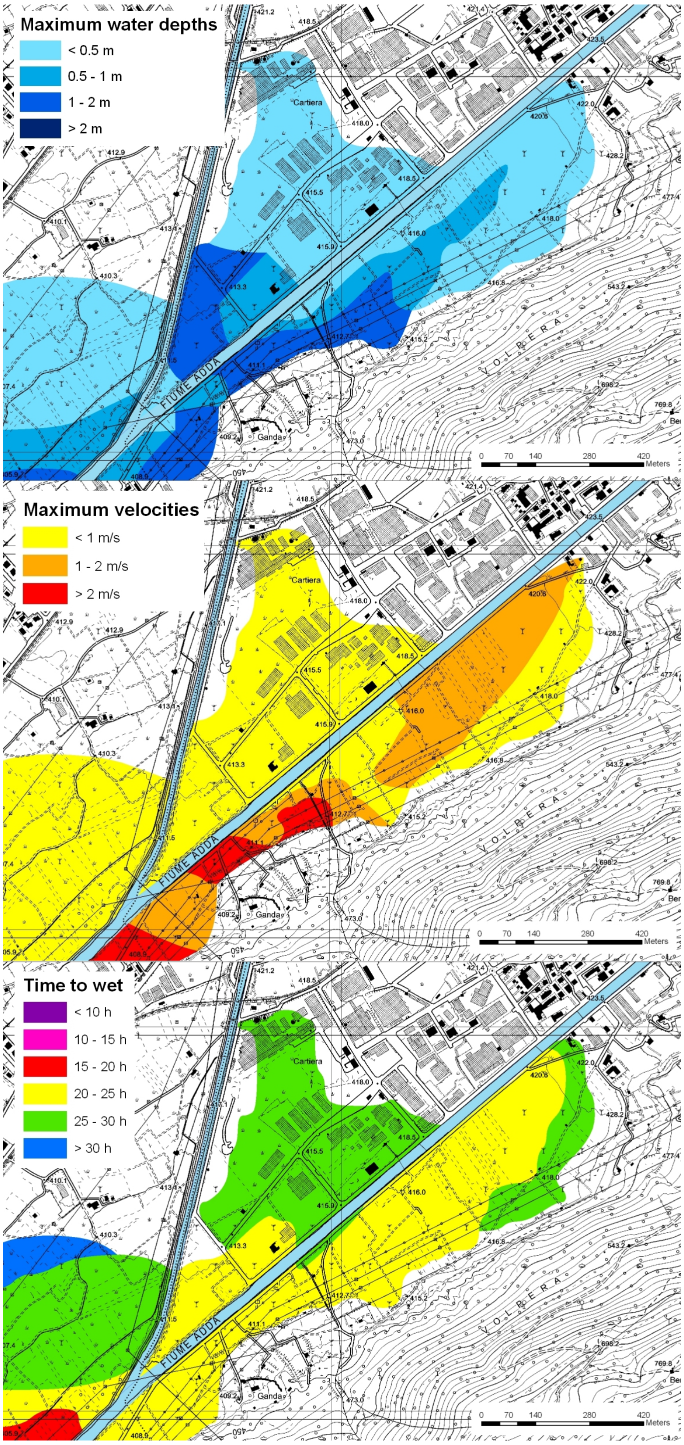


Figure 8.8 – Detailed event scenarios maps for emergency planning.

CHAPTER 9
Research conclusions

In Lombardy Region, as in many other contexts all over the world, hazard maps does not have a precise legislative confirmation. Despite this, they are necessary to support activities such as disaster and management planning and local urban planning.
Literature describes several possible approaches to be used for flood modelling in a river system, and provides a wide series of study cases. Anyway, these applications are strongly influenced by the specific research objectives, analysis scale, data availability, and accessible computational resources. The choice of a particular model and its correct application, therefore, are not trivial issues, and comparative tests are generally required. Moreover, the correctness of these models can often be only qualitatively evaluated, because sufficient data for calibration and validation are lacking.
Finally, in order to ensure the possibility to be included in local planning instruments, hazard analysis should be as in compliance as possible with the legislative framework in force.

The presented PhD research aimed to propose an approach for hazard analysis and mapping that fits the Lombardy Region legislation, but introduces a level of experimental modelling. It was applied to a study area located in Valtellina di Tirano (Alps, northern Italy) which has quite complex hydrological, hydraulic and topographic characteristics. Several approaches and software packages for flood modelling were tested, and modelling results were converted into hazard maps, making use of an innovative methodology.

9.1 Main scientific findings

Two-dimensional and combined one-two-dimensional flood models were constructed and applied to the study area, through SOBEK (1D, 2D or 1D2D), FLO-2D (1D2D or 2D) and FloodArea (2D) software packages. Main necessary data relate to topographical description, roughness setting and boundary conditions of models. The first two, especially, have a major influence on model behaviour. Comparison and analysis of results allowed to formulate the following conclusions.

- 2D modelling is not a suitable approach for the whole study area, since it requires a spatial resolution which is too high for the available data; moreover, extremely long calculation times makes the approach unfeasible for multiple runs, which are indispensable to test model capabilities.
- When performing experimental 2D simulations, channel roughness values had to be severely reduced compared to 1D values to get quite the same results in terms of water arrival time: for SOBEK 2D, n had to be set equal to 0.015 while the 1D value was equal to 0.03; for FloodArea k_{St} had to be reduced even more (reaching a value of 174). This means that the expression of roughness does not represent anymore only the physical characteristics of the river channel (which 1D coefficients are instead appropriate to), but it includes model uncertainties and, probably, implicit errors. This is in accordance with the experience of some authors (even if they did not provide a quantitative definition of the roughness reduction required by 2D models compared to 1D ones), and represents a major limit for 2D modelling, since the definition of appropriate coefficients still needs research.
- The calibration performed on roughness coefficients with the aim to try to make 2D channel hydraulic behaviour as close as possible to 1D one showed that coefficients are not conservative towards water depth and discharge; in fact, coefficients resulting from a calibration for low flows does not perform equally well for high flows. This is another complex issue regarding models roughness setting.
- The construction of an appropriate elevation model to describe the topographical characteristics of both the channel and the floodplain is a very complex issue, especially if only low density data (e.g. from aero photogrammetric surveys) are available. Main problems are the correct reproduction of linear features and the averaging implicit in coarse structured meshes. Anyway, it was proved that TIN provides a more useful and correct representation of a combined man-made and natural topography, compared to usual raster interpolation methods.
- It was proved that the 1D2D approach which treats in 1D the channel and in 2D the floodplain is the most appropriate to model a river system with complex morphology and artificial characteristics: roughness coefficients applied to the channel benefit from a well-established knowledge and experience and so uncertainties in their

definition are reduced; floodplain roughness proved to have a minor influence on the prediction of flood extent, so their uncertainties are far less important. 1D2D modelling, anyway, has some drawbacks, mainly related to the relationship among 1D and 2D resolutions, and the lost of detail in floodplain representation compared to fully 2D models.

- Software packages for flood modelling are as useful and proficient as they are understandable and practical, and if they are able both to capture and represent main flow processes basing on not optimal data and to communicate the causes of modelling problems to the user. Difficulties encountered in the application of FLO-2D and FloodArea led to prefer SOBEK.
- It is not an easy task to convert modelling results into hazard maps: the process should be scientifically based but, at the same time, clear and useful for the end-users' needs. A joint expression of uncertainty should be also provided. Two maps could be more appropriate than a single one, if the information content is expressed in a more immediate and understandable way. The proposed methodology accomplishes both the requests: considering indexes related to spatial probability of inundation (presence of water), temporal probability (return time), and intensity (expressed as water depths), two complementary flood hazard maps were created, which represent a useful combined tool to visually understand which areas could be mostly affected by floods and what would be the expected intensities. These maps are an improvement both of usual hazard maps which do not include any expression of modelling uncertainties, and of the current delimitation of flood prone areas in the study area.
- The approach presented is an effective method for hazard mapping, which could helpfully support urban planning, also in prospect of the national application of the Directive 2007/60/EC. It could thus be a reference method for similar contexts and objectives.

9.2 Research constraints

The heterogeneity of the study area, both for physical characteristics and data availability, allowed to test the approaches on a context far from ideal, and this had advantages since it allowed to explore different kinds of problems, but it also determined limitations. To list the main, these limitations are:

- the lack of historical data and hydraulic measures from operational gauging stations did not allow to perform a complete evaluation of the quality of modelling results, which have a degree of uncertainty that it was not possible to define quantitatively;
- the Adda basin complexity and the unavailability of a recent and trustful physically-based hydrological study did not allow to relate river discharges to expected rainfalls, and thus to link event scenarios to meteorological forecasts; institutional peak discharges for different return times were therefore applied to derive input hydrographs;
- problems often encountered while applying the software packages (e.g. intelligible errors or failed runs), did not allow to explore all the capabilities of the models;
- a more high-resolution elevation data set would have reduced the difficulties in reproducing an acceptable topographical description for the models, and the uncertainties in the final representation;
- specific objects that could have an influence on channel flow dynamics, e.g. bridges, were not modelled due to their intrinsic difficulties, which would have required an engineering support. Their effect on modelling results, therefore, was not tested;
- Adda river, similarly to other mountain rivers, when flowing downvalley, and especially in case of severe rainfall events, can convey considerable quantities of sediment and debris, which could increase hazard conditions; this effect, anyway, was not analysed.

9.3 General remarks and recommendations

No model is able to provide “perfect” results. It is important to be aware of this limitation and to consider cautiously hazard maps, being aware of models, in general, and the applied model, in particular, intrinsic limits. Problems include the degree of uncertainty that can be associated with model results owing to the choice of the model used, and the role of error in the input data and how it effects the outcomes. The success of a modelling initiative, anyway, should be assessed in the context of improved decision-making, which again highlights the importance of the cooperation among researchers and local planners or territorial managers.

Flood hazard modelling is usually performed by hydraulic engineers, even if it seems, from the experience gathered during this research, that the issue should be treated from a more interdisciplinary point of view. In fact, many other skills are required: hydrologists, topographical analysts, GIS experts, geomorphologists, social scientists, and last but not least, environmental scientists, which should have the capability to link all these competences in a proficient way.

To conclude the Chapter and the report, a sentence is quoted from Cunge (2003), which should not be forgotten whenever a model is applied to analyse a physical problem:

"*The modelling problem is always a cognitive problem and the data, quantitative or qualitative, including the model results, are rarely useful in their raw forms. There is an obvious need for their interpretation: and it is only through this interpretation that we are truly in the presence of a model – or «the model comes to presence».*"

Acknowledgements

I would like to thank my tutor, Mattia De Amicis, and the working group at the Department of Environmental and Territorial Sciences of the University of Milano-Bicocca for their support and company in these three years; David Alexander, my external supervisor, for his always useful yearly suggestions; Jan Blahut, Carolina Garcia and Simone Frigerio from the European Project Mountain Risks (2007-2010) for their friendship and useful scientific discussions which allowed me to feel the pleasure to work with foreign researchers; Giovanni di Trapani from the Mountain Consortium of Valtellina di Tirano for his support during field trips and availability to satisfy all my data and documentation requests; Fabio Luino and Franco Godone from the CNR-IRPI of Turin, for allowing me to cooperate with their scientific activities and to learn from their experience; Dinand Alkema from the ITC Institute of Eschede (The Netherlands) for his support during my two-months period of research at the Institute; Luigi Bertoli from the Adda Consortium, the personnel of the Hydrographic Office at Arpa Lombardia, and Marco La Veglia from AIPO (Cremona Office) for their kindness and availability to supply data and reports; Sonny de Wit (SOBEK) and Fernando Nardi (FLO-2D) for their e-mail support to software problems; Samuel Hagos from the Regional Civil Protection Operational Hall, for the useful information he provided me.
I am also grateful to all the people I forgot to mention here but who helped me to improve the quality of my work.
Last but not least, I would like to thank my family and Corrado (they know why).

References

Alemseged T.H. and Rientjes T.H.M. (2007): Uncertainty issues in hydrodynamic flood modeling. Proceedings of Spatial Data Quality 2007, 5th International Symposium, 13-15 June 2007, ITC, Enschede, The Netherlands.

Alkema D. (2007): Simulating Floods: on the application of a 2D-hydraullic model for flood risk assessment, PhD Thesis, International Institute for the Geo-information Science and Earth Observation, Enschede.

Apel H., Aronica G.T., Kreibich H., Thieken A.H. (2009): Flood risk analyses – how detailed do we need to be?. Natural Hazards (2009) 49 :79-98.

Arcement G.J. and Schneider V.R. (1989): Guide for selecting Manning's roughness coefficients for natural channels and flood plains. United States Geological Survey Water – supply Paper 2339.

Asselman N. et al. (2009): Flood inundation modelling – Model choice and proper application. FLOODsite Project Report N. T08-09-03.

Bates P.D. and De Roo A.P.J. (2000): A simple raster-based model for flood inundation simulation. Journal of Hydrology (2000) 236: 54-77.

Bengtson H. (2010): Introduction to the Manning equation for uniform open channel flow calculations. Available online at: http://www.brighthub.com/engineering/civil/articles/52905.aspx.

Beven K.J. (2000): Uniqueness of place and process representations in hydrological modelling. Hydrology and Earth System Sciences (2000) 4(2): 203-213.

Beven K.J. (2006): A manifesto for the equifinality thesis. Journal of Hydrology (2006) 320: 18-36.

Beven K.J. and Binley A.M.(1992): The future of distributed models: model calibration and uncertainty prediction. Hydrological Processes (1992) 6: 279–298.

Büchele B. et al. (2006): Flood-risk mapping: contributions towards an enhanced assessment of extreme events and associated risks. Natural Hazards and Earth System Sciences (2006) 6: 485-503.

Chow, V.T. (1959): Open channel flow, McGraw-Hill (ed), 680 pp.

Cook A. and Merwade V. (2009): Effect of topographical data, geometric configuration and modelling approach on flood inundation mapping. Journal of Hydrology (2009) 377: 131-142.

Cowan W.L. (1956): Estimating hydraulic roughness coefficients. Agricultural Engineering v. 37, no. 7, p. 473-475.

Cunge J. A. (1975): Two-dimensional modeling of flood plains. In Unsteady Flow in Open Channels (ed. K. Mahmood & V. Yevjevich), Chapter 17.9, pp. 731–750. Water Res. Publications.

Cunge J.A. (2003): Of data and models. Journal of Hydroinformatics 05.2 (2003).

Cunge J.A., F.M. Holly Jr., and Verwey A. (1980): Practical Aspects of Computational River Hydraulics. Pittman Advanced Publishing Program, London, UK.

David E., Erlich M., Masson A. (2009): Benefits of 2D modelling approach for urban flood management. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Ferrier N. and Haque E. (2003): Hazards risks assessment methodology for emergency managers – A standardized framework for application. Natural Hazards (2003) 28: 271-290.

Frank E., Ostan A., Coccato M. and Stelling G.S. (2001): Use of an integrated one dimensional-two dimensional hydraulic modelling approach for flood hazard and risk mapping. River Basin Management book, edited by: R. A. FALCONER, Cardiff University, United Kingdom and W. R. BLAIN, Wessex Institute of Technology, United Kingdom.

Gilvear D.J. (1999): Fluvial geomorphology and river engineering: future roles utilizing a fluvial hydrosystems framework. Geomorphology (1999) 31: 229-245.

Goodchild M.F., Guoqing S., Shiren Y. (1992): Development and test of an error model for categorical data. International Journal of Geo-graphical Information Systems 6, 87–104.

Guzzetti F, Cardinali M, Reichenbach P. (1994): The AVI Project – A bibliographical and archive inventory of landslides and floods in Italy. Environmental Management (1994) 18:623-633.

Hagemeier-Klose, M. and Wagner K. (2009): Evaluation of flood hazard maps in print and web mapping services as information tools in flood risk communication. Natural Hazards and Earth System Sciences (2009) 9(2): 563-574.

Haile A.T. and Rientjes T.H.M. (2005): Effects of Lidar DEM resolution in flood modelling: a model sensitivity study for the city of Tegucigalpa, Honduras. Proceedings of ISPRS WG III/3, III/4, V/3 Workshop "Laser scanning 2005", Enschede, the Netherlands, September 12-14, 2005.

Haile A.T. and Rientjes T.H.M. (2007): Uncertainty issues in hydrodynamic flood modeling, in: Proceedings of the 5th International symposium on Spatial Data Quality SDQ 2007, Modelling qualities in space and time, ITC, Enschede, The Netherlands, 13-15 June, 2007.

Hardy R.J., Bates P.D., Anderson M.G. (1999): The importance of spatial resolution in hydraulic models for floodplain environments, Journal of Hydrology (1999) 216: 124-136.

Holub M. and Hübl J. (2008): Local protection against mountain hazards – state of the art and future needs, Natural Hazards and Earth System Sciences (2008) 8: 81–99.

Horrit M.S. (2006): A methodology for the validation of uncertain flood inundation models. Journal of Hydrology (2006) 326: 153-165.

Horrit M.S. and Bates P.D. (2001): Effects of spatial resolution on a raster based model of flood flow, Journal of Hydrology (2001) 253: 239-249.

Horritt M.S. and Bates P.D. (2002): Evaluation of 1D and 2D numerical models for predicting river flood inundation, Journal of Hydrology (2002) 268: 87-99.

Hunter G.J., Goodchild M.F., Robey M. (1994): A toolbox for assessing uncertainty in spatial databases. In: Masters, E.G. (Ed.), Proceedings of the 22nd Annual Conference of the Australasian Urban and Regional Information Systems Association Inc., Sydney.

Hunter N.M., Bates P.D., Horrit M.S., De Roo P.J., Werner M.G.F. (2005): Utility of different data types for calibrating flood inundation models within a GLUE framework. Hydrology and Earth System Sciences (2005) 9(4): 412-430.

Hunter N.M., Bates P.D., Horritt M.S., Wilson M.D. (2007): Simple spatially-distributed models for predicting flood inundation – A review. Geomorphology (2007) 90: 208-225.

Jones J.L., Haluska T.L., Williamson A.K., Erwin M.L. (1998): Updating flood maps efficiently – Building on existing hydraulic information and modern elevation data with a GIS. U.S. Geological Survey Open-File Report 98-200, from URL http://wa.water.usgs.gov/reports/floodgis/, HTML format.

LAWA (2002): Flood hazard map guidelines of the German Working Group of the Federal States on Water Issues (Bund / Länder-Arbeitsgemeinschaft Wasser (LAWA)).

Luino F. (2005): Sequence of instability processes triggered by heavy rainfall in the northern Italy. Geomorphology (2005) 66: 13–39.

Lumbroso D.M., Mens M.J.P. and van der Vat M.P. (2009): A framework for decision support systems for flood event management – application to the Thames and the Schelde Estuaries. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Markus A.A., Courage W.M.G., van Mierlo M.C.L.M. (2010): A computational framework for flood risk assessment in the Netherlands, Scientific Programming (2010) 18(2): 93-105.

McCarthy S., Tunstall S., Parker D., Faulkner H., Howe J. (2007): Risk communication in emergency response to a simulated extreme flood. Environmental Hazards (2007) 7(3): 179-192(14).

Mens M.J.P., van der Vat M., Lumbroso D. (2009): A comparison of evacuation models for flood event management – application on the Schelde and Thames Estuaries. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Merwade V., Cook A., Coonrod J. (2008): GIS techniques for creating river terrain models for hydrodynamic modeling and flood inundation mapping, Environmental Modelling and Softwares (2008)23: 1330-1311.

Merwade V., Maidment D. R., Goff J. A. (2006): Anisotropic considerations while interpolating river channel bathymetry, Journal of Hydrology (2006) 331: 731-741.

Morvan H., Knight D., Wright N., Tang X. and Crossley A. (2008): The concept of roughness in fluvial hydraulics and its formulation in 1D, 2D and 3D numerical simulation models. Journal of Hydraulic Research (2008) 46 (2): 191-208.

Mueller M. et al. (2009): Short-range plain flood forecasting and risk management in the Bavarian Danube basin. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Murphy P.N.C., Ogilvie J., Meng F.R., Arp P. (2008): Stream network modelling using Lidar and photogrammetric digital elevation models – a comparison and field verification. Hydrological Processes (2008) 22: 1747–1754.

Nardi F., O'Brien J.S., Garcia R., Cuomo G, Grimaldi S. (2009): Updating flood maps using 2D models in Italy – A case study. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Néelz S., Pender G., Villanueva I., Wilson M.D., Wright N., Bates P.D., Mason D. and Whitlow C. (2006): Using remotely sensed data to support flood modelling. Proceedings of the Institution of Civil Engineers, Journal of Water Management (2006) 159 (1): 35-43.

Néelz S. and Pender G. (2007): Sub-grid scale parameterisation of 2D hydrodynamic models of inundation in the urban area. Acta Geophysica vol. 55, no. 1, pp. 65-72.

Néelz S. and Pender G. (2009(1)): Desktop review of 2D hydraulic modelling packages, Science Report SC080035, Joint UK Defra / Environment Agency Flood and Coastal Erosion, Risk Management R&D Program, 63 pp.

Néelz S. and Pender G. (2009(2)): Grid resolution dependency in inundation modelling – A case study, in: Flood Risk Management: Research and Practice, Samuels et al. (eds), Taylor and Francis Group, London, ISBN 978-0-415-48507-4.

Pappenberger F., Beve K., Horrit M., Blazkova S. (2005): Uncertainty in the calibration of effective roughness parameters in HEC-RAS using inundation and downstream level observations. Journal of Hydrology (2005) 302_ 46–69.

Pappenberger F., Matgen P., Beven K.J., Henry J.B., Pfister L., de Fraipont P. (2006): Influence of uncertain boundary conditions and model structure on flood inundation predictions. Advances in Water Resources (2006) 29: 1430-1449.

Plate E.J.(2002): Flood risk and flood management, Journal of Hydrology (2002) 267: 2-11.

Prinos P. et al. (2008): Review of flood hazard mapping. FLOODsite Project Report N. T03-07-01.

Reese S. and Smart G. (2009): High resolution inundation modelling as part of a multi-hazard loss modelling tool. Flood Risk Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Romang H. and Wilhelm C. (2009): Emergency management of flood events in Alpine catchments, in: Flood Risk Management: Research and Practice, Samuels et al. (eds) © Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Srinivas K., Werner M., Wright N. (2009): Comparing forecast skill of inundation models of differing complexity – The case of Upton upon Severn, in: Flood Risk

Management: Research and Practice – Samuels et al. (eds) © 2009 Taylor & Francis Group, London, ISBN 978-0-415-48507-4.

Tarekegn T. H., Haile A. T., Rijentes T., Reggiani P., Alkema D. (2010): Assessment of an ASTER-generated DEM for 2D hydrodynamic flood modeling, International Journal of Applied Earth Observations (2010) 12: 457-465.

Todini E. (1999): An operational decision support system for flood risk mapping, forecasting and management. Urban Water (1999) 1: 131-143.

Verwey A. (2001): Latest developments in floodplain modelling – 1D/2D integration. Proceedings of the Conference on Hydraulics in Civil Engineering, The Institution of Engineers, Hobart (Australia), 28-30 November 2001.

Verwey A. (2005): Hydroinformatics support to flood forecasting and flood management. Proceedings of the Fourth Inter-Celtic Colloquium on Hydrology and Management of Water Resources. Guimaraes, Portugal, July 11-14, 2005.

Vidal J.P., Moisan S., Faure J.B., Dartus D. (2007): River model calibration, from guidelines to operational support tools. Environmental Modelling and Software (2007) 22: 1628-1640.

Werner M.G.F. (2004): A comparison of flood extent modelling approaches through constraining uncertainties on gauge data. Hydrology and Earth System Sciences (2004) 8 (6): 1141-1152.

Werner M.G.F., Hunter N.M. and Bates P.D. (2005): Identifiability of distributed floodplain roughness values in flood extent estimation. Journal of Hydrology (2005) 314: 139-157.

Wise S. (2000): Assessing the quality for hydrological applications of digital elevation models derived from contours, Hydrological Processes (2000) 14: 1909-1929.

Zerger A. (2002): Examining GIS decision utility for natural hazard risk modelling. Environmental Modelling and Software (2002) 17: 287-294.

AIPO – Interregional Agency for the Po river (2008): Lavori di sistemazione della confluenza fiume Adda e torrente Poschiavino nel Comuni di Tirano, Villa di Tirano e Bianzone, Relazione tecnica.

Lombardy Region and ANAS S.p.a. (2002): Progettazione definitiva, comprensiva del quadro progettuale e del quadro ambientale, dello studio di V.I.A. per procedura regionale della SS. 38, variante di Tirano, dallo svincolo di Stazzona (compreso) allo svincolo di Lovero (con collegamento alla dogana di Poschiavo) – Relazione idrologica e idraulica.

Magistrato per il Po (1997): Lavori di adeguamento della sezione di deflusso e consolidamento delle opere spondale in Comune di Tirano, Relazione Idrologica. Ufficio di Cremona, 1997.

Merizzi G. and Baldini D. (2007): Studio idraulico e geologico finalizzato al tracciamento delle fasce fluviali alla scala dello strumento urbanistico comunale e alla ridefinizione delle norme geologiche all'interno della fascia fluviale C, Relazione idrologica e idraulica.

Govi M., Turitto O. (1994): Ricerche bibliografiche per un catalogo sulle inondazioni, piene torrentizie e frane in Valtellina e Valchiavenna. Associazione Mineraria Subalpina, Quaderni di Studi e di Documentazione n. 16.

Guida M., Iaccarino G., Metcalf G., Vallario A. (1979): Bibliografia delle frane dal 1900 al 1978. Vol. XL, CNR, Roma.

CNR-GNDCI (1983): Eventi alluvionali e frane nell´Italia Settentrionale: Periodo 1972-74. CNR-GNDCI, Pubblicazione n. 1897, Torino.

Govi M., Luino F., Turitto O. (1996): Tipologie dei processi e sequenze di sviluppo: due esempi recenti. In: Luino F (ed) La prevenzione delle catastrofi idrogeologiche: il contributo della ricerca scientifica, Convegno internazionale, Atti Vol. 2, CNR-GNDCI, Alba, Pubblicazione n. 1600, pp. 261-278.

Cardinali M. et al. (1998): Catalogo delle informazioni sulle località italiane colpite da frane e da inondazioni. CNR-GNDCI, Pubblicazione n. 1799, Perugia.

Tropeano D. et al. (1999): Eventi alluvionali e frane nell´Italia Settentrionale: Periodo 1975-1981. CNR-GNDCI, Pubblicazione n. 1927, Torino.

L. 183/89: Norme per il riassetto organizzativo e funzionale della difesa del suolo, Italian National Law, 1989.

PAI (2001): Piano stralcio per l'assetto idrogeologico per il bacino idrografico di rilievo nazionale del fiume Po, adopted with Deliberazione 18/2001, Autorità di Bacino del fiume Po, available at: http://www.adbpo.it/on-multi/ADBPO/Home/Pianificazione/Pianistralcioapprovati/PianostralcioperlAssettoIdrogeologicoPAI/ Pianovigente.html.

L.R. 11/03/2005, n. 12: Legge per il governo del territorio, Lombardy Region Law.

D.G.R. 22/12/2005, n. 8/1566: Criteri ed indirizzi per la definizione della componente geologica, idrogeologica e sismica del Piano di Governo del Territorio, in attuazione dell'art. 57, comma 1, della l.r. 11 marzo 2005, n. 12, Lombardy Region Decree.

Directive 2007/60/EC: On the assessment and management of flood risks, European Parliament and the Council of the European Union.