FLOOD HAZARD ASSESSMENT IN A FLOOD PRONE URBAN AREA USING HYDRODYNAMIC MODELS AND GIS TECHNIQUES Case of study in Pergamino, Argentina

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Case of Study in Pergamino, Argentina

by

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Disclaimer

To my parents Jorge and Carmen

Abstract

The use of hydrodynamic models to assess flood events and levels has been applied all over the world for already many years. However, in developing countries flood hazard assessments has to overcome limitations of the information and lack of technical assessment. This work focuses on the flood hazard assessment in a flat area of Pergamino (Argentina) using two hydrodynamic models with the support of GIS techniques. The research attempts the evaluation of flood characteristics of the river using 1D model HecRas and the assessment of the flood propagation in the urban area using model Flo-2D. The work is based on data collected from previous studies and validation values taken during fieldwork.

HecRas model allowed the calibration and validation of the rating curve for the only city liminimeter (Florencio Sanchez). The wave celerity and the travel time of the wave were also calculated. The limnigraph stations Alfonzo y Urquiza located upstream and downstream of the urban area were evaluated as source of data for the modelling process, however, a backwater effect found in the area of Alfonzo plus the scarcity of topographic information were determinant to restrict their application. A DTM generated from points measured during a topographic survey was used as main data. Major attention is place in the construction of the DTM and advices exposed. The three origin of flood events were simulated: a) Heavy rainfall, b) Overflow from the channel (riverine event) and c) the combination of both events for 10 years of return period. During the simulations the volume conservation and numerical stability were evaluated, and a process considering changes in the numerical stability criterion was performed. Furthermore a comparison between the maximum simulated flow depth for a past event and the values recorded locally was attempt.

The results showed a properly performance of Hec Ras model for the river channel in steady flow conditions, a hazard map with the water depths under the levees have failure problems was presented. Furthermore, the simulations with Flo-2D allowed the generation of flood hazard maps as first approach or draft for the city. In the case of scenario a), the three hours total rainfall did not generate high level of flood hazard, a medium hazard level was predominant in the case of scenario b) and c). The comparison between the simulated maximum flow depth for a past event and the values recorded in Pergamino showed the need to improve the data collection with instrumentation in a systematic manner to allow further validations.

Key words: Flood hazard, hydrodynamic models, Hec Ras, Flo-2D, Pergamino city.

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1. INTRODUCTION

1.1. Background

Flood, is defined as an overflowing of water onto land that is normally dry. Flood events have existed and will continue to exist around the world. They are considered one of the most recurrent natural hazards in urban and rural areas and although efforts continue, nowadays the number of people affected by this phenomenon does not decrease. Only between 1900 and 2005 around 17 million people were killed by floods and droughts and more than 5 billion were affected in the world (Cooley *et al.*, 2006).

At the moment of assessing flood events, the attention focuses in regions where human presence and prone land areas combine. The developing of urban cities implies the change in the land use that generally results in more runoff. According to the CEOS (CEOS, 2003) the runoff in an urban area increases from two to six times when compared with the natural terrain. Moreover, the geomorphology of the area will strongly influence the flood characteristics and the way to approach its assessment. For example, in flat areas where the water tends to stay long, flood duration and water depth are the main responsible for the damage; whilst in mountain regions water velocity and erosive power of discharge are the main factors.

The traditional approach to simulate flow in river channels is done with one-dimensional (1D) hydrodynamic modeling. However, in cases when speed and localized forces have to be known more accurately in other 2D or 3D hydrodynamic models are being use (Merwade *et al.*, 2008). In these cases, the principal constraint is the input data availability.

Therefore, the compilation of the available data and the post processing analysis of the modeling results might be supported by the use of GIS techniques. Besides that, from the experience in previous researches, a GIS–based flood assessment allows the compensation of the data scarcity and assist in the validation of the modelling results (Peters, 2008).

This study focuses on the assessment in one of these developing flood prone areas, Pergamino (Argentina). This region was affected for several flood events throughout its history. Between 1912 and 2002, there were 113 floods of different degrees of severity. Of these, 35 resulted in water levels reaching heights that demanded the partial evacuation of the local population (Herzer, 2006). One of the greatest flood events that caused major damage to Pergamino happened in 1995. After two days of rainfall medium intensity, a storm of 6 hours duration, struck with over 300 mm of rainfall, a case with no precedents in the history of the region. This caused overflow of streams, and urban flooding which was aggravated by the backwater from the Pergamino river into the drainage system of the city. As a result, four people died and the city suffered significant economic losses (IATASA and ABS, 2008).

The flood events in Pergamino city happen from three sources: the first one related to the river overflow as the flow from the upstream subcatchments produces the rise of the water level in the channel and the water exceeds the banks. The second one is caused by heavy local rainfalls; the rain and the insufficient hydraulic capacity of the sewage system combines produce the flood. The last one is the combination of the overflowing in the river and the heavy rainfall in the city.

Furthermore, the assessment of the flood hazard in Pergamino is not an isolated issue. Other surrounding cities with similar characteristics in terms of topography and hydrologic suffer similar flood problems. For example, in December 2009, Arrecifes and San Antonio de Areco (located downstream of Pergamino) had a flood event without precedents (Cornejo, 2009).

Figure 1 Recent flooding in San Antonio de Areco (December 2009). Pergamino dikes resisted with less than 90 cm of overtopping.

In this context, several efforts to understand the floods and prevent more damage in Pergamino have been made by the Municipality, Neighborhoods Association (CO.S.S.O.PER) and the Province Government through institutions as the Water National Institute (INA); this research intends to contribute with that from a modeling approach and the assessment of results in terms of flood hazard.

1.2. Problem statement

In developing countries, the flood hazard assessment has not been evaluated in full magnitude yet. When a flood event happens in a region, in most of cases, the attention is focused only in the humanity help to the affected after the event. However, the data collection and correct compilation, during and after the event (e.g. water levels signs, peak discharges, duration of the flood, etc), are not done properly for the authorities. This inaction reduces the chances of accessing vital information. The lack of the technical assessment in flood events can be overcame using hydrodynamic models and GIS techniques; however, the limitation on the information available will limit the accuracy of the results. Understanding what is possible to achieve during flood assessments under data scarcity constraints helps to propose strategies to face the problem. One and two dimensional hydrodynamic models are proposed to asses flood events. The knowledge of the model requirements and its performance are crucial before deciding on its used for a specific area.

As discussed in the background section, Pergamino region faced several flood events in its history. The topographic characteristics (rolling, local name: “pampa ondulada”) makes the region prone to suffer flood events. The authorities are developing a project to construct two dams upstream the city (IATASA and ABS, 2008), but a combination between political and economic problems are postponing the execution. This project will also be considered in this thesis and the results will contribute to some extra information for decision makers. The flat character of the area suggests the used of two dimensional models to asses the flood events; however, the acquisition cost and the lack of experience in the applications of these models become a constraint. The evaluation of the model performance, in this case with FLO – 2D, will give reference to the level of analysis that it is possible to achieve with the available data. Furthermore, the use of Hec Ras, can also contribute in the evaluation of the flood problem considering the attention only in the river channel and next-to-river floodplain areas. The premise of this study is to demonstrate the level of assessment that can be achieved with the combination of both models.

1.2.1. Main objective

The main objective in this study is to assess the flood hazard in the urban area of Pergamino using hydrodynamic models supported by ground information, and GIS techniques.

In order to achieve the main objective, the following specific objectives are formulated:

1. To asses the flood characteristics in Pergamino River using 1D model Hec Ras.
2. To study the flood propagation in Pergamino city using 2D model Flo-2D.
3. To compare the models results with the information provided in the area.
4. To generate a flood hazard map for the Pergamino city.

1.2.2. Research questions

1. What are the main characteristics of the Pergamino stream in terms of water depth and other water dynamics?
2. What is the flood propagation produced in the urban area of Pergamino for the studied event?
3. What is the difference between the models results and the information provided in the area?
4. What is the flood hazard in Pergamino city?

1.3. Thesis outline

The research begins with the Introduction Chapter, which presents the problem background as a description of the historical and current situation. Furthermore, describes the problem statement with the objectives and the research questions. An overview about the bibliography is presented at the end of this chapter.

Chapter 2 has two parts, the first one related to the description of the study area, location, climate and relation between historical rainfall and flood events. The second one focuses in the description of the available data like topography and hydrologic.

The methodology applied in the research is developed in Chapter 3. Both models, Hec Ras and Flo-2D are described in individual section, focusing in the introduction of the input data and the assessment during the processes.

Chapter 4 presents the results. The main results of the simulations are discussed and the comparison with information provided in the area is also done.

The conclusions and recommendations are presented in Chapter 5.
An appendix chapter is added with the fieldwork information and a memory that includes some process.

1.4. Literature review

Water is essential for the development of any civilization. Since the beginning of the history, human beings settled their cities and developed their activities near water sources. For centuries, several flood events have showed the power of this resource, flooding has been one of the most devastating disasters both in terms of property damage and human casualties (Mujumdar, 2001). Flooding as a natural phenomenon does not mean death and destruction, that kind of disaster event happen when human lives or property values are affected (Dworak and Hansen, 2003). A hazard is defined as a phenomenon that may cause disruption to humans or their property and infrastructure. The hazard assessment determining the type of hazardous phenomena that may affect the area, their frequency and magnitude, and representing on a map which areas are likely to be affected (Van Westen, 2000).

However, the flood assessment will depend on the kind of event because all floods are not alike. A flood travels along a river as a wave, with velocity and depth continuously changing with time and distance(Mujumdar, 2001). Some floods develop slowly, sometimes over a period of days; in this case the attention will be focus in the duration and depth of the water more than the velocity. But flash floods can develop quickly, sometimes in just a few minutes and without any visible signs of rain, in this cases the velocity will be the factor to cause more damage (Fema, 2005).
Many urban areas are developed close to the rivers. However, the flood risk will depend on the vulnerability of the city to the hazard event. (Imperviousness due to asphalts on streets decreases the soil infiltration capability and the surface runoff increases)

1.4.1. Modelling flood events

The representation of hazard phenomenon in the real world is possible using models; their performance will help to understand it and to raise mitigation strategies. For the case of inundation in urban areas, modelling enables to calculate the magnitude of the event in terms of water depth and velocity. However, the complexity in the urban environment and the lack of high resolution topographic and hydrologic data compromise the implementation of those models in developing countries (Chen *et al.*, 2009).

Many approaches have been developed in order to understand and forecast the hydrodynamic response of the rivers. Conventionally, the flood damage assessments have been doing through the use of one dimensional (1D) models. This kind of models are very useful to asses the response of the river (Alkema *et al.*, 2007). They do not fully consider the effect of cross section shape changes, bends, and other two-dimensional and three-dimensional aspects of flow. All flow is parallel to the direction of the main channel (Tennakoon, 2004). Although this assumption is not theoretically correct, it is suitable for most open channel hydraulic work (Dyhouse *et al.*, 2003). These models are usually applied to study flood levels and discharges in river systems, and have been applied successfully in modelling flood routing at river reach scales from tens to hundreds of kilometers (Werner, 2004).

One dimensional models are capable of calculating flood levels and discharges quite accurately in applications where the flow path is mainly “linear”. However, in urban areas two dimensional (2D) models have more sounding theoretical basis for flooding simulations. These models require dedicated and continuous representation of terrain topography (mainly the preservation of narrow water containing structures degradable during DEM generation) and provide information about: flood water depth, velocity and spatial distribution, variation of extent and duration over a user defined time frame (Peters, 2008). The output produced by them can be easily transferred to decision makers as they match maps and GIS, However, their application, most of the time, requires considerable cost and time for data collection.

1.4.2. River and Flood routing

Flood routing or stream routing is solved using mathematical procedures. Nowadays, is approached by the use of numerical techniques (Brutsaert, 2005). During a flood event, depth and velocity are assessed. Both flow properties change with the time, so the flood flow is considered unsteady and gradually varied. The principles which governs the wave movement are the Saint Venant equations: Continuity (Conservation of mass) (Equation 1) and Momentum (Newton`s second law of motion) equation (Equation 2).

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

Equation 1

$$\overbrace{\left(\frac{1}{A}\right)\frac{\partial Q}{\partial t}+\left(\frac{1}{A}\right)\partial\left(\frac{Q^2}{A}\right)\Big/\partial x+\overbrace{g\frac{\partial y}{\partial x}\overbrace{-g(S_o-S_f)}^{\text{Kinematic wave}}}^{\text{Diffusion wave}}}^{\text{Dynamic wave}}=0$$

Local Acceleration term	Convective Acceleration term	Pressure Force term	Gravity Force term	Friction Force term

Equation 2

Where ‘Q’ is the discharge (m^3/s), ‘A’ is the area, ‘q’ is the lateral flow per unit length of the channel ($m^3/s/m$), ‘x’ is the distance along the channel, ‘y’ is the depth of flow, ‘g’ the Earth gravity, ‘S_o’, the bed slope of the channel and ‘S_f’ is the friction slope. Some flow routing equations are generated by using the Equation 1 and neglecting some terms of the Equation 2, this is according to the accuracy level desired or simplifications that can be done after analyzing the flow type (Mujumdar, 2001).

As it is not possible to solve the above equations analytically (except in some very simplified cases), numerical solutions are possible to obtain the variation of discharge and depth with time, along the length of the water body. The numerical methods start at time= 0 with an initial condition and boundary conditions. The boundary conditions describe the exchange of water mass between the study area an the rest of the universe during the model run (Alkema *et al.*, 2007). Upstream boundary conditions are specified commonly as discharge hydrograph, on the other side, downstream conditions might be specified as stage or discharge hydrograph, stage-discharge relationship, or hydraulic fall (Tamiru, 2005).

1.4.3. Impact of the grid size in flooding assessment

There are two basic data requirements to estimate an extent of flooding map using hydrodynamic models: the elevation model and the cross sections lines of the area under study (Werner, 2001). Digital Elevation Models (DEM's) are used to parameterize a 2D hydrodynamic flood simulation algorithm and predictions are compared with published flood maps and observed flood conditions. Recent and highly accurate topographic data should be used for flood inundation modeling. These conditions proved crucial in this research. Therefore, these elevation models should be used cautiously in the context of flood zone mapping for they may cause a systematic underestimation of flood risks (Sanders, 2007). Cross section information on river channels usually comes from ground surveys, sometimes the data are typically not dense enough to capture all channel features however the interpolation process inside the models made feasible their use (Merwade *et al.*, 2008).

The grid size governs the running time and the accuracy of the results. As a result, big grid size will decrease the simulation time but it will generate results where some narrow or small structures are omitted. Small pixel sizes will take in count more details but the time of simulation increases exponentially. However, the simulation time will depend also on the computer machine characteristics. The challenge is to find a balance between acceptable computation times and accurate representation of the surface topography (Alkema *et al.*, 2007).

Some previous studies can be considered as reference. According to Tennakoon (2004), in the flood hazard mapping for Naga city, a high resolution (better than 7.5 m DTM) is required for studies related to exploration of flow conditions around individual structures. Furthermore, a resolution of 10 m pixel size is sufficient for generating realistic urban flood hazard maps. However, these values refer to the use SOBEK model. For the case of FLO-2D, in previous experiences, grid size is not lower than 30 m. For example, in the Development of the Middle Rio Grande the U.S. Army Corps of Engineers (Army Corps of Engineers, 2002) used 152 m (500 ft) and in the Preliminary Flood Study for Tract Map 6731, the consulting civil engineers (Cornerstone Enineering, 2007) used 30.48 m (100 ft) for their simulations. The consideration of affections of structures inside the pixel is done by shape and volume factors.

1.4.4. One dimensional model Hec Ras

The mass conservation and momentum conservation equations (Equation 1 and Equation 2) are solved in the one dimensional model Hec Ras by an implicit linearized system of equations using Preissman's second order box scheme. In a cross section, the overbank and channel are assumed to have the same water surface, though the overbank volume and conveyance are separate from the channel volume and conveyance during the use of conservation of mass and momentum equations. The simultaneous system of equations generated for each time step (and iterations within a time step) are stored with a skyline matrix scheme and reduced with a direct solver developed specifically for unsteady river hydraulics (Hicks and Peacock, 2005).

The model application to conduct a flood routing and water level simulation requires the following input: channel geometry, boundary conditions, tributary inflows and channel resistance. The water level output in every cross section shows the performance of the model in terms of flooding events (Brussel, 2008). Even though the model does not work by itself in a spatial distribution, the results allow an accurate interpretation of the real flood situation. Furthermore, interfaces with GIS tools improve the use of these model (Knebl *et al.*, 2005).

1.4.5. Two dimensional model Flo2D

FLO-2D is a grid-based physical process model which routes precipitation-runoff and flood hydrographs over unconfined surfaces and channels using either a kinematic, diffusive or dynamic wave approximation to the momentum equation (Equation 2) (Hübl and Steinwendtner, 2001). During the simulation, the model routes flows in eight directions (Figure 2). The spatial and temporal resolution is dependent on the size of the grid elements and rate of rise in the hydrograph (O'Brien *et al.*, 2009).
The Flo-2D flood routing scheme follow , in a brief summarize, the follow steps: (For more detail refers to Flo-2D Manual (O'Brien *et al.*, 2009)) :

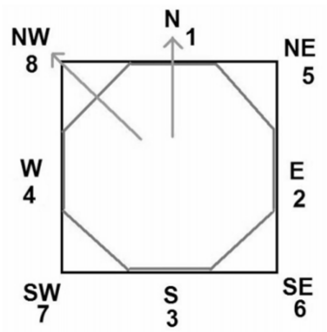

Figure 2 Work schematic of each grid

1. The average flow geometry, roughness and slope between two grid elements are computed.
2. The flow depth for computing the velocity across a grid boundary for the next timestep is estimated from the previous timestep.
3. The first estimate of the velocity is computed using the diffusive wave equation. (Equation 2).
4. The predicted diffusive wave velocity for the current timestep is used to solve the full dynamic wave equation for the solution velocity.
5. The discharge across the boundary is computed by multiplying the velocity by the cross sectional flow area.
6. The incremental discharge for the timestep across the eight boundaries is summed and the change in volume is distributed over the available storage area to determine an incremental increase in the flow depth.
7. The numerical stability criteria are then checked for the new grid element flow depth. If any of the stability criteria are exceeded, the simulation time is reset to the previous simulation time, the timestep increment is reduced, and all the previous timestep computations are discarded and the velocity computations begin again.
8. The simulation progresses with increasing timesteps until the stability criteria are exceeded.

In this computation sequence, the grid system inflow discharge and rainfall is computed first, then the channel flow is computed, next the overland flow in 8- directions is determined (Figure 2). The model verified three stability criteria to avoid volume conservation problems (error allowed lower than 0.001%). These criteria are checked by the model in a sequence of steps:

1. First, the percentage change in depth (the value suggested by the manual is equal to 0.2).

2. Then, Courant-Friedrich-Lewy (CFL) criterion, which relates the floodwave celerity to the model time and spatial increments. The physical interpretation of this criterion is that a particle of fluid should not travel more than one spatial increment Δx in one timestep Δt:

$$\Delta t = C\Delta x/(\beta V + c)$$ *Equation 3*

Where 'C' is the Courant number (the model works with a value equal to 1.0 and it does not allow to the user to change it), 'Δx' is the square grid element width, 'V' is the computed average cross section velocity, 'β'is a coefficient (5/3 for a wide channel) and 'c' is the computed wave celerity (O'Brien *et al.*, 2009).

3. Finally the Dynamic wave stability criteria governed which is an extension of the Courant Criteria.

$$\Delta t < \xi S_o \Delta x^2 / q_o$$ *Equation 4*

Where 'Δx' is the grid width, 'q_o' is the discharge, 'S_o' is the bed slope and 'ξ' is the dynamic stability coefficient "Wavemax". The model verifies the numerical stability criteria in every grid element at each timestep to ensure that the solution is stable. The model by default used 1.0 for Wavemax, however, it is possible and sometimes necessary to modified the value (O'Brien *et al.*, 2009). The understanding of the stability procedure is essential to run FLO-2D, at it might save days of processing.

- Wavemax = 0.1 to 1.0 (typical value = 0.25): Dynamic wave stability criteria increments and decrements the computational timestep when Wavemax is exceeded, the model runs more slowly but is stable.
- Wavemax = -0.1 to -1.0 (typical value = -0.25): the model does not consider the third criterion and the increment or decrement of the timestep is according to the two first criteria. The floodplain roughness values are incremented when the stability criteria exceed, but the timestep is not decreased.
- Wavemax >100 (typical value = 100.25): the timestep are incremented or decremented only by the two first criteria, but in this case, there is no n-value adjusted.

Flo-2D starts with a minimum timestep equal to 1 second and increases it until one of the three numerical stability condition is exceeded, and then the timestep is decreased. If the stability criteria continue to be exceeded, the timestep is decreases 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.

1.4.6. Flood warning system proposed for Pergamino city

The project "Defense works and storm drains in the Pergamino city"((IATASA and ABS, 2008) formulates, some strategies to manage the flood events in the urban area. The proposal considers as strategic to control the flood through the construction of two regulation dams upstream of the city.

This measure is complemented by the suggestion of non structural measures related to the reduction of the vulnerability in the city as the establishment of a flood warning system. The warning system will be divided in two control areas, the first one related to the monitoring of the correct operation of the dams and the second one related to the warning system for the city. In general terms, this system considers the use of heavy rainfall forecast from the National Meteorological Service in Argentina and the installation of three stations to measure precipitation, water level, velocity and quality water, locate in the catchments upstream of the city, between the dams and the city. However, as the project has not been executed yet, nowadays the city is prone to any flood event without previous warning.

2. STUDY AREA

2.1. Main Characteristics

2.1.1. Location

The Pergamino catchment is located in the rolling Pampas region (local name: pampa ondulada), in Argentina (South America). With an area of 2092 km^2, it is subdivided in four sub catchments: the Upper, Upper-middle, Lower-middle and Lower basin.

The study area is located between the Upper-middle and the Lower-middle basin (province of Buenos Aires, Argentina) between S33.88º, S33.92º of latitude and W60.60º, W60.56º of longitude (Figure 3).

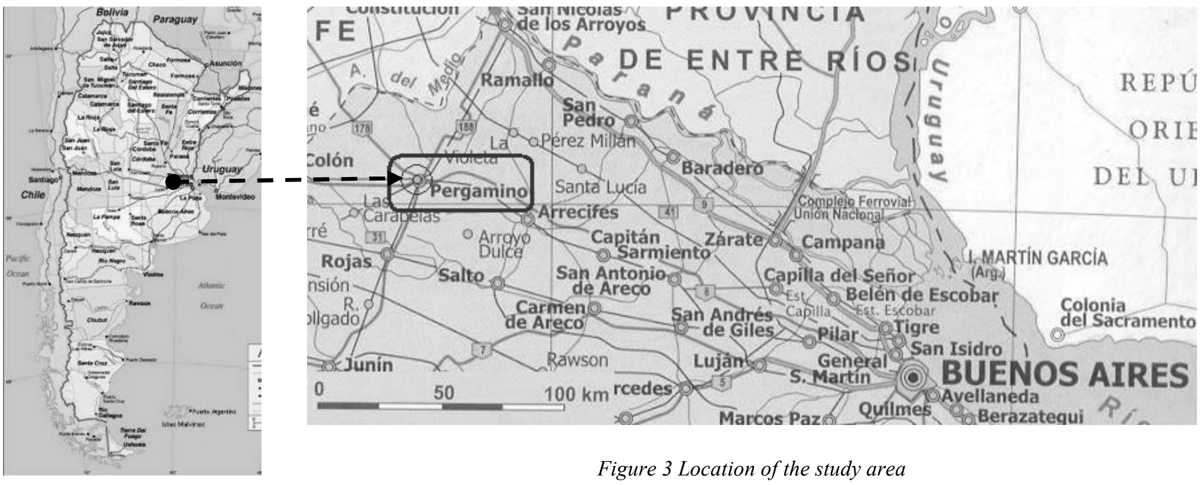

Figure 3 Location of the study area

Pergamino city has a population of about 85,000 inhabitants and it is considered 8th city in Buenos Aires province. Pergamino is considered the Argentina's main agricultural region, with high value of land. In terms of economic importance, this area has a high level with the production of soya, corn and wheat. In 2007, more than one fifth of Argentine exports of about US$56 billion were composed of unprocessed agricultural primary goods, mainly soybeans, wheat and maize.

2.1.2. Climate

The temperature average annual is 15ºC (10ºC in winter and 22º in summer). And, the average annual precipitation is around 1000 mm (range in 1961 – 2007: 588mm a 1562 mm). The average annual Potential Evapotranspiration is around 1000 mm The situation of average balance of rainfall and evaporation is biased by cyclic periods of droughts and excess, common in the Province of Buenos Aires (INA, 2004).

2.1.3. Relation between historical rainfall and flood events

As it was mention in background section, Pergamino city has a long record of flood events during its history. In the Figure 4 the flood events from 1933 until 2000 are presented. Each event was classified according to the level of impact:

- Slight level means that flooding happened in specific areas caused by rainfall without overflow of the river.
- Moderate level means flooding of extend areas in the city without evacuation of the population.
- High level means flooding with evacuation.
- Very high level means big impact in the population in terms of duration and water extend.

This classification was done after newspaper reports and because the information was not always accurate or complete, these categories are relatively subjective (Centro, 2000).

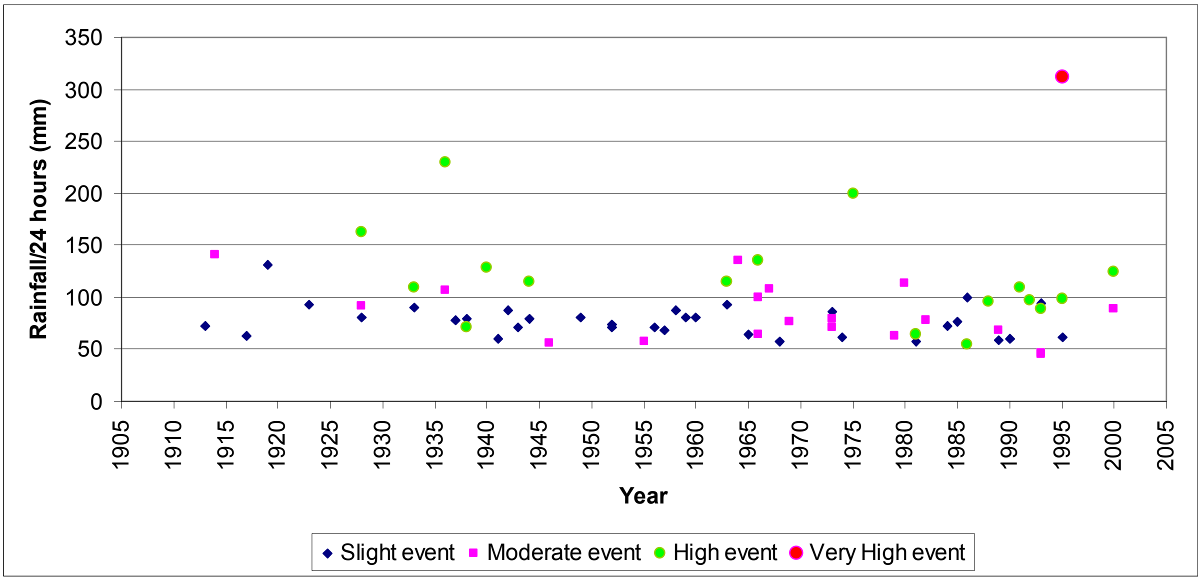

Figure 4 Flood events in Pergamino
Source: Centro estudios sociales y ambientales,2000

Slight events was caused by similar precipitation (range around 50 mm and 100 mm), however in the same range of rainfall, some moderate and high events happened, which means that similar rainfall events can produce different impact in the same area. Moreover, the records show that there is no similar recurrence between events with the same class, e.g. in the first years of the records (1915 to 1925) there were not events categorized as High level, then the next years (1925 to 1944) those kind of events appear, later on, (1945 to 1960) not floods with that category were recorded but during the last period (1980 to 1995) the high events occurred more frequently than before.

2.1.4. Recurrence Period for Precipitation in Pergamino

The Figure 5 present below was generated from the Maximum Precipitation in 24 hours record measured in INTA Station, Pergamino. According to that, for 10 years of return period the Maximum Precipitation is around 150 mm. However, this study considers for the modelling process a total precipitation in three hours calculated by IATASA as part of the project “Defense works and drainage for Pergamino city”.

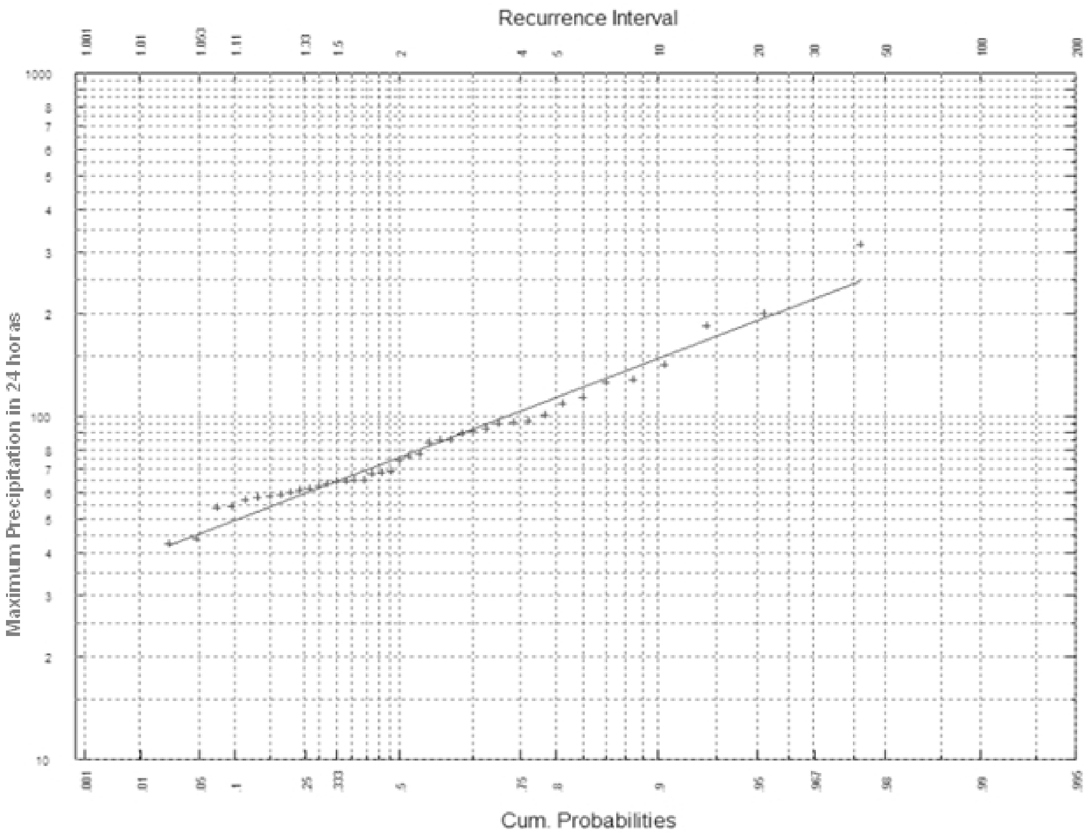

Figure 5 Gumbel probability for the precipitation record in Pergamino (1967 – 2007)
Source: INTA, 2009

2.2. Data availability

Pergamino River, has a historic past related with flooding problems, because of that, several studies were made by different organizations in the last years. From these studies some of the data were available and collected. Part of that information was available and assessed during the pre – fieldwork and the rest was completed during the fieldwork.

The Figure 6 presents the areas where were collected the data. The figure is quiet important for the understanding of this thesis as the areas with different level of data availability are defined as the geographical position with respect to Pergamino. Alfonzo Station is upstream and Urquiza station downstream of the Thesis area.

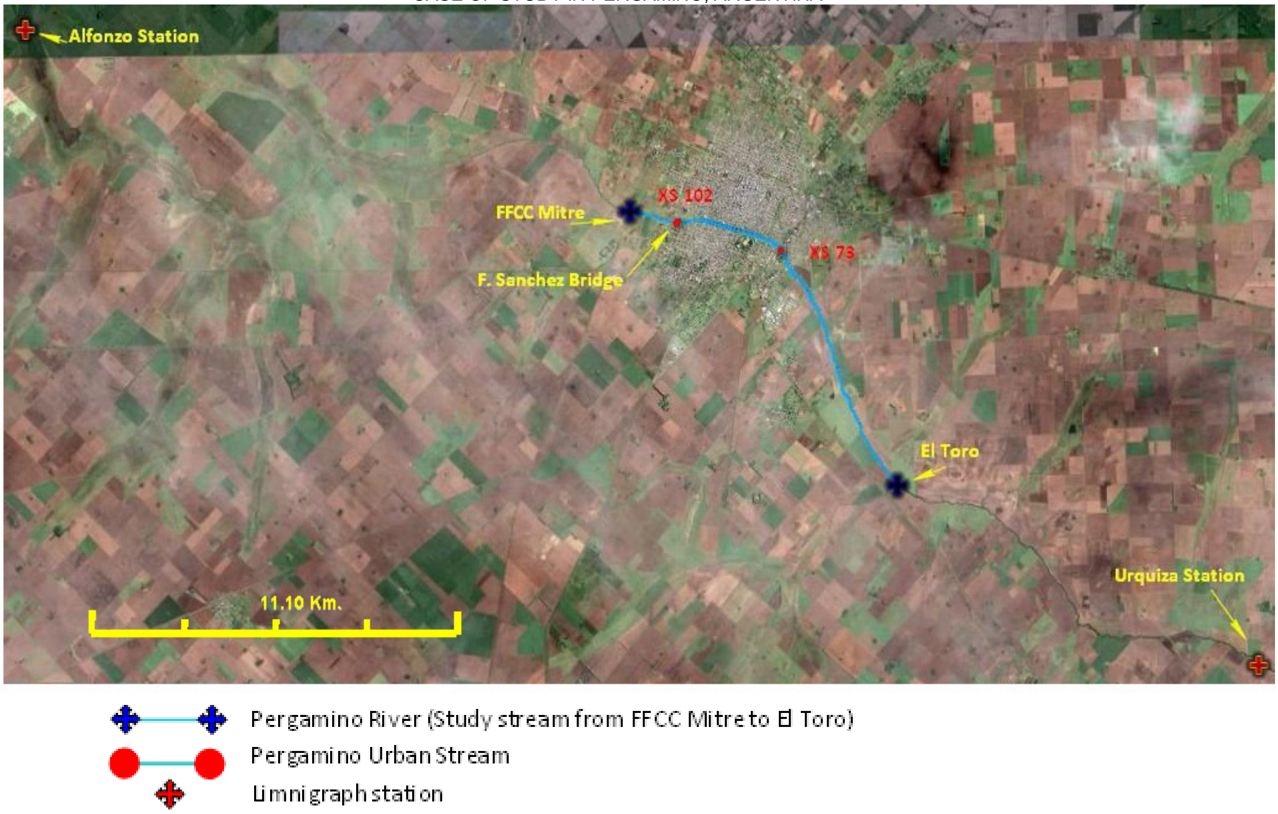

Figure 6 Location of the areas assessed during the study

Most of the data were collected for the study area between FFCC Mitre and El Toro, as begin and end of the study stream. However, some data available were also collected from the two new linmigraph stations located at Alfonzo station and at Urquiza Station of the study area.

As it will be explain in further chapters, during the first part of the research, for the assessment using Hec Ras model, all the area showed in Figure 6 between FCC Mitre and El Toro were taken into consideration as river cross sections were available. For Flo-2D model the study area was focused only in the urban region.

2.2.1. Topography data available

a) Cross sections: 118 river and floodplain cross sections between FFCC Mitre (Cross Section 118) to El Toro (Cross Section 0),with a distance between each other in a range of 60 – 150 m (Figure 7).
b) Planview blueprint: With the location of the cross sections and the principal civil structures like a bridges and main streets (Figure 8).
c) Contours map of Pergamino city: with an interval of 0.5 m (datum: Campo Inchauspe, Projection Gauss Krugger).
d) Topographic Points: Coordinates (X, Y, Z) of 7156 points distributed in Pergamino city. (datum: Campo Inchauspe, Projection Gauss Krugger).
e) Toposheet: Scale 1:50000, source IGM (1958). Includes geodetic points and contours.

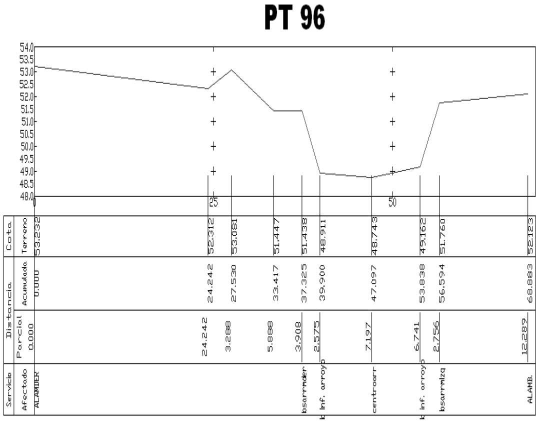

Figure 7 Cross Section example

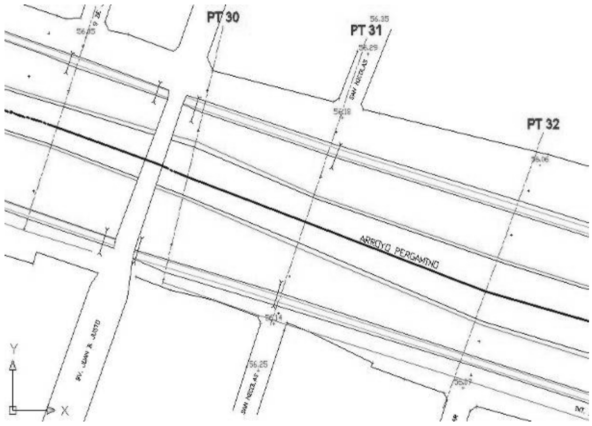

Figure 8 Plan example

f) Cross Sections at the Limnigraphs stations areas: 8 for Alfonzo Station and 3 for Urquiza Station.

g) Bridges characteristics: dimensions of the bridges located in the study stream and in Alfonzo Station.

2.2.2. Hydrological data

a) Rating curve at **F. Sanchez** bridge [H= F(Q)]: The rating curve was built after individual and isolated points measured for several discharges since 1967 to 2004 at the limnimeter located in F. Sanchez Bridge (urban area, close to the cross section 102). No points are available during moderate to high river levels (See Figure 9).

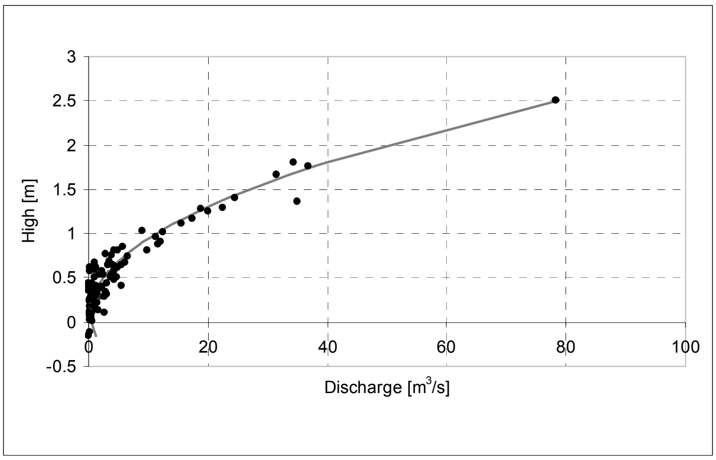

Figure 9 Rating curve Discharge- High curve at F. Sanchez Bridge

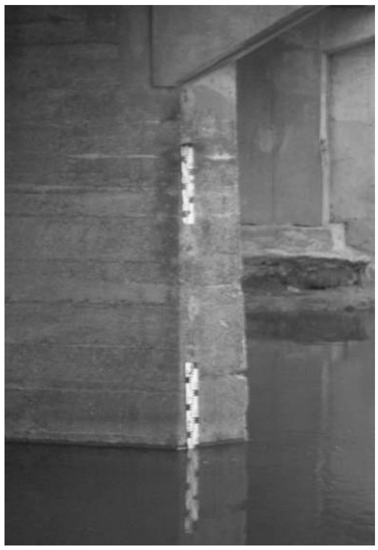

Figure 10 Linmimeter in F Sanchez Bridge

These values were measured systematically, therefore a frequency analysis is not possible to achieve, more over, the extreme flood events occurred in the urban area of Pergamino were not recorded through this limnimeter.

The best-adjusted just curve is presented below:

$$Q = 13.63949 * H^2 - 2.784557 * H + 0.4895915$$ *Equation 5*

Source: INA, 2004

Where Q is discharge in m^3/s and H is water depth in 'm' (value read in the limnimeter).
During fieldwork the water depth value in the limnimeter was measured. It will be later used as calibration point.

b) Water level values from the two limnigraph stations at Alfonzo and Urquiza: because of the stations were established in 2008, the record is only available for that period.
c) Hydrographs: according to the objectives formulated, the hydrologic study of the area is not part of this research, mainly due to limited resources. However, a hydrograph to run a hydrodynamic model is necessary as an input. In a previous study "Study of defense projects and flood control Pergamino stream" the follow design hydrographs to the Pergamino city were determined for different return periods.

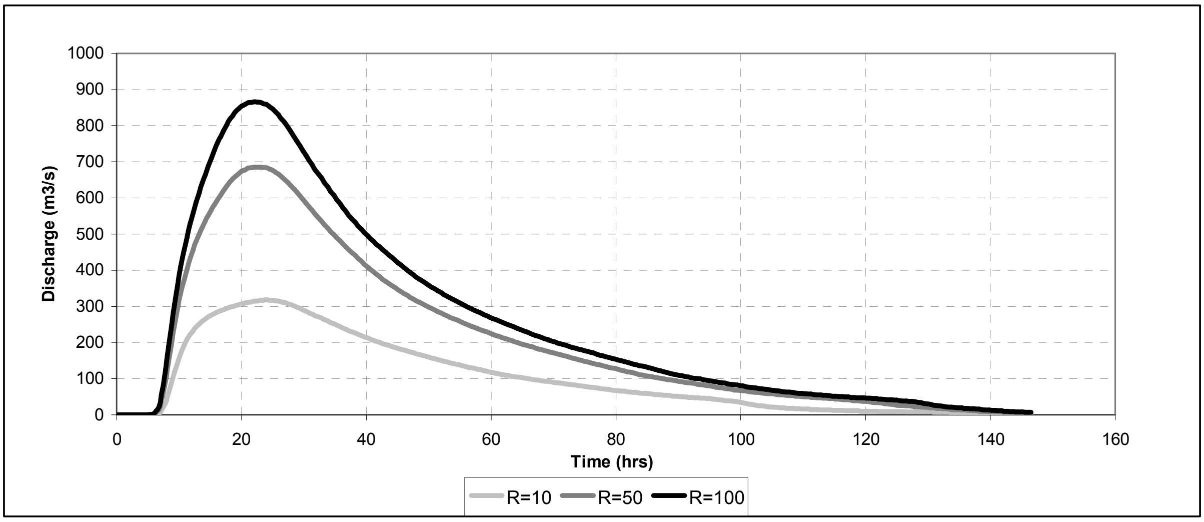

Figure 11 Income hydrographs to the Pergamino city
Source: INA, 2004

d) Discharges values: during fieldwork three values of discharge were measured. One close to F. Sanchez Bridge (cross section 102), the second in Alfonzo Station and the last one at Urquiza Station.(The detail of the measurements and calculations are presented in Appendix A1).
Furthermore, in the same hydrologic study, after a recurrence assessment, INA[1] determined the follow peak discharges at the begin of the urban stream (FFCC Mitre Bridge).

Table 1 Maximum Discharges for several recurrences

Recurrence (years)	Peak discharge (m3/s)
2	66
5	163
10	317
20	487
50	682
75	696
100	847

Source: INA, 2004

[1] INA: (Instituto Nacional de Agua) is the official water board at national level in the country

e) Rainfall data: a hyetograph of 3 hours of duration and 10 years return period from the "Study of defense projects and flood control Pergamino stream" (INA, 2004). From the hyetograph was determined a Total Precipitation equal to 76.41 mm. (See Appendix B2). This is a design hyetogram proposed by the methods of alternating blocks after the IDF curves. As such this rainfall does not represent any real past event.

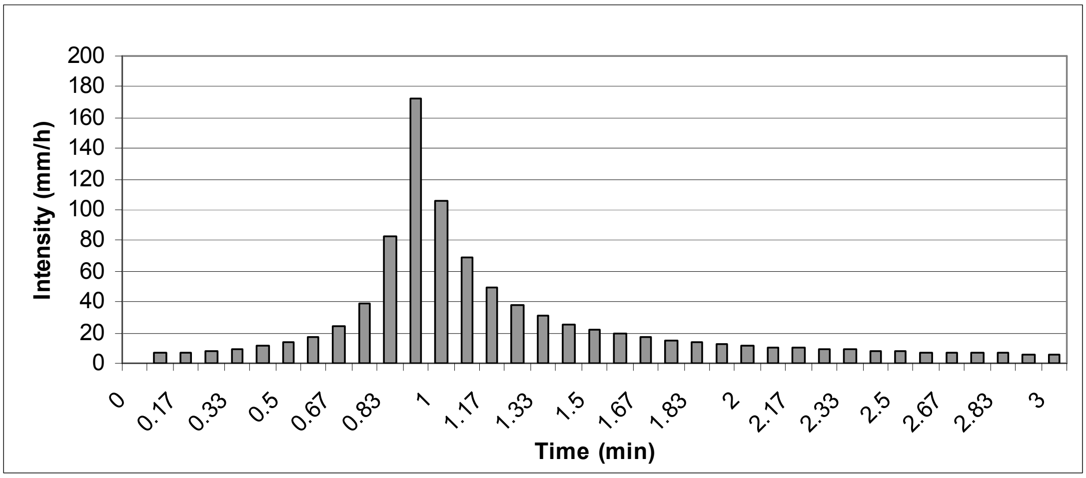

Figure 12 Hyetograph of 3 hours storm, recurrence period 10 years

Source: INA, 2004

f) Historical record of past events: in 2001 the Pergamino neighbors association (CO.S.S.O.PER.) made, on their own initiative, a quantitative record of the water level reached in previous rainfall events. They selected five heavy rainfall events; the closest at the moment of the evaluation, and after several meetings, the neighbors recalled each event and filled a form where they indicated the water levels reached in the streets closer to their houses. Although this form was sent to all the districts in the city, unfortunately not all the district reply it, however, the information collected was sent to INA, who compiled and presented the data in Autocad format. From that information, it was selected for this research the rainfall event occurred in February 9, 2001, with a total precipitation of 113.7 mm in three hours, because according to the study made by IATASA, that kind of value corresponds to 10 years return period (IATASA and ABS, 2008). This precipitation was used to simulate a past event in Flo-2D.

2.2.3. Remote sensing and other GIS data

a) QuickBird image of the study area: This image was taken from Google Earth Pro (2009), after that a georeferenced process was done in Ilwis in based of control points obtained from the Google Earth and the Topography sheet.
b) Cadastral map of the Pergamino city at block level (Source: Pergamino Municipality, 2007, Autocad format)
c) Streets and main routes in Pergamino city map (Source: Pergamino Municipality, 2009, Autocad format)

3. METHODOLOGY

In this section a description of the methods and data collected during fieldwork will be introduced. Hypothesis, calculations and restrictions that influence the model setting are discussed. The analysis of results is done in the next chapter.

3.1. Hec Ras model

The analysis of the flood characteristics in the Pergamino river, between Mitre and El Toro, using the 1D model Hec Ras was done in two parts. The first one is to the steady simulation of the channel in order to calibrate the current rating curve generated by INA. Then, a sensitivity analysis of roughness coefficients was also done. Finally, with the results of the simulation, potential sectors along the channel with overflow hazard were identified. Furthermore, in the second part, after the assessment and simulation of the data from the limnigraphs stations, an unsteady simulation was performed to determine the wave celerity.

3.1.1. River System schematic

The river system schematic was introduced to the Hec Ras model for the study area between FFCC Mitre to El Toro (Figure 6). As a first step in Hec Ras, a QuickBird image was imported to visualize the features and to support the introduction of the geometric data. Then, the river scheme was digitalized and the cross sections were introduced one by one in the model. Moreover, the location of the banks and levees were also introduced for every cross section and finally, a verification of the cross sections width and the location of the banks was done using as reference the image imported. As major improvement, the geometric characteristics of five bridges, were also introduced.

Figure 13 River system schematic

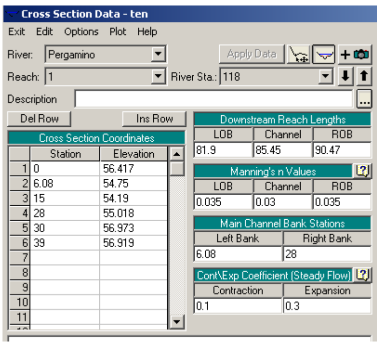

Figure 14 Input cross section data

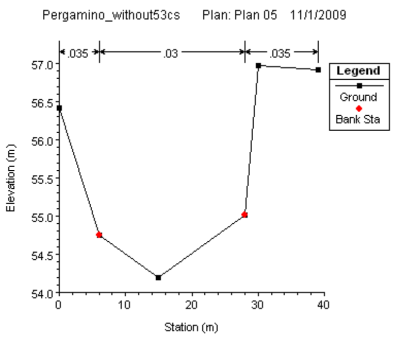

Figure 15 Cross Section schematic

3.1.2. Steady flow assessment

As was explained in the previous chapter, the current rating curve F. Sanchez was generated from field measurements where the maximum water depth value recorded was 2.5 m (water elevation 55.78 m) corresponding to a discharge of 78.41 m^3/s, this can be consider a limitation at the moment to assess extreme flow events in the urban area. Therefore, in order to improve that constrain, a steady flow assessment described below was done to obtain a calibrated rating.

To generate a set of adequate steady flow data, the current rating curve F. Sanchez was used as source. Thirteen water depths from 0.25 to 5.0 m were assumed as possible scenarios from low flow to overflow in the channel; those values were introduced to the rating curve to obtain the discharges values for the simulations (Table 2). The table shows the values of the discharge calculated from the available rating curve. Values above 70 m3/s were interpolations. HECRAS requires values of discharge in steady flow. From these values, the water height is calculated by the model.

Table 2 Flow data input for the Steady flow assessment

Scenario [2]name	0.25m	0.8 m	1.0 m	1.5m	2.0 m	2.5 m	3.0 m	3.5 m	4.0 m	4.5 m	4.6 m	4.8 m	5.0 m
Discharge (m^3/s)	0.65	6.99	11.34	27	49.48	78.78	114.89	157.83	207.58	264.16	276.29	301.38	327.55

The river hydraulics was simulated in steady flow considering the discharges values calculated in Table 2 as upstream boundary condition and the normal depth (0.0003845 m/m) as downstream boundary condition. The assessment of the results was made in the cross section 102 (closest to limnimeter F. Sanchez). The comparison between the results generated by the model and the values calculated directly from the rating curve equation, in terms of water depth brought in first instance the evaluation of the model performance. Moreover, based on the results obtained from the simulation, a new rating curve was formulated for the F. Sanchez gauge. Finally, the rating curve proposed was validated using the values obtained in fieldwork.

Furthermore, with the model calibrated a simple roughness sensitivity analysis was done. The previous simulation was performed using a roughness coefficient equal to 0.03, value suggested by the literature for natural stream channel, clean and straight (Brussel, 2008). Moreover, three more simulations were performed, for roughness in the channel changed to 0.025, 0.04 and 0.05. The last value was calculated from measures made during fieldwork (Table 3).

Based on Manning Equation

$$n = 1/Q * R^{2/3} * S^{1/2}$$ *Equation 6*

Where 'Q' is discharge in m^3/s, 'A' is area in m^2, 'P' is wet perimeter in m, 'S' slope in m/m and 'n' is the roughness coefficient.

[2] The high values presented in the table are just an indication of the scenario (profile) name in a Hec Ras run and they do not refer to the exact water level.

Table 3 Determination of the roughness coefficient in field

	XS 102	XS Middle	XS 101	Average
Area m2	9.02	10.09	9.32	9.17
Hydraulic Ratio m	0.54	0.62	0.87	0.70
Wet Perimeter m	16.73	16.23	16.25	16.49
Difference water slope between XS			0.02	m
Discharge center cross section			1.62	m3/s
Velocity center cross section			0.10	m/s
Lenght between XS102 and XS 101			83.50	m
Slope (S)			0.00019	m/m
High limnimeter			0.87	m
Roughness coefficient	0.05			

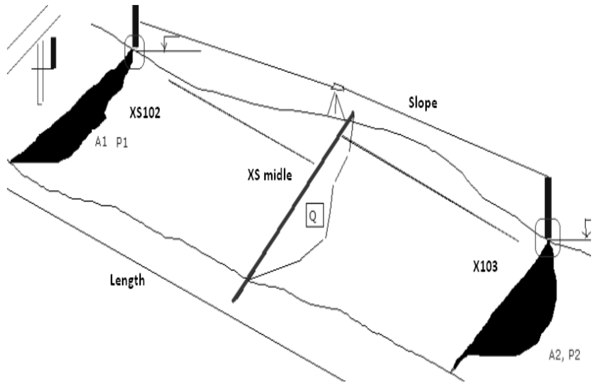

Figure 16 Scheme of roughness coefficient measured in field

The water depth simulated in every cross section shows the critical sectors along the channel where the flow may exceed the levees. It was considered that as soon as the water depth exceed the levee's height, that element is under flood hazard. Using new simulations for different discharges each cross section was evaluated and classified in based of the water depth value that generate a flood hazard situation. Similar evaluation was done for the bridges to determinate the water depth which can generate flood hazard situations. Finally the results were plotted in maps using Ilwis tools.

3.1.3. Unsteady flow assessment

The input data required by Hec Ras in unsteady flow simulation is a hydrograph. To generate that input the following hypothesis were adopted:

- It is possible to generate the hydrograph with the Alfonzo Station (upstream) data to use it as an input to run Hec Ras model, in unsteady flow condition.
- As the hydrograph is few kilometers upstream the first cross section with data, the assumption was that this hydrograph could be transported downstream till the beginning of this first cross section available in study area near Pergamino (FFCC[3] Mitre).

The hydrograph in Alfonzo did not have a rating curve. So in order to prove and eventually use the data for Alfonzo as mentioned above, this rating curve had to be built. A simulation in steady flow was done with the geometric data and random values of discharges in Alfonzo hydrometric Station area (Figure 17) as an attempt to reproduce the rating curve. The influence of the bridge located downstream was also considered for the simulation (the geometric characteristics of the bridge were taken during fieldwork) (Figure 18).

[3] FFCC means Railway in Spanish.

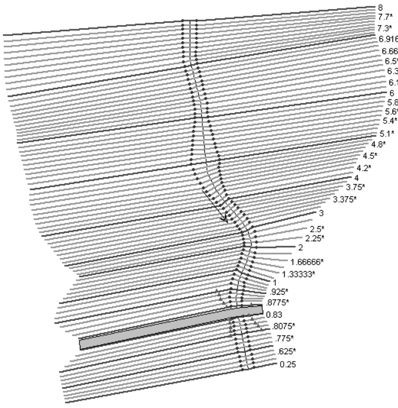

Figure 17 Geometry Data Alfonzo Station

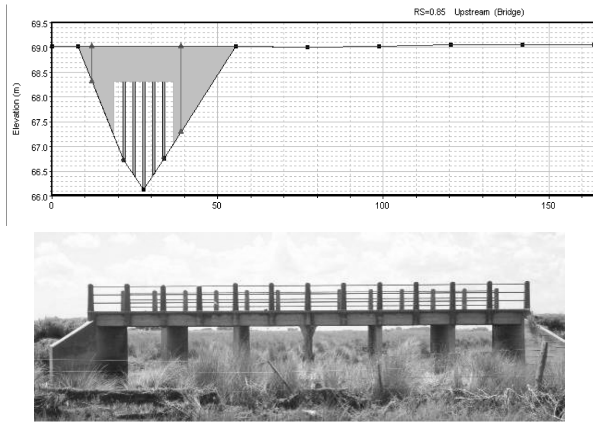

Figure 18 Bridge located downstream of Alfonso Station

However, during the evaluation in fieldwork, the area of Alfonzo station (and its measurements) was suspected to be under a strong backwater effect. The water level measured by the limnigraph was 67.36 m, much higher than the value simulated by the model 66.72 m when considering for the simulation the discharge measured in field (0.96 m^3/s) and normal condition as downstream boundary (See Appendix A1). The existence of backwater effect indicates that a rating curve cannot be obtained without linking the position of the station to places where the backwater effect generates. Because of the operational and time limitations, that option was not feasible, so the only alternative was to prove the existence of backwater. The previous hypothesis should be rejected if backwater effect exists and then the rating curve could not be built.

In order to prove its existence, in a new simulation, the boundary condition "Normal depth" was changed to "Water Surface Known". Values close to the water level recorded in field by the limnigraph were used as a boundary condition in the new simulation. The value of discharge measured in field was also introduced. The objective of this simulation was to demonstrate, that it was only possible to achieve that measured water level and discharge when a backwater effect is happening downstream. Entering this data the model reproduced exactly the heights and discharges measured at the field, what confirmed the backwater effect downstream of the station.

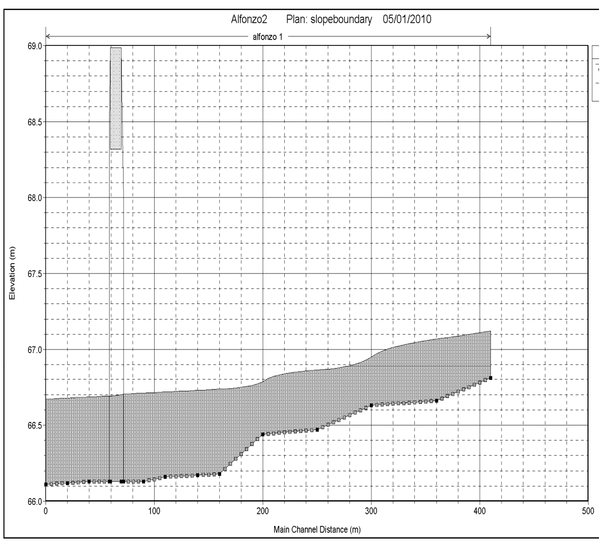

Figure 19 Simulation with Normal Depth boundary condition. This would reproduce the condition without back water

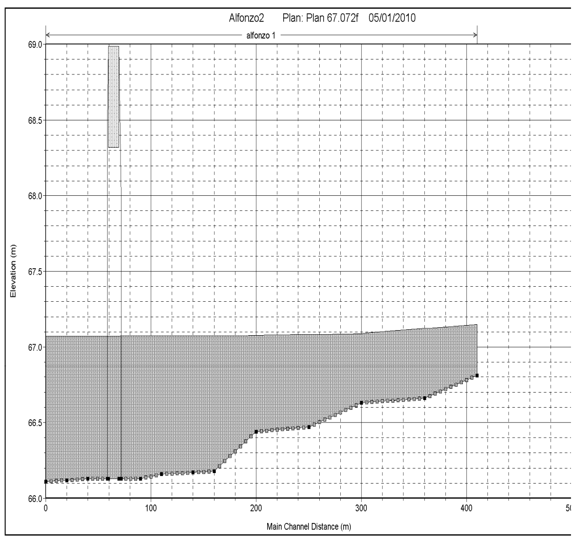

Figure 20 Simulation with W.S. Known boundary condition. This reproduces the effect with backwater. This was the real situation at Alfonzo.

This backwater effect is caused by the topographic characteristics downstream of the bridge, which makes possible an unusual rising of the water level upstream (Figure 20). Although HecRas can model this kind of situation, the scarcity of the cross sections data downstream the bridge made impossible to handle it for the period of this research.

Concluded the analysis, the hypothesis formulated to generated the hydrograph from the Alfonzo station was rejected. However, the geometric data for the study area (FFCC Mitre – El Toro) was validated in the steady flow simulation. Then, to achieve the wave celerity analysis in unsteady flow condition the hydrographs determined by INA (Figure 11) were introduced in the model. The simulations were done for each period of recurrence and the hydrographs results from the first and the last cross section were assessed.

In spite of the situation found during the analysis in Alfonzo Station, a simulation in steady flow was made also for the area of Urquiza Station to generate a rating curve. However, the scarcity in the number of cross section used (only three) did not allow to continue with the assessment. Obtaining more section was limited by accessibility and high risk during fieldwork.

3.2. Flo- 2D model

The flood hazard assessment with Flo 2D was only focused in Pergamino urban area. The analysis is presented in two sections. The first section relates effort in data input as it was the most time consuming task of the model, and the one proven more sensitive: the generation of the digital terrain model, grid size assessment and roughness coefficient. The second part describes the flood run performed, the problems faced during the process and the improvements made until achieve the final simulation.

3.2.1. Input data description

a) Digital terrain model generation

The most influential input to simulate a flood event using a 2-D model is without any doubt the DTM. The time spent in the design of an FLO-2D-compatible DTM will certainly save hundreds of hours of erroneous calculations. This is one of the main conclusions of this research in terms of data input.
In some developing areas the possibility to work with accurate DTM's from LIDAR data is low as it is very costly. There is one company in Argentina able to flight LIDAR, but the airplanes and equipment need to be transported from Brazil, and the cost can only be covered in well funded projects. In most cases, the sources to generate a DTM must come from topography surveys. In any case, the quality of that data will (extremely) condition the results. In the Pergamino case, the DTM was generated from the topography survey made in 2007, as it was considered the most up-to-date and sensible information. (See subtitle 2.2.1). However the goal of that survey was not to develop a model for Flood Hazard, and soon this became clear after the first analysis. The development of a DTM-for-flood model is a methodology by itself that requires the survey of structures that affect water flow rather than a standard striped-pattern survey for cadastral or road planning.

The first approach was developing a DTM from contours with an interval of 0.5 m provided by the Pergamino Municipality. After importing from Autocad format, the first DTM was generated using the spatial interpolation tool in Ilwis. However, some erroneous features were generated and found in the results. An unreal embankment shape off the river channel was created as the contours delineation confused the upper cord (pavement) of the bridges as part of the natural terrain. To improve that, the only solution possible was to neglect those problematic contours.

The following procedure was selected for the river area:

1. Digitalization of the river stream using as a base the QuickBird image (georeferenced high resolution Google Earth image (dry period))
2. Measurement of the distance from the main river axis to every elevation point in the cross sections available after the field survey (Figure 21).
3. Those points were plotted in the available contours map (Figure 22). Then, the new points were used to produce an interpolation and, as a result, new contours were built (Figure 23).
4. With the contour map improved, a new DTM was generated using the spatial interpolation tool in Ilwis.
5. The optimized DTM process was executed in Ilwis using the hydroprocessing tools available. The optimization process requires the values of buffer distance, smooth drop and sharp drop, to introduce those values as best representation of the river, this was considered in four segments (Figure 24).

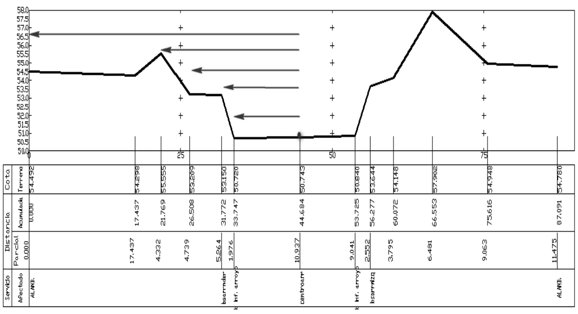

Figure 21 Distance and Elevation measurement in the Cross Sections

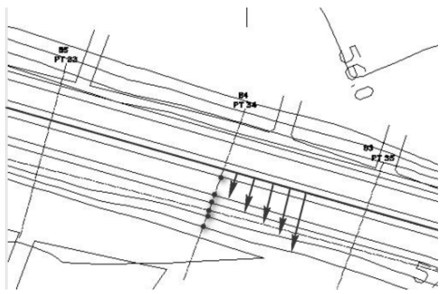

Figure 22 Location of the new points and new contours lines interpolated

Figure 23 Contour map modified

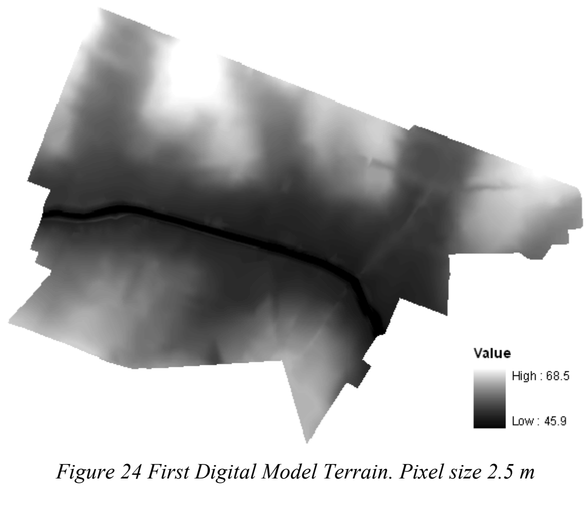

Figure 24 First Digital Model Terrain. Pixel size 2.5 m

During the first FLO-2D simulations some problems of model instability were faced (See 3.2.2). To overcome that, the review of the DTM was needed, a deeper analysis of the contours map showed more mistakes (Figure 25), as the contours did not represent properly the natural terrain. It was concluded that the DTM did not have the properties or quality for 2D modeling of the Pergamino city. The alternative source was the original elevation points measured during the topography survey, that in turn were used to build the previous DTM. To avoid error of redundancy, the points were reviewed carefully and those that could not represent the real terrain (e.g. elevation bridges) were not considered in the new set. (Figure 27)

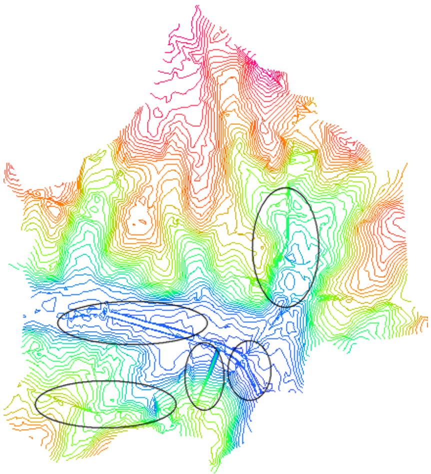

Figure 25 Some mistakes found in the contours map interval 0.5 m

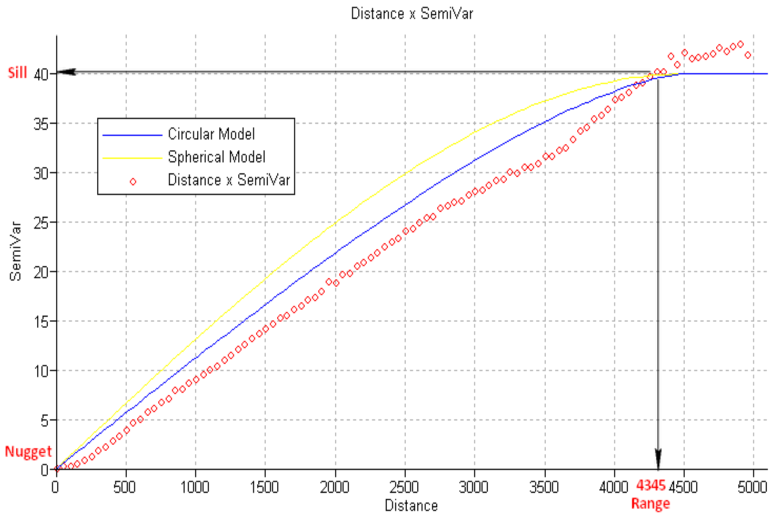

Figure 26 Parameters used to Kriging Interpolation

With the new data set of points, the Kriging method was chosen for the interpolation in Ilwis as an error map was part of the process. The spatial correlation analysis was used to evaluate the values of Sill, Nugget and Range. The circular model was the best fit curve to adjust the relation between the values and the distance between the points (Figure 26). The interpolation was done; as a result a new DTM was generated and the specific area corresponding to the river was selected to perform new simulations (Figure 28).

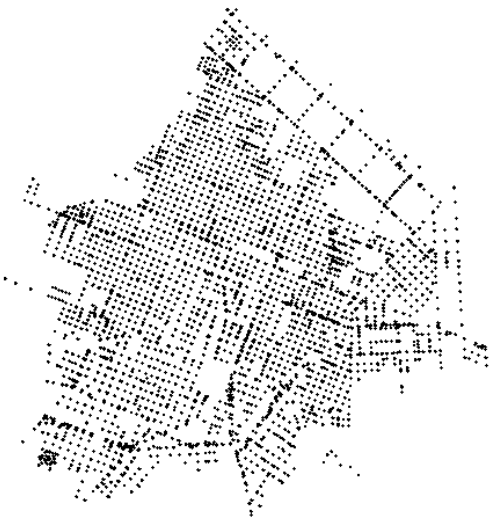

Figure 27 Points set modified

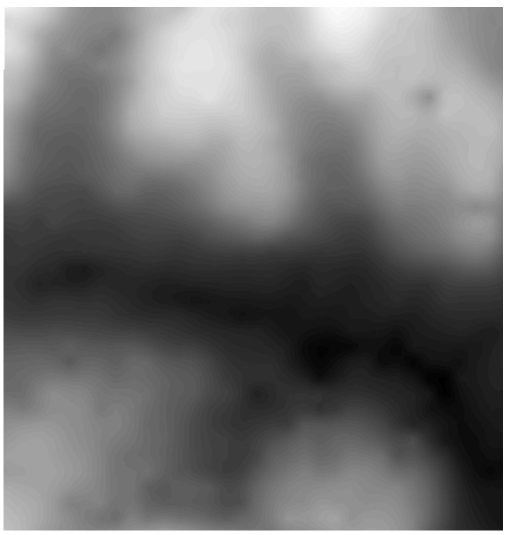

Figure 28 Final Digital Terrain Model Pixel size 2.5 m

Finally, as the standard source of topographic information in developing countries is the contours of the toposheets, a third Digital Terrain Model was generated in Ilwis from the digitalized contours (Figure 29) and (Figure 30). The comparison between Figure 28 and Figure 30 clearly shows the unavoidable error of the different DTM's generated from different sources.

Figure 29 Topography sheet. Contours interval 2.5 m

Figure 30 Digital Terrain Model from topography sheet source Pixel size 2.5 m

b) Grid Size and Boundary conditions for the flood routing simulation

When the DTM is introduced as input data in the model, the grid size must be simultaneously defined. Every next step is handled by the model through each grid, after the grid size is defined. An interpolation process is done by the model in order to assign an elevation value to every grid element or cell, understanding that the grid size is coarser than the pixel size. The following relation (O'Brien *et al.*, 2009) was carefully considered at the moment of selecting the grid size:

$$0.03\frac{m^3}{s-m^2} < \frac{Q}{A} < 0.3\frac{m^3}{s-m^2}$$

Equation 7

Table 4 Grid size selection

Q m^3/s	Grid size m	Grid Area m^2	Q/A
317	20	400	0.79
317	35	1225	0.26
317	50	2500	0.13

Where Q is the discharge expressed in m3/ s and A is the grid area expressed in m^2.

Considering the peak discharge equal to 317 m^3/s from the hydrograph of 10 years recurrence period (Figure 11), according to the Equation 7 the minimum grid size value should be 35 m to guarantee a properly computational time. The first simulations were ran with that value; however, that grid size was inadequate when considering the river width. Due to the river channel performance plays an important role during the flood simulation, a 20 m grid size was selected as a final value for the rest of the simulations.

A QuickBird image (from Google Earth) was used to visualize and to introduce the main elements (e.g. channel River, upstream node, etc). The hydrograph for 10 years of recurrence was input as a boundary condition in one node upstream of the channel river (Figure 31). All the grid-cells located in the downstream boundary were considered as outflow plain, which means that the flow is allowed to pass out of the system through those grids (Figure 32). This is standard procedure in Flo-2D (O'Brien *et al.*, 2009).

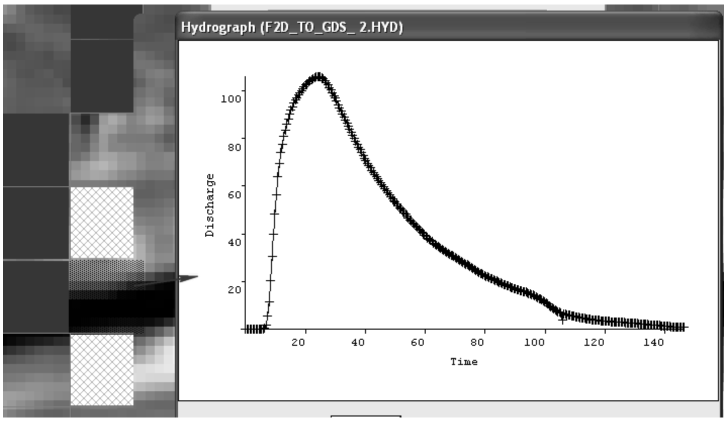

Figure 31 Inflow input, upstream condition

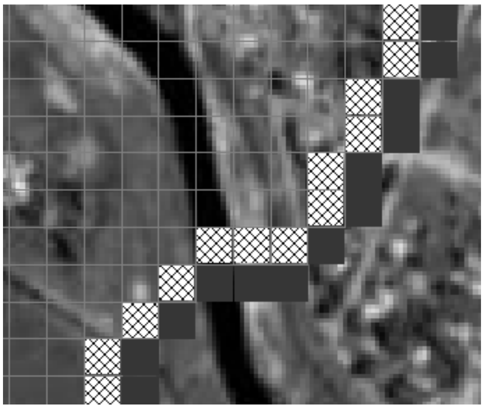

Figure 32 Outflow input, downstream condition

c) River channel

The river channel in the model is defined by the cross section data. First, the channel was digitalized in the model using as a base the city image that was imported. During the digitalization each point, which represented the channel, matched with the center of each cell, however, as the cell does not contain perfectly the river, some reaches are not properly represented. The river cross sections from the urban area were imported from the HEC RAS model using the proper FLO-2D options, then each cross section was located and linked with its respective grid taken the left bank as a reference. After that, the model established the localization of the right banks based on the cross section width. Finally, as every grid-cell of the channel must have a defined cross section; an interpolation process available in the model was selected. (Figure 33) (Figure 34)

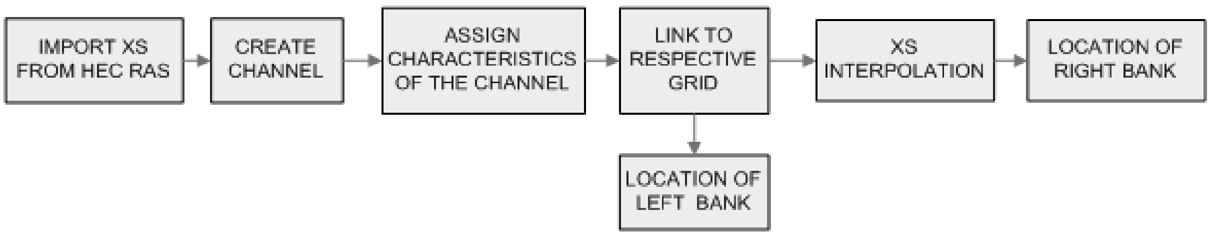

Figure 33 Scheme of channel input

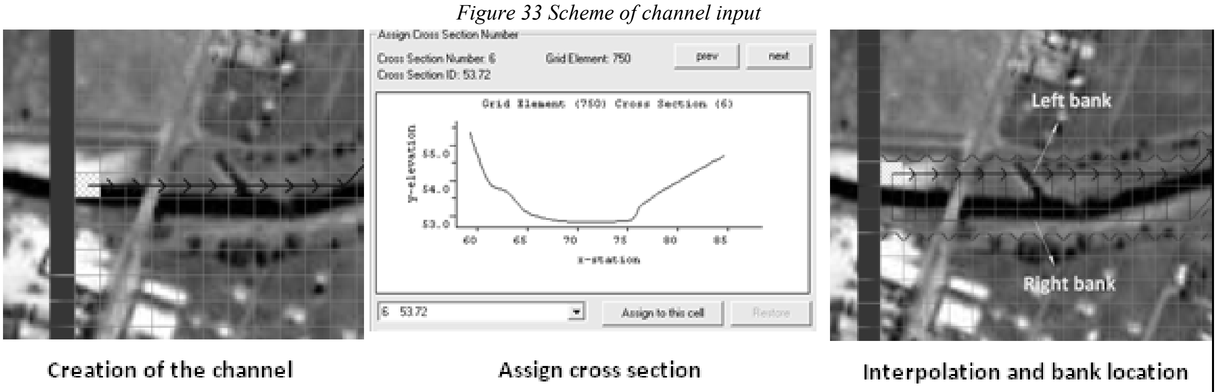

Figure 34 Steps in the process of the channel input

c) Roughness coefficient

A roughness coefficient map was imported to the model with the values from the literature for the channel, green areas (e.g. grass cover: parks) and for the rest of the floodplain. The model assigns those values to each grid-cell. It is possible to modify those values in the grid-cell interactively. The Table 5 shows the base values considered for the simulations (Table 5).

Table 5 Roughness coefficient

	Roughness coefficient
Floodplain *	0.035
Grass cover *	0.200
Channel**	0.040

** Guide Manual Flo 2d*

*** Hec Ras Reference Manual*

The roughness coefficient for the channel 0.04 was selected as a reference by simply average between the value obtained in one point measured in fieldwork 0.05 and the value used during Hec Ras simulation 0.03.

c) Rainfall

The rainfall data file with the precipitation per interval entered as a percentage from the total rainfall (76.41 mm) was generated in base of a three hours duration storm corresponding to a 10 years recurrence period (INA, 2007). The storm was introduced at the same time to the peak of the hydrograph in order to simulate the worst scenario, using an alternative block diagram.

3.2.2. Trial simulations

Three scenarios were considered for the simulation of the flood events in Pergamino city:

1. Scenario A considers the flood event caused only by the rainfall.
2. Scenario B considers the flood event caused only for the overflow in the channel.
3. Scenario C simulated the combination of the two previous cases.

The first part of the assessment described below was considering the scenario B), later on, taking in count the rainfall data, the scenario C) is evaluated.

Several simulations using the DTM generated from the elevations points (Figure 28) were performed as trials to improve the model performance. During the simulations two variables were under surveillance: the computation speed (established by the reduction of the time step to compel with stability) and the conservation of mass. The first goal was to achieve a reasonable simulation time without volume conservation problems; which is the first objective in model stability.

One of the important problems that 2D models have is the running time as the time step automatically decreases when some stability criteria is not fulfilled (See subtitle 1.4.5). The learning curve required to work with FLO-2D implies that the undesirable errors that reflects in calculation time can easily be in order of days for the novice user. Several runs were attempted with different upstream boundary conditions loads. The hydrograph for 10 years return period was distributed in two and three hydrographs, each of them were introduced at the same time in different cells upstream. However, the results, in terms of improving the speed, were not satisfactory and introducing the hydrograph only in one node proved better[4].

Furthermore, as the model ran only with the channel represented by the DTM, the river channel section data was introduced (See subtitle 3.2.1, c). However, running with this new characteristic did not solve the speed problem and a volume conservation problem appeared.

The model performs by default with a stability coefficient (Wavemax) equal to 1.0 (For further explanation see Subtitle 1.4.5). However, some changes in that value can modify significantly the simulation time. A Wavemax coefficient equal to -0.25 allows the model to adjust the input roughness coefficients but the timestep is not decreased, which means, the model does not take long time for the simulation. New simulations were made with that value; the volume conservation problem persisted, but the simulation time was reduced. The results allow focusing in the volume conservation problem. Typically a volume conservation error greater 0.001 percent is an indication that the model could be improved (O'Brien *et al.*, 2009), the results showed a volume conservation error over 1 % during the simulations. The value increased enormously when it reached the peak of the hydrograph and the simulation time was very long as the time step changed automatically to adjust the stability.

The model performance forced to reconsider the analysis of the main input data: Hydrograph and DTM. An external DTM from different area (San Pedro, Guatemala) was used to evaluate the performance of the input hydrograph. The simulation was shorter in time and the conservation volume error lower that 0.001%. The conclusion was that the upstream boundary condition was correct. As a consequence, the assessment was focused again in the DTM (See subtitle 3.2.1,a), new simulations were perform with the three different DTMs (Figure 24,Figure 28 and Figure 30), in spite of the improvements in the DTM the volume conservation problem error continued. (The analysis continued considering the DTM generated from the elevation points (Figure 28)

As most of the volume conservation problems are because of input channel errors (O'Brien *et al.*, 2009), every cross sections was reviewed from the main file and abrupt changes in shape and slope were avoided. The improvement in the channel showed reasonable volume conservation in the channel. As a conclusion is important to notice that bed pools and back-slopes are not handled by FLO-2D and a process similar to “FILL-SINKS” in 2D hydroprocessing is required in the channel. As the overall conservation problem remained, the conclusion was that the problem was in the floodplain. Furthermore, in order to detect the source of the volume conservation problem, a simulation without taking into account the storm (scenario B) was done. The performance without conservation problem concluded that the rainfall data file needed to be reviewed. A simulation considering the scenario A confirmed that conclusion. Then, a new simulation using an improve rainfall data, and a Wavemax coefficient equal to -0.250 showed a satisfactory model performance with a volume conservation error lower that 0.001%.

[4] During the modeling period, tenths of emails were exchanged with the helpdesk FLO-2D who assisted with suggestions. This exchanged implied also time consuming from the research.

As the volume conservation problem was overcame for that Wavemax coefficient, the next step was to calibrate the roughness coefficient adjusted by the model. This was done in a series of simulations. The scheme presents in Figure 35 shows the steps followed in the next simulations.

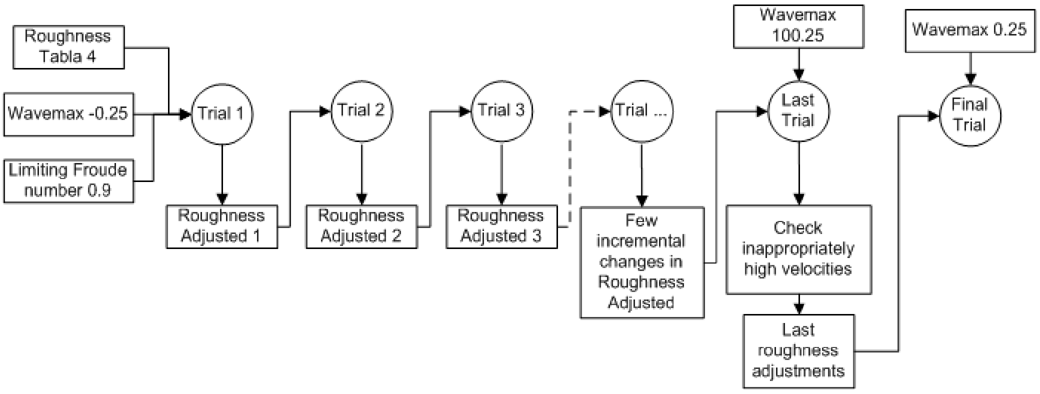

Figure 35 Simulation process

Considering as Wavemax[5] -0.250 and as limiting Froude number[6] 0.9, for the floodplain and the channel, the first trial was done. During the simulation, the model adjusted the roughness coefficient in those cells where the limiting Froude number was exceeded; as a result, a new set of roughness coefficients were generated as outputs. Then the maximum velocities happening in few unstable grid-cells were carefully reviewed, the grids with unusual high velocity values were identified, in those cases, the roughness values for the vicinity grids of those nodes were increased, and the rest of the adjusted roughness values were accepted. In the next trial the roughness set improved was used as new input and the steps described above were repeated in several trials until only few incremental roughness values appeared in the results. A last trial was performed with a Wavemax 100.25 to speed up the model and to check in the results any possible high velocity (O'Brien *et al.*, 2009). Finally, with the roughness coefficients calibrated and considering Wavemax[7] 0.250, the Final trial was done. In all these scenarios the conservation mass was observed within the recommended values.

The sequence described above was repeated for the three scenarios. For the case of the scenario A, after the final trial, a new simulation was performed considering the rainfall event similar to the one occurred in Pergamino city on February 9, 2001, in this case, the value of total precipitation registered by INTA station 113.7 mm (IATASA,2007) was introduced to the model considering the same hyetograph shape used in the previous simulations (Return period 10 years). As it was the only source of comparison available, the results of the model, in terms of maximum flow depth were compared with the areas where the neighbours gave values after the event (See subtitle 2.2.2. f).

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[5] Assign WAVEMAX=-0.250 increase the roughness values for those elements that are numerically unstable resulting in unreasonable velocities. (O'Brien,2009)
[6] Assign Limiting Froude number has the purpose to make sure that the computed velocities are reasonable. (O'Brien,2009) The simulation made with HEC RAS gave as a result Froude number lower than 1 (subcritical flow), in based of that a limiting value of 0.9 was selected for the case of Flo-2D simulations.
[7] Wavemax = 0.250 Dynamic wave stability criteria increments and decrements the computational timestep when Wavemax is exceeded. Model runs more slowly but is stable. (O'Brien,2009)

d) Summary of the simulations

During the flood assessment using Flo-2D model, as it is explained above, several simulations (trials) were performed. Different alternatives of inputs for the model were tested and experience gained. The Table 6 presents only the final runs for which their results will be part of the assessment in the corresponding analysis chapter (See Chapter 4).

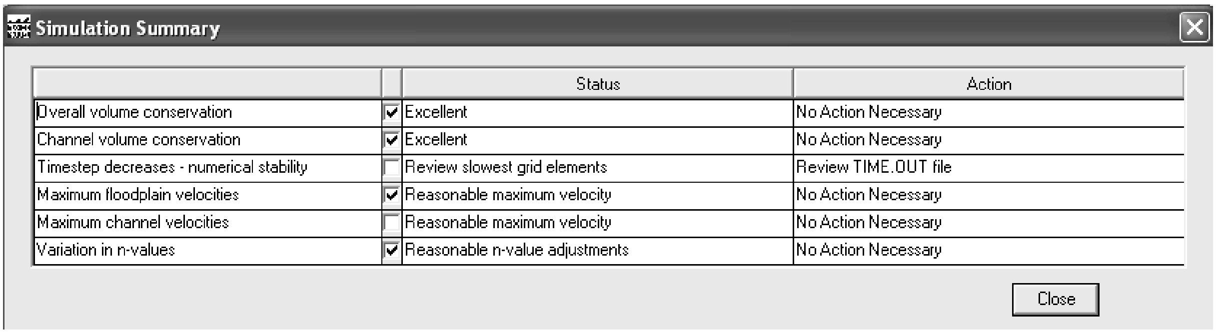
Table 6 Summary simulations

Scenario	**Trial name**	**Scenario Description**
A)	Final_rain	3 hours Storm, Recurrence period of 10 years
	Final_Cossoper	Rainfall event February 9, 2001, Recurrence period of 10 years
B)	Final_flow	Flow in the channel
C)	Final_both	3 hours Storm and Flow in the channel at the same time.
General characteristics		Grid size: 20 m; hydrograph input: one node; Return period Rainfall and Flow: 10 years; DTM: generated from the point map

3.2.3. Model performance

Flo2D allows following the performance process during the simulation time, the user can select the option to follow the simulation by graph display or by text screen. In both cases the conservation volume error is presented for each output interval chose. Besides that, when the graph display is selected, it is possible to visualize the simulation time, the process along the hydrograph and the depth for that simulation time. The graph display option was selected during the simulations of the scenario B and C; in the case of scenario A, as there is not hydrograph as data, the model only displays the process through the text screen.

When the model finishes the simulation, a result simulation summary shows the overall performance on volume conservation, channel volume conservation, numerical stability, maximum velocities (floodplain and channel) and variation in n-values. This screen gives a first perception of the performance and it was used as guide to review the output files.


Simulation Summary

		Status	Action
Overall volume conservation	☑	Excellent	No Action Necessary
Channel volume conservation	☑	Excellent	No Action Necessary
Timestep decreases - numerical stability	☐	Review slowest grid elements	Review TIME.OUT file
Maximum floodplain velocities	☑	Reasonable maximum velocity	No Action Necessary
Maximum channel velocities	☐	Reasonable maximum velocity	No Action Necessary
Variation in n-values	☑	Reasonable n-value adjustments	No Action Necessary

Close

Figure 36 Example of the summary table reported by the Model after finishing the simulation

Flo-2D model gives the output in several files, which can be read in *.txt format, at the same time, the results can be visualized using the post-processor program MAPPER (that is incorporated into Flo-2D model) which brings the map results of inundation area, maximum flow depth, maximum flow velocity and others, these maps are in shape format, so, further analyzes can be done using GIS programs (e.g. Arc Gis or Ilwis).

To further analyze, the maps results were exported to Ilwis, then using the cross map option, every map result from the three scenarios (inundation area, maximum flow depth and maximum flow velocity) was related with the cadastral map and the street map in order to identify the areas affected by the flood.

3.3. Flood hazard assessment

Flo-2D has incorporated inside the post-processing program MAPPER, the routine to generate hazard maps from the results obtained in the simulation. The methodology for this delineation is based on Swiss Standards (O'Brien *et al.*, 2009), based on the intensity of the event, which is a function of the flow depth and the maximum flow velocity. The model establishes three zones to identify the potential hazard (This methodology was proposed in the PREVENE project[8]). The potential flood hazard is then defined by the model as a discrete combined function of the event intensity (severity of the event) and return period (frequency) (Garcia *et al.*, 1999). According to the Swiss method, the intensities are defined in terms of the maximum water depth generated throughout the event and the product of the maximum velocity multiplied by the maximum depth (Table 7).

Table 7 Flood hazard level used by Flo-2D model

Water flood event intensity	Maximum depth h (m)		Product of maximum depth h times maximum velocity v (m^2/s)
High	$h > 1.5$ m	OR	$v\,h > 1.5\ m^2/s$
Medium	0.5 m $< h < 1.5$ m	OR	$0.5\ m^2/s < v\,h < 1.5\ m^2/s$
Low	$h < 0.5$ m	AND	$v\,h < 0.5\ m^2/s$

The user can select the option to introduce the results from three return periods or to work with the current data from the last simulation. As this project was focused on the assessment in the return period of 10 years, the last option was selected. The final simulation for each scenario were post-processed in MAPPER, then, the potential flood hazard maps were generated.

Furthermore, the results for each scenario were also evaluated base on the hazard levels established by FEMA[9] . Three zones are classified according to the relation between water depth and water velocity: Low danger zone, Judgment zone and High danger zone (Figure 37).The two equations that govern the zones were calculated by an optimized interpolation procedure (See Appendix B9) followed by a process in Ilwis to build the hazard. Finally those maps were related to the cadastral map in order to identify areas affected.

[8] PREVENE = Contribution of the prevention of Natural disasters in Venezuela. Project developed as part of the Swiss cooperation to Venezuela (South America) during the period of 2000 to 2001.
[9] FEMA= Federal Emergency Management Agency part of the U.S. Department of Homeland Security (USA).

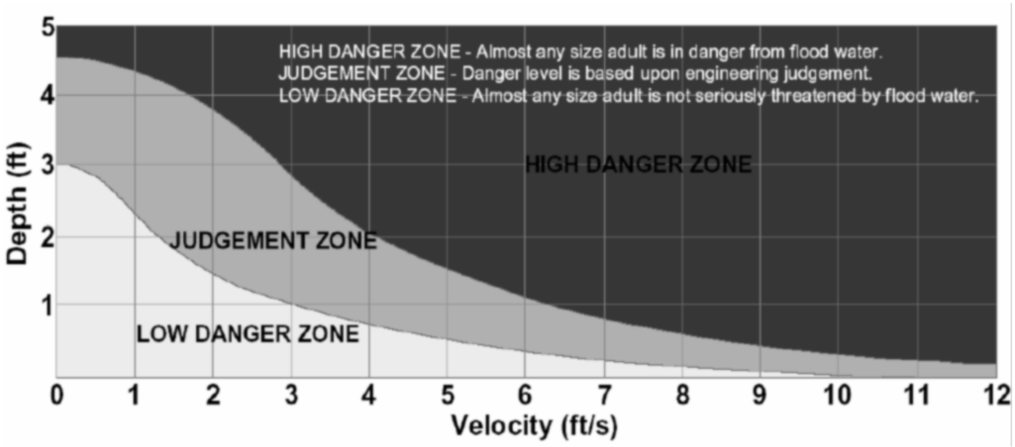


Figure 37 Flood hazard zones
Source: FEMA, 2009

4. RESULTS AND DISCUSSION

This chapter summarizes the results found of the scenarios and attempts explained in the previous chapter.

4.1. Hec Ras performance

Steady flow assessment

4.1.1. Model calibration

The results from the first simulations before fieldwork showed that the geometric data obtained from third party survey provides a good representation of the real river situation.

The final simulation allowed a comparison between water depths simulated and estimated from the current rating curve respectively (Figure 38). This comparison was made in the cross section closest to the limnigraph F. Sanchez Bridge (number 102) (Table 8)

Table 8 Results from simulation in Steady flow conditions F. Sanchez gauge

(1)	(2)	(3)	(4)
	Water Surface	Water depth	
Discharge	Hec Ras	Hec Ras	F. Sanchez
m3/s	m	m	m
0.65	53.48	0.73	0.78
6.99	54.05	1.30	1.33
11.34	54.26	1.51	1.53
27.00	54.78	2.03	2.03
49.48	55.33	2.58	2.53
78.78	55.85	3.10	3.03
114.89	56.34	3.59	3.53
157.83	56.89	4.14	4.03
207.58	57.55	4.80	4.53
264.16	58.46	5.71	5.03
276.29	58.68	5.93	5.13
301.38	58.79	6.04	5.33
327.55	58.88	6.13	5.53

Where:

(1) Discharge introduced as input in the model.
(2) Results of the Hec Ras simulation
(3) Water depth values calculated from the results of the model.
(4) Water depth values calculated with the rating curve F. Sanchez. Note the current rating curve was obtained with a maximum measured value of 78.4 m^3/s. Other values are extrapolation from the original equation

(To see an example of the calculation steps refers to Appendix B2)

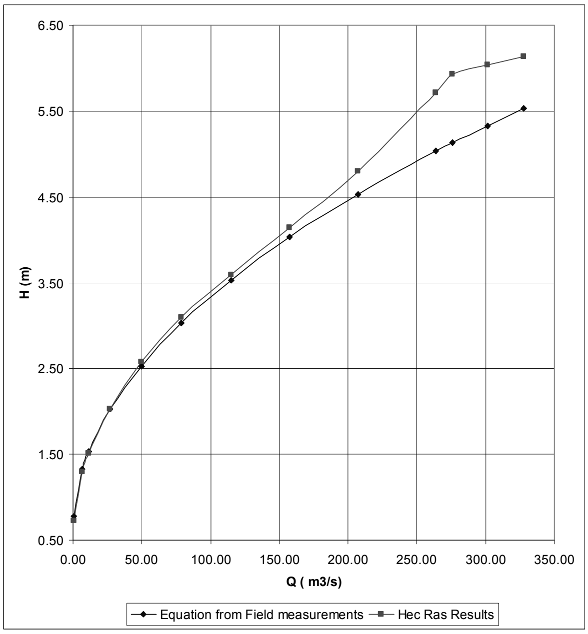


Figure 38 Comparison between water depths generated by Hec Ras model and values determined by the rating curve F. Sanchez.

Hec Ras gave as result of the simulation Water Surface values (column 2, Table 8). The difference between those values and the lowest elevation point in the river bed cross section assessed (corresponding to F. Sanchez Bridge) gave the water depth (column 2) to compare with the values calculated directly from the rating curve (Equation 5). The deviation between the two curves is for the effect of the bridge structure as water approaches the upper cord Figure 38 This effect is not considered by the original rating curve, as data for the rating curve were obtained for values well below from those reaching the bridge crone.

The performance of the model was following the original rating curve till water depths lower than 4.0 m. (Figure 38). Water depths higher than that value showed an increase in the difference. The Florencio Sanchez Bridge was simulated with 5.0 m of height (elevation 57.75 m). As soon as the water level past that value, that bridge begins to work under pressure conditions, as a result the hydraulic conditions change downstream. Because of that, the calibration of the model fits satisfactorily the measured ranting curve only until the threshold 4.0 m. Therefore, from the results assessed the best curve adjusted for that range is presented below for F. Sanchez limnimeter:

$$Q = 12.5 * H^2 - 14.6 * H + 4.8 \qquad \textit{Equation 8}$$

Where ‘Q’ is the discharge in m^3/s and ‘H’ is water depth in m.
Note in the Equation 8 the discharge value is expressed as a function of the water depth, because this parameter measures direct from the limnimeter. However, to expresses the results of the curve in Water levels all the values must be refer to 52.75 m as the lowest point elevation of the cross section. (e.g. Water depth of 1.0 m is equal to water level 53.75 m)

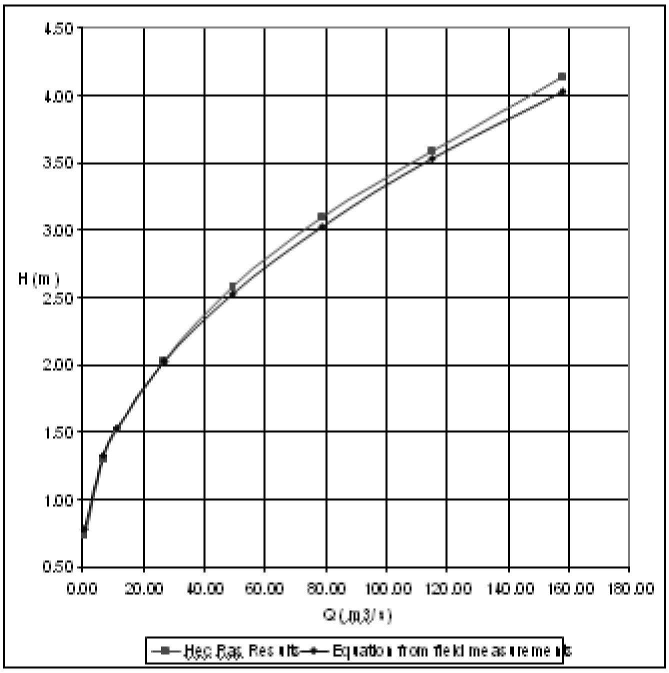


Figure 39 Rating curve adjusted from the calibrated model

Although the improved rating curve generated needs more than one point value to be validated, the comparison between the only discharge calculated from measurement during fieldwork and the value obtained using curve adjusted (Equation 8) shows a good agreement:

Water depth measured in the limnimeter= 0.87 m
Field discharge = 1.62 m^3/s
(See Appendix A1)

Calculated discharge = 1.52 m^3/s

The presented rating curve is a better estimation of the values for depth water compared to a mere extrapolation of the measured rating curve. Furthermore, the use of Hec Ras allows seeing the effect of the bridge in the flow, condition that happened in the past but it was not measured.

4.1.2. Roughness coefficient

In order to assess the model performance as a function of the roughness coefficient used, the results in terms of water depth were evaluated in the cross section located near to F. Sanchez Bridge for the fourth simulations performed (Figure 40).

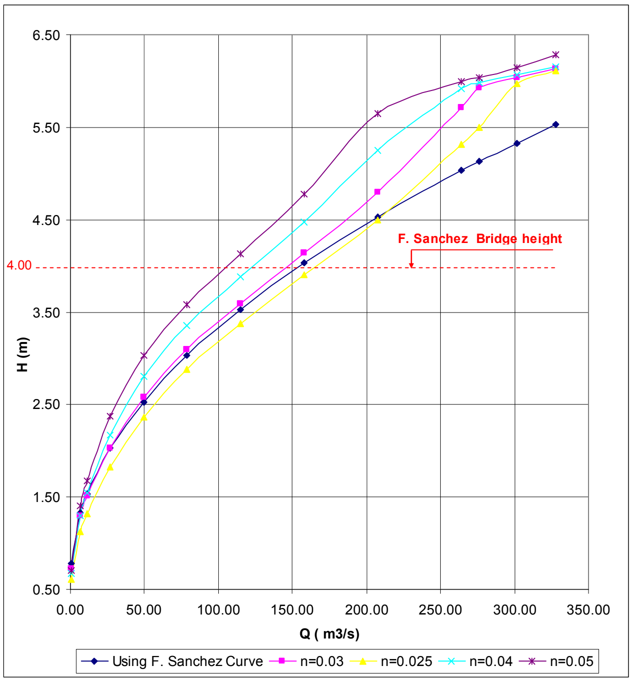


Figure 40 Roughness coefficient sensitivity analysis

The roughness coefficient suggested by the literature for the kind of riverbed (0.03) gave the best results to the water depths matching the F. Sanchez rating curve. The value determined in the field 0.05 (one point measurement, using the slope-area method) did not allow good performance of the model, the difference is bigger. The roughness coefficient is a parameter with an important sensitivity for the Hec Ras performance, the incorrect use of it can generate unrealistic results. Therefore, for the case of Hec Ras model, the simulation with the coefficient 0.03 for the river bed was accepted as the best one and used in subsequent analysis for long term analysis, understanding that the roughness condition of the channels changes with time. The value measured in field will be considered during the assessment with Flo-2D model.

4.1.3. Levees exceeded hazard

As it was explained in chapter 3, the flood hazard was assessed for water levels exceeding the levee height. The lowest water depth simulated before the water pass over the dike was identified in each cross section[10]. As a result, flood hazard maps considering the overflow in the channel were generated (Figure 41, Figure 42, Figure 43 and Figure 44). These maps can be considered as useful information to prevent and mitigate possible failures in the levees along the channel.

Considering the river bottom as reference, lower depth values, between 2.0 m and 3.0 m generated critical situations in the sectors located upstream and downstream of the city (Figure 41) where the levee is less maintained. This is also supported because in that sector the channel shape is natural (with minor or without modifications). In the range upper than 3.0 m the hazard increases downstream (Figure 42). However, it is only since the water depth achieves values above 4.0 m that the hazard appears in the urban area. (Figure 43)

[10] A database with the information in terms of cross section geographic location, street affect, water level which causes the flood hazard and dike side affected was generated. (See Appendix B3)

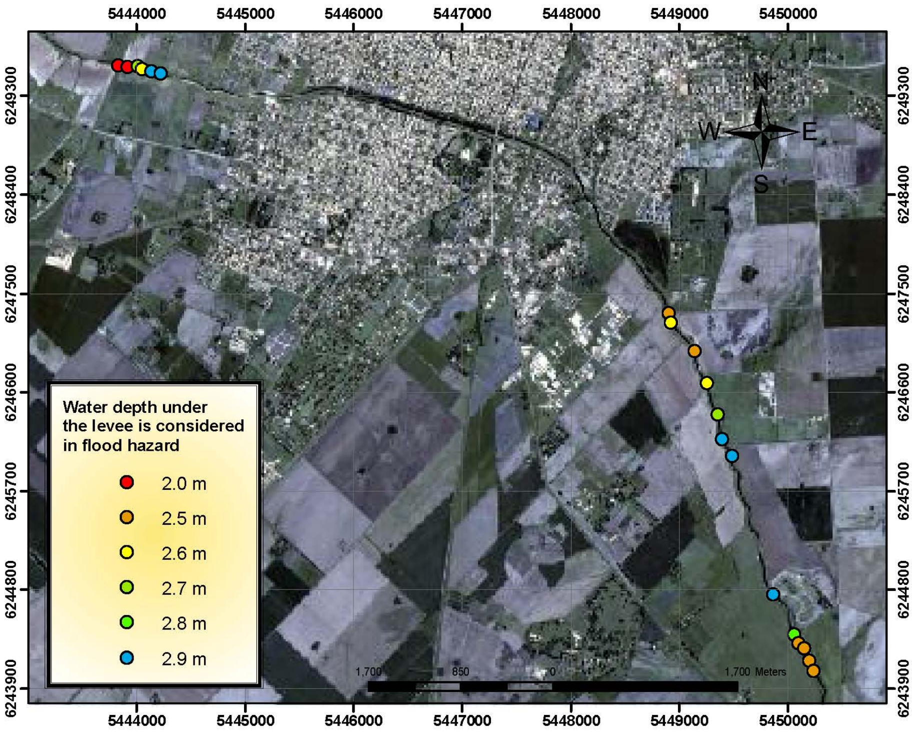


Figure 41 Cross sections under overflow hazard for water depths in the ranges from 2.0 m to 2.9 m

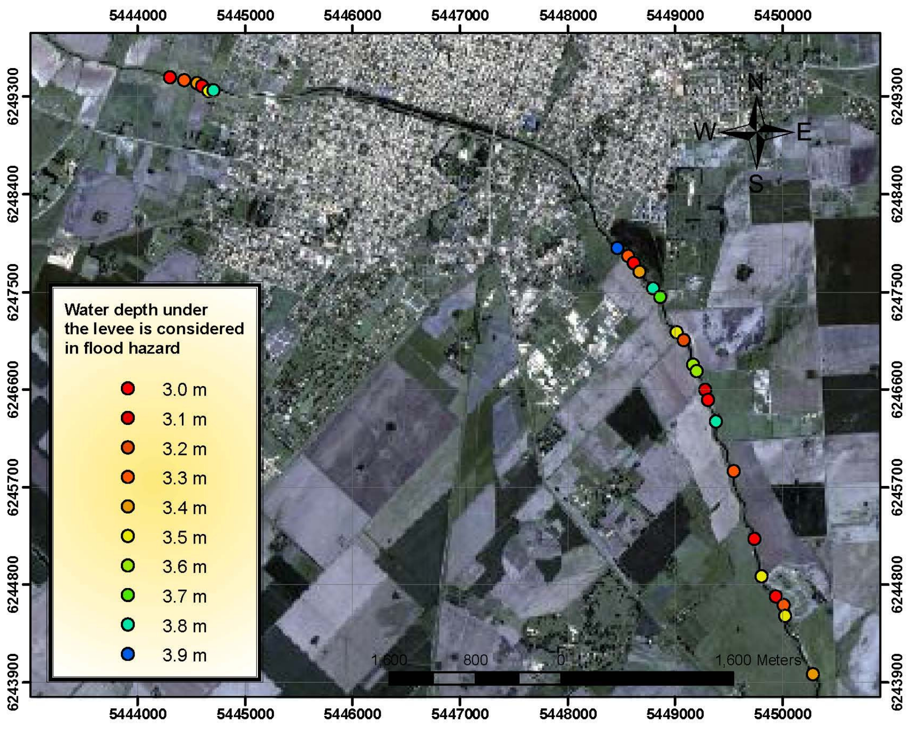


Figure 42 Cross sections under overflow hazard for water depths in the ranges from 3.0 m to 3.9 m

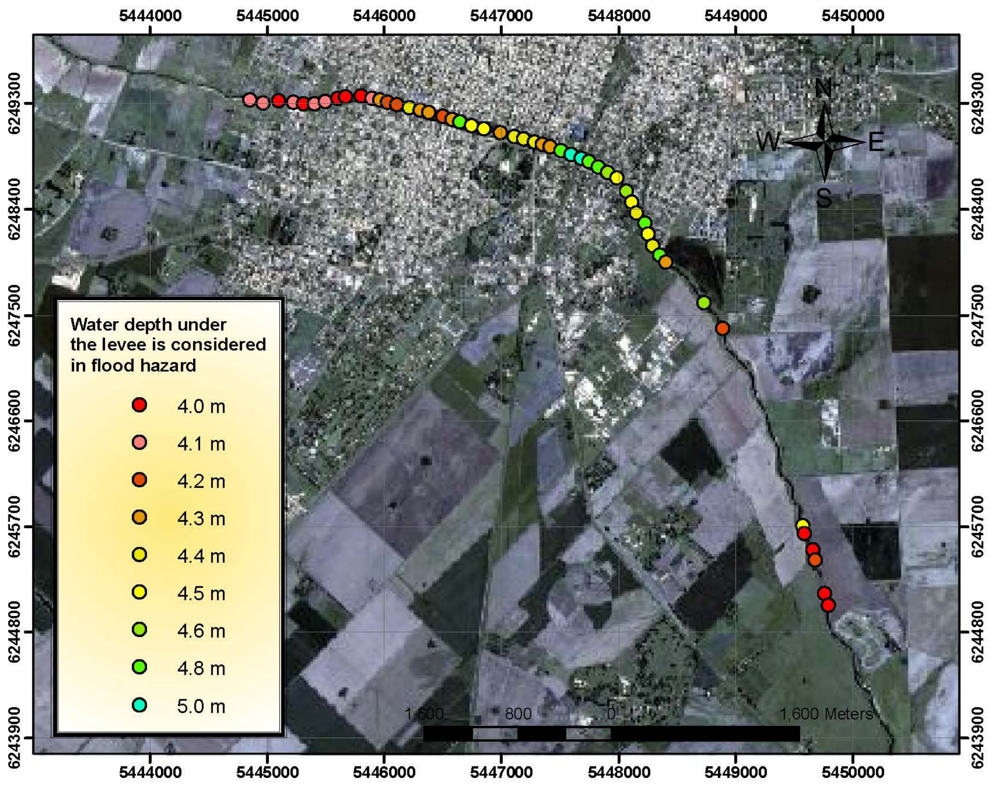


Figure 43 Cross sections under overflow hazard for water depths in the ranges from 4.0 m to 5.0 m

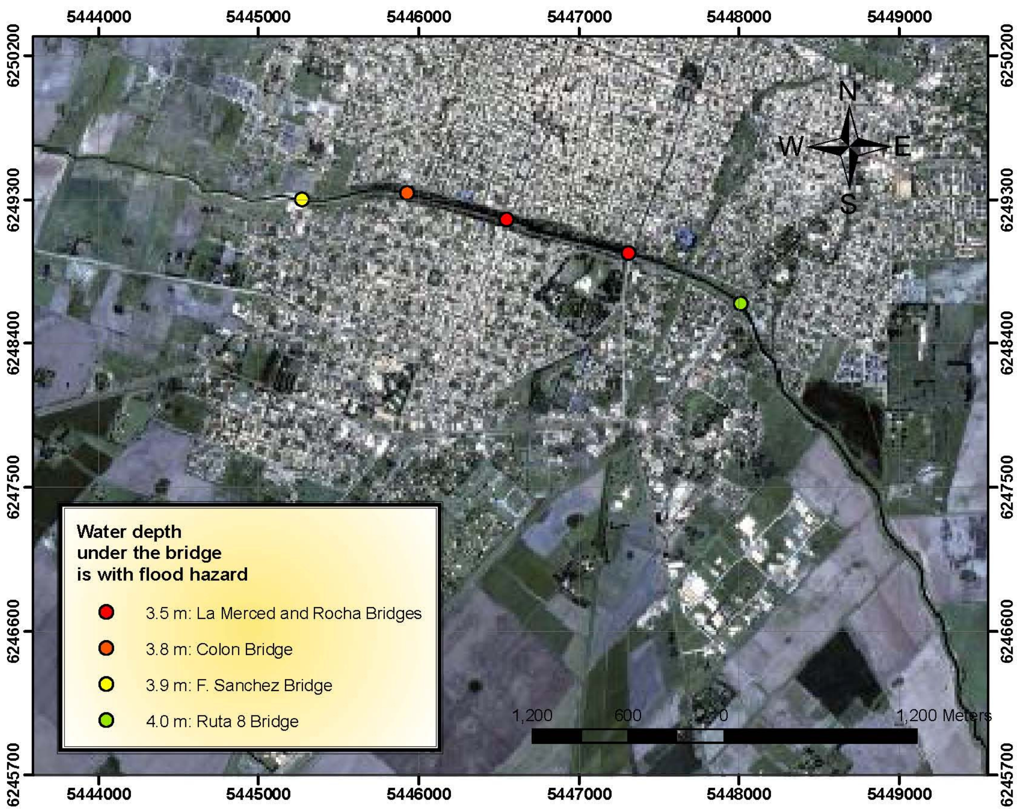


Figure 44 Water depths to generate flood hazard to the bridges

The first kilometers of the city (from 2+270 to 3+638 m[11]) (See Appendix B3) has the highest hazard to suffer the overflow with water levels between 4.0 m and 4.3 m. The path between the Rocha and Ruta 8 Bridges (from 3+480 to 4+068) presented lower hazard with water levels between 4.4 m and 4.6 m. (See water levels in Appendix B3).

The overtop danger in the dikes is not in all the cases for both sides of the cross sections. Some times one bank is affected first. This could be calculated from HECRAS but these results were not presented in the figures above. The determination of that along with the location of the cross section and the streets affected is presented in Appendix B3.

La Merced and Rocha Bridges (Figure 44) are under hazard when the water depths arises 3.5 m (water levels 55.8 m. and 54.8 m respectively). At this height, the bridges work under pressure.

According to the assessment, the streets more threatened are Matheu, Rivadavia and 9 de Julio with a water depths of 4.2 m (water levels 56.9 m 56.7 m respectively).

Unsteady flow assessment

4.1.4. Flood celerity analysis

The backwater effect downstream Alfonzo station was demonstrated using the Hec Ras model. In this context the measured water level values cannot be used to elaborate a rating curve which in turns prevents to generate the wave celerity assessment using the hydrograph generated.

However, the model showed properly performance in steady flow conditions for the main (central) study stream longitudinal profile. A simulation using the design hydrograph calculated by INA 2007 (Figure 11) was done. In order to evaluate the celerity of the flood wave the results hydrographs from the first (FFCC Mitre) and the last cross section (El Toro) were evaluated (See Figure 45). For this section there is confidence on the cross sections and longitudinal profile.

The time of travel the flood wave through the reach is the time between the centroids of the inflow and outflow hydrographs. (Brutsaert, 2005) The time that the flood wave needs from the beginning (FFCC Mitre) to the end (El Toro) of the study stream is about 3 hours (Figure 45). As the longitude of the river in the study area is 11.5 Km the celerity wave is 1.06m/s. A time of three hours does not leave much room for warning. In this regards, is by far insufficient and the idea of building the temporal storage reservoirs upstream Pergamino is then supported.

[11] Nomenclature which expresses the location of the cross section in terms of length, e.g. 3+480 means 3.48 km as from Mitre station.

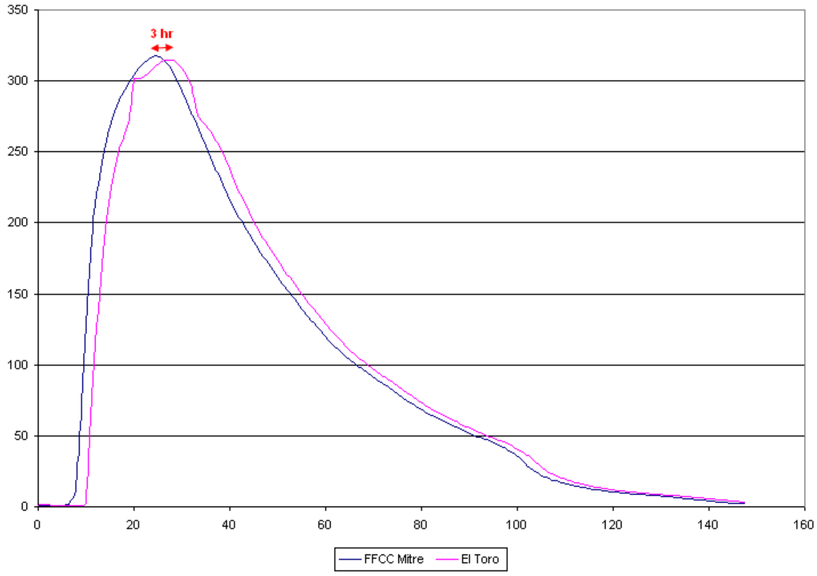


Figure 45. Wave time for 10 years recurrence period

Table 9 Wave celerity for different recurrence periods

Recurrence period years	Wave Time hr	Wave celerity m/s
10	3	1.06
50	2	1.28
75	3	1.06
100	3	1.06

Similar analysis was done for recurrence periods of 50, 75 and 100 years. The results are similar in terms of wave celerity (Table 9).

It is noticeable the effect that the recently constructed dikes produce over the Pergamino stream. As there is no room for water expansion, then there is no peak reduction. The plan formulated in the Municipality project is the construction of two dams upstream Pergamino, however as today, the general vulnerability of the city is compromised.

The dikes are being maintained and the structural stability of them is not compromised by the speed of the water in the stream during floods, as long as the water remains in the stream.

4.1.5. Limnigraphs analysis

One of the most relevant findings in this thesis came after the analysis of the data from the recently installed hydrographs.

As rating curves could not be derived from water level records due to persistent backwater effect at Alfonso, the short-recorded information from the limnigraphs stations was not used as source in this research. The backwater situation at Alfonso Bridge discovered during this research was confirmed by technical staff from INTA in February 2010 after a short verification campaign following heavy rains. However, during the assessment, the peaks in terms of water levels of both limnimeters were analyzed as the screening showed some interesting facts (Figure 46).

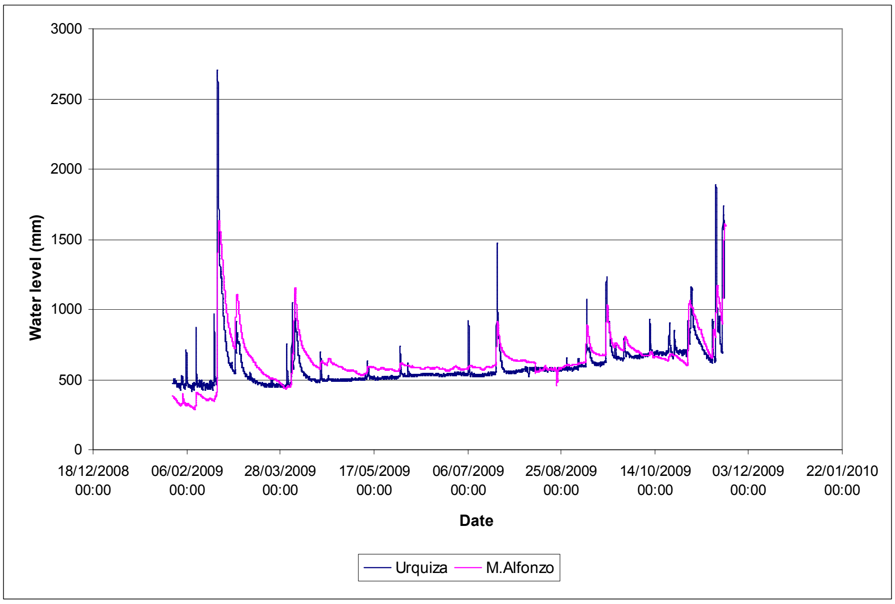


Figure 46 Comparison between values from limnigraphs stations

The peak at Urquiza Station (downstream) appears before the one at Alfonzo Station (upstream), this situation it is not usual. That can be observed clearly in February and it has been repeated in several showers (Figure 47). This singularity can only be explained considering the contributions of water discharges from small subcatchments downstream Alfonzo and close to the city and from the city itself. Those contributions make it to Urquiza faster than the upstream water from Alfonzo does.

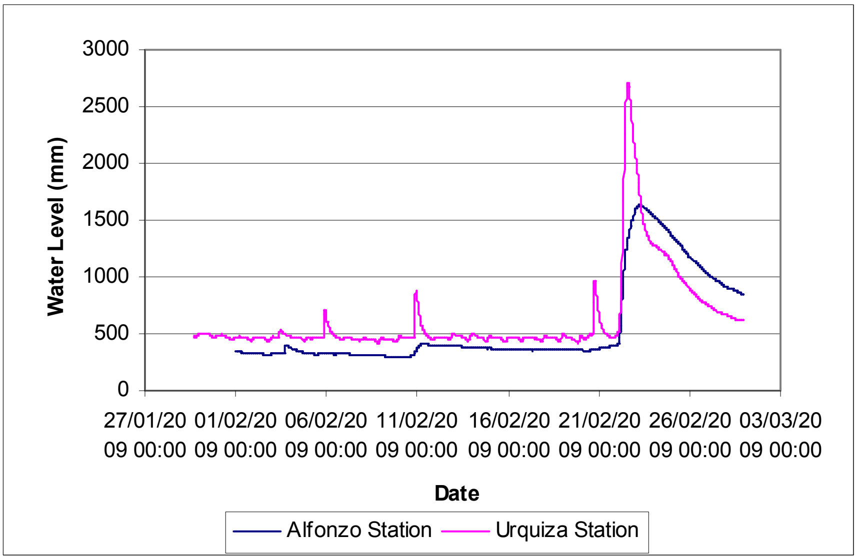


Figure 47 Comparison between peak values from the limnigraphs station, February 2009

The Alfonzo limnigraph is located upstream of an embankment bridge that is acting as linear reservoir in case of a flood. The influence of this small elevated road was not in the safety plans for Pergamino but the effect is not negligible. As such a study should be carried out to account for the storage and the beneficial effect of Alfonzo bridge to the downstream city.

Figure 48: Upstream view over Alfonzo bridge during a low risk flood (February 2010). It shows the limnigraph station, the backwater effect that had been foreseen by HECRAS and the large inundation plain the will be inundated in case of extreme flooding. Reducing the peak to Pergamino

Figure 49: Downstream Alfonzo bridge.

4.1.6. Improving in the existing Flood warning system

The results obtained in this research, showed two influential terrain feature aspects that were not analyzed in the project mentioned above. The situation is graphically depicted in Figure 50. The future location of the two dams is upstream Alfonzo station, close to Del Pescado lagoon. The limnigraph readings at Alfonso (upstream Pergamino) show that the peak of discharge appears systematically later than the peak at Urquiza station (downstream Pergamino city).

Second, the bridge at Alfonzo station builds a considerable retention in the storage area produces by the road embankment (Figure 50) and the second large storage happens at the location of Pescado lagoon immediately downstream Alfonzo, where the dam are planned. The combination of these two effects is considerable. This effect was discovered and tested after a HECRAS analysis over the limnigraph curve from Alfonzo Station. A simulated backwater effect and bridge damming was reproducing the measurements at Alfonso, proving this effect. As a consequence the peak at Alfonso is independent from the peak at Urquiza which supports the hypothesis that a review study of the dam projects could be consider the existence of a damping effect in Alfonzo station area, that could be eventually end up by reducing the projected height design of the dam (the current project consider a 9.0 m high (Figure 50)).

The quick peaks at the hydrograph in Urquiza do not come mainly from Alfonzo as it was supposed in the warning system design, but more locally from the subbasins surrounding Pergamino that are out of this present study. The conclusion is that a more adequate hydrological model calibrated for the damping effects now not considered should end in a cheaper solution (using the favorable effects of Alfonzo and Del Pescado that allows reducing the dam specifications) and more attention should be given to local flows from Pergamino vicinity.

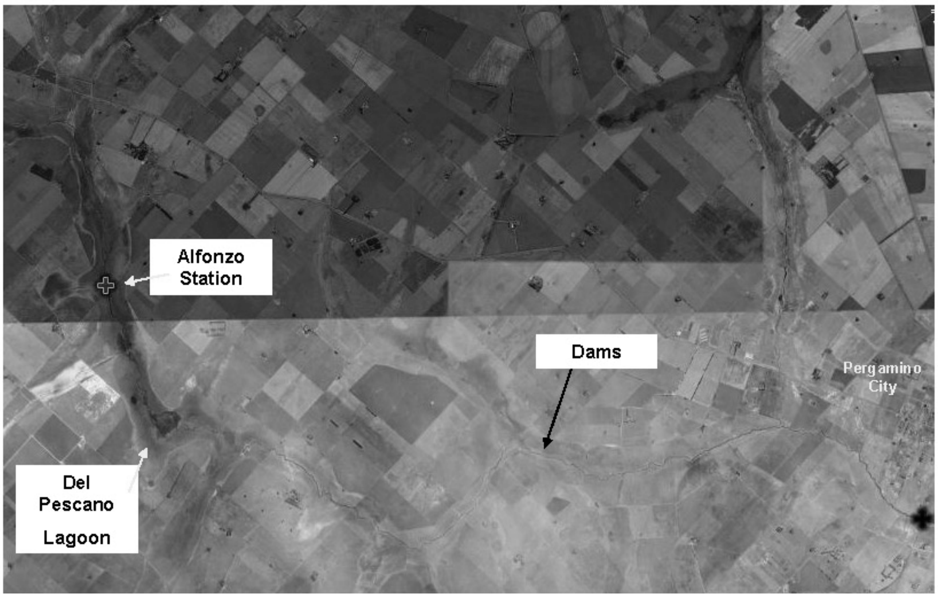


Figure 50 Lagoon Del Pescado location, between the Alfonzo Station and Pergamino City

4.2. Flo2D model performance

4.2.1. Grid Size constrain

As it was explained in the chapter 3, the spatial and temporal resolution of the FLO-2D model depends on the size of the grid elements and the rate of rise in the hydrograph (discharge) (Equation 7). With that relation, the highest grid size suggested to run the model was 35 m, however, considering some important features to be assessed, it was decided the attempt with 20 m grid-cell accepting that the model will run slowly. The main considerations were the average width of the channel (around 30 m) and the average width of the streets (around 8 m).

Due to stability constraints, to perform the model with this higher resolution the peak discharge value must be lower, some tries to reduce the value dividing the peak in two or three hydrographs was done, however, the constrain in the size grid at the input cell nodes was notorious.

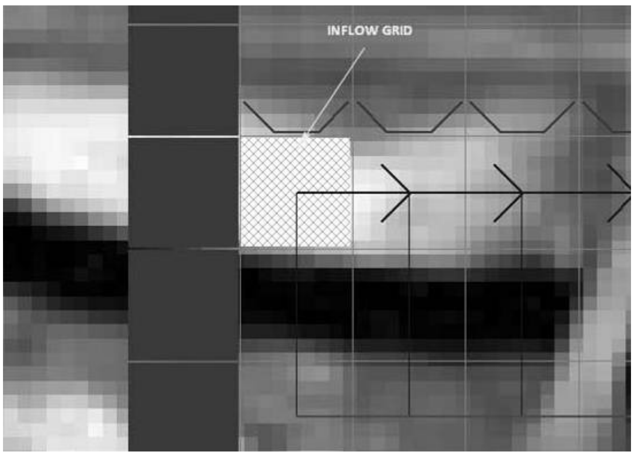


Figure 51 Constrain at the moment to introduce the inflow data

Two grids-cell (40 m) apparently could represent the width of the channel (around 35 m), however, the grids locations over the georeference image did not coincide exactly to represent the channel. As a result, the model did not consider the second hydrograph as part of the channel. In base on that analysis the introduction of the inflow data in two or even three nodes to achieve the stability the discharge was rejected.

During the introduction of the parameters in a flood simulation, the trend is to have as higher level of detail as possible; however, the grid size limits becomes a constrain. The ideal scenario for the flood modelling in an urban area would be with the introduction of roughness coefficients for each specific feature, considering the differentiation between the gardens, the material of the street (asphalt, soil, etc), and the houses. However, in this specific study, it was not possible to introduce the information with that level of detail, because the grid size and lack of information. Therefore, as first approach, all the simulations were performed considering only three types of areas for the introduction of the roughness coefficient: floodplain, channel, and parks or green areas. For the same reasons, the representation of the buildings was not achieved in this study.

Flo-2D deals with storage and conveyance at cell level by assigning the percentage of the area that cannot be occupied by surface flows (e.g. impervious buildings) and the percentage of flow width lost due to obstruction, meaning the direction where the flow will not pass, using the Area Reduction Factor and the Width Reduction Factor respectively (O'Brien *et al.*, 2009)) . The assignment of these factors will modified the performance of the model, as the flow does not calculate for the 8 directions and for the total grid area, so it would be give different results. Even when the attempt to use this options was done, the higher resolution of the image and the grid size did not allow completing successfully for the final simulations (different features inside the same grid, e.g. in the same grid there were buildings, gardens and street) The difficulties found to achieve model stability constrained the original expectation in the research, but the experience proved valid to establish a clear criterion on what spend time and effort before running the model.

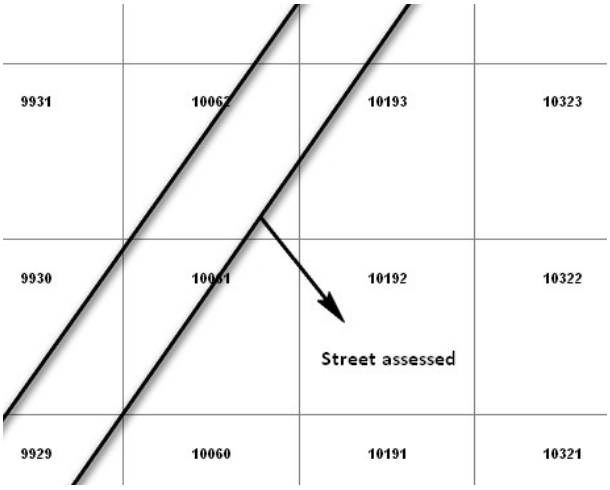


Figure 52 Example of constrain in the identification of the features

Furthermore, at the moment to analyze the model results by grid, the grid size influence was notorious. e.g. some streets catalogued with potential flood problems did not fit exactly in the width of some grids. As a result the street belongs to partial grids at the same time (Figure 52).

4.2.2. Failed simulation

Most of the problems in the first simulations were focus in the way to introduce the input data, even thought the model offers a tutorial, moving away to a real case with limited data proved an issue. The data set had to be verified, therefore, more simulations were done, as the use of the DTM was evaluated from threes sources, the first one from the contours map offered by the Municipality faced several problems of numerical stability criteria, the computation time were in these simulations several days without results. The third DTM generated from the topography sheet (scale 1:50000, interval 2.5m, source 1958) did not offer improvement at the moment to solve the volume conservation problems, the computation, the computer run time was similar than the previous simulations. As a result the second DTM generated from the points map provided by survey topography was chose to do next simulations.

4.2.3. Roughness coefficient calibration

Scenario A

From the scheme process described in previous chapter (Figure 35) , four trials were performed until achieve the final simulation.(Table 10). After each simulation concluded, the maximum floodplain velocity and the roughness coefficient adjusted for each grid were analyzed from the output files. To assess the maximum flow velocities reported by the model, a threshold equal to 2.5 m/s was established, this value was selected base on the assumption that flat areas (average slope 0.045%) will not report higher values in terms of velocity, furthermore, former flood modeling studies in other regions reported velocities lower than the threshold, e.g. in Ocotopeque, Honduras, the mean maximum velocity is reported in the range of 2.0 to 3.0 m/s (Ahti, 2007); in Tegucigalpa, Honduras, the mean maximum velocity was 2.21 m/s, considering for the modelling a DTM with resolution 2.5 m (Tamiru, 2005); and in Naga, Philippines the range of maximum velocities was reported lower than 1.0 m/s.
In the following we show the results after the recommended procedure to run FLO-2D in an interactive manner to detect unstable cells producing high speeds.

Table 10 Trials performed for scenario A

Trial	Wavemax coefficient	Maximum Flow Velocity	Number of grid adjusted
Rain	-0.250	< 2.5 m/s	2691
Rainv2	-0.250	< 2.5 m/s	2397
Rainv3	-0.250	< 2.5 m/s	1962
Rainv4	100.25	< 2.5 m/s	0

The first trial did not report maximum flow velocities higher than 2.5 m/s, from that results, according to the sequence process (Figure 35), it would be possible to run the model setting the Wavemax coefficient equal to 100.25 for the final verification of the velocities, however, in order to see the performance in the number of n-values adjustments, two more trials were done (Trial 2 and Trial 3) as a previous step with the same Wavemax coefficient.

In terms of roughness coefficient, Figure 53 shows the values introduced as input in each simulation, in Trial 1 graph the values correspond to the literature (Table 5), the Trial 2 graph shows the values adjusted by the model during the simulation of trial 1, the range between the values (0.030 to 0.234) represent the floodplain area, the model kept the roughness value for the channel in 0.040. The Trial 3 graph shows that during the simulation in Trial 2 the values were increased in some cells until 0.237, even though the change looks not significant, the number of cells adjusted was reduced (Table 10). No more increased in roughness values were done by the model during the simulation in Trial 3, only the number of cells adjusted was reduced. The Trial 4 graph presents the roughness values used for the final simulations.

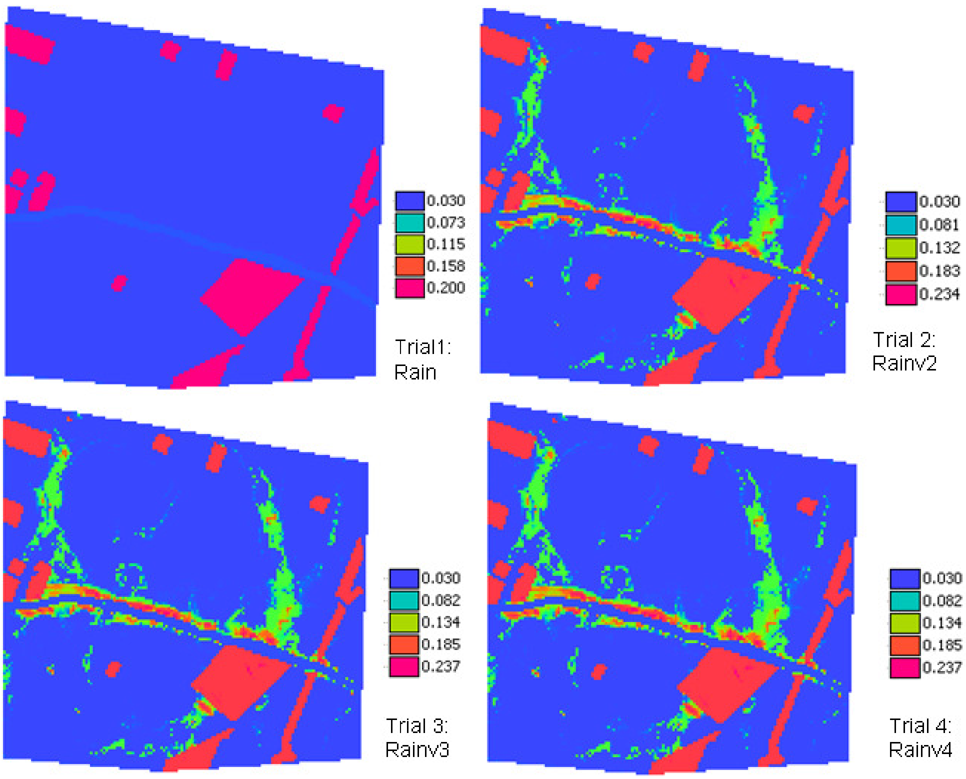


Figure 53 Roughness coefficients introduced as input for the simulation considering Scenario A

Furthermore, one parameter which affects the adjustment of the roughness coefficient during the simulations is the Limiting Froude Number. Despite that the model presents by default a value equal to 0 (representing Froude can vary freely) previous simulations using that value showed a high increase in the roughness values during the adjustment (it was recorded as the highest value 0.918). The same indications were reported by the authors of FLO-2D who indicated that those values did not correspond to characteristics of the area. The literature presented n-values lower than 0.5 (O'Brien *et al.*, 2009). A Limiting Froude Number equal to 0.9 (sub-critical conditions) was considered acceptable for the trials presented in Figure 53.

The results presented in the Figure 53 were assessed considering the difference between the original roughness coefficients (introduced as input in the simulation) and the final roughness values (adjusted by the model during the simulation and reported in the output files. The difference values (Figure 54) represent the grids where the model identified potential stability problems (the dynamic wave stability criteria were exceeded) and defaulted to an increase the roughness values. If that difference was significant, a new simulation was required to reduce it. During the simulation of the Trial 1, the difference values were in the range between 0.00 to 0.199, the grids with zero difference shows no need of adjustments, in the Trial 2 the range between 0 to 0.109 which meant better performance of the model, finally as the Trial 3 gave the range between 0 to 0.072, the roughness values adjusted were accepted to continue the process (Figure 35). The next simulation (Trial 4) did not report any higher velocity that could be responsible of any possible instability problems, as a result the roughness values were considered as calibrated for the final simulation.

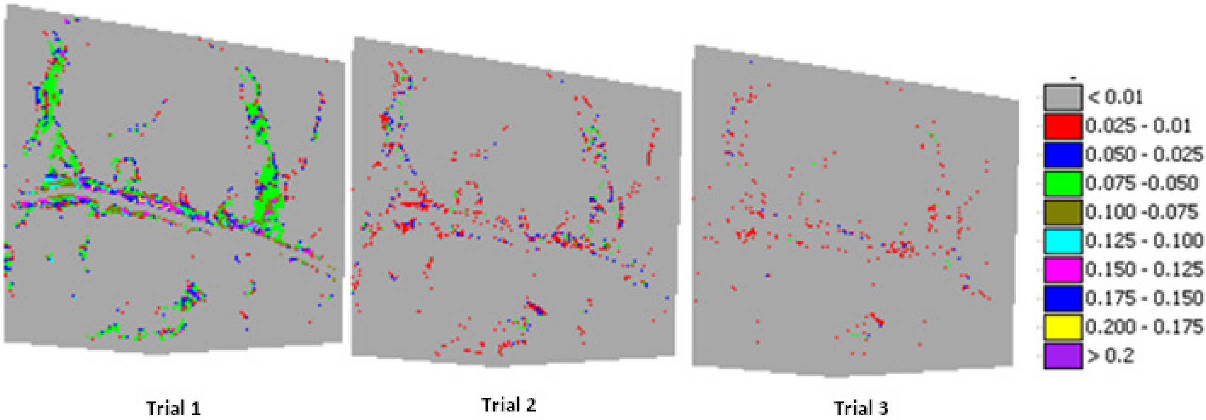


Figure 54 Difference between the roughness coefficients introduced as input and the roughness coefficients adjusted by the model during each simulation Scenario A

Scenario B

In the case of scenario B, similar procedure as described above was done to calibrate the roughness coefficients, four trials were made until the final one (Table 11). The verification of high velocity values was done after each simulation, and a difference with the scenario A was noticeable after the first trial, as some cells showed velocities higher than the threshold. Therefore, the roughness coefficient adjusted by the model and also an external adjustment process was done, the cell number with high velocity was identified and the roughness values for the eight neighbouring cell-grids were increase as suggested by the methods.

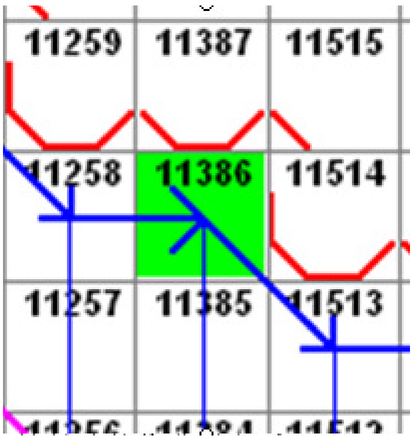


Figure 55 Example of grid with high velocity

For example in the first trial results, the grid 11386 showed a velocity 2.84 m/s over the threshold established, this is related to a numerical surging (mismatch between flow area, slope and roughness (O'Brien *et al.*, 2009). Therefore, the roughness coefficients in the 8 elements in the vicinity were increase for the next trial.

The Table 11 shows the number of grids adjusted in the internal process by the model and the number of grid adjusted due to highly velocities

Table 11 Trials performed for the scenario B

Trial	Wavemax coefficient	Number of grid adjusted by internal process	Number of grid adjusted by high velocity
Flow	-0.25	3535	3
Flowv2	-0.25	1206	0
Flowv3	-0.25	743	0
Flow4	100.25	0	0

Table 12 Example of the difference n-values (Final n – originl n) calculated from the output results

GRID	MAX n	FINAL n	ORIG. n	Difference
17605	0.121	0.12	0.035	0.085
17606	0.082	0.082	0.035	0.047
17607	0.063	0.035	0.035	0.000
17719	0.11	0.04	0.04	0.000
17721	0.108	0.108	0.035	0.073
17722	0.091	0.035	0.035	0.000
17830	0.084	0.035	0.035	0.000
17835	0.104	0.04	0.04	0.000
17837	0.093	0.093	0.035	0.058
17950	0.11	0.04	0.04	0.000

As for the scenario A, the internal adjustment process was also evaluated calculating the difference between the roughness values introduced and final values presented by the model at the end of each simulation. (See e.g. Table 12). The first trial present increments in the values in the range of 0 to 0.270, the grids changed are surrounding the channel where the flood happens. In the next trial, the range of increasing from 0 to 0.163 and finally in the third trial performed the difference decrease until the range of 0 to 0.105. Therefore, the roughness values adjusted in the trial 3 were considered acceptable for the next step that is, the verification of high velocities using Wavemax coefficient equal to 100.25 (Figure 35)

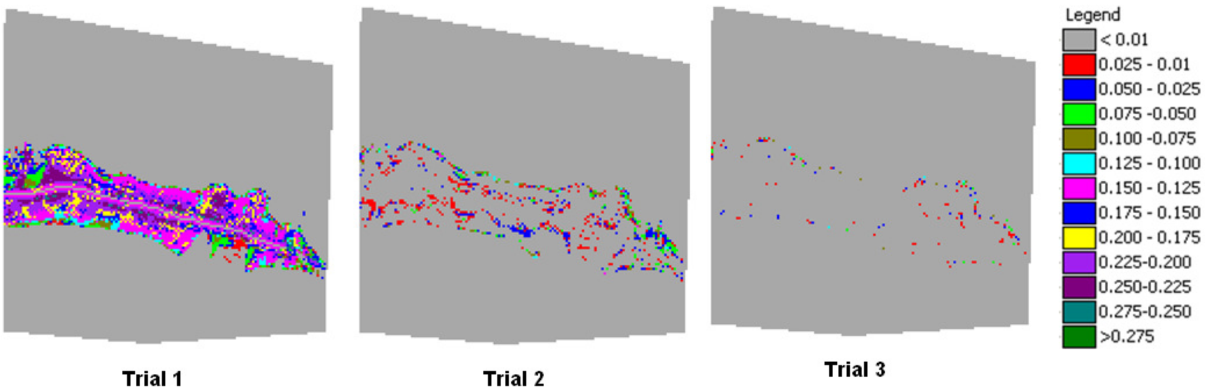


Figure 56 Difference between the roughness coefficients introduced as input and the roughness coefficients adjusted by the model during each simulation Scenario B

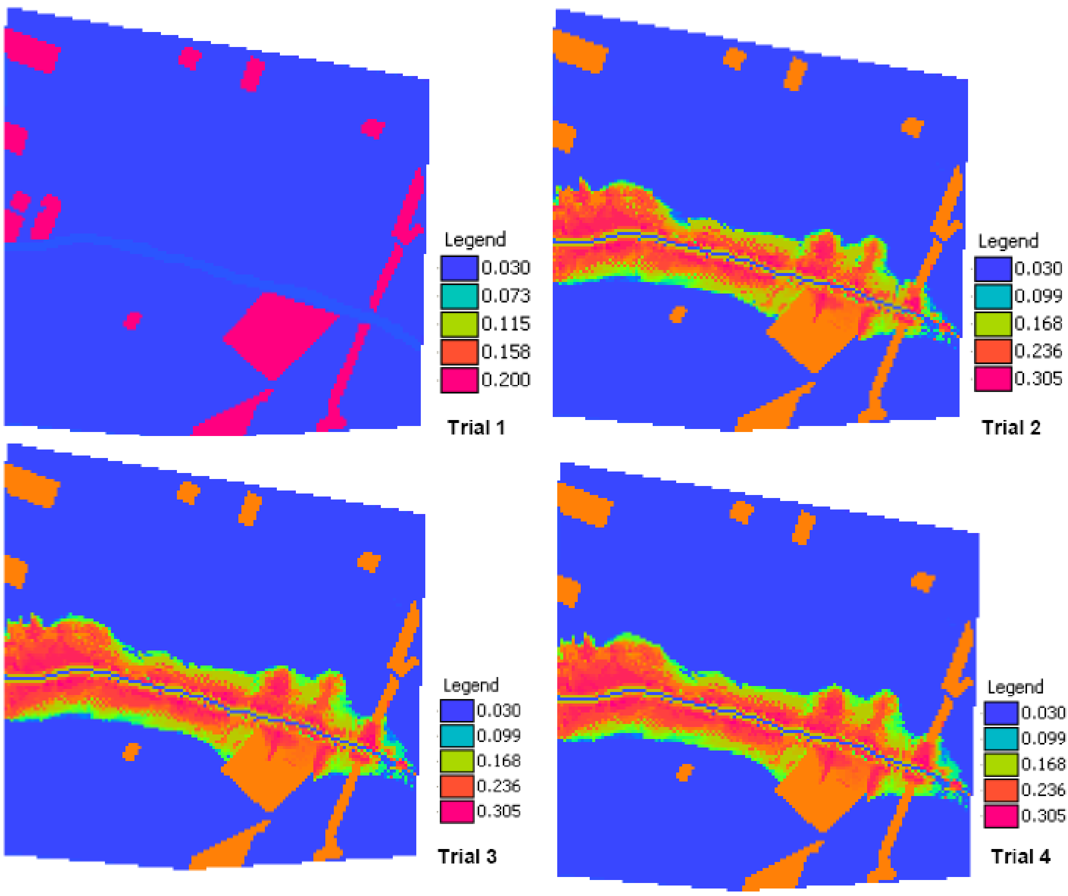


Figure 57 Roughness coefficients used during the assessment in Scenario B

The roughness coefficient used during the different trials (Figure 57) are in the range from 0.030 to 0.305, the adjustments correspond to the floodplain areas most of the time, even though it was detected some channel grids to be adjusted, at the end of the simulations all the roughness values for the channel were kept by the model in the original value 0.04, those values are in the range suggested by the literature, more over, the green areas near to the channel where also adjusted by the model increasing the values from 0.2 to 0.305.

Scenario C

Scenario C is the most compromised, similar process was followed in this scenario, four trials were done to obtain the adjusted roughness coefficients set for the final simulation. In all the trials was necessary to increase the roughness coefficient in the vicinity grids of those with high velocities (> 2.5 m/s) (Table 13).

Table 13 Trials performed in Scenario C

Trial	Wavemax coefficient	Number of grid adjusted by internal process	Number of grid adjusted by high velocity
Both	-0.25	4130	7
Bothv2	-0.25	2027	5
Bothv3	-0.25	1133	3
Bothv4	100.25	0	4

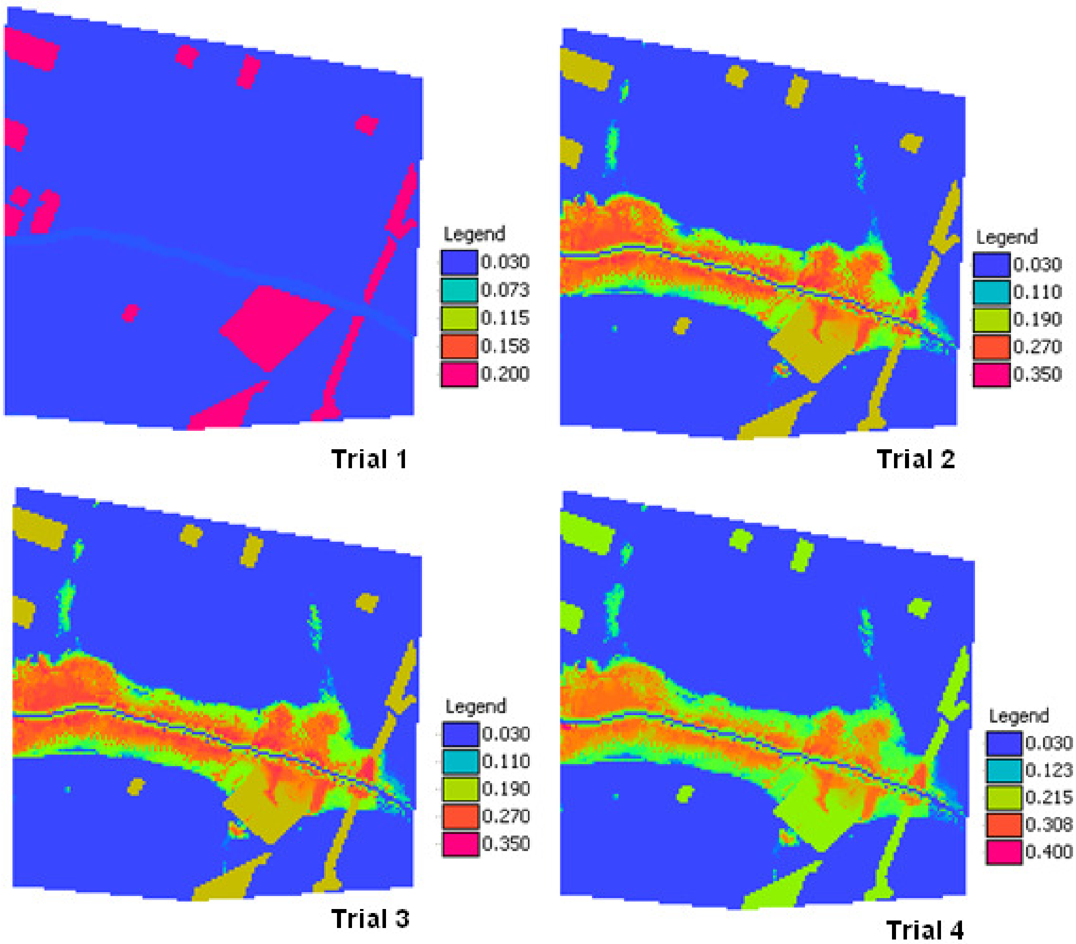


Figure 58 Roughness coefficients used during the assessment in Scenario C

The roughness coefficient adjusted in the first trial were between 0.030 to 0.350, this value was kept for the model as maximum adjusted in the second trial, however, for the last trial, the model increase the roughness in some grids until 0.400 (Figure 58).

As it was done in the previous scenarios, the difference between the roughness coefficients introduced to the model as input and the roughness values adjusted by the model after every simulation were assessed.

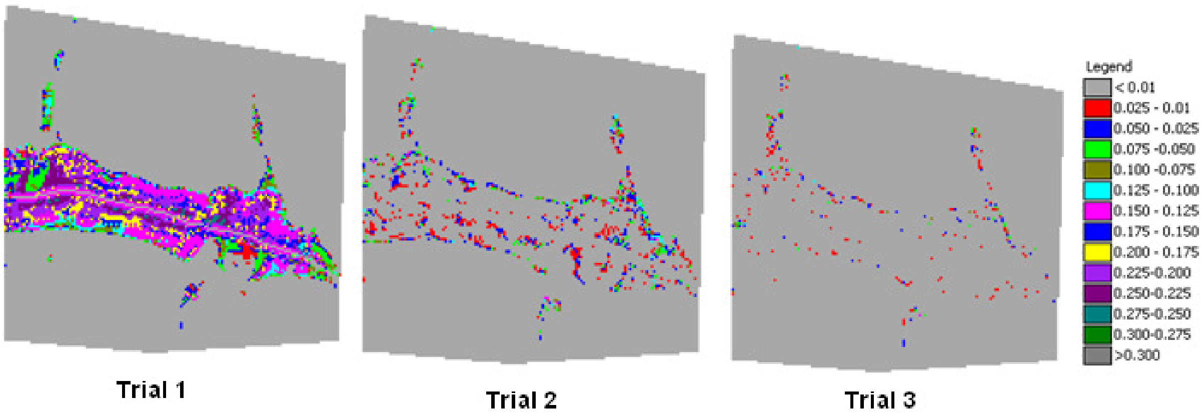


Figure 59 Difference between the roughness coefficients introduced as input and the roughness coefficients adjusted by the model during each simulation Scenario C

In Trial 1 (Figure 59) the difference in the adjustment were in all the area surrounding the channel, the maximum increase performed for the model in some cells were 0.315, in the second trial not only the number of cells adjusted was reduced but also the increment value (maximum 0.150). In the third trial the maximum value increase in some cells of the floodplain was 0.133.

During the performance of the trials a comparison between the maximum flow velocity and maximum flow depth was done. An example of this process is presented in Appendix B7. Despite the results of flow depth and velocity from the trials gave similar values, there were variations in terms of computational time and an inundation area.

4.2.4. Flood patterns

Scenario A

The final simulation performed for the scenario A, is presented in terms of Inundation Area in Figure 60, this inundation area map was generated by the model considering the cells where was reported values of maximum flow depth during the simulation.

The total study area modelled was 9.87 Km^2, the model reported as inundation area 3.5 Km^2, which means 35 % of the total area assessed. Considering the cadastral map of the city, the districts (or neighborhoods) corresponding to this area were identified using Ilwis tools, previous process of exporting the maps from Autocad format. The districts Trocha, Vicente Lopez, Belgrano, Centro and Centenario present the most extended area affected by the flood. However, considering in the analysis not only the extend area but also the mean maximum flow depth and the mean maximum flow velocity by district, Trocha registered 0.23 m, Centenario 0.20 m, Centro 0.36 m. Vicente Lopez and Belgrano registered 0.51 m and 0.42 m respectively. Furthermore, Cueto with small extend of area affected registered a depth equal to 0.34 m.

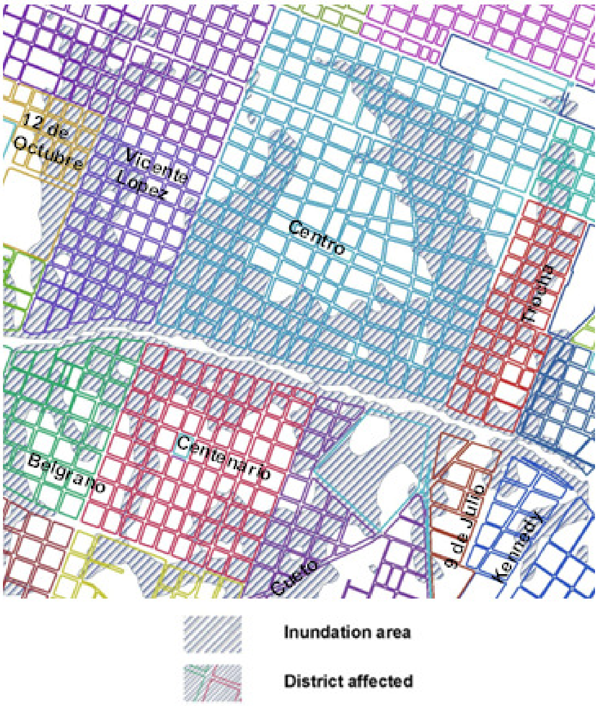
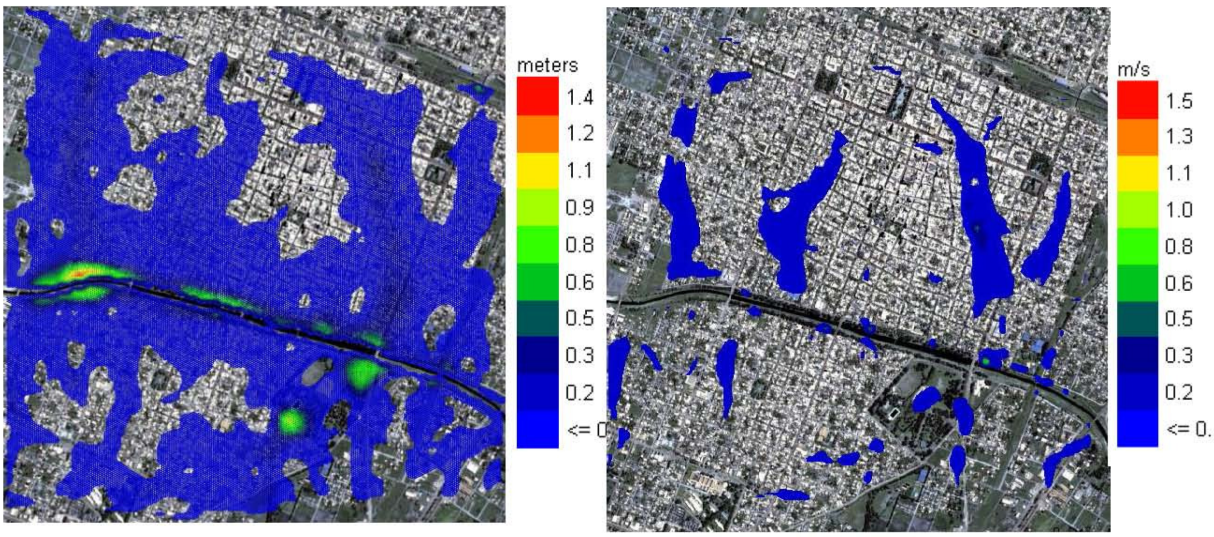


Figure 60 Inundation Area for the Scenario A

Table 14 Districts affected Scenario A

District	Area affected m2	Percentage of District
12 de Octubre	105850	42
27 de Noviembre	47331	28
9 de Julio	32813	21
Belgrano	190569	48
Centenario	253131	44
Centro	762419	48
Cueto	192613	38
Desiderio de la Fuente	32500	26
Kennedy	45319	11
Mariano Moreno	16625	10
Martín Illia	106106	17
Martín M. de Güemes	19613	20
Trocha	180169	68
Vicente Lopez	251081	63
Villa Fernández	74356	29
Villa Progreso	106	0.2
Lotes	6838	28
FFCC	6719	4
Plazas Parques	134950	39
Calles	1044631	38
total	3503738	35

The model simulated the maximum flow depth 1.4 m in Vicente Lopez District near to the channel (Intendent Biscayart street between Vicente Lopez and Francia Street) (See in the Figure 61 , a). The maximum flow velocity simulated is 1.46 m/s at 2.55 hours since the beginning of the rainfall, this velocity corresponds to a 0.16 m of depth, the point is located in Trocha Distric (Makintach Street, beetween Moreno and Gonzalez Street).



a) **b)**

Figure 61 a) Maximum Flow depth b) Maximum Flow velocity

Historical Rainfall: February 9, 2001

The areas where CO.S.S.O.PER[12] supplied information from the rainfall event occurred in February 9, 2001, were identified in the grid system used by the model for the simulation. The values recorded for the neighbours were compared with the maximum flow depth simulated by the model (Table 15).

Table 15 Comparison between the water depth reported by CO.S.S.O.PER and the values simulated by the model

		CO.S.S.O.PER	FLO-2D				
Area	Location	Flow depth (m)	Flow depth average (m)	Flow depth Grid Selected	Time hr	Flow depth Max (m)	Flow depth Min (m)
0	Jauretche Street	0.30	0.41	0.32	2.34	0.32	0.10
1	Villegas e/ Jauretche and Lavalle Street	0.45	0.33	0.43	2.08	0.77	0.14
2	25 de Mayo e/ Magallanes and Balboa Street	0.30	0.13	0.21	1.42	0.21	0.10
3	Magallanes e/ 25 de Mayo and Villegas Street	0.30	0.28	0.30	1.63	0.37	0.18
4	Int Biscayar, 3 de Febrero and Castelli e/ Rocha and Azcuenaga.	0.25	0.36	0.25	2.13	0.41	0.25
5	Int Biscayar, 3 de Febrero and Castelli e/ Rocha and Azcuenaga	0.25	0.39	0.27	1.86	0.47	0.27
6	Solis e/ 25 de Mayo and Jauretche	0.20	0.12	0.17	1.89	0.17	0.07

The values reported by the neighbours were not taking in direct measures during the event, the source of these values is from unsupervised meetings where the neighbours recalled the event and filled a form with theirs memories about it (See e.g. Appendix A5). During fieldwork some marks in the doors or walls were search to support those values, unfortunately without any results. Therefore, the values reported by CO.S.S.O.PER have to be considered only as reference to evaluate the model performance.

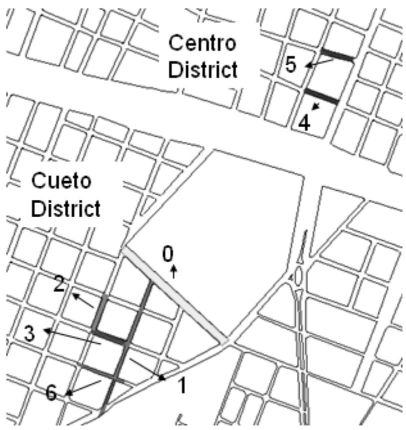


Figure 62 Location of the areas affected
Source: CO.S.S.O.PER,2009

The comparison between the mean maximum depth and the CO.S.S.O.PER values (Table 15) shows an apparently variation in the model performance. However, as the maximum depth in every grid is simulated at different times of the event, the average value is not a right representation of the area. Therefore, from all the grids inside the affected area, it was selected the one which have similar value to the one reported by CO.S.SO.PER. The time where those values were simulated is presented in the Table 15, also as reference are presented the maximum and minimum value simulated considering all the grids inside the affected area.
In short all the values should be seen as a reference that depends on the event considered, but the model showed a consistency in the area affected.

4.2.5. Scenario B

In the case of the Scenario B, the model simulated 1.70 Km^2 as inundation area (Table 16). The districts with more extend area affected are Vicente Lopez and Belgrano, Although Martin M de

[12] They are o group of good-will neighbors who decided to take some records during the flood and locate them in maps. Despite that the data was quantitatively doubtful this association is the only resource of records in the area.

Guemes has almost 50% of area affected, the value is not considered as reference as its total surface is not inside the boundaries of the study area.

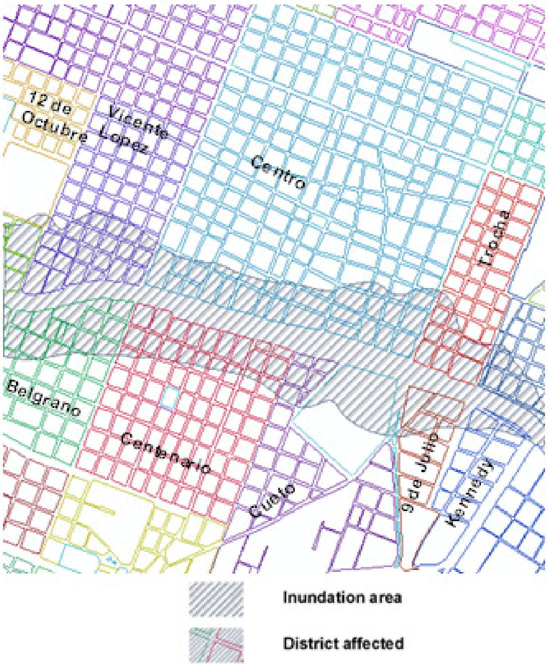


Figure 63 Inundation Area for the Scenario B

Table 16 Districts affected Scenario B

District	Area affected m2	Percentage of district
12 de Octubre	4425	2
27 de Noviembre	49225	29
9 de Julio	47463	30
Belgrano	137594	35
Centenario	146394	25
Centro	261306	16
Cueto	36088	7
Kennedy	17069	4
Martín M. de Güemes	43281	45
Trocha	76988	29
Vicente Lopez	155131	39
Plazas Parques	96988	28
total	1071950	21

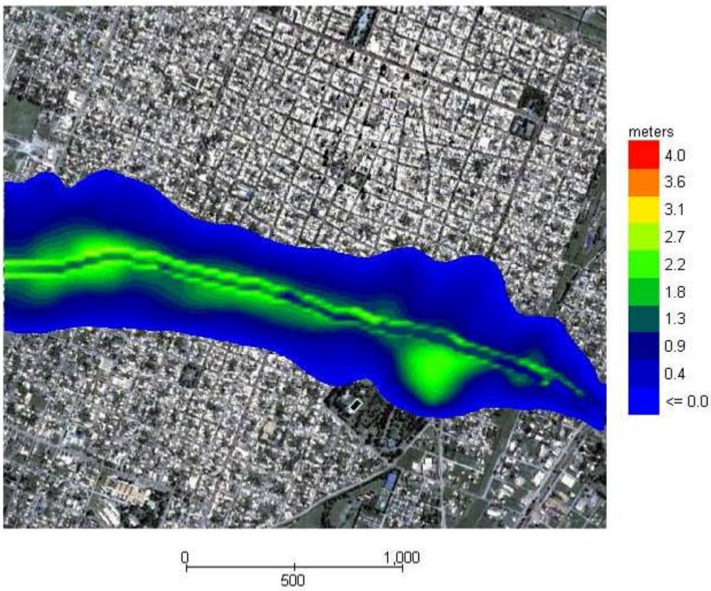


Figure 64 Maximum flow depth Scenario B

The maximum flow velocities were simulated in the areas surrounding the channel (maximum value 1.70 m/s near to the upstream boundary). The maximum flow depth predominant in the floodplain is 0.030 m, however, near to the channel higher values were simulated as 2.8m or 3.15m, the maximum flow depth average is 1.8 m,

4.2.6. Scenario C

The final simulation for the scenario C, combined the effect of the rainfall event and the overflow in the channel, gave the largest flood area equal to 1.9 Km^2 (Figure 65).

The districts affected in this case are presented in the Table 17; the worst case is Vicente Lopez District with more than 50% of its surface cover by flood, then Trocha, 9 de Julio and Belgrano present high values of affected areas. Even though, Martin M Guemes and 27 de Noviembre Districts report more around 50% of their surface affected, those values needs to be under later assessment because only a part of those districts belong to the study area.

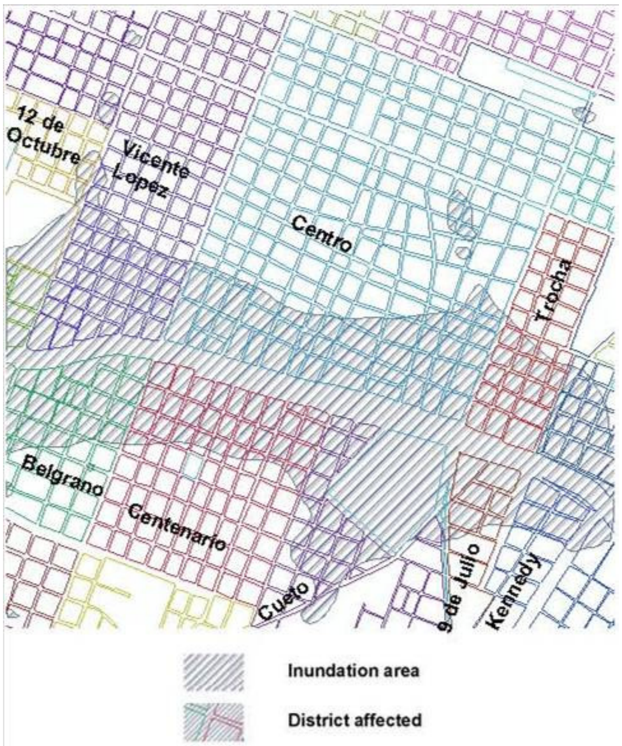


Figure 65 Inundation Area for Scenario C

Table 17 Districts affected in Scenario C

District	Area affected m2	Percentage of district
12 de Octubre	53731	21
27 de Noviembre	96038	57
9 de Julio	74269	47
Belgrano	191875	49
Centenario	213600	37
Centro	445094	28
Cueto	141056	28
Kennedy	63381	15
Mariano Moreno	1394	1
Martín Illia	12563	2
Martín M. de Güemes	47100	48
Trocha	127556	48
Vicente Lopez	234081	58
Lotes	9844	41
FFCC	2013	1
Plazas Parques	188050	55
total	1901645	31

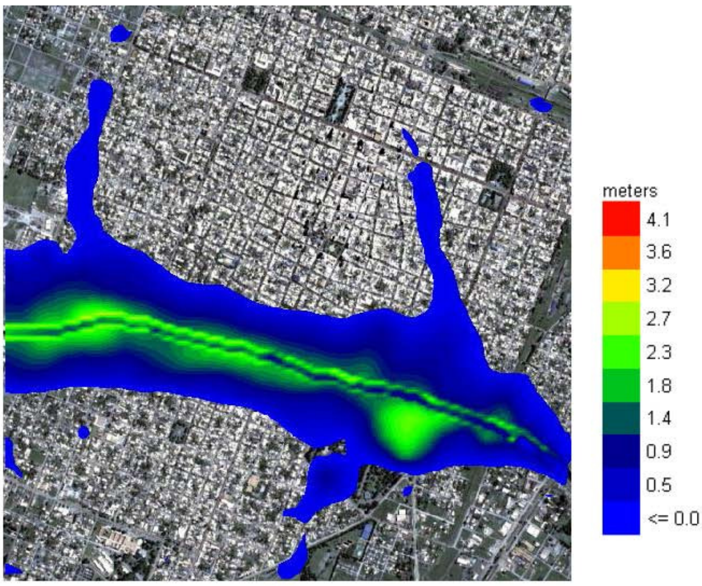


Figure 66 Maximum flow depth in Scenario C

Furthermore, in this case simulation gave high depth values in the areas surrounding the channel (around 3.0 and 4 m), the average maximum flow depth for the floodplain is 1.8 m and the predominant value is 0.04 m.

The maximum flow velocity value simulated is 1.84 m/s, with an average maximum value equal to 0.67 m/s in the floodplain.

4.2.7. Flood propagation

As the scenario C represents the worst event, the flood propagation was assessed in terms of water depth every hour. The graphs in Figure 67 shows some relevant screening during the simulation (See Appendix B8 for the rest). At 9 hours from the beginning of the simulation time, some sectors along the channel began to show overflow problems. The overflow along the channel is completed after 14 hours. The rainfall event began at 20 hours and the worst situation is around 22 hours when the combination of rainfall and overflow in the channel acted together. After that peak, the inundation area is defined by the model and the rest of the hours the area affected become to decrease.

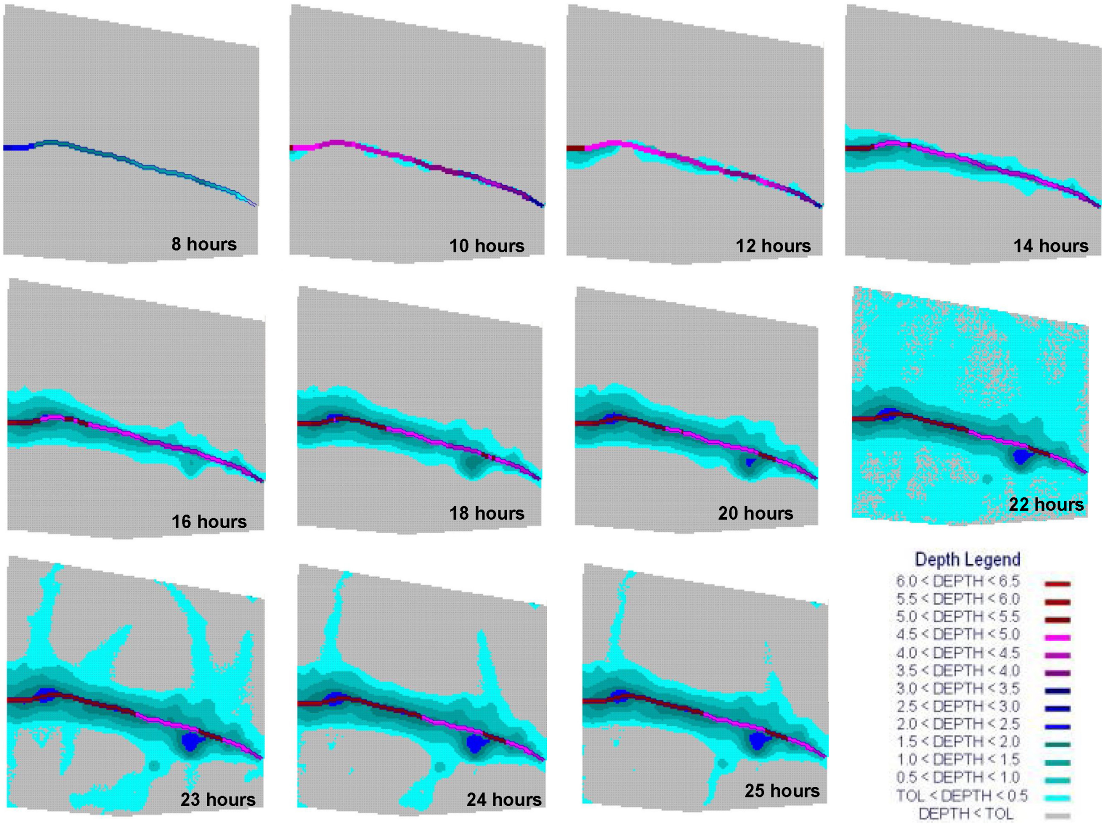


Figure 67 Flooding propagation for Scenario C

4.2.8. Applicability of the results

During the flood assessment the maximum flow depth in a specific area and the time when that value happened was considered. The results that Flo-2D model offers at the end of the simulation enable to assess those values in specific areas or cells. To show the assessment that is possible to achieve with the results, only one sector was chosen as example: Dr. Leandro N. Alem Street, located in the Centro District between Biscayart Street and Trincavelli Street. The grids inside the area were identified in the system scheme generated by the model. Since the grid size did not match exactly with the shape and width of the street. (Figure 68) in this analysis were considered those cells with at least 50% of the area inside the study surface. (Number grids selected: 12152, 12280, 12281, 12282, 12283, 12410 and 12411)

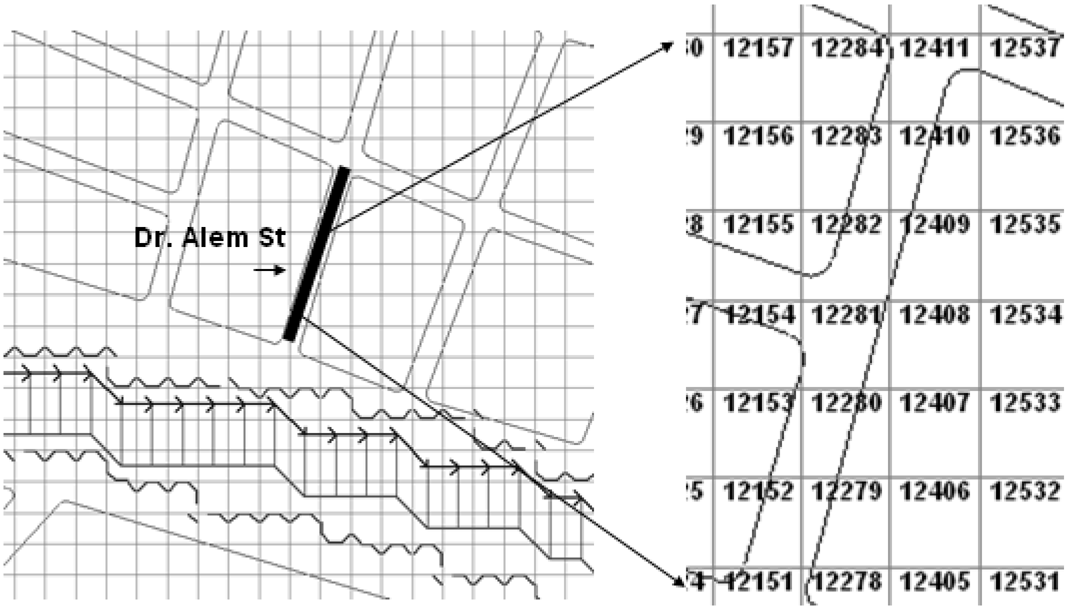


Figure 68 Street selected for the depth and velocity assessment

The values simulated by the model for those cells were plotted for each scenario. In (Figure 69) is presented the maximum flow depth and the time when those values were simulated, more over, the velocity on that grid for that specific time. The biggest depth values in the street, are between 0.21m to 0.43 m, and were simulated at the end of the rainfall event; however, at the middle of the rainfall event (1.5 hr) the maximum depth was simulated in the north part of the street.

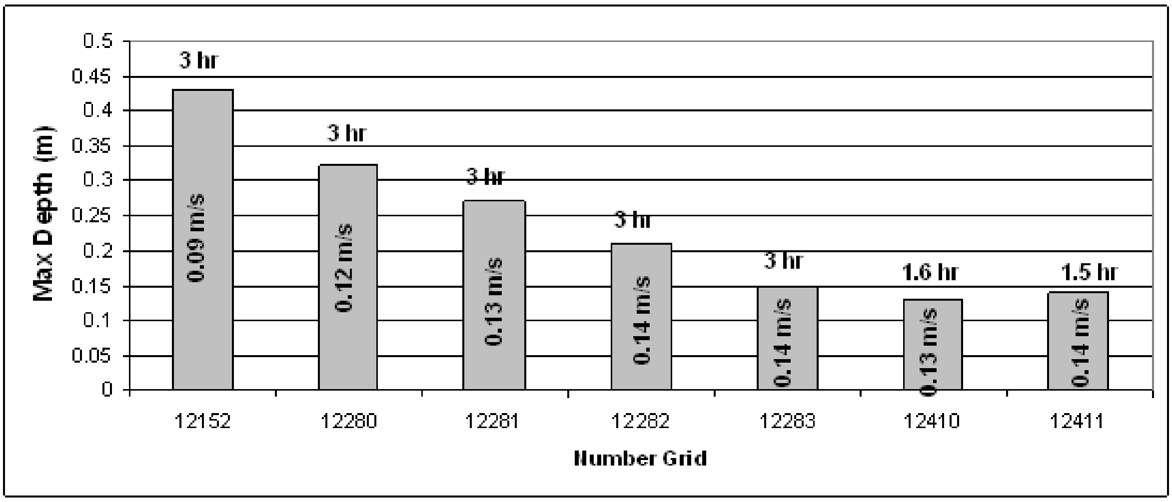


Figure 69 Maximum flow depth simulated in the Scenario A in Dr. Alem Street

The maximum flow depth increased more than 1.0 m when was considered the scenario B and C in comparison with the scenario A, (Figure 70). In the case of the specific street assessed, the velocities values for those water depths are quite similar for both scenarios. In scenario C the time when the maximum depth is simulated is shorter than in scenario B, this is because a more water volume of water (combination of rainfall and overflow) is presented in the first one. The comparison with the scenario A (Figure 69) apparently shows a higher influence of the flow in the channel than the rainfall over the city.

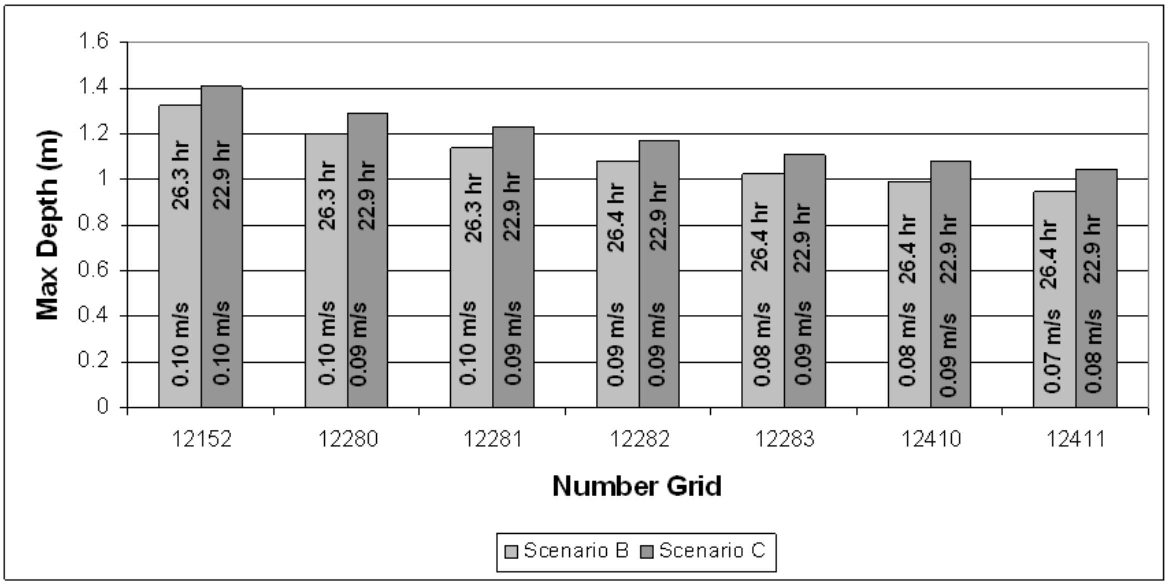


Figure 70 Maximum Flow depth simulated in the Scenario B and C for Alem Street

4.3. Flood hazard assessment

Despite some difficulties in the model run, the simulation with Flo-2D model, is presented as first approach to a potential flood hazard maps for Pergamino city with 10 years return period.

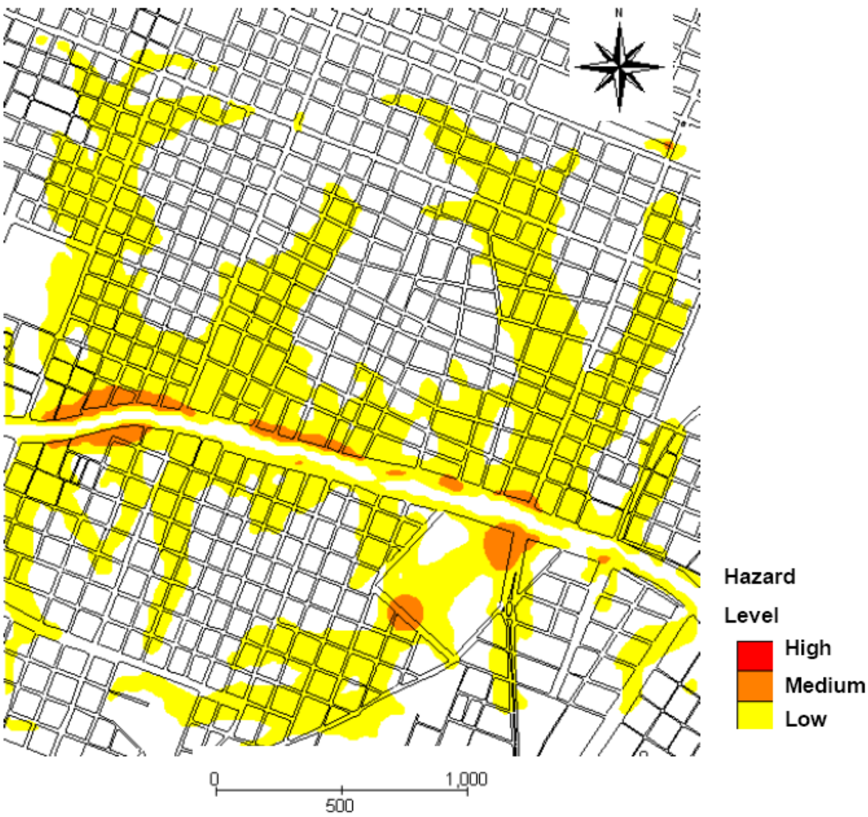


Figure 71 Flood Hazard map in the case of Scenario A : Rainfall event

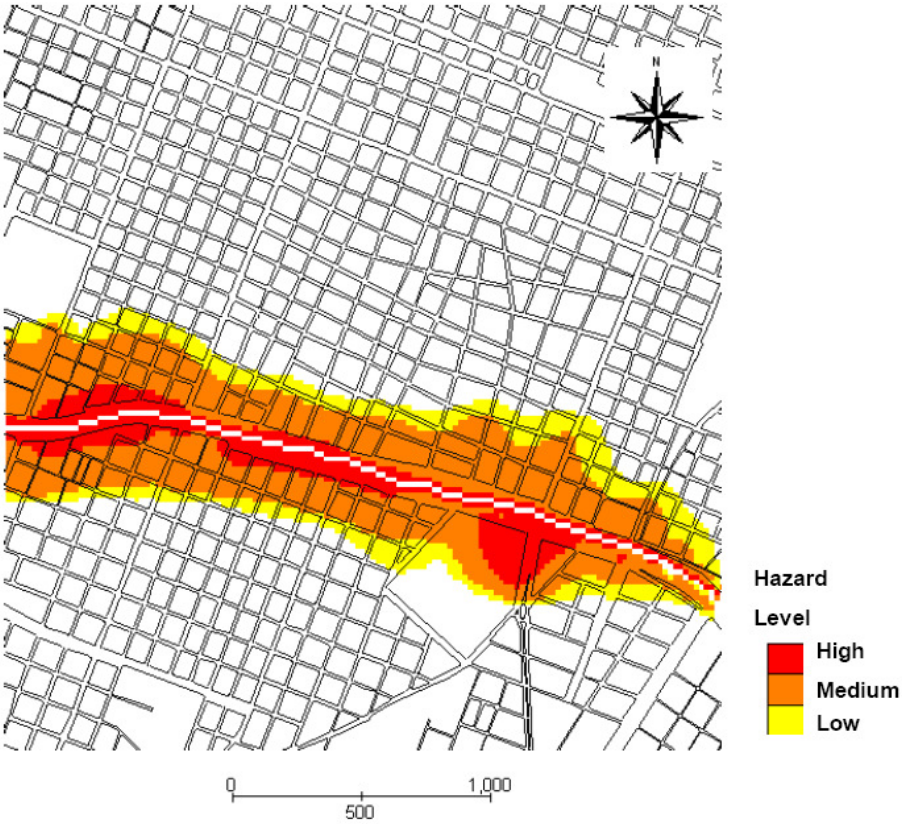


Figure 72 Flood Hazard map in the case of Scenario B: Overflow in the channel

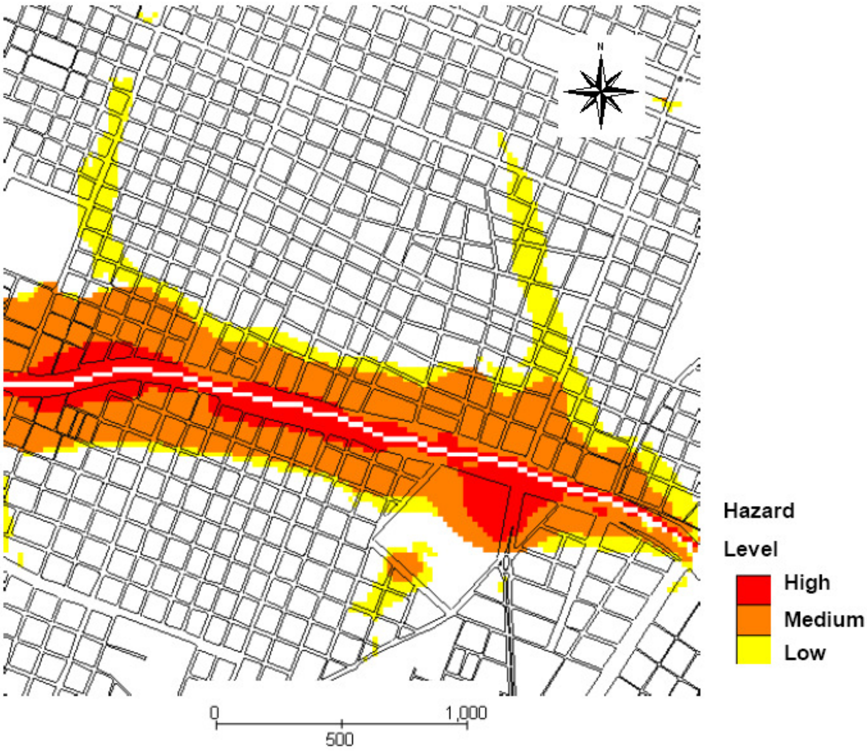


Figure 73 Flood Hazard map for Scenario C: Rainfall and Overflow in the channel

According to the hazard delineation performed by the model a High Hazard Level (red colour in the maps) means "Persons are in danger both inside and outside their houses. Buildings are in danger of being destroyed", then, a Medium Hazard Level (orange colour in the maps) means "Persons are in danger outside their houses. Buildings may suffer damage and possible destruction depending on construction characteristics". Finally Low Hazard Level (yellow colour in the maps) means "Danger to persons is low or non-existent. Buildings may suffer little damages, but flooding may affect houses interiors" (O'Brien *et al.*, 2009).

Despite that the flood propagation in scenario A represents a bigger area in comparison with the other two scenarios, the hazard level identified is low almost everywhere, only near the channel the flood hazard increases to medium level. In the scenario B, a medium hazard level is presented in most of the inundation area and the high level appears surrounding the channel and in the area corresponding to the Municipality Park. Furthermore, even when the inundation area in scenario C was bigger (Figure 60), the hazard map generated by Flo-2D represents smaller area (Figure 73), this is because the threshold used by the model (Table 7)

Furthermore, according to the danger zone classification made by FEMA (Figure 37), three maps were generated (Figure 74, Figure 75 and Figure 76). The zones identified with this methodology are different in comparison with the hazard maps generated by Flo-2D. For example the medium hazard level, (analogue to judgment zone) presents more extension in area for the maps generated by Flo-2D. This is because the criterion used related velocity and depth is different. However, in each scenario, both maps can be useful for the decision makers as this raise the concern that a local criteria needs to be built.

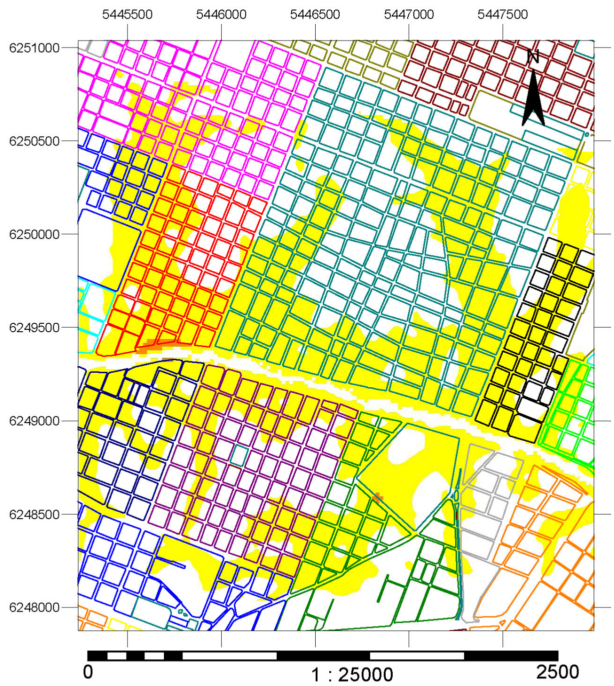


Figure 74 Flood Hazard map for Scenario A (According FEMA classification)

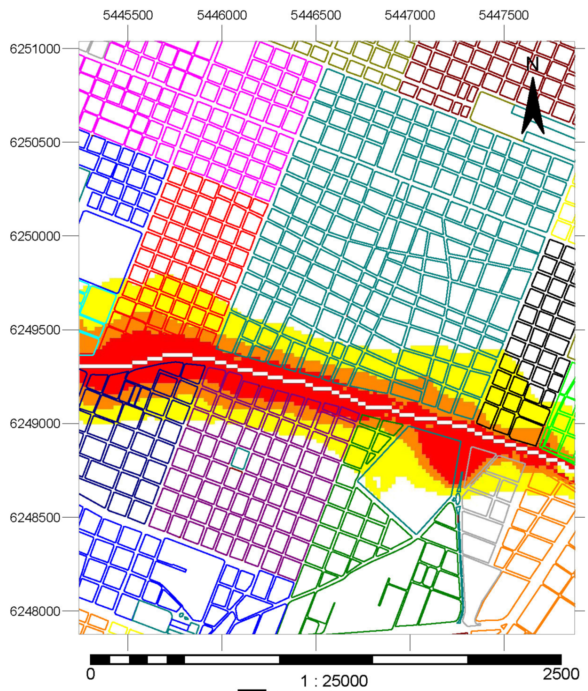
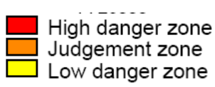


Figure 75 Flood Hazard map for Scenario B (According FEMA classification)



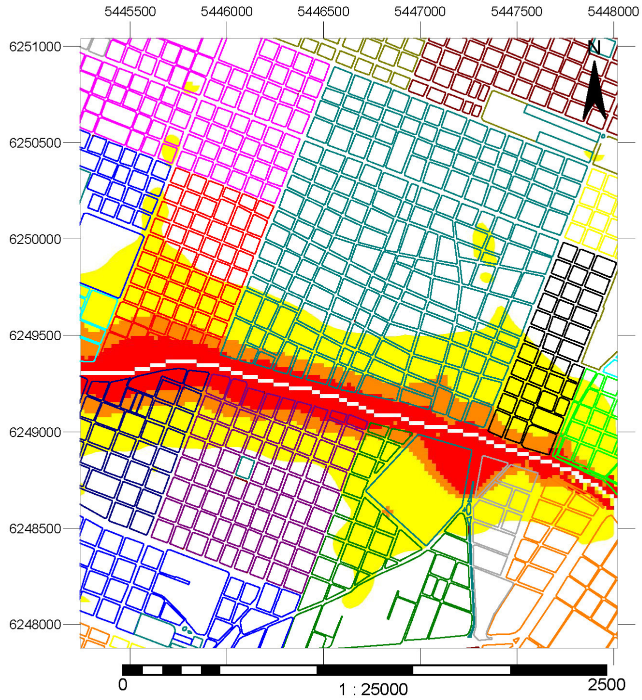
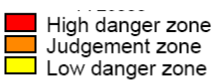


Figure 76 Flood Hazard map for Scenario C (According FEMA classification)



- High danger zone means “almost any size adult is in danger from flood water”,
- Judgment zone means “Danger level is based upon engineering judgment”
- Low danger zone means “Almost any size adult is not seriously threatened by flood water”

Finally, a summary of the areas affected according to the zone identified is presented (Table 18).

Table 18 Areas according to Hazard zone identified

	Area in m2		
	Scenario A	**Scenario B**	**Scenario C**
High danger zone	0	204130	231835
Judgement zone	5480	306440	326120
Low danger zone	2418875	675110	1331530

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

Developing countries need to improve the flood assessment using technical tools as hydrodynamic models, however, the available data is the main constrain when they are applied. The research was done based on the available data in terms of terrain representation and hydrologic features to evaluate the level of flood hazard assessment of Pergamino city as was possible. The conclusions that can be drawn from this study are:

1. Flood characteristics in the Pergamino River using 1D model Hec Ras
What are the main characteristics of the Pergamino stream in terms of water depth and other water dynamics?

The simulations made using Hec Ras model solved satisfactorily the steady flow conditions for Pergamino River. The results in terms of water depth enabled to improve the rating curve (Discharge – High) for the limnimeter at F. Sanchez located in the urban area. This will allow the determination of calibrated discharges values for further assessments in the channel instead of using directly extrapolation from the measured values.

The unsteady flow assessment showed a wave celerity in the river of 1.1 m/s, with travel time equal to 3 hours (between FFCC Mitre Bridge and El Toro). This time is short for an immediate answer from the urban area. The results support the criterion formulated by the municipality project to control the flow volume upstream of the city. Furthermore, an independency of the hydrographs peaks between the limnigraphs stations was found; therefore, further assessments need to consider the independent systems for the analysis. One related to the area upstream from the city and the other one relate to the urban area itself. This might lead to more in-deep hydrological modeling.

2. Flood propagation in Pergamino city using 2D model Flo-2D
What is the flood propagation produced in the urban area of Pergamino for the studied event?

The flood propagation assessment in Pergamino urban area using Flo-2D model considering 10 years as return period gave the extend area affected for three scenarios: 3.5 Km^2 , 1.1 Km^2 and 1.9 Km^2 (flood caused by Rainfall, Overflow in the channel and the combination of the previous ones respectively). The performance of the Flo-2D model presented during the simulations problems related to numerical instability and volume conservation that were overcome through the adjustment of the roughness coefficient in several simulations.

3. Comparison of the model results with information provided in the area
What is the difference between the models results and the information provided in the area?

The comparison between the maximum flow depth results simulated by the model for a past rainfall event (February 9, 2001) and the values recorded by the neighborhood association (CO.S.S.O.PER) showed agreement when the assessment was not considering the average of grid-cells inside the study area but the selection of them in a specific time of the event. However, when the comparison is related to the maximum or minimum value simulated (between cells) the model overestimated and underestimated the depth value.

4. Flood hazard maps in Pergamino city
What is the flood hazard in Pergamino city?

Three flood hazard maps were generated from the simulations results using Flo-2D model, in the case of scenario A, the three hours total rainfall does not generate high level of flood hazard in the city. However, for the case of scenario B and C, the medium level of flood hazard is predominant in the inundation area; high levels of flood hazard are presented in the areas close to the channel and in the Municipality Park.

5.2. Recommendations

The applicability of hydrodynamic models to assess hazard flood in developing areas was accomplished throughout the overcome of several constrains. The flood hazard maps are generated as first approach for the city. Furthermore, from the experience gained during this research some recommendations are formulated below.

Recommendation for the data input in FLO-2D

- All the input data are introduced to Flo-2D using the preprocessor program GDS, however, the consuming time to introduce all the information (e.g. Rainfall, levees, channel data, Cross Section, hydrograph data) become a limitation at the moment to do the adjustments and improvements for next simulations. Therefore, the GDS program is recommended in the first simulation, but then at the moment to do the adjustments, it is better to work directly with the ASCII files and a text editor.

- In Flo-2D, when the simulation represent a combined event of rainfall and discharge with different times of occurrence (e.g. rainfall time shorter than hydrograph time), the model works with a total precipitation expressed in percentage. The rainfall data must be kept in 1.0 until the end of the discharge event; otherwise a volume conservation problem will be faced.

- Before the import process of the cross section from Hec Ras to Flo-2D, all the levees in every cross section must be deleted; because those elements must be introduce to Flo-2D as independent process. The abrupt changes of shape and slope between cross section must be reviewed carefully in order to avoid possible stability problems.

Recommendation for the model validation

- The information collected by CO.S.S.O.PER was useful as first approach and it should be maintained, supported and augmented as an important social service. However, as it was not come from a direct measure the accuracy of the model results could not be demonstrated. Therefore, the validation of the results obtained by Flo-2D model needs to be completed. This requires a systematic and technical system of data collection in terms of water depth and duration of the event need to be carried out by the Municipality under the support of CO.S.S.O.PER.

Recommendation for future studies and continuation of the research

- The lack of cross section data did not allow the use of the data from the limnigraphs stations to generate the hydrographs, then a bathymetry and a topographic survey must be done in the area located between the Alfonzo Station and FF.CC Mitre. Besides that, it is necessary to evaluate the location of both stations in terms of elevation because during fieldwork it was observed the vulnerability of the instruments for water level increase.

- The DTM generated in this research needs to be improved throughout a specific topography survey that consider not only the urban area but also the upstream area. Despite that extending the study area upstream could required more computational time, it would counteract some undesirable effects closer to the area of interest. In this way the instability and inaccuracies are kept away of the modeled area. Negative slopes due to pits or depressions should be strictly avoided in the channel.

- During the simulations performed with Flo-2D, the numerical stability problem was overcome with the modification of the Wavemax coefficient related to the third stability criterion. However, a sensitivity analysis about the first stability criterion Percentage change in flow depth (this study kept the default value in 0.2) can give a second option to evaluate numerical instability problems.

- As next step in the simulations with Flo-2D, the use of Reduction Area and Reduction Width factors must be evaluated. The comparison between the results obtained in this research and new results using those factors can prove or reject their use as alternative to avoid roughness coefficient adjustments throughout the modification of Wavemax coefficient. The use of these alternatives requires a multidisciplinary project to evaluate the influence of the actual structures and buildings in the model.

- As the levees along the channel were introduced as independent element in this study, changes in their grid-cell location and elevation values must be tested in order to evaluate their influence in the channel volume conservation.

Recommendation for Pergamino Municipality

- After this research and the effort of INTA, INA and the Municipality rating curve in F. Sanchez bridge is one of the most reliable information in the system. As such it is strongly suggested that a full hydrometric station is place there. This station would be valuable to understand all the flow mechanism on the Pergamino Stream.

- The existence of a damping effect in Alfonzo station and Del Pescado lagoon area must be considered in the review study of the dams project formulated by the Municipality, in order to take in count the favorable effect in the height design of the dams.

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APPENDICES

Appendix A

FIELDWORK

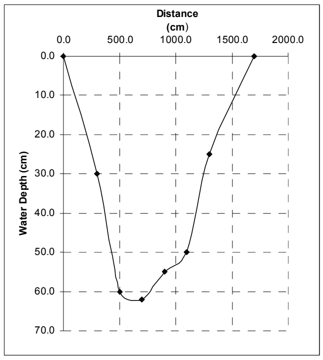
A1. Discharges determination

- Were done three points of the river: Alfonzo Limnigraph Station, Limnimeter F. Sanchez. and Urquiza Limnigraph Station. Furthermore, the water levels were read at the same moment of each measure.

a) Discharge in Alfonzo Limnigraph station

Vertical Number	Distance from base [cm]	Width [cm]	Depth [cm]	Veloc. depth [cm/s]	Area [cm2]	Discharge [m3/s]	Average Velocity [cm/s]	Time [s]	n
1.0	0.0	150.0	0.0	0.0	1125.0	0.0	0.0	0.0	0.0
2.0	300.0	250.0	30.0	7.6	7500.0	0.1	7.6	704.0	0.1
3.0	500.0	200.0	60.0	17.3	12000.0	0.2	17.3	256.0	0.4
4.0	700.0	200.0	62.0	20.0	12400.0	0.2	20.0	217.0	0.5
5.0	900.0	200.0	55.0	19.4	11000.0	0.2	19.4	225.0	0.4
6.0	1100.0	200.0	50.0	13.8	10000.0	0.1	13.8	334.0	0.3
7.0	1300.0	300.0	25.0	13.2	7500.0	0.1	13.2	352.0	0.3
8.0	1700.0	200.0	0.0	0.0	1250.0	0.0	0.0	0.0	0.0

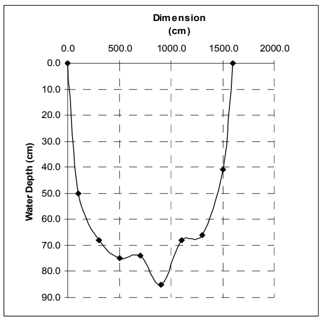
$Q = 0.962\ m^3/s$



b) Discharge in F. Sanchez Limnimeter

Vertical Number	Distance from base [cm]	Width [cm]	Depth [cm]	Veloc. depth [cm/s]	Area [cm2]	Discharge [m3/s]	Average Velocity [cm/s]	Time [s]	n
1.0	0.0	50.0	0.0	0.0	625.0	0.0	0.0	0.0	0.0
2.0	100.0	150.0	50.0	15.0	7500	0.1	15.0	453.0	0.3
3.0	300.0	200.0	68.0	15.7	13600	0.2	15.7	428.0	0.4
4.0	500.0	200.0	75.0	16.1	15000	0.2	16.1	418.0	0.4
5.0	700.0	200.0	74.0	15.8	14800	0.2	15.8	425.0	0.4
6.0	900.0	200.0	85.0	15.7	17000	0.3	15.7	428.0	0.4
7.0	1100.0	200.0	68.0	18.6	13600	0.3	18.6	353.0	0.4
8.0	1300.0	200.0	66.0	15.6	13200	0.2	15.6	432.0	0.3
9.0	1500.0	150.0	41.0	15.0	6150	0.1	15.0	453.0	0.3
10.0	1600.0	50.0	0.0	0.0	513	0.0	0.0	0.0	0.0

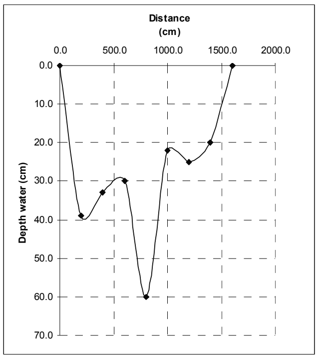
$Q = 1.620\ m^3/s$

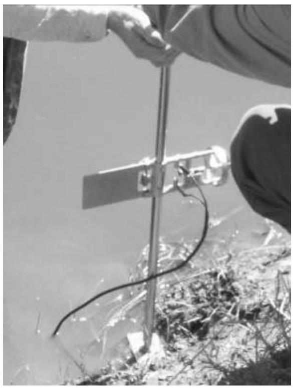


c) Discharge in Urquiza Limnigraph station

Vertical Number	Distance from base [cm]	Width [cm]	Depth [cm]	Veloc. depth [cm/s]	Area [cm2]	Discharge [m3/s]	Average Velocity [cm/s]	Time [s]	n
1.0	0.0	100.0	0.0	0.0	975.0	0.0	0.0	0.0	0.0
2.0	200.0	200.0	39.0	11.6	7800.0	0.1	11.6	408.0	0.2
3.0	400.0	200.0	33.0	18.4	6600.0	0.1	18.4	239.0	0.4
4.0	600.0	200.0	30.0	31.5	6000.0	0.2	31.5	132.0	0.8
5.0	800.0	200.0	60.0	34.7	12000.0	0.4	34.7	119.0	0.8
6.0	1000.0	200.0	22.0	36.2	4400.0	0.2	36.2	114.0	0.9
7.0	1200.0	200.0	25.0	33.4	5000.0	0.2	33.4	124.0	0.8
8.0	1400.0	200.0	20.0	31.1	4000.0	0.1	31.1	134.0	0.7
9.0	1600.0	100.0	0.0	0.0	500.0	0.0	0.0	0.0	0.0

$Q = 1.267\ m^3/s$




Velocimetry


Cross section measurements


Slope measurement

Three cross section were considered for the determination of the roughness coefficient in the area located downstream of the F. Sanchez limnimeter.

A2. Measurements of water level values:

Water level measured by the Alfonzo limnigraph = 0.916 m
Water level measured at the F. Sanchez limnimeter = 0.870 m
Water level measured by the Urquiza limnigraph = 0.636 m


Limnigraph equipment


Record from the limnigraph


Alfonzo Station

During the fieldwork there was evidenced that the Alfonzo Station location were suspect to be under backwater effect. Furthermore, the Urquiza Station location was not working at the moment of the visit because in previous days the water overpass it, as a result all the equipment was affected. To measure the water level in that point was momentarily moved the equipment from Alfonzo Station.

A3. Inspection of the catchment upstream and dowstream of the river.

The basin is large and wide. Upstream of the stream under study, the catchment is a typical lowland-flat prone area with non defined network drainage. The Pergamino rivers is excavated and marked at the urban area. All the channel of the urban area was altered by human activities.
Downstream of the urban area, the catchment changes, even though the river bed is lower, the network drainage is more incised in a natural channel until Urquiza Station. This difference between downstream and upstream is attributing to the Pedalogy (soil study).

A4. Inspection of the river located in the urban area.

The stream corresponding to the urban area was visited; the dimension values of the bridges introduced in Hec Ras model before fieldwork were checked.

A5. Collection of data from the Pergamino Municipality and CO.S.O.PPER.

Two meetings were made with authorities of the Pergamino Municipality and CO.S.O.PPER, to obtain information about historical floods and to complete the data for the simulations.

Floods historical records

Gps measurements

Meeting with CO.S.O.PPER

Furthermore, direct interviews with formers citizen affected for the flooding events in 1995 and 2001 helped to understand not only the technical magnitude but also the social impact in the population.

Appendix B

B1. Alfonzo Station simulation

Q m3/s	Water Surface m	H m
0.1	66.39	0.23
0.25	66.49	0.33
0.5	66.59	0.43
0.75	66.67	0.51
0.962	**66.72**	**0.56**
1	66.73	0.57
5	67.11	0.95
25	67.79	1.63
50	68.44	2.28
100	69.48	3.32
150	69.64	3.48
200	69.78	3.62
250	69.91	3.75
300	70.21	4.05
350	70.74	4.58

Low point Cross section 1 = 66.20 m

Discharge measured in fieldwork= 0.962 m^3/s
Results of the simulation with that discharge:
H=0.56 m

Therefore:

Elevation water simulated= 66.72 m <67.36 m =
Elevation water field

B2.Steady flow assessment

The example of the calculations steps for the table Table 8 is presented below:
The water depth assumed to obtain the discharge value was 0.25 m. Replacing that value in the current rating curve F. Sanchez

$$Q = 13.63949 * H^2 - 2.784557 * H + 0.4895915$$

$$Q = 0.65 \ m^3/s$$

This value was simulated by the model and the result obtained, in terms of water surface, for the cross section corresponding to F. Sanchez Bridge is = 53.48 m
As the lowest elevation point in that cross section is 52.75 m (bottom of the river), the water height (53.48 m – 52.75 m) is 0.73 m.

Before the comparison between the depth values obtained by the model and the values calculated from the rating curve was necessary to consider the limnimeter level. The instrument was not installed in the level 52.75m (bottom of the river) but in 53.48 m, which means that the depth value measured from the limnimeter must be adjusted with a delta height equal to 0.53 m. Therefore, the depth water value considered for the comparison must be 0.78 m (0.25 m + 0.53 m).

Finally both values 0.73 m from the simulation and 0.78 m were plotted to assess the model performance.

B3.Detail of the flood hazard assesed in every cross section

Cross Section	Location X	Location Y	Location Prog	Street	Hazard Water Hight (m)	Hazard Water Level (m)	Dike Side
118	5443837	6249570	0+000		2.0	56.2	Left
117	5443920	6249556	0+085		2.0	55.9	Left
116	5444013	6249563	0+179		2.7	56.7	Both
115	5444054	6249537	0+227		2.6	56.4	Right
114	5444144	6249521	0+318		2.9	57.2	Left
113	5444230	6249501	0+407		2.9	56.7	Right
112	5444307	6249473	0+489		3.0	56.7	Both
111	5444439	6249446	0+622		3.3	56.4	Left
110	5444561	6249423	0+747		3.4	56.6	Left
109	5444604	6249395	0+797		3.0	55.8	Left
108	5444665	6249350	0+876		3.5	56.1	Left
107	5444711	6249352	0+922		3.8	56.5	Left
106	5444856	6249328	1+072		4.1	56.8	Right
105	5444974	6249297	1+195		4.1	56.9	Both
104	5445104	6249320	1+328		4.0	56.5	Left
103	5445229	6249307	1+454		4.1	56.7	Left
102	5445318	6249296	1+543		4.0	56.8	Left
101	5445410	6249294	1+635		4.1	57.2	Both
100	5445501	6249314	1+728		4.1	57.1	Right
99	5445607	6249339	1+837		4.0	57.1	Left
98	5445676	6249355	1+909		4.0	57.1	Right
97	5445806	6249361	2+038		4.0	57.0	Left
96	5445902	6249343	2+137		4.1	57.0	Both
95	5445965	6249328	2+201		4.3	57.1	Right
94	5446032	6249310	2+270	Matheu	4.2	56.9	Right
93	5446111	6249288	2+352	Rivadavia	4.2	56.7	Right
92	5446218	6249261	2+463	Estrada	4.4	56.8	Right
91	5446309	6249239	2+556		4.3	56.7	Both
90	5446385	6249218	2+635	Italia	4.3	56.8	Both
89	5446505	6249186	2+760	9 de Julio	4.2	56.4	Right
88	5446581	6249162	2+839		4.3	56.6	Left
87	5446648	6249140	2+910	San Nicolas	4.8	57.1	Right
86	5446753	6249106	3+020		4.5	56.9	Right
85	5446858	6249079	3+129	25 de Mayo	4.5	56.5	Left
84	5446997	6249045	3+272	Dr. Alem	4.3	56.4	Left
83	5447113	6249014	3+392	Moreno	4.4	56.5	Left
82	5447197	6248993	3+480	Azcuénaga	4.4	56.4	Left
81	5447289	6248967	3+574		4.4	56.4	Left
80	5447351	6248951	3+638		4.3	55.6	Left
79	5447419	6248928	3+710	L. Moreno	4.3	55.5	Left
78	5447510	6248896	3+806	Gonzales	4.8	55.9	Left
77	5447598	6248864	3+899	Menéndez	5.0	56.4	Right
76	5447686	6248832	3+994		5.0	56.4	Left
75	5447755	6248801	4+068	Chile	4.8	56.2	Left
74	5447833	6248759	4+158	Mar del Plata	4.8	56.2	Both
73	5447914	6248712	4+252	Issac E Annan	4.6	55.9	Right
72	5447992	6248661	4+345		4.5	55.8	Right
71	5448039	6248616	4+412				
70	5448072	6248547	4+489		4.6	55.7	Left
69	5448114	6248456	4+588	Gálvez	4.5	55.2	Left
68	5448158	6248364	4+691		4.4	55.5	Right
67	5448232	6248275	4+807	Aº Chu - Chu	4.8	55.7	Right
66	5448255	6248183	4+901		4.5	55.6	Right
65	5448294	6248086	5+005	Ghiraldes	4.4	55.4	Left
64	5448357	6248008	5+186	Colodrero	4.8	55.8	Left
63	5448412	6247949	5+261		4.3	55.3	Right
62	5448468	6247902	5+383		3.9	54.9	Right
61	5448566	6247829	5+467		3.3	54.3	Right
60	5448620	6247763	5+567		3.0	53.6	Left

Cross	Location			Hazard	Hazard	Dike
Section	X	Y	Prog	Water Level (m)	Water Level (m)	Side
59	5448677	6247681	5+667	3.4	54.2	Right
58	5448736	6247601	5+768	4.6	55.3	Right
57	5448803	6247526	5+868	3.8	54.5	Right
56	5448865	6247447	5+938	3.7	54.4	Right
55	5448893	6247382	6+010	4.2	54.9	Right
54	5448904	6247313	6+094	2.5	53.0	Left
52	5448928	6247231	6+094	2.6	53.7	Left
51	5449018	6247124	6+233	3.5	53.9	Left
50	5449086	6247052	6+333	3.2	53.5	Left
49	5449144	6246970	6+435	2.5	52.7	Left
48	5449174	6246826	6+580	3.6	53.3	Right
47	5449205	6246766	6+646	3.6	53.4	Both
46	5449256	6246679	6+747	2.6	52.3	Left
45	5449286	6246593	6+839	3.1	52.8	Left
44	5449312	6246501	6+935	3.0	52.7	Left
43	5449356	6246393	7+051	2.7	52.2	Left
42	5449386	6246299	7+149	3.8	53.3	Left
41	5449395	6246167	7+282	2.9	52.3	Left
40	5449492	6246014	7+465	2.9	52.3	Left
39	5449503	6245924	7+555			
38	5449553	6245842	7+652	3.3	52.7	Left
37	5449579	6245706	7+790	4.5	54.0	Left
36	5449592	6245634	7+864	4.0	53.5	Right
35	5449666	6245499	8+017	4.0	53.4	Left
34	5449680	6245414	8+103	4.2	53.5	Right
33	5449692	6245325	8+194			
32	5449741	6245221	8+307	3.0	52.3	Right
31	5449766	6245125	8+407	4.0	53.4	Both
30	5449795	6245024	8+511	4.0	53.4	Left
29	5449809	6244875	8+661	3.5	52.7	Left
28	5449867	6244752	8+798	2.9	52.0	Left
27	5449943	6244690	8+896	3.0	52.1	Left
26	5450016	6244605	9+007	3.3	52.4	Left
25	5450031	6244508	9+108	3.5	52.4	Left
24	5450059	6244392	9+227	2.8	51.8	Right
23	5450101	6244307	9+321	2.5	51.3	Right
22	5450153	6244261	9+390	2.5	51.2	Both
21	5450198	6244150	9+512	2.5	51.3	Both
20	5450242	6244058	9+612	2.5	51.3	Left
19	5450290	6243973	9+710	3.4	52.2	Left
18	5450353	6243864	9+836	3.6	52.3	Left
17	5450352	6243713	9+989	2.5	51.1	Left
16	5450421	6243610	10+112	3.4	51.9	Left
15	5450501	6243489	10+257	3.4	51.8	Left
14	5450551	6243406	10+355	3.4	52.0	Left
13	5450605	6243313	10+460	3.8	52.4	Both
12	5450653	6243240	10+548	3.6	52.2	Right
11	5450705	6243154	10+649	3.3	51.7	Right
10	5450790	6243104	10+747	2.7	51.2	Left
9	5450826	6243047	10+813	3.3	51.9	Right
8	5450874	6242980	10+895	3.5	51.9	Right
7	5450939	6242895	11+005	3.1	51.4	Right
6	5450995	6242806	11+108	2.7	51.0	Right
5	5451091	6242691	11+257	3.2	51.3	Right
4	5451159	6242599	11+373	2.5	50.3	Right
3	5451259	6242488	11+523	2.5	49.9	Right
2	5451321	6242397	11+632	2.6	50.5	Right
1	5451386	6242333	11+724	2.5	50.3	Right
0	5451434	6242285	11+792	2.5	50.3	Right

Water level that caused flood hazard in the bridges

Bridge	Water depth m	Water level m
F. Sanchez	3.9	56.7
Colon	3.8	56.6
La Merced	3.5	55.8
Rocha	3.5	54.8
Ruta 8	4.0	55.3

B4.Results from sensitivy analysis performed by Hec Ras

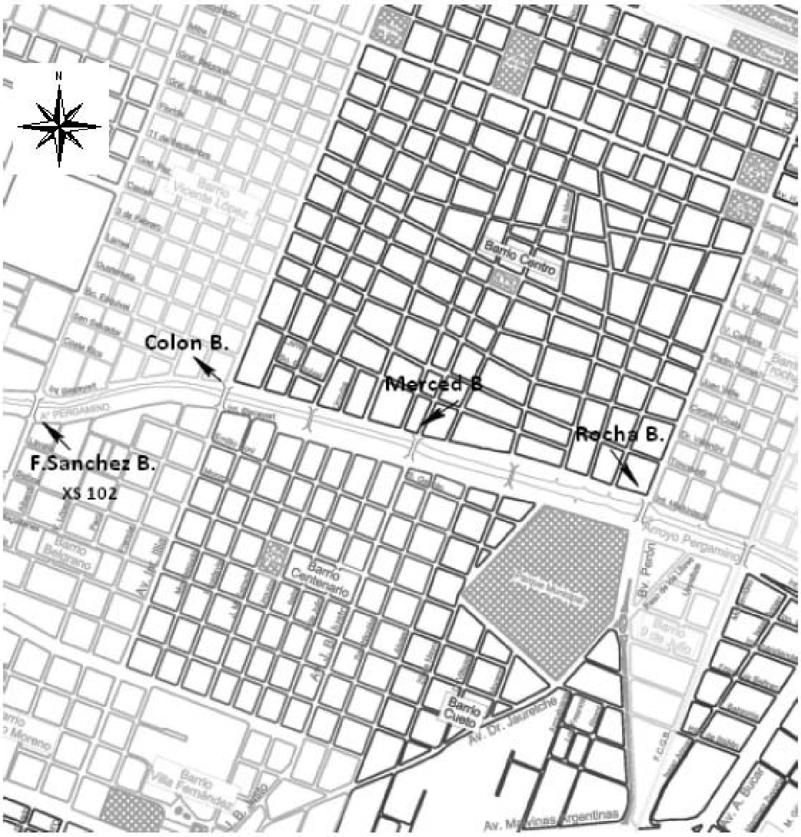
Discharge m3/s	Water depth Rating curve F.Sanchez	Water depth results different roughness coeficient			
		0.03	0.025	0.04	0.05
0.65	0.78	0.73	0.61	0.67	0.71
6.99	1.33	1.30	1.13	1.30	1.40
11.34	1.53	1.51	1.32	1.54	1.67
27.00	2.03	2.03	1.83	2.17	2.37
49.48	2.53	2.58	2.36	2.80	3.03
78.78	3.03	3.10	2.88	3.35	3.58
114.89	3.53	3.59	3.38	3.88	4.13
157.83	4.03	4.14	3.90	4.48	4.78
207.58	4.53	4.80	4.50	5.25	5.65
264.16	5.03	5.71	5.31	5.92	5.99
276.29	5.13	5.93	5.50	5.98	6.04
301.38	5.33	6.04	5.97	6.07	6.14
327.55	5.53	6.13	6.11	6.16	6.28

B5.Determination of the rainfall percentage (Input data for Flo2D simulations)

Interval [hs] 0.083

Time Acum hs	Intensity [mm/h]	Precipitation Interval [mm]	Total [mm]	Per. Inter %	Per. Acum %
0.00	0.00	0.00	0.00	0.00	0.00
0.08	6.54	0.54	0.54	0.01	0.01
0.17	7.30	0.61	1.15	0.01	0.02
0.25	8.27	0.69	1.84	0.01	0.02
0.33	9.55	0.80	2.64	0.01	0.03
0.42	11.30	0.94	3.58	0.01	0.05
0.50	13.83	1.15	4.73	0.02	0.06
0.58	17.77	1.48	6.21	0.02	0.08
0.67	24.63	2.05	8.27	0.03	0.11
0.75	38.95	3.25	11.51	0.04	0.15
0.83	82.21	6.85	18.36	0.09	0.24
0.92	172.45	14.37	32.73	0.19	0.43
1.00	105.23	8.77	41.50	0.11	0.54
1.08	68.48	5.71	47.21	0.07	0.62
1.17	49.52	4.13	51.34	0.05	0.67
1.25	38.24	3.19	54.52	0.04	0.71
1.33	30.89	2.57	57.10	0.03	0.75
1.42	25.78	2.15	59.24	0.03	0.78
1.50	22.04	1.84	61.08	0.02	0.80
1.58	19.21	1.60	62.68	0.02	0.82
1.67	17.00	1.42	64.10	0.02	0.84
1.75	15.23	1.27	65.37	0.02	0.86
1.83	13.79	1.15	66.52	0.02	0.87
1.92	12.59	1.05	67.57	0.01	0.88
2.00	11.58	0.96	68.53	0.01	0.90
2.08	10.72	0.89	69.42	0.01	0.91
2.17	9.98	0.83	70.26	0.01	0.92
2.25	9.33	0.78	71.03	0.01	0.93
2.33	8.76	0.73	71.76	0.01	0.94
2.42	8.26	0.69	72.45	0.01	0.95
2.50	7.82	0.65	73.10	0.01	0.96
2.58	7.42	0.62	73.72	0.01	0.96
2.67	7.06	0.59	74.31	0.01	0.97
2.75	6.73	0.56	74.87	0.01	0.98
2.83	6.43	0.54	75.41	0.01	0.99
2.92	6.16	0.51	75.92	0.01	0.99
3.00	5.92	0.49	76.41	0.01	1.00
Total		**76.41**		1.00	

B6.Pergamino Street map with the location of the bridges



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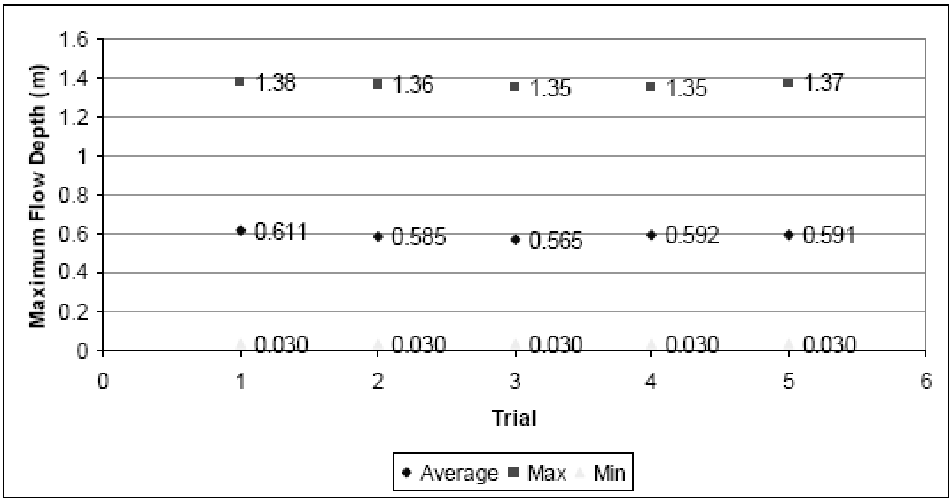
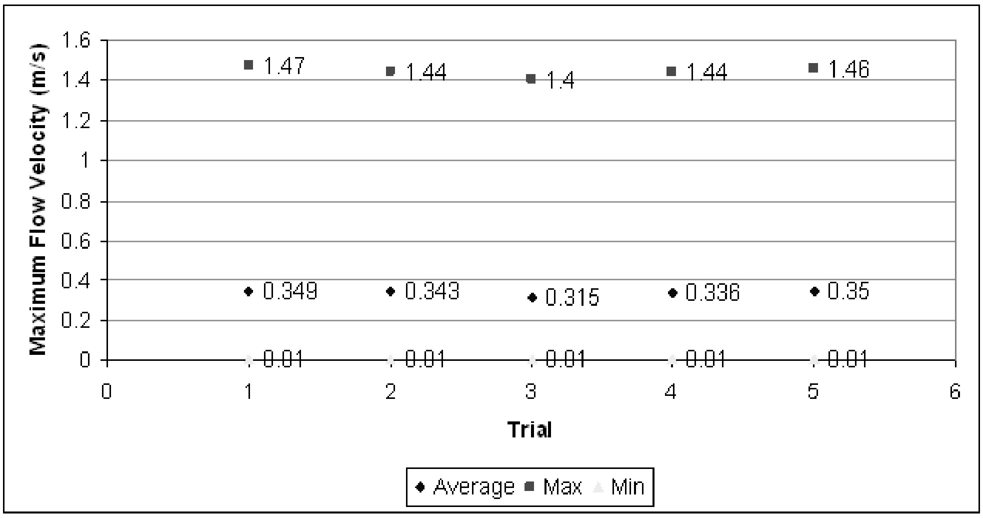
Source: Pergamino Munipality,2009

B7.Comparison between the results obtained with Flo-2D model

In order to see the influence in the adjustments of the roughness coefficient during the trials simulations, the maximum flow velocity and maximum flow depth for the Scenario A were exported into Ilwis. Then the average, maximum, minimum and predominant values were calculated. The comparison of those results, for the scenario A, are presented:



Comparison between Maximum Flow Depth in the floodplain obtained from the trials simulated in Scenario A



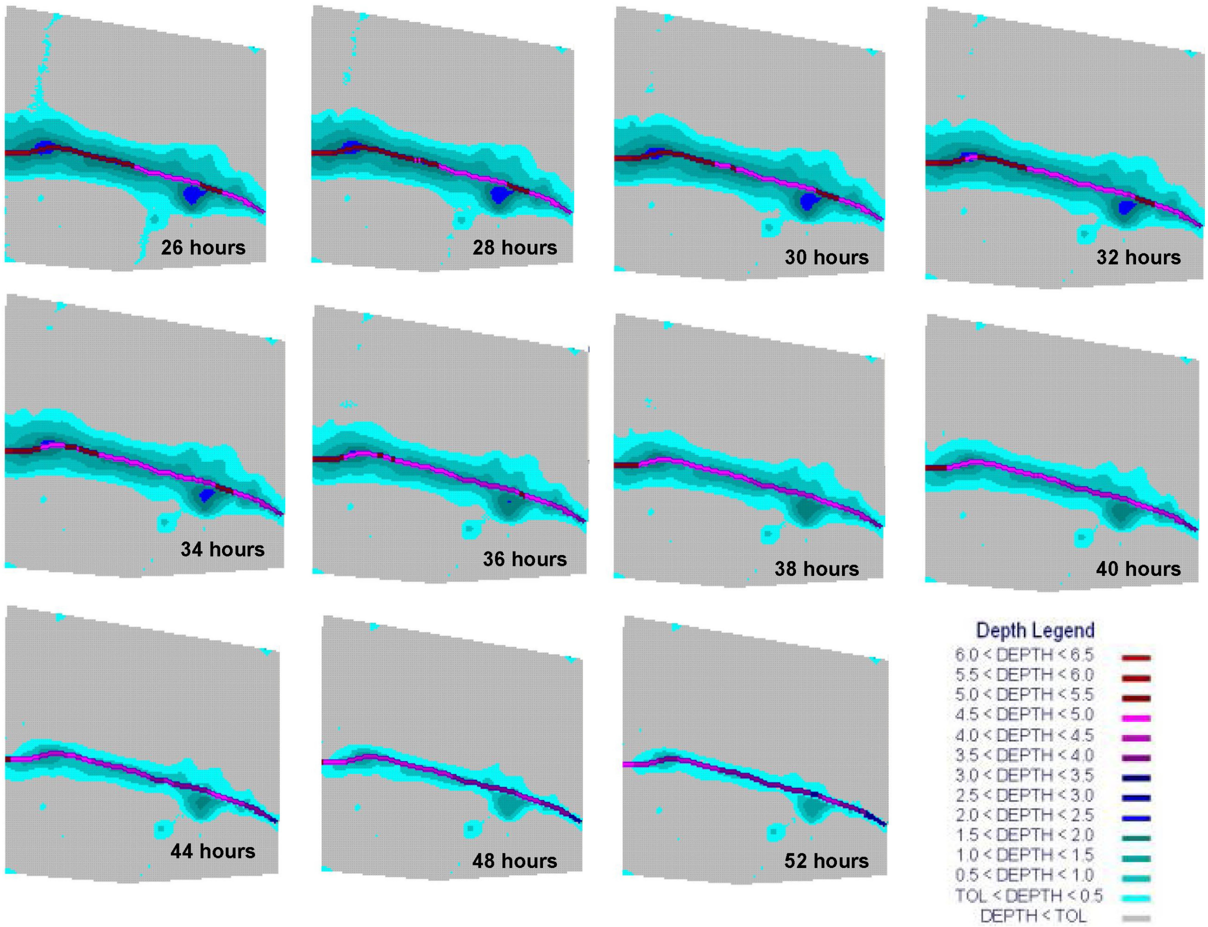
Comparison between Maximum Flow Velocity in the floodplain obtained from the trials simulated in Scenario A

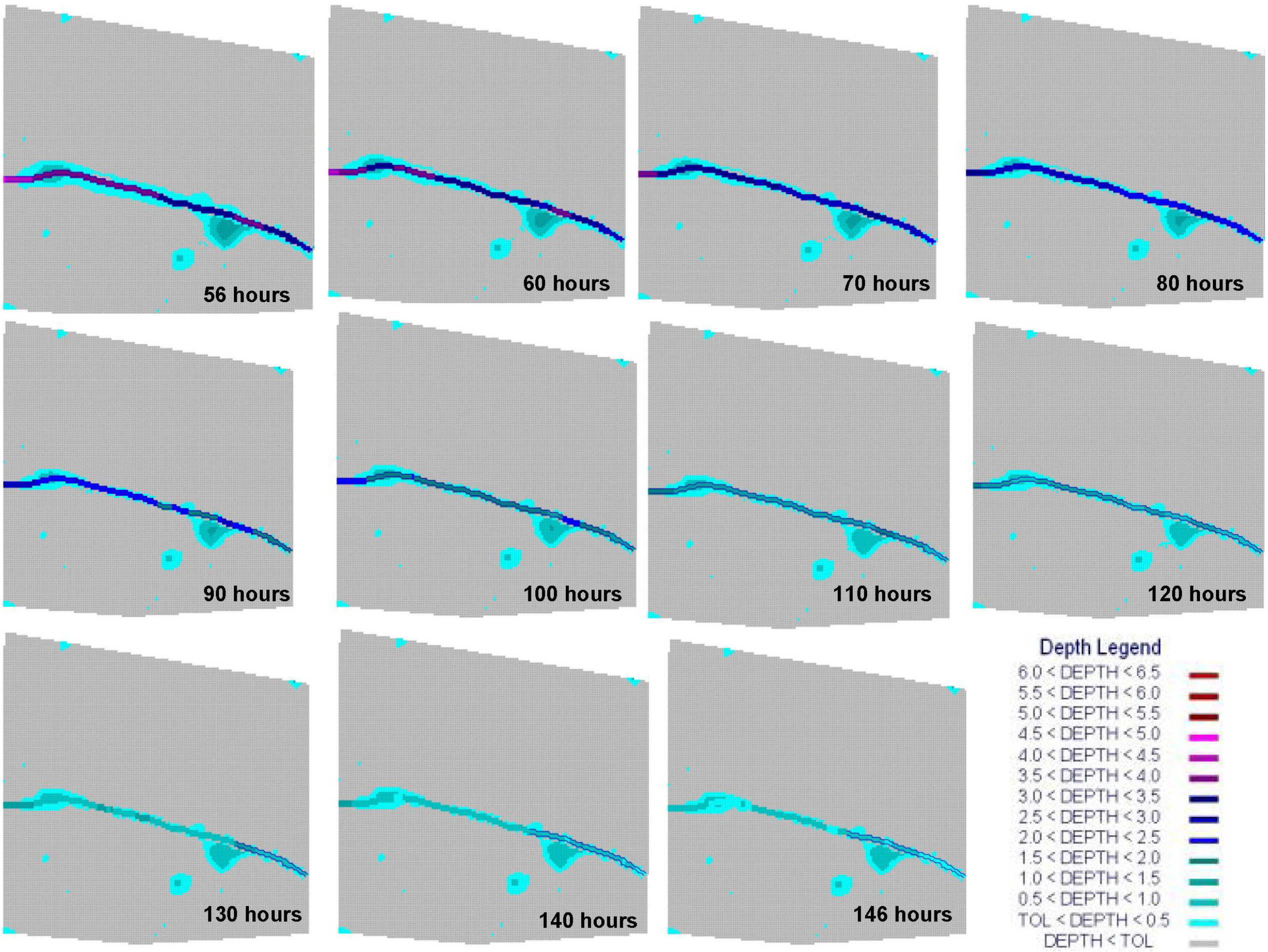
In both cases the minimum value did not change through the simulations, in the three first trials performed with the Wavemax -0.250 was observed a decrease in the values, however, as soon as the model performed with a positive Wavemax coefficient (100.25 and 0.25 respectively) the values increase. The figures above apparently show that the final simulation gave similar to the first trial, however, the adjustment influence is observed in the variation of the run time and the inundation area:

Comparison between the results from the trials in scenario A

Name	Run time (hr)	Inundation area (m^2)
Rain	0.049	3503976
Rainv2	0.049	3503744
Rainv3	0.048	3503899
Rainv4	0.045	3503721
Rainv5	3.952	3503738

B8. Flooding propagation scenario C





B9. Flood hazard map by FEMA

The follow equations were calculated in base on the graph presented by FEMA (Figure 37)

Low danger zone under this curve:

$$\text{Depth} = 3.2677*0.3048*e^{-0.36377685*V/0.3048}$$

Judgment zone under this curve:

$$\text{Depth} = 0.3048/(0.21943529+0.079357487*(V/0.3048)^{\wedge}2.480132)$$

Then the follow relation was used in Ilwis process to generate the maps:

Flood_hazard: iff(d1<depth1,"Low danger zone",iff(d1<depth2,"Judgement zone","High danger zone"))

The areas affected by district area presented below:

Scenario A	High danger zone	Judgement zone	Low danger zone
District	Area affected m2		
12 de Octubre			105850
27 de Noviembre			45050
9 de Julio			32813
Belgrano		350	188695
Centenario			253130
Centro			758350
Cueto		20	191580
Desiderio de la Fuente			32500
Kennedy			40120
Mariano Moreno			10615
Martín Illia			98240
Martín M. de Güemes			19240
Trocha			180020
Vicente Lopez		4160	246925
Villa Fernández			74110
Villa Progreso			106
Calles			6840
FFCC			2390
Plazas Parques		955	132310

Scenario B	High danger zone	Judgement zone	Low danger zone
District	Area affected m2		
Barrio 12 de Octubre			6680
Barrio 27 de Noviembre	3750	10175	45470
Barrio 9 de Julio	11230	10915	28990
Barrio Belgrano	52265	41510	50100
Barrio Centenario	44820	41795	68100
Barrio Centro	10370	94420	180200
Barrio Cueto	1160	9715	26790
Barrio Kennedy	544	1840	22055
Barrio Martín M. de Güemes	1005	13625	30490
Barrio Trocha	205	18820	68815
Barrio Vicente Lopez	39825	43200	82725
Calles		20430	465
Plazas Parques	38955		64245

Scenario C	High danger zone	Judgement zone	Low danger zone
District	Area affected m2		
Barrio 12 de Octubre			53730
Barrio 27 de Noviembre	6305	9800	74740
Barrio 9 de Julio	11630	12775	49860
Barrio Belgrano	52925	40850	95450
Barrio Centenario	48725	43800	121075
Barrio Centro	21675	103045	320375
Barrio Cueto	1795	10065	129200
Barrio Kennedy	1410	2665	57570
Barrio Mariano Moreno			1155
Barrio Martín Illia			11800
Barrio Martín M. de Güemes	2190	14350	29595
Barrio Trocha	855	23860	102845
Barrio Vicente Lopez	42725	43770	147590
Calles			9845
FFCC			1400
Plazas Parques	41595	21150	125305