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Design hydrological event and routing scheme for flood mapping in urban area

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Abstract: *Design hydrological event and routing scheme for flood mapping in urban area.* Definition of flood risk maps is a task to which modern surface hydrology addresses a substantial research effort. Their impact on the government of the flood prone areas have increased the need for better investigation of the inundation dynamics [Fema 2002]. This identifies open research problems such as: the definition of the design hydrograph, the identification of the surface boundary conditions for the flood routing over the inundation plan, the choice of the hydraulic model that is the most close to the physical behaviour of the flood routing in the specific environment, such as urban areas or river valley. Most of academic and commercial mathematical models resolving the De Saint Venant equations in mono or bidimensional approach, fail on complex topography. Steep slopes, geometric discontinuities, mixed flow regimes, initially dry areas are just the main problems an hydraulic model should solve. In this study, we address two points: the definition of the critical event for an inundation area and a flood routing modelling technique for a highly urbanized flat area. For this latter we show that, in urban areas, a modelling scheme of a network of connected channels and storages, gives a better representation of surface boundary conditions such as aggregation of buildings and road network and sufficient accuracy for flood risk mapping purpose respect to a real 2-D hydraulic routing model.

Key words: design hydrograph, distributed model, flood hazard maps, quasi-2D model, urban flood.

INTRODUCTION

Flooding is one of the most common environmental hazard, due to the widespread geographical distribution of river valleys and the attraction of human settlements to these areas.

Floods can be generally considered in two categories [Castelli. 1994]: flash floods, the product of heavy localized precipitation in a short time period over a given location, and general floods, caused by precipitation over a longer time period and over a given river basin.

Although flash flooding occurs often along mountain streams, it is also common in urban areas where much of the ground is covered by impervious surfaces. Fixed drainage channels in urban areas may be unable to contain the runoff that is generated by relatively small, but intense, rainfall events.

Many modelling approaches exist to simulate floods [Leopardi et al. 2002]. The choice of the simulation approach depends on the questions to be answered [Horrit and Bates 2001, Ferrante et al. 2000]. In this work we focus on determination of flood hazard map for an urban area located in the Liguria province in Italy. Traditional approach is based on determination of flood extent for a given frequency event, using steady flow 1-dimensional model. A new rule has been recently introduced for highly urbanized area; it identifies the hazard according to both flow depth and flow velocity, claiming the necessity to use more accurate unsteady flow numerical models.

The aim of the work is to assess the accuracy of a quasi-2D model versus pure 2D models for flood prediction in urban area.

THE CASE STUDY

The study area is located in the western Italy, in the province of Liguria (Fig. 1). The area approaches the sea and has an extension of about 1.8 km^2 in which 6 rivers are encountered: Gorleri, Varcavello, S. Pietro, Pineta, Rodine and Madonna. The drainage basin ranges from 0.32 km^2 of the river Rodine to 18.05 km^2 of the river S. Pietro. The maximum elevation is reached in the Evigno Peak at 988.5 m. a.s.l.

The morphology is characterized by steep slopes in the upper part of the basins, decreasing while approaching the sea. The restricted coastal plain has attracted tourism activity with the effect of an exponential growth of urban density. As a consequence the rivers often have been channelized into artificial drainage. In this situation, we can frequently observe, during storm event, water overtopping the levees and flowing into main roads and districts, dragging people and cars. The target area is crossed by a railway that divides it in two parts: the upper with a slope of about 1.5% and the lower with an average slope of about 0.8%.

THE DIRECTIVES FOR BASIN PLANNING

For the purpose of land use planning and management, flood risk maps are required. The Italian legislation leave the task of the assessment of flood risk maps to the River Basin Authority with

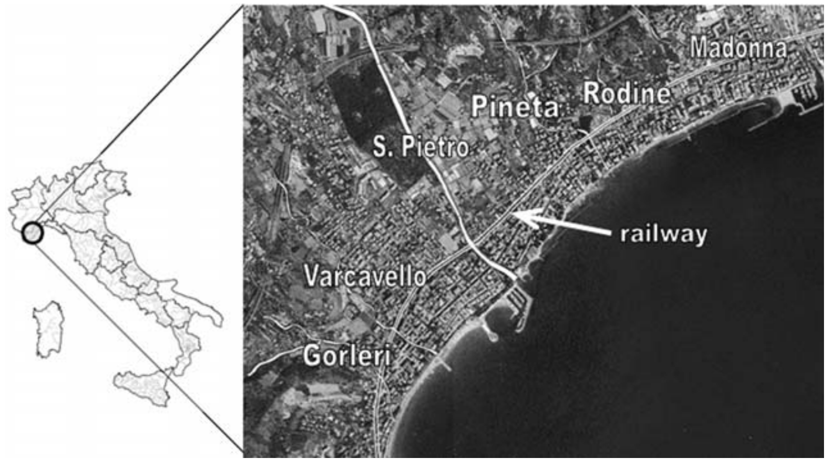

FIGURE 1. Aerial photo of the study area. The six rivers and the railway are visible

the Basin Plan study. The traditional approach simulates the inundation processes with a steady 1-dimensional model. The analysis is iterated for 50 and 200 year return period peak discharge. The 50 year flood extent is classified as class-A hazard and 200 year flood extent is classified as class-B hazard (Fig. 2a). Class-A areas are subject to more restrictive rules than class-B areas.

A new scheme recently proposes [FEMA, 2002; Rosso 2003] to determine the hazard map on the basis of both hydraulic depth and flow velocity. According to this scheme, if an area is frequently flooded but with low depth and low velocity, the level of hazard is reduced (Fig. 2b). Moreover a new hazard class is introduced: according to the new scheme, an area can be classified as class-A, class-B and class-B0 hazard. The class-B0 area is the less restrictive one.

The necessity to employ flow velocity data, requires the use of more sophisticated hydraulic model, with the ability to simulate unsteady flow in complex urban terrain.

THE DESIGN STORM HYDROGRAPHS FOR INUDATION MAPS

There are many types of design hydrographs that have been developed over the years [Maidment 1993; Chow et al. 1988], and the debate is open on the frequencies of the flood map due to the difference between the frequencies of the peak discharge and the hydrograph volume [De Michele et. al. 2005; Salvadori and De Michele 2004]. Moreover, if a rainfall runoff model is used for hydrograph computation, the hypothesis of the equivalence among the return period of the peak discharge and the rainfall is false too.

Inundation maps are generally computed using peak discharge for given return period as input variable and simple steady 1-dimensional analysis

a b

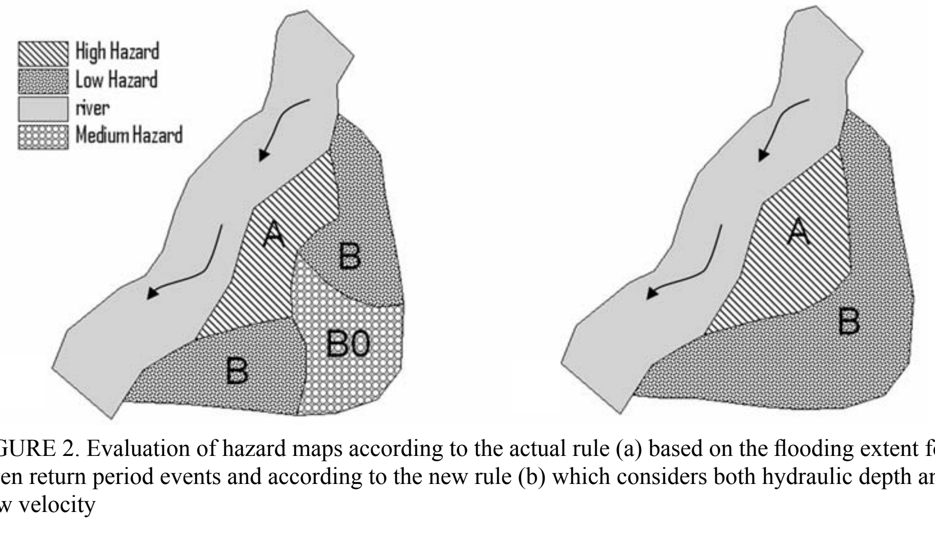

FIGURE 2. Evaluation of hazard maps according to the actual rule (a) based on the flooding extent for given return period events and according to the new rule (b) which considers both hydraulic depth and flow velocity

for the river channel, that means that the hydrograph volume and the routing on the inundated areas are not considered. As well known, especially for plain urban area, the storaging volume can significantly affects the routing and the extension of the inundated areas.

According to this we think that it is more correct, in the assessing of inundation map, to take into account the inundation volume and its statistics as the key variable for map characterization.

For ungaged basin, when the hydrograph is determined from rainfall-runoff transformation, we propose a methodology for the inundation volume definition based on research of the critical rainfall event for an inundation area. This is defined as the one that gives the maximum value of inundation volume for a given return period of the rainfall in the hypothesis that the intensity duration frequency (IDF) curve represents the rainfall behaviour. The maximum inundation volume is defined as the maximum value of the integral of the difference between the incoming hydrograph and the bankfull discharge.

For this purpose, rainfall runoff distributed model, FEST [Mancini 1990; Montaldo et al. 2002; Rulli and Rosso 2002; Montaldo et al. 2003; Montaldo et al. 2004; Salandin et al. 2004], was employed. FEST is a distributed hydrologic model especially developed at the Politecnico di Milano focusing on flash flood event simulation. As a distributed model, FEST can manage heterogeneity in hillslope and drainage network morphology (slope, roughness, etc..) and land use [Rosso 1994].

From a family of IDF curves it is possible to obtain an hydrograph for any duration at a given frequency and so a series of hydrographs for every return period (Fig. 3). Given the bankfull discharge for the examined river branch, the hydrograph that presents the maximum inundation volume identifies the critical rainfall event (Fig. 4) for the

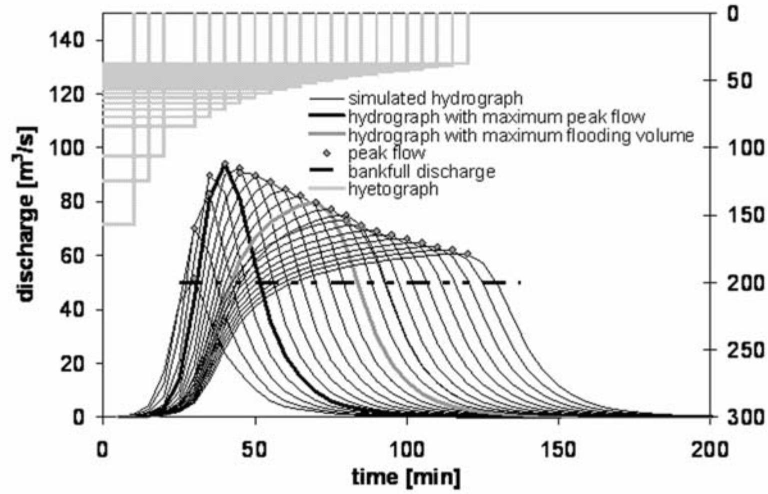

FIGURE 3. Procedure for the search of the critical event defined as that event which is characterized by the maximum potential inundation volume. The results refer to river Varcavello for the 50 years return period

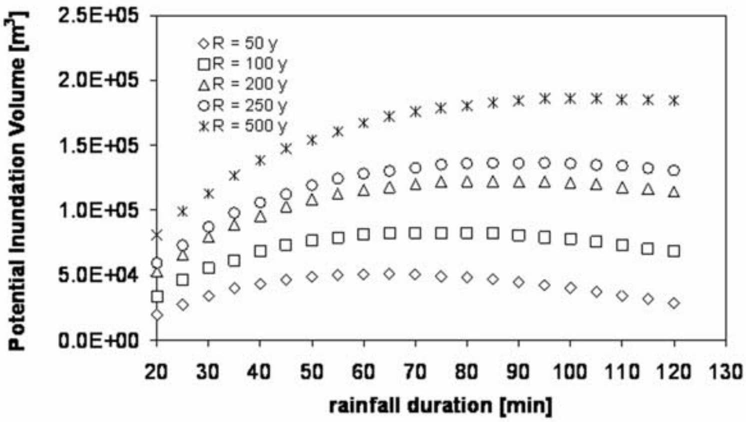

FIGURE 4. Series of potential inundation volume for different return period. The design hydrograph for a given return period is the one characterized by the maximum value of the potential flooding volume

inundation map. In the end, according to retention pool analysis [Artina et al.1997], the critical event for a flood mapping is different from the critical event for the maximum peak discharge (Fig. 3) and the correlated hydrograph peak presents different return period for a given rainfall frequency (Tab. 1) .

DESCRIPTION OF THE HYDRAULIC MODEL

Urban areas are usually characterized by streets and aggregation of buildings (in the following termed blocks), that can be schematized, from a flood routing point of view, as network of channels where the flow velocity is greater than zero, and storages where the velocity is about zero. This latter hypothesis derives from the consideration on the friction induced by the macro roughness of the manmade obstacles present in a block. So that the implemented network model presents three main unit: the main rivers, the channels along the main streets and the storages for the aggregation of buildings.

De Saint Venant equations are then integrated along the river branches and street channels using the Preissman implicit numerical scheme [Wallingford Software 2005]. Energy and continuity equation is verified at nodes given by the channel intersection. Water discharge can flow in every direction in the channel network according to the hydraulic gradient. Reservoir equation is used to model diffusion in the blocks.

The connection between storages and channels are discretized in the specific nodes and the weir equation controls the water flux to and from the reservoir according to the difference of water level.

The river branches feed the channel network where the cross sections are insufficient respect to the flood discharge. When the riverbanks are overtopped, water enters the street channels.

The channel network model is a very good representation of urban flood routing when the hydraulic depth

TABLE 1. Comparison between maximum peak discharge and peak discharge of the hydrograph with the maximum inundation volume computed for a given rainfall frequency. In the last column the return period of the peak discharge of the hydrograph with the maximum inundation volume is reported

Rainfall return period (years)	Maximum peak discharge (m^3/s)	Peak discharge of the hydrograph with the maximum inundation volume (m^3/s)	Return period of the peak discharge of the hydrograph with the maximum inundation volume (years)
50	94.04	79.82	25.5
100	111.75	86.37	38
200	132.82	95.61	54
500	161.24	99.7	64.5

and energy is lower than the height of the surrounding buildings which act as impervious boundary elements.

The described channel network model was implemented (Fig. 5) in the Infoworks-CS software [Wallingford Software 2005]. Rivers and main roads are represented by conduits. Manholes allow the exchange of water between river and roads and are used to represent crossroads. Conduits are linked with storages through weirs.

The main model parameters are the roughness coefficient which controls flow velocity in the channels and the weir discharge coefficient which controls the amount of water exchanged between channels and storages. The adopted set of values are reported in Table 2.

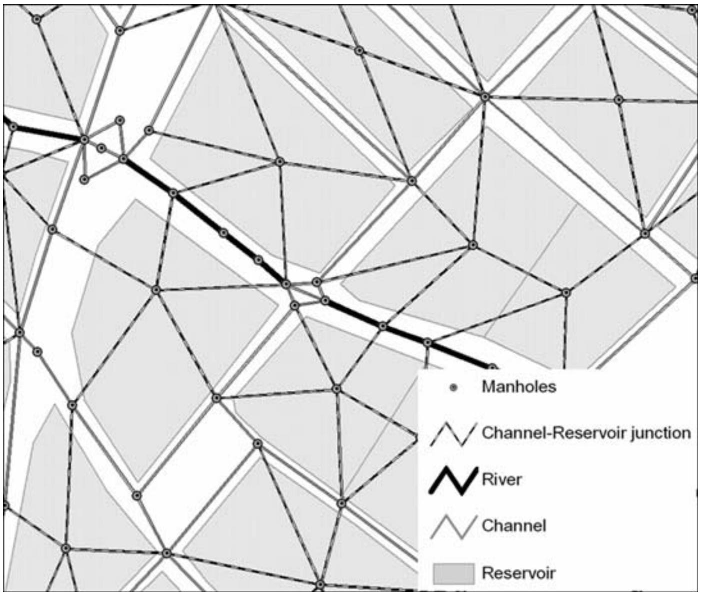

FIGURE 5. Network quasi-2D hydraulic model representing urban area: detail of the river Varcavello

TABLE 2. Parameters describing components of the network quasi-2D model

Parameter	**Value**
Natural river channel Strickler roughness	30 $m^{1/3}s^{-1}$
Concrete river channel Strickler roughness	50 $m^{1/3}s^{-1}$
Not asphalted street Strickler roughness	30 $m^{1/3}s^{-1}$
Asphalted street Strickler roughness	50 $m^{1/3}s^{-1}$
Weir discharge coefficient for districts upstream the railway [(a)]	0.1
Weir discharge coefficient for districts downstream the railway [(a)]	0.28
Weir length for districts upstream the railway [(a)]	50 m
Weir length districts downstream the railway [(a)]	20 m

[(a)] Weir formula adopted for discharge computation in the model: $Q = C_d B D_u \sqrt{g(D_u - D_d)}$, where Q is the discharge [m^3/s], Cd is the discharge coefficient, B is the width of the weir [m], Du is the upstream depth with respect to the crest [m], Dd is the downstream depth with respect to the crest [m] and g is the acceleration due to gravity [m/s^2].

THESIS VERIFICATION USING 2-DIMENSIONAL MODEL

The basic assumption of the network model is that velocity of flood flow over the urban area is greater than zero in the main streets while, in the blocks, velocity can be neglected. To verify this assumption, we analysed flood dynamic in the blocks, by means of a high resolution 2-D model. A subset of urban land in proximity to river Varcavello, has been extracted from the main domain. It is composed by two main blocks: one upstream the railway and the other downstream. The upper one is characterized by an average slope of about 1.5%, the lower by a slope of about 0.8%. These subsets are considered to be representative of the whole study area. A full 2-D model was implemented using the SMS software [www.bossintl.com] (Fig. 6). A steady analysis has been performed. Hydraulic depths deriving from channel network simulation have been taken as boundary condition of the blocks.

Buildings have been modelled in two different ways: as impervious area (buildings are excluded from the model domain) and as a high roughness surface (Strickler coefficient equal to 0.01 $m^{1/3}s^{-1}$). The latter way means that water is free to move in the whole domain, but the flow through buildings is made difficult because of an high value of roughness parameter. Two cases for the Strickler coefficient for gardens surrounding buildings were considered: in the first case its value was fixed to 5 $m^{1/3}s^{-1}$and in the second case to 10 $m^{1/3}s^{-1}$.

Velocity field deriving from simulations has been mapped to a regular grid and the cumulative frequency of the velocity values has been evaluated (Fig. 7). In the upstream block (Fig. 7a), we can note that most of the cells (67%) has a value of velocity less than 0.6 m/s even for the simulation with Strickler coefficient for gardens equal to 10 $m^{1/3}s^{-1}$ and impervious buildings.

In the other simulations, the percentage of cells with velocity under 0.6 m/s increases nearly to 100%. Both

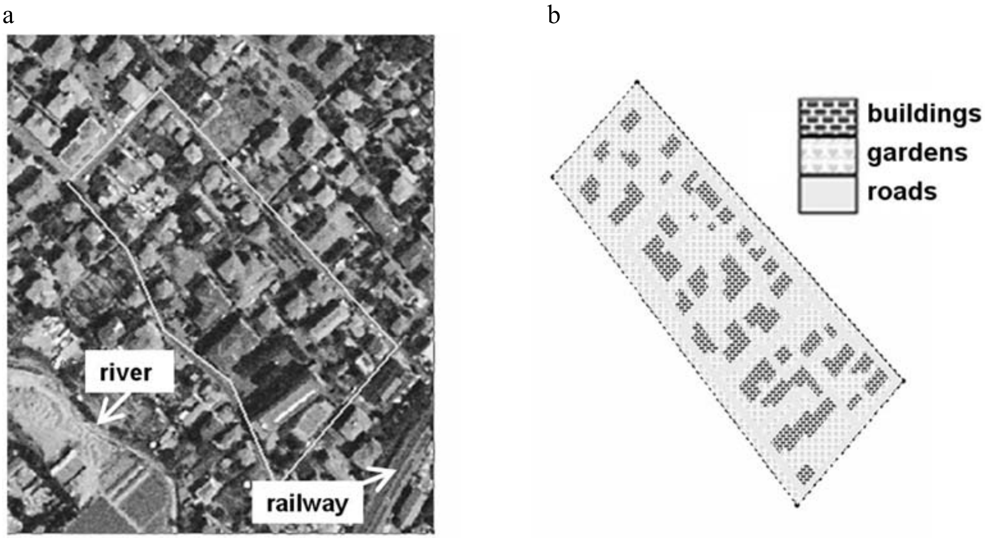

FIGURE 6. Subset of the study area (a) near river Varcavello upstream the railway and (b) representation in the 2-D model

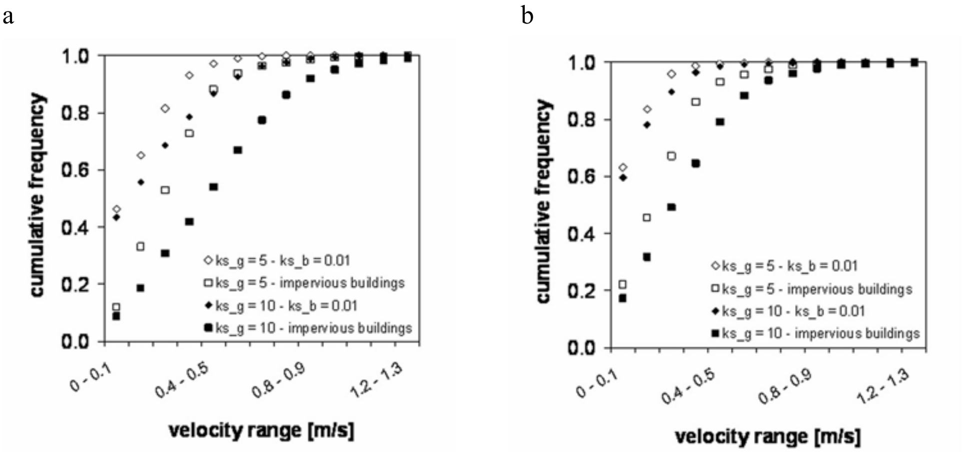

FIGURE 7. Cumulative frequency of velocity magnitude simulated by means of full 2D model in (a) upstream district and (b) downstream district; ks_g and ks_b denote, respectively, the Strickler roughness coefficient for gardens and buildings

considering buildings impervious or as high roughness surface, the peak flow velocity reaches, anyway, a maximum value of 1.3 m/s in just a few cells.

The observations are valid for the downstream block (Fig. 7b) too. As a consequence of a milder slope, flow velocity is lower indeed.

We can conclude that motion of water in the aggregation of buildings is characterized by low values of velocity. The assumption to simulate blocks as storages in the channel network model seems reasonable.

THE FLOOD HAZARD MAPS

Flow velocity and hydraulic depth values, computed respectively in the channels and in the nodes of the network model, have been interpolated over the entire study area to obtain a continuous map. For this purpose the borders of districts have been considered as barriers. The overlay of the velocity with the depth map, produced the hazard map (Fig. 8), according to the new directive of the River Basin Authority (§ 3).

A comparison is made with the previous study performed by Basin Authority [Regione Liguria] which made use of steady flow computation model for delimitation of flood extent, not regarding flow velocity.

Total amount of flooded area has increased in this new study (Fig. 9) from 0.97 to 1.27 km^2. The class-A areas and class-B areas have decreased, respectively from 0.56 to 0.24 km^2 and from 0.42 to 0.16 km^2. The complementary area is included in the class-B0 area which was not defined in the previous analysis.

The differences in the framework of the present study respect to the previous one, have counterpart in the resulting flooding map (Fig. 8). In the network quasi-2D model, overtopping water is routed along main roads as far as distant blocks. This is good explanation for those areas that, in the previous study, are not even water logged by 200 year flood event: the pure 1D steady model can't predict flood processes in urban area.

The new framework can also predict water logging due to manmade obstacles orthogonal to the flow direction. The railway divides the city on north-south direction and behaves as an impervious levee to water flow. This does not seem to be completely represented by the 1D model.

CONCLUSIONS

A procedure is presented for estimation of design hydrographs. It is based on the search for the critical event which maximizes the potential flooding volume. The iterative process makes use of the FEST model for the rainfall-runoff transformation. The distributed model permits to well represent basins characterized by heterogeneous

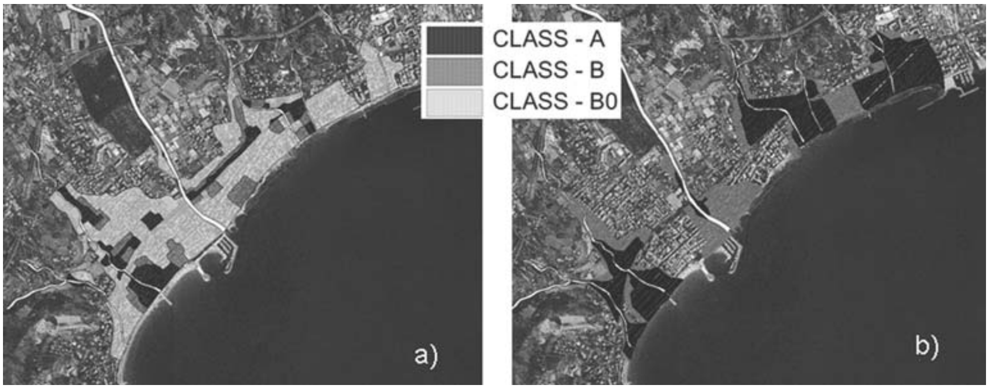

FIGURE 8. (a) Flood hazard map resulting in this study, compared to (b) actual hazard maps published by Basin Authority

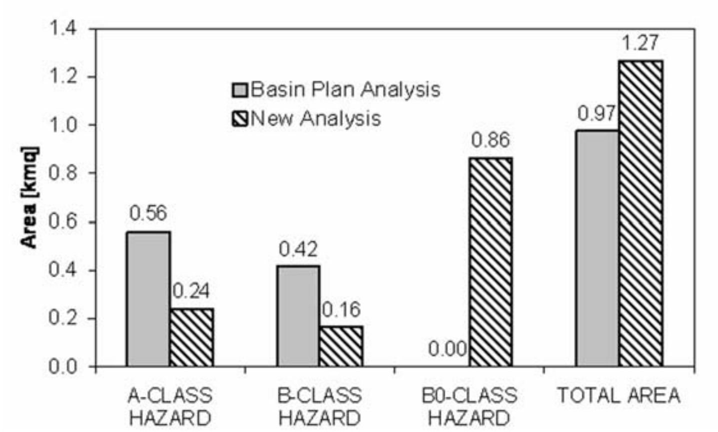

FIGURE 9. Extension of class-hazard area as resulting from this study, compared to previous study performed by Basin Authority

morphology and land use, as the ones in this work. The critical event guarantees that the hydrograph with the maximum effect on territory is assumed, even if peak discharge is lower than the maximum peak discharge for the given rainfall frequency.

Classical one dimensional models are poor tools for flood analysis in urban area. They can be used as long as the main stream is not overtopped, as they fail to simulate flow component other than along river direction. Two dimensional models, on the other hand, are time consuming and, for unsteady flow simulations on complex topography, they fail on steep slopes, geometric discontinuities, mixed flow and initially dry areas.

This work proposes an hybrid approach. The urban and drainage system is modelled by means of a network in which both rivers and roads are modelled as channels linked by nodes. The basic assumption is that high density urban blocks can be modelled as storages in which flow velocity is null. The assumption is verified by a 2D model results which confirm that flow velocity in the blocks is negligible.

The channel network model seems to well represents flood routing in urban area, it is not computationally expensive as 2D model and, above all, is much more stable on complex topography. On the other hand it requires a deep knowledge of the territory and a good skills of the modeller to set up the hydraulic sketch.

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Streszczenie: *Konstruowanie hydrogramów i schematów obliczeniowych do odwzorowania obszarów zagrożonych powodziami na terenach zurbanizowanych.* Wyznaczanie map ryzyka powodziowego jest jednym z ważniejszych zagadnień we współczesnej hydrologii. Ich wpływ na zarządzanie terenami zalewowymi zyskał duże

znaczenie, co spowodowało potrzebę badań nad dynamiką powodzi. Było to również przyczyną intensyfikacji badań nad następującymi zagadnieniami: zdefiniowanie hydrogramu projektowego, identyfikacja powierzchniowych warunków brzegowych do obliczeń zasięgu powodzi, wybór modeli jak najlepiej opisujących zasięg powodzi w specyficznym środowisku, jakim jest obszar zurbanizowany lub doliny rzeczne. Większość modeli matematycznych zarówno szkoleniowych, jak i komercyjnych, rozwiązuje równania De Sait Venant'a w jednym lub dwu wymiarach, nie sprawdza się jednak dla skomplikowanych topograficznie powierzchni. Duże spadki, nieciągłości geometryczne, różne zmieniające się reżimy przepływu, początkowo suche obszary są głównymi problemami, które powinny być rozwiązywane przez modele hydrauliczne. W tych badaniach skupiono się na dwóch zagadnieniach: zdefiniowane zdarzenia krytycznego dla terenów zalewowych i technice modelowania terenów zalewowych dla silnie zurbanizowanej płaskiej powierzchni. W tym przypadku schemat połączonych kanałów i zbiorników retencyjnych dał lepsze odwzorowanie powierzchniowych warunków brzegowych, takich jak kompleksy budynków, sieć ulic i wystarczająca dokładność w określaniu obszarów zagrożonych powodziami w porównaniu do dwuwymiarowego modelu hydraulicznego.

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