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RESEARCH ARTICLE

Design hydrograph and routing scheme for flood mapping in a dense urban area

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Definition of flood risk maps is a task to which modern surface hydrology devotes substantial research effort. Their impact on the management of flood-prone, dense, urban areas has increased the need for better investigation of inundation dynamics. The problems associated with the aforementioned topics range from the definition of the design hydrograph and the identification of the surface boundary conditions for the flood routing over the inundation plan, to the choice of the hydrodynamic model to simulate urban flooding. Most of academic and commercial mathematical models, solving the De Saint Venant equations, fail on complex topography. Frequently encountered difficulties concern steep slopes, geometric discontinuities, mixed flow regimes, and initially dry areas. In the present paper, flood routing modelling approaches in urban areas and principles for the definition of the design flood events are outlined. The paper shows how urban flooding can be simulated by a quasi-2D hydrodynamic model that makes use of a network of connected channels and storages to simulate flow, respectively, on the streets and into the building blocks. Furthermore, the paper shows that, when flood hazard is assessed by considering flood extent, water depth and flow velocity, an in-depth analysis of the use of design hydrographs that maximise peak flow or inundation volume is needed.

Keywords: design hydrograph; distributed hydrological model; flood hazard maps; quasi-2D hydraulic model; urban flood

1. Introduction

Flooding is one of the most common environmental hazards, due to the widespread geographical distribution of river valleys and the attraction of human settlements to these areas. Floods can be generally arranged in two categories (Castelli 1994): flash floods, the product of heavy localised precipitation in a short period over a given location, and general floods, caused by precipitation over a longer 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. Watercourses in urban environment are often artificially constrained in narrow channels so that they may be unable to contain the runoff that is generated by relatively small, but intense, rainfall events (Aronica *et al.* 2005).

Water flowing on the urban surface may cause direct damage, indirect damage (e.g. traffic disruptions, production losses, etc.) and social consequences (negative long-term effects such as decrease of property values) (König *et al.* 2002, Mark *et al.* 2004).

For the optimal planning and management of flood prone urban areas an invaluable tool is the flood hazard map. This will enable more informed decision-making when looking at addressing existing problems, as well as planning for future development.

In this work we focus on determination of flood hazard map for a dense urban area located in the Liguria Region in Italy.

The traditional approach is based on determination of flood extent for a given frequency event, using a steady-flow 1-dimensional model. A new method has recently been proposed for highly urbanised areas; it identifies the hazard according to both water depth and flow velocity, claiming that it is necessary to use more accurate unsteady-flow numerical models (FEMA 2002, Rosso 2003).

Many modelling approaches exist to simulate floods (Goodrich *et al.* 1991, Labadie 1992, Inoue *et al.* 2000, Leopardi *et al.* 2002). The choice of model depends on problem complexity (Ferrante *et al.* 2000, Horrit *et al.* 2001).

The aim of this work is to assess the accuracy of a quasi-2D model against pure 2D models for flood

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hazard assessment in an urban area. Furthermore, the paper investigates the method for the definition of design hydrographs. If the objective of the analysis is to assess the maximum extent of the flooded area, regardless of the flow velocity, one possible strategy is to assume the design hydrograph that maximises the inundation volume. When the hazard map is assessed by considering both water depth and flow velocity in a dense urban area with complex topography, the design hydrograph that maximises the peak flow has to be taken into account too.

The structure of the paper is as follows. In the second section the urban system within the study area is described, which includes the definition of flood hazard maps adopted by the Floods Directive (2007/60/EC). In the third section the proposed procedure for evaluation of the design storm hydrographs is presented. In the fourth section the proposed hydrodynamic model for flood simulation in a dense urban area is presented, together with the hypotheses required. In the fifth section a few results of the flood hazard maps are presented, and finally, in the last section, conclusions are drawn.

While the study focuses on a specific region, the overall approach is generic and may be applied for elsewhere.

2. The case study

The study area is located in western Italy, in the Liguria Region (Figure 1). The area approaches the sea and has an extension of about 1.8 km^2 in which, from the west to the east, six 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 above sea level.

The morphology is characterised by steep slopes in the upper part of the basins that tend to attenuate in the downhill zone and only flatten out in the vicinity of the outlet. The uphill zones are scarcely urbanised, while the restricted coastal plain has attracted tourism activity, resulting in an exponential growth of urban density. As a consequence the rivers often have been channelled into artificial drainage. In this situation, overflow of the watercourses and subsequent inundation can be frequently observed during storm events.

The study area is divided into two parts by a railway embankment that runs all the way parallel to the coastline: the upper part with a slope of about 1.5% and the lower with an average slope of about 0.8%.

In the study area rainstorm records are only available in five rain gauges outside but in the vicinity of the hydrographic basin of the six watercourses. Neither river stage measurements nor inundation maps nor flood marks for historical storm events are available.

2.1. *The directive for basin planning*

The Italian laws leave the task of the assessment of flood risk maps to the River Basin Authority by means of the Basin Plan study. The traditional approach simulates the inundation process with a steady

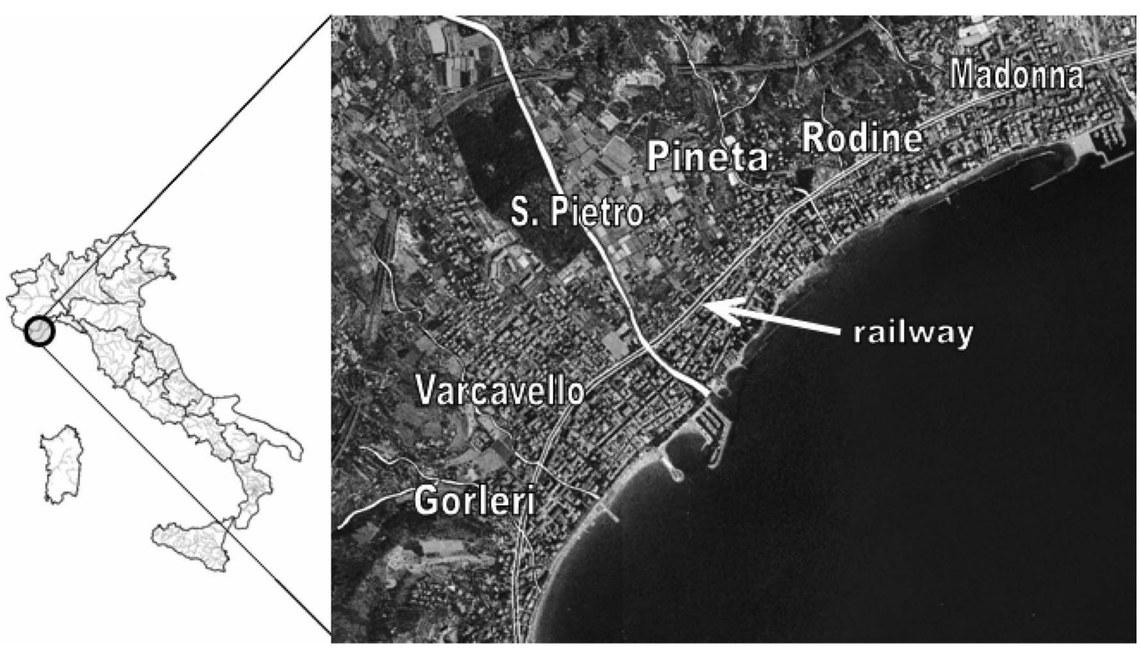

Figure 1. Aerial photo of the study area. The six rivers and the railway are visible.

1-dimensional model. The analysis is iterated for 50 and 200 years return period peak discharges. The 50-year flood extent is classified as a class-A hazard and the 200-year flood extent is classified as a class-B hazard (Figure 2a). Class-A areas are subject to more restrictive rules than class-B areas.

In order to determine the hazard map, a new scheme has been recently proposed in the Liguria Region, based on threshold values of water depth and flow velocity that define three hazard conditions: low, medium, and high (Figure 3). According to this scheme, if an area is frequently flooded but with low depth and low velocity, the class of hazard is reduced (Figure 2b). Moreover a new hazard class has been introduced: according to the new scheme, an area can be classified as a class-A, class-B or class-B0 hazard. The class-B0 area is less restrictive.

The steps for the classification of the inundated area are as follows:

(1) The area inundated by the 50-year return period hydrograph is classified into an A0 region (corresponding to low hazard conditions) and an A region (corresponding to medium and high hazard conditions) (Figure 3);
(2) The area inundated by the 200-year return period hydrograph, external to the 50-year return period flood extent previously classified, and the A0 region, is classified into a B0 region (corresponding to low and medium hazard conditions) and a B region (corresponding to high hazard conditions).

The assumption is that the expected total damage is related to the physical properties of the flood, i.e. flood extent, water level above ground level, and flow velocity.

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

3. The design hydrographs for inundation maps

Many approaches for the assessment of the design flood, in terms of peak discharge or hydrograph, have been developed over the years (Sokolov *et al.* 1976, Chow *et al.* 1988, Maidment 1993, Wong *et al.* 1999, Ashfaq *et al.* 2000, Yue *et al.* 2002, Nasri *et al.* 2004), and the debate is open on the frequency of the flood map due to the difference between the frequency of the peak discharge and the volume of the hydrograph (Salvadori *et al.* 2004, De Michele *et al.* 2005). Moreover, if a rainfall runoff model is used for the computation of the hydrograph, the hypothesis of the equivalence in the return period of the peak discharge and the rainfall is questionable too.

Inundation maps are generally computed using peak discharge for a given return period as an input variable to simple steady 1-dimensional hydrodynamic models of the main watercourses, which means that the actual volume of the hydrograph and the routing on the inundated area are not considered. However, the storage volume can significantly affect the routing and the extension of the inundated area, especially in an urban environment, where the effect is enhanced by the presence of obstacles, storage in the buildings and complex hydraulic flow patterns. When the storage effect becomes relevant, the traditional approach is not sufficient to characterise the flood extent and the design

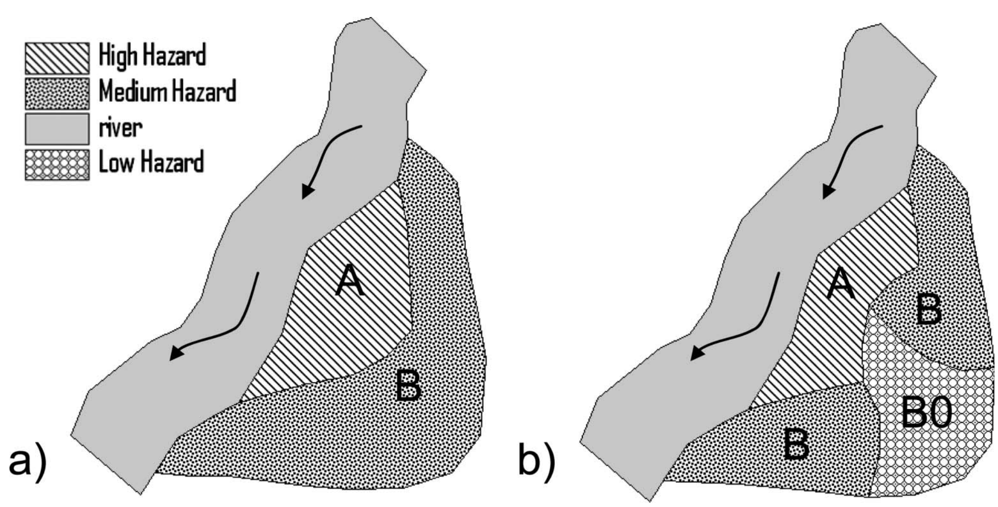

Figure 2. Evaluation of hazard maps according to the actual rule (a) based on the flooding extent for given return period events and according to the new rule (b) which considers both hydraulic depth and flow velocity.

hydrograph has to be considered with unsteady flow modelling.

In ungauged basins, the design hydrograph can be assessed from rainfall-runoff transformation, under the assumptions that the intensity duration frequency (IDF) curve characterises the rainfall regime and assuming the critical flood design criterion. According to this, the design hydrograph is the one that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in a drainage area.

If the objective of the analysis is to assess the maximum extent of the flooded area, one possible strategy is to assume the design hydrograph that maximises the inundation volume. When the flood hazard is assessed by considering both water depth and flow velocity, the design hydrograph that maximises the peak flow has to be taken into account too.

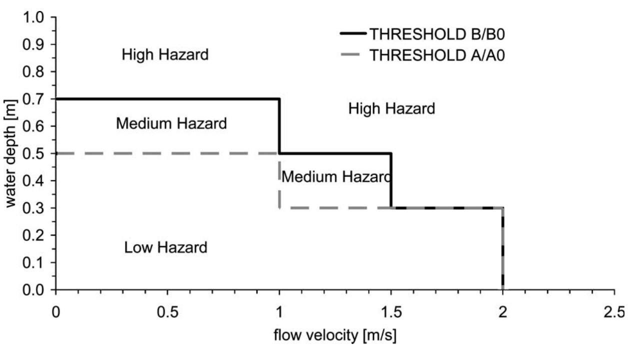

Figure 3. Threshold values of water depth and flow velocity that define the hazard condition. Points that lie above the solid line are in high hazard conditions, points below dashed line are in low hazard conditions, and points in the middle are in medium hazard conditions.

In this work, for the rainfall-runoff transformation, the FEST model was employed (Mancini 1990, Montaldo *et al*. 2002, Rulli *et al*. 2002, Montaldo *et al*. 2004, Montaldo *et al*. 2007, Rabuffetti *et al*. 2008). FEST is a distributed hydrological model developed at the Politecnico di Milano focusing on flash-flood event simulation. As a distributed model, FEST can manage the spatial distribution of meteorological forcings, and heterogeneity in hillslope and drainage network morphology (slope, roughness, etc.) and land use (Rosso 1994). Given the small extent of the basins, in the present study rainfall was considered to be uniformly distributed over space and time. The calculation time step is 5 minutes.

3.1. *The probable maximum peak design flood (PMPDF)*

From a family of IDF curves, by transforming rainfall into runoff, it is possible to obtain a hydrograph for any duration at a given frequency and, finally, a series of hydrographs for each return period (Figure 4). According to the critical flood design criterion, assuming the peak flow as the key variable that characterises the inundation on the urban surface,

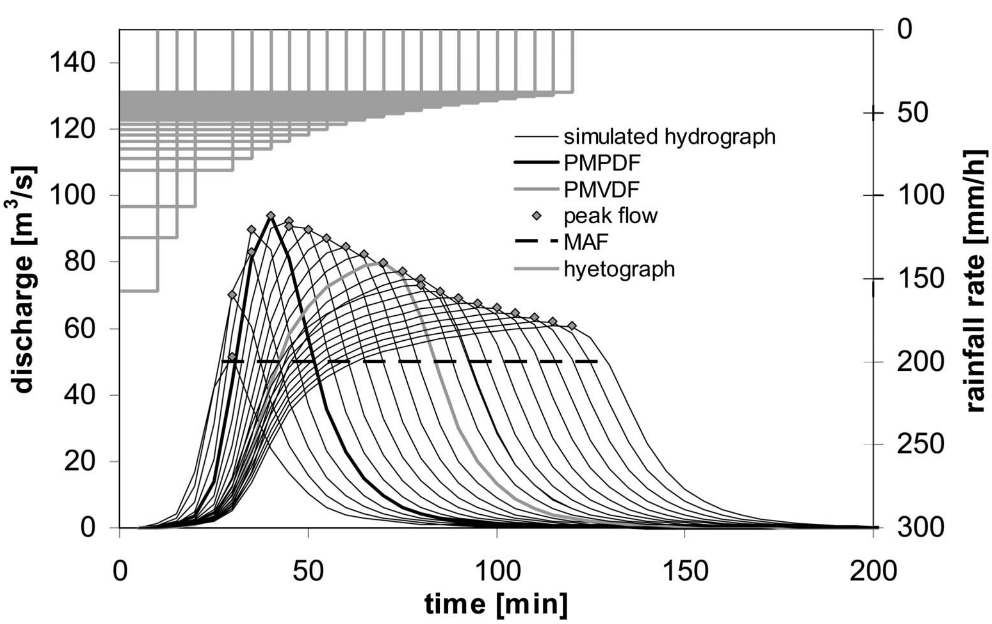

Figure 4. Procedure for the search of the design flood defined as that event which is characterised by the maximum key variable (potential inundation volume or peak flow). The probable maximum peak design flood (PMPDF), the probable maximum volume design flood (PMVDF) and maximal acceptable flow (MAF) are reported. The results refer to the river Varcavello for the 50-year return period.

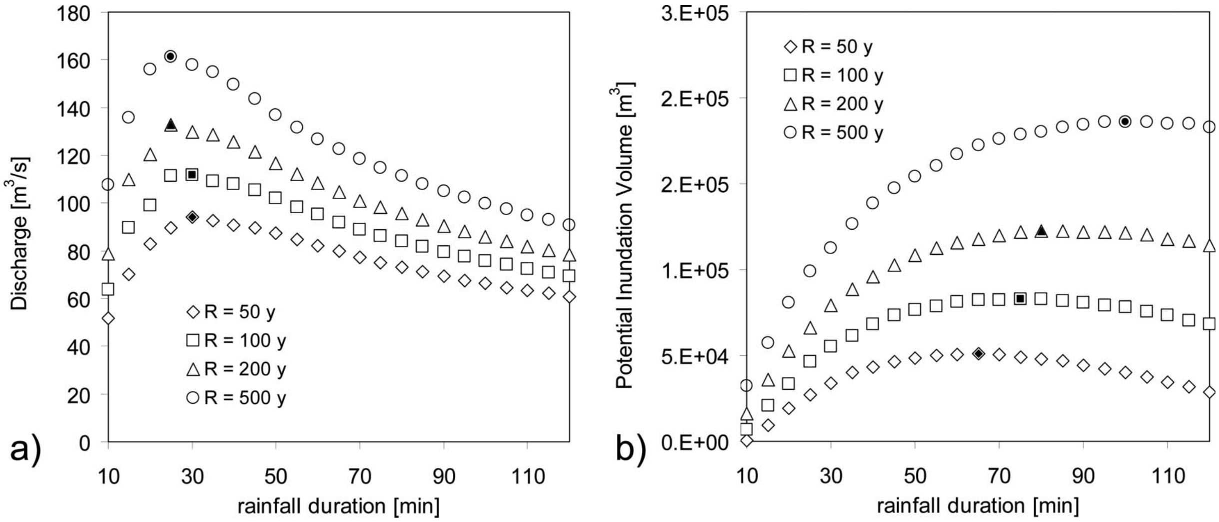

Figure 5. Series of peak discharge (a) and potential inundation volume (b) for different return periods for the river Varcavello. The design hydrograph for a given return period is the one characterised by the maximum value of the key variable (solid black mark).

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

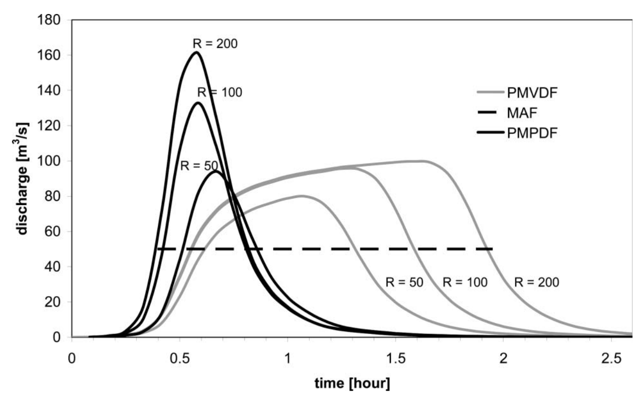

Figure 6. The probable maximum peak design flood (PMPDF), the probable maximum volume design flood (PMVDF) and maximal acceptable flow (MAF) for the river Varcavello and three return periods: 50, 100, 200 years.

the probable maximum peak design flood (PMPDF) for a given return period is the one related to the rainstorm duration (d) that causes the hydrograph with the maximum peak discharge (Equation (1)).

$$PMPDF : Max\{Q(d,t)\} \tag{1}$$

where Q is the discharge varying with time (t) and rainstorm duration.

In Figure 5a the series of peak discharges for some return periods are reported for the river Varcavello. The maximum value, marked with solid black, is the one associated with the PMPDF.

3.2. *The probable maximum volume design flood (PMVDF)*

In this section the procedure to evaluate the probable maximum volume design flood (PMVDF) is reported.

Starting from the same IDF curves in the previous subsection, if we assume as the key variable for flood characterisation the flood volume that can overflow the watercourses potentially, the PMVDF for a given return period is the one related to the rainstorm duration that causes the hydrograph with the maximum potential inundation volume (Equation (2)).

$$PMVDF : Max\left\{\int (Q(d,t) - MAF)dt\right\} \tag{2}$$

where MAF is the maximal acceptable flow, defined as the maximum value of the discharge that can be conveyed by a river reach. In this work, the MAF for the urban reach of the six watercourses was evaluated with a steady 1D analysis, considering detailed cross-sectional profiles and the geometry of obstructions (bridges and inline structures in general).

In Figure 5b the series of potential inundation volumes for some return periods are reported for the river Varcavello. The maximum value, marked with solid black, is the one associated with the PMVDF.

According to retention pool analysis (Artina *et al.* 1997), the flood that maximises the peak flow is different from the one that maximises the inundation volume and the related peak is characterised by different return periods for a given rainfall frequency (Table 1).

In Figure 6, the resulting PMPDFs and PMVDFs for the river Varcavello are shown.

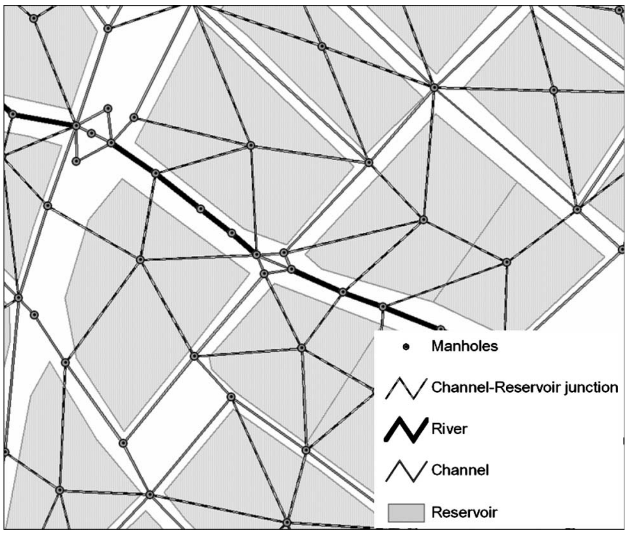

Figure 7. Network quasi-2D hydraulic model representing an urban area: detail of the river Varcavello.

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 storages upstream of the railway[a]	0.1
Weir discharge coefficient for storages downstream of the railway[a]	0.28
Weir length for districts upstream of the railway[a]	50 m
Weir length districts downstream of 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), C_d is the discharge coefficient, B is the width of the weir (m), D_u is the upstream depth with respect to the crest (m), D_d is the downstream depth with respect to the crest (m) and g is the acceleration due to gravity (m/s^2).

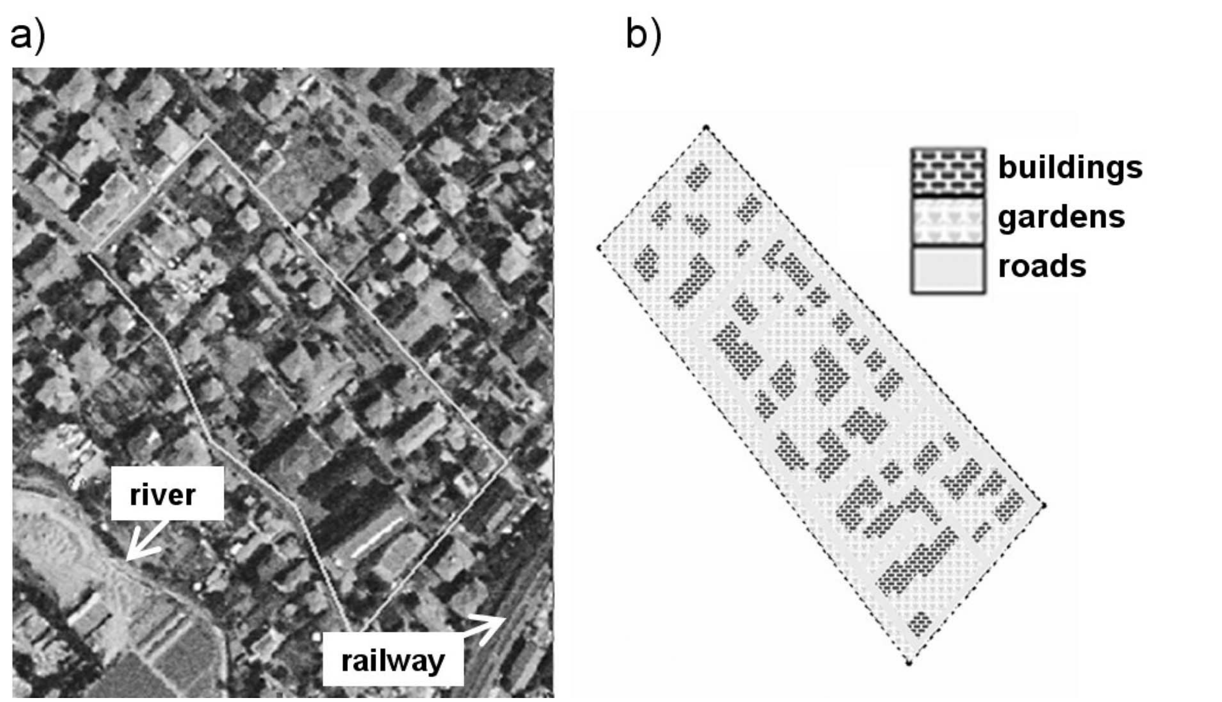

Figure 8. Subset of the study area (a) near the river Varcavello upstream of the railway and (b) representation in the 2D model.

4. Description of the hydraulic model

In dense urban areas, during flood events, it can be assumed that the majority of the water passes through the streets with a one-dimensional flow (Mignot *et al.* 2006). The building blocks (densely concentrated buildings close together) can be considered impervious or, alternatively, can be modelled as a porous area with a storage capacity. The urban environment can, hence, be schematised as a network of 1D channels where the flow velocity is greater than zero, and storages where the velocity is null (as discussed in the next subsection).

A network mathematical model was thus implemented based on three main elements: the main rivers, the channels along the main streets and the storages to simulate building blocks.

The De Saint Venant equations are integrated along the river branches and the street channels using the Preissman implicit numerical scheme (Wallingford Software). The flow along the street can be in either direction according to the hydraulic gradient. The connection between storages and channels is simulated by means of the weir equation. When flooding takes place, water in excess from the watercourses may flow into the streets.

The described quasi-2D network model was implemented (Figure 7) in the Infoworks-CS software (Wallingford Software). Rivers and main streets are represented by conduits. The street junctions and the connection between watercourse and street are represented by means of manholes. Conduits are linked to storages through weirs. Bridges are described by means of two parallel conduits, one to model flow under the bridge and one to take into account the possibility that the flow passes over the deck of the bridge.

The underground sewage network was not taken into account in this work, assuming that the exchanges between the surface and the network could be ignored compared to the flow in the streets.

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 is reported in Table 2. Natural watercourses and un-asphalted streets are assumed to have comparable roughness and so also are concrete channels and asphalted streets. The discharge coefficient and weir length are different in the building blocks upstream and downstream of the railway to take into account the actual geometry of the weir crest due to the different urban landscape morphology.

4.1. *Hypothesis verification using a 2-dimensional model*

The basic assumption of the implemented network model is that the velocity of flood flow over the urban area is greater than zero in the main streets while, in the blocks, velocity can be ignored. To verify this assumption, we analysed flood dynamics in two representative building blocks, by means of a high-resolution 2-D model. A subset of urban land in proximity to the river Varcavello was simulated. It is composed of two main blocks: one upstream of the railway and the other downstream. The upper one is

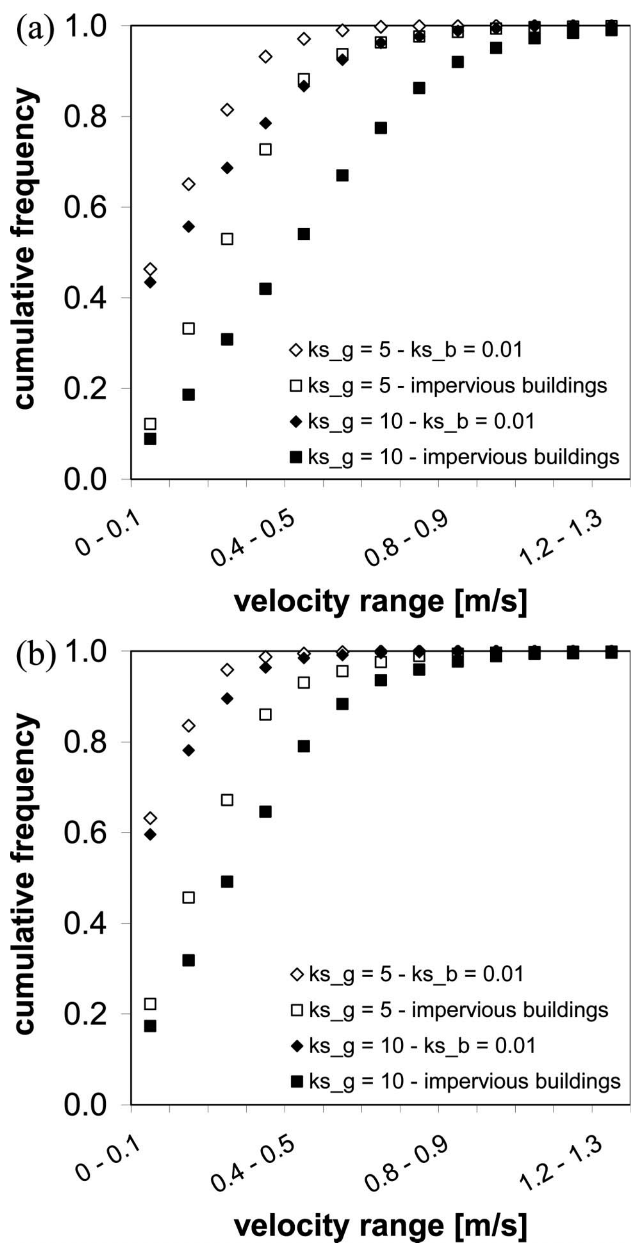

Figure 9. Cumulative frequency of velocity magnitude simulated by means of full 2D model in (a) an upstream building block and (b) a downstream building block; ks_g and ks_b denote, respectively, the Strickler roughness coefficient for gardens and buildings.

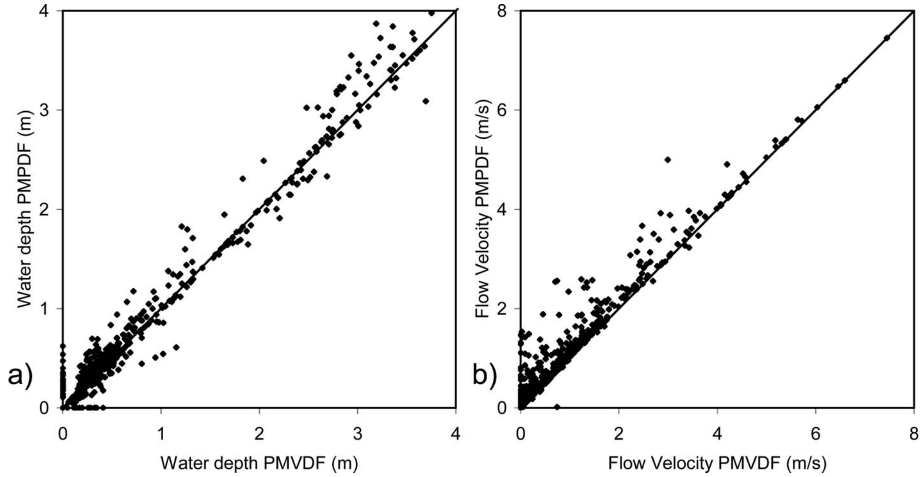

Figure 10. Comparison between the results in the computational nodes of the simulation that consider PMPDFs and PMVDFs for the 200-year return period.

characterised by an average slope of about 1.5%, the lower by a slope of about 0.8%.

A fully 2D model was implemented using the SMS software (Boss International) (Figure 8) to perform a steady flow computation. Hydraulic depths from quasi-2D simulation were taken as boundary conditions.

Buildings were modelled in two different ways: as an 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 method implies that water is free to move in the whole domain, but the flow through buildings is made difficult because of a high roughness coefficient.

The Strickler roughness coefficient for garden areas around the buildings was assumed to vary from 5 $m^{1/3}\ s^{-1}$ to 10 $m^{1/3}\ s^{-1}$ to take into account the presence of vegetation and, in general, obstacles of varying shapes and lengths.

The velocity field was mapped to a regular grid and the cumulative frequency of the velocity values was evaluated (Figure 9). In the upstream block (Figure 9a), we can note that the velocity of most of the cells (67%) is less than 0.6 m/s even for the simulation with garden Strickler coefficient equal to 10 $m^{1/3}\ s^{-1}$ and impervious buildings.

In the companion simulation, the percentage of cells with velocity under 0.6 m/s increases nearly to 100%. In both simulations the maximum flow velocity reaches, anyway, a value of 1.3 m/s in just a few cells.

The same remarks are valid for the downstream block (Figure 9b) where the velocity is even lower due to a smaller surface slope.

We can conclude that the water flow in the building blocks is characterised by low values of velocity. The assumption to simulate blocks as storages in the channel network model seems reasonable.

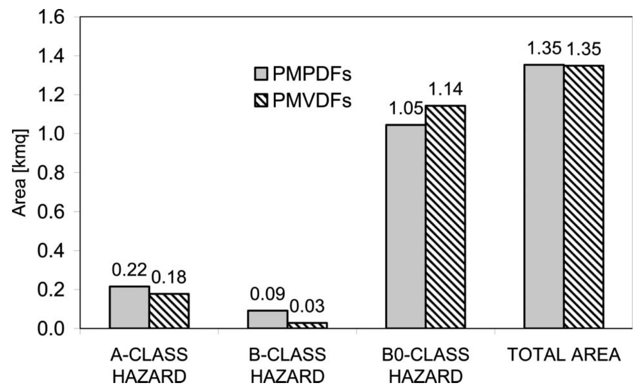

Figure 11. Extent of class-hazard area as resulting from analysis based on PMPDF and PMVDF hydrographs.

5. The flood hazard maps

The maximum flow velocity and hydraulic depth values, computed respectively in the channels and in the nodes of the network model, were interpolated over the entire study area to obtain a continuous map. For this purpose the borders of building blocks were considered as impervious barriers. From the flow velocity and water depth maps it is possible to produce the flood hazard map, according to the new directive of the River Basin Authority (§2.1). Both types of design flood were considered in this analysis. In Figure 10 the comparison between the results in the computational nodes of the simulation that consider PMPDFs and PMVDFs is reported for the return period of 200 years (similar results were attained for the 50-year return period). It is shown that flow velocity is mostly higher when considering PMPDFs. Water depth for PMPDFs is generally higher, except in some computational nodes where water is higher for PMVDFs.

Figure 12. (a) Flood hazard map resulting from this study, compared to (b) actual hazard maps published by the Basin Authority.

The effect of these differences in the resulting flood hazard map is shown in Figure 11, where a comparison of the extent of different class hazard areas for PMPDFs and PMVDFs is reported. Higher values of water depth and flow velocity induced by PMPDFs lead to an increment in the extent of class-A and class-B areas. The total flooded area is comparable even if local differences in inundation extent are seen.

To take into account both types of design flood, the final flood hazard map is taken considering, in every point of the computational domain, the more restrictive class resulting from the PMPDF and PMVDF hydrographs (Figure 12a).

A comparison is made with the previous study performed by the Basin Authority (Regione Liguria) based on the traditional approach (steady 1-dimensional analysis of the watercourses) (Figure 12b). The total amount of flooded area has increased in this new study (Figure 13) from 0.97 to 1.40 km^2. The class-A and class-B areas have decreased, respectively from 0.56 to 0.22 km^2 and from 0.42 to 0.1 km^2. The complementary area is included in the class-B0 area, which was not defined in the previous analysis.

The differences encountered with the new approach have their counterpart in the resulting flooding map. When the network quasi-2D model is used, water in excess from the watercourses is free to flow into the streets and to reach those distant localities that, in the previous study, were not even flooded by the 200-year event: the pure 1D steady model is a poor instrument to predict flood inundation in a dense urban area with complex topography.

The new modelling approach can also predict the effect on flood of manmade obstacles orthogonal to the flow direction. The railway divides the city in a north–south direction and behaves as an impervious levee to water flow. This does not seem to be completely represented by the steady 1D model.

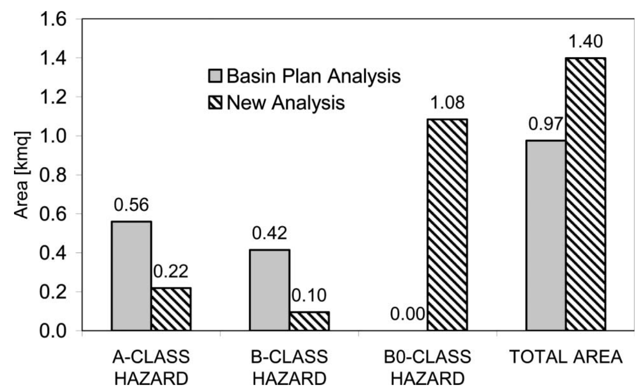

Figure 13. Extent of class-hazard area resulting from this study, compared to the previous study performed by the Basin Authority.

6. Conclusions

A procedure is presented for the estimation of design hydrographs. It is based on the search for the critical event that maximises the effect on the territory in the form of a flood hazard map. When the hazard map is assessed by considering both water depth and flow velocity in a dense urban area with complex topography, it is important to take into account both the peak flow and the potential inundation volume. Only in this way can we guarantee that the most severe combination of critical meteorological and hydrological response is considered.

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

This work proposes a hybrid approach (quasi-2D model). The urban system is modelled by means of a network in which both rivers and roads are simulated 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. This assumption is verified by the results of a 2D model which confirm that flow velocity in the blocks is negligible.

The quasi-2D model seems to better represent flood routing in urban areas than the 1D model, is less computationally expensive than 2D models and, most importantly, is much more stable on complex topography. On the other hand it requires deep knowledge of the territory and good modelling skills in designing the hydraulic sketch.

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