Indirect Estimation of Design Flood in Urbanized River Basins Using a Distributed Hydrological Model

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Introduction

A number of approaches are available for estimating design floods (Ashfaq and Webster 2000; Nasri et al. 2004; Yue et al. 2002; Bocchiola et al. 2003). In cases in which long records of measured streamflow data are available, a direct statistical analysis of the data may be feasible. However, the streamflow data series are often too short to perform robust statistical inference. Benson (1962) indicated that reliable quantile estimates can be obtained only for return periods $<2n$ as a rule of thumb, where n denotes the number of years of observations. Also, Hosking et al. (1984) showed that reliable quantile estimates are obtained only for nonexceedance frequencies $<1-1/n$, which corresponds to a return period n.

In many circumstances, no measured streamflow data are available at the site of interest. Moreover, in urban areas, when natural development of the watercourse is significantly altered by anthropic constraints such as bridges, detention ponds, and levees, design flood estimates may result significantly lower than natural discharge, with dangerous impact on downstream sections in case of further modifications of the upstream river.

Under such conditions, the design flood can be assessed from rainfall-runoff transformation under the assumptions that the depth duration frequency (DDF) curve characterizes the rainfall regime and assuming the critical flood design method. According to this, the design hydrograph is the one that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a drainage area (Ravazzani et al. 2009).

The most diffused approach to hydrological modeling has been the lumped conceptual one. In lumped models, the whole catchment is considered as a single entity, spatial variations are averaged, and basin response is evaluated only at the outlet. Starting from the first studies by Freeze and Harlan (1969), research activities focused on the importance of the time-space variability either of the soil characteristics and of the rainfall field combined with the processes governing catchment response to precipitation (Rosso 1994). Thus distributed hydrological models have become very common in research activities for their capability to describe spatial variability of processes, input, boundary conditions, watershed characteristics, and output, but it is still not widely diffused in the engineering practice (Beven 2001).

This paper presents the application of distributed hydrological model FEST (flash-flood event-based spatially distributed rainfall-runoff transformation, including reservoirs system) for the assessment of design flood of the Olona River basin, a small watershed in northern Italy. The significant heterogeneity of the Olona River basin is enhanced by the presence of a mixture of forest, natural landscapes, and highly urbanized areas that, despite the small extent of the basin, require use of a spatially distributed hydrological model.

Case Study

Olona river is the main stream of a group of water courses in the north of Milan, Italy, most of which flow through built-up areas causing damage to the population especially during the rainy seasons. Olona is a 71-km-long river, which runs mainly through the provinces of Varese and Milan. After passing through the deep Olona Valley, cut in the porous soils of the upper Po Valley, the Olona River flows in the plain until Milan, Italy, usually contained in narrow artificial banks.

The area at the considered closing section (Lozza, Italy) is 94.5 km^2 (Fig. 1). Its elevation ranges from 271 m above sea level

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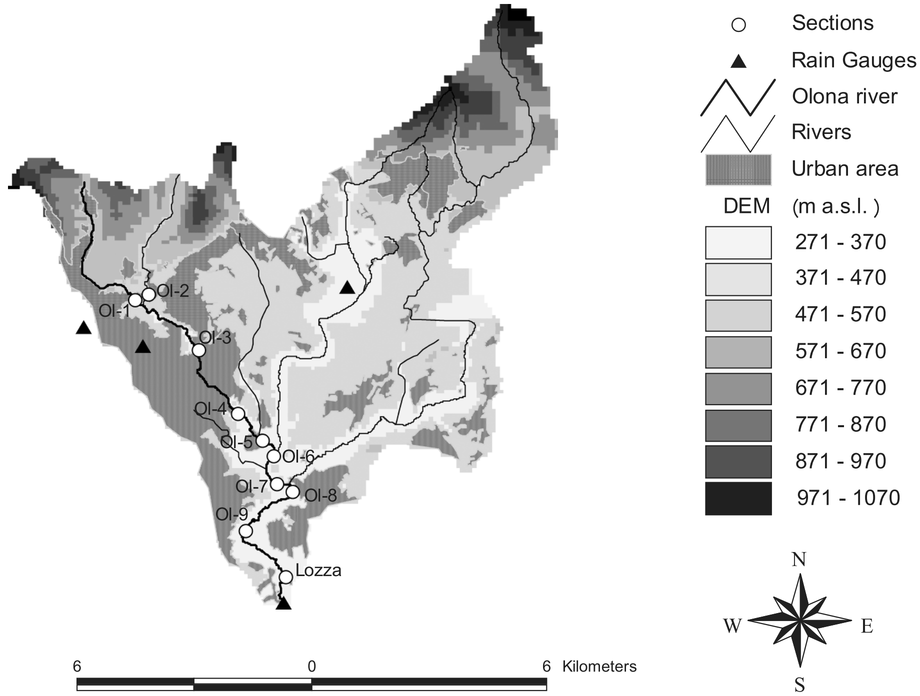

Fig. 1. Olona River basin showing locations of the rain gauges, hydrometric stations, and cross sections considered in the design flood assessment

at the outlet to approximately 1,070 m above sea level at the Tre Croci crest. The average elevation is 455 m above sea level. Land cover is heterogeneous including broadleaf forest (38%), mixed forest (16%), agricultural (20%), and urban (26%).

Climate conditions are typically humid, characterized by higher precipitations in autumn and spring and lower in winter. The total annual precipitation is approximately 1,600 mm.

The Olona River experienced significant floods since 1584, the year of the first reported event, with an increase of flood frequency in recent years (five major floods in the last 20 years).

Meteorological and hydrologic hourly data were collected by the telemetric monitoring system of Regione Lombardia. Data of four rain gauges were available from January 1, 2003, to December 31, 2010. River discharge measurements at Lozza, Italy (basin outlet) were available from January 1, 2002, to December 31, 2010. The locations of the rain gauges and Lozza hydrometric station are shown in Fig. 1.

Depth duration frequency curves parameters were obtained from the Regional Environmental Protection Agency of Lombardia (ARPA Lombardia). They were computed by fitting the general extreme value (GEV) probability distribution function to annual maximums of 1- to 24-h precipitation amounts, using the scale invariance concept (Burlando and Rosso 1996). GEV parameters for ungauged sites were obtained by spatial interpolation using the kriging method with isotropic spherical variogram. Precipitation depth, $h_T(d)$ (mm), for a given return period, T (years), and given duration, d (hours), is thus obtained applying Eq. (1)

$$h_T(d) = a_1 w_T d^n \tag{1}$$

where a_1 = hourly precipitation coefficient; n = power exponent; and $w_T = T$th GEV quantile of a normalized random variable.

Precipitation depth was spatially averaged over the river basins by applying the areal reduction factor (ARF) (De Michele et al. 2001)

$$\mathrm{ARF} = \left[1 + \bar{\omega}\left(\frac{A}{d}\right)^b\right]^{-b/v} \tag{2}$$

where A = basin area (km^2); and ω, b, and ν = parameters equal to 0.09, 0.54, and 0.484, respectively.

Ten river sections along the Olona River were considered in this analysis, including the Lozza, Italy, basin outlet. Basin area, percentage of river basin covered by urbanized area, hourly precipitation coefficient, power exponent, and GEV quantile of normalized random variable of representative depth duration frequency curves are reported in Table 1.

Description of Distributed Hydrological Model FEST

In this paper, for the rainfall-runoff transformation, the FEST model was employed (Montaldo et al. 2004, 2007; Rabuffetti et al. 2008; Pianosi and Ravazzani 2010; Corbari et al. 2011). FEST is

Table 1. Cross Sections Considered in the Design Flood Assessment

Section	Area (km^2)	Urban (%)	a_1 (mm/h^n)	n (—)	w_{10}	w_{100}	w_{500}
Ol-1	5.9	26.78	34.36	0.354	1.432	2.166	2.682
Ol-2	4.8	8.96	34.29	0.352	1.432	2.167	2.685
Ol-3	17.2	33.08	34.46	0.349	1.433	2.174	2.694
Ol-4	19.6	41.12	34.69	0.347	1.434	2.180	2.704
Ol-5	27.5	36.32	34.57	0.345	1.433	2.178	2.701
Ol-6	45.5	27.54	34.31	0.342	1.431	2.170	2.690
Ol-7	50.5	30.89	34.55	0.340	1.432	2.176	2.699
Ol-8	89.5	24.69	33.96	0.339	1.427	2.156	2.670
Ol-9	91.9	25.14	34.16	0.337	1.428	2.161	2.676
Lozza, Italy	94.5	25.71	34.16	0.337	1.428	2.161	2.676

Note: Code of cross section, extent of basin area in square kilometers, percentage of river basin covered by urbanized area, hourly precipitation coefficient (a_1), power exponent (n), and GEV quantile of normalized random variable for 10 years (w_{10}), 100 years (w_{100}), and 500 years (w_{500}) return period of depth duration frequency curve.

a distributed hydrologic raster-based model developed at the Politecnico di Milano. As a distributed model, FEST can manage spatial distribution of meteorological forcings and heterogeneity in hillslope and drainage network morphology (e.g., slope and roughness) and land use (Rosso 1994). FEST is developed with modular structure so the user can activate components according to the aim of the application and availability of data. FEST can be used as a conceptual simple model focusing on flash-flood event simulation or as a fully physically based model capable of simulating hydrological processes in complex and heterogeneous river basins as other models do, such as topographic kinematic approximation and integration (TOPKAPI) (Ciarapica and Todini 2002), distributed hydrological model for the special observation period (SOP) (DimoSOP) (Ranzi et al. 2003), and AFFDEF (Moretti and Montanari 2007). In this application, a basic version of FEST is used.

The basic version of the FEST model has three principal components (Fig. 2). In the first component, the flow path network is automatically derived from the digital elevation model using a least-cost path algorithm (Ehlschlaeger 1989). It assigns flow from each pixel to one of its eight neighbors without the necessity of removing pits in the elevation data. For hillslope and channel network definition, the model uses the constant minimum support area concept. It consists of selecting a constant critical support area that defines the minimum drainage area required to initiate a channel (Montgomery and Foufoula-Georgiou 1993).

In the second component, the surface runoff (or rainfall excess) is computed for each elementary cell using the Soil Conservation Service curve number (SCS-CN) method (Soil Conservation Service 1986) in its differential form (Mancini and Rosso 1989). The depth of rainfall excess, P_n^j, of each elemental cell at the jth time step is computed as

$$P_n^j = \begin{cases} 0 & \text{if } P^j < I_a \\ \frac{(P^j - I_a)^2}{(P^j - I_a + S)} & \text{if } P^j \geq I_a \end{cases} \tag{3}$$

where P^j = cumulative depth of rainfall at the jth time step; I_a = initial abstraction before ponding and is equal to cS with constant c in the range 0.1 to 0.3 (Ponce 1989); and S = potential maximum soil retention related to the curve number parameter, CN, which ranges from 1 to 100 [see, e.g., Ponce (1989), Eqs. (5)–(7), p. 157].

The SCS-CN method distinguishes three levels of antecedent moisture condition (AMC I, AMC II, and AMC III), depending on the total rainfall in the 5 days preceding a high rainfall event.

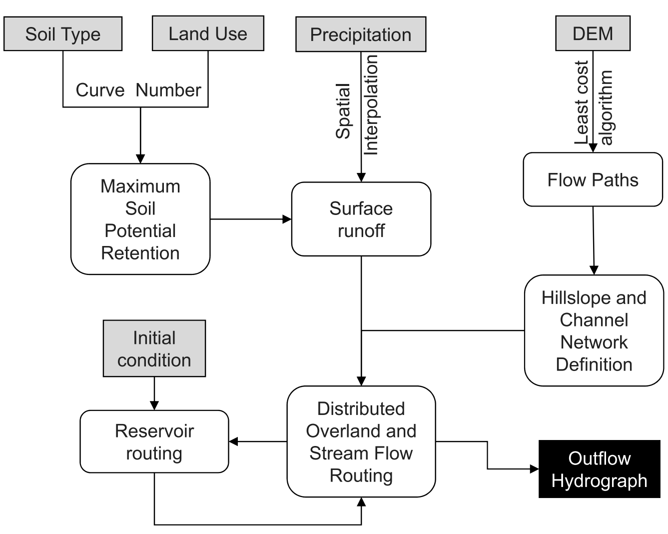

Fig. 2. FEST model structure

Recent work demonstrated the influence of the topographic index on 5-days precipitation amount to distinguish between three AMC classes (Miliani et al. 2011). The values of CN for the AMC II condition are tabulated by the Soil Conservation Service on the basis of soil type and land use. The values of CN for AMC I and AMC III are expressed in terms of the values of CN for AMC II through empirical relationships [see, e.g., Ponce (1989), Eqs. (5–13) and (5–14), p. 160]. In this way, different values of CN (i.e., soil retention capacity) can be assigned to the cells of the basin according to the antecedent moisture conditions. Finally, the surface runoff rate from the elemental cell, as integrated in time from $(j-1)$ to j, is computed at the jth time step as

$$P_e^j = P_n^j - P_n^{j-1} \tag{4}$$

The third component performs the runoff routing throughout the hillslope and the river network and the flow routing through the reservoirs. The runoff routing throughout the hillslope and the river network is performed through a diffusion wave scheme based on the Muskingum-Cunge method in its nonlinear form with the time variable celerity (Ponce and Yevjevich 1978; Ponce 1989; Ponce and Chaganti 1994). The model computes the surface runoff in each cell and propagates it along any reach of the basin flow path.

The routing scheme is given by

$$Q_{i+1}^{j+1} = C_1 Q_i^{j+1} + C_2 Q_i^j + C_3 Q_{i+1}^j + q_{i+1}^{j+1} \tag{5}$$

with i and $i+1$ = up and down-stream nodes of the generic channel (hillslope) crossing a grid cell; j and $j+1$ = discrete time steps; q_{i+1}^{j+1} = lateral inflow rate to the elemental channel; and C_r $(r = 1, 2, 3)$ = routing coefficients. These are obtained by use of the continuity equation for the channel (hillslope) between the i and $i+1$ nodes and with the assumption that the storage volume W is a linear function of inflow (Q_i) and outflow (Q_{i+1}) discharges

$$W = t[\varepsilon Q_i + (1-\varepsilon)Q_{i+1}] \tag{6}$$

where τ = coefficient related to the mean time taken by the wave to propagate along the channel; and ε = dimensionless weighting factor. The routing coefficients C_r are expressed as

$$C_1 = \frac{\Delta t - 2\tau\varepsilon}{2\tau(1-\varepsilon) + \Delta t} \tag{7}$$

$$C_2 = \frac{\Delta t + 2\tau\varepsilon}{2\tau(1-\varepsilon) + \Delta t} \tag{8}$$

$$C_3 = \frac{2\tau(1-\varepsilon) - \Delta t}{2\tau(1-\varepsilon) + \Delta t} \tag{9}$$

where $\tau = \Delta x/\chi$; $\varepsilon = (1/2)[1 - Q/(B\chi S_0 \Delta x)]$; $\chi = (dQ_i^j)/(d\Omega)$ is the kinematic wave celerity; Δx = incremental length; S_0 = channel bed slope; B = channel width; Ω = cross-sectional flow area; and Δt = time interval used for numerical integration. The lateral inflow rate to the elemental channel, q_{i+1}^{j+1}, is expressed as

$$q_{i+1}^{j+1} = A_0 \frac{P_{e_{i+1}}^{j+1}}{\Delta t} \tag{10}$$

with $P_{e_{i+1}}^{j+1}$ denoting direct runoff rate from the elemental cell $(i+1)$ with area A_0 as integrated in time from $j\Delta t$ to $(j+1)\Delta t$, and estimated by Eq. (4). The wave celerity $\omega_{i,j}$ is assumed to be variable in time because

$$\omega_{i,j}(t) = 5/3 V_m(t) \tag{11}$$

with V_m (m/s) denoting the mean velocity in the reach at a given time step as estimated from the Manning-Gauckler-Strickler friction equation

$$V_m = c_s R^{2/3} S_0^{0.5} \tag{12}$$

where R (m) is the hydraulic radius; and $c_s(\mathrm{m}^{1/3}\ \mathrm{s}^{-1})$ is the apparent Gauckler-Strickler roughness coefficient. For rectangular channel cross sections, Eq. (12) can be expressed as (Montaldo et al. 2004)

$$V_m = \left[k_s \left(\frac{r_f}{r_f+2}\right)^{2/3} S_0^{0.5}\right]^{0.75} \left(\frac{Q}{r_f}\right)^{0.25} \tag{13}$$

with r_f denoting the ratio between cross-section width and flood flow height; and Q is estimated through a three-point average method (Ponce and Yevjevich 1978). Therefore, by assuming rectangular channel cross sections, the wave celerity is characterized by only two coefficients relative to channel geometry and roughness: r_f and c_s.

Flow routing through a reservoir is described using the third-order Runge-Kutta method (Carnahan et al. 1969; Chow et al. 1988) for the classical level pool scheme. This is based on the continuity of mass equation

$$\frac{dS_r}{dt} = I(t) - Q(t,H) \tag{14}$$

where S_r = water storage in the reservoir; $I(t)$ = reservoir inflow as a function of time t; and $Q(t,H)$ = reservoir outflow as a function of time and water elevation H. Because the reservoir water surface A_r is a function of the water elevation, the change in storage dS_r due to a change in elevation is equal to $A_r(H)d(H)$. Thus, the continuity equation for the reservoir flow can be rewritten as

$$\frac{dH}{dt} = \frac{I(t) - Q(t,H)}{A_r(H)} = f(t,H) \tag{15}$$

where t is the independent variable and H is the dependent variable. When one elemental cell corresponds to a reservoir instead of a channel or a hillslope, Eq. (15) is substituted for Eq. (5) in the routing scheme.

To solve Eq. (15) using a third-order integration scheme, three small increments of the independent variable, time, using known values of the dependent variable H are made. The water elevation H at the $(j+1)$th time step is expressed as

$$H_{j+1} = H_j + \frac{1}{4}(\Delta H_1 + 3\Delta H_3) \tag{16}$$

where the three successive approximations are estimated as

$$\Delta H_1 = \frac{I(t_j) - Q(H_j)}{A_r(H_j)} \Delta t \tag{17}$$

$$\Delta H_2 = \frac{I(t_j + \frac{\Delta t}{3}) - Q(H_j + \frac{\Delta H}{3})}{A_r(H_j + \frac{\Delta H}{3})} \Delta t \tag{18}$$

$$\Delta H_3 = \frac{I(t_j + 2\frac{\Delta t}{3}) - Q(H_j + 2\frac{\Delta H}{3})}{A_r(H_j + 2\frac{\Delta H}{3})} \Delta t \tag{19}$$

In case of inline reservoir, relationship between reservoir water level and outflow is assigned as a lookup table for a finite number of values. Intermediate values are found by linear interpolation. In case of lateral reservoir, two tables are defined: the first for the relationship between river flow and reservoir inflow, the second for the relationship between reservoir water level and outflow.

Calibration and Validation

The model was subjected to a process of calibration and validation by comparison of simulated and observed discharge. Flood events that caused widespread inundation were excluded from the calibration and validation process because the objective of this paper is to compute design flood under the assumption that discharge is not affected by anthropic constraints. Furthermore, the observed discharge hydrograph was subdivided into its superficial and deep flow components (Pilgrim and Cordery 1993) because the hydrologic distributed model cannot simulate baseflow. Two flood events were included in calibration, the one that occurred between April 26 and 30, 2009, and the one that occurred between June 6 and September 6, 2009. The flood event that occurred between March 28 and 31, 2009, was used for validation.

Classic indexes of the goodness of fit of simulations were used: peak discharge relative error, $\varepsilon_{q\ \max}$; flood volume relative error, $\varepsilon_{\mathrm{vol}}$; root mean square error, RMSE (Willmott 1982); and Nash and Sutcliffe (1970) efficiency, η. The values of the indexes are given in Table 2 and simulation results are shown in Fig. 3.

After calibration and validation, the hydrological model was used for simulation of an intense major convective thunderstorm that occurred in July 2009 that caused extensive inundation along the watercourse, with strong impact on discharge measurement at basin outlet [Fig. 3(d)]. Because the hydrological model was calibrated to simulate natural discharge, peak flow and flood volume overestimation was approximately 60% and 115%, respectively. These differences can be assumed as an indicator of the severe artificial alteration experienced by Olona in the past years that reduced river conveyance.

Indirect Design Discharge Assessment

From a family of depth duration frequency 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 (Fig. 4). According to the critical flood design criterion, the probable maximum peak design flood (PMPDF)

Table 2. Goodnes of Fit Indexes Obtained in Calibration and Validation of the Hydrological Model

Flood	$Q_{\mathrm{obs}}^{\max}$ (m^3/s)	V_{obs} (m^3)	$\varepsilon_{q\ \max}$ (—)	$\varepsilon_{\mathrm{vol}}$ (—)	RMSE (m^3/s)	η (—)
April 26–30, 2009	52.29	4.01×10^6	−0.11	−0.11	6.45	0.74
June 6–September 6, 2009	9.70	1.43×10^5	−0.17	−0.15	0.77	0.65
March 28–31, 2009	8.70	3.58×10^5	−0.05	0.30	0.73	0.78

Note: Indexes obtained are observed peak flow, $Q_{\mathrm{obs}}^{\max}$; volume of observed discharge hydrograph, V_{obs}; peak discharge relative error, $\varepsilon_{q\ \max}$; flood volume relative error, $\varepsilon_{\mathrm{vol}}$; root mean square error, RMSE; and Nash and Sutcliffe (1970) efficiency, η.

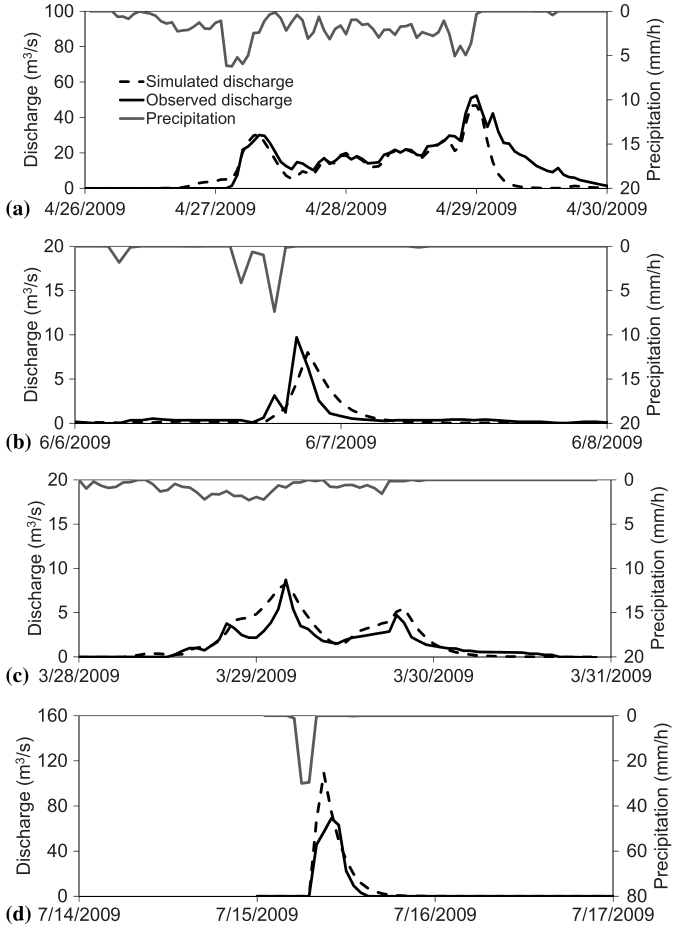

Fig. 3. Flood events occurred on the Olona River used for (a and b) calibration; (c) validation; (d) July 2009 event, not included in calibration and validation, is shown as example of flood that caused inundation

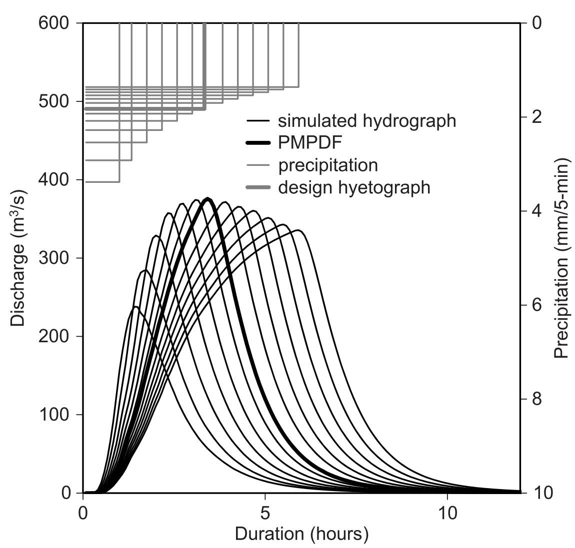

Fig. 4. Procedure for the search of the design flood defined as that event that is characterized by the maximum peak flow; the probable maximum peak design flood (PMPDF) and design hyetograph are reported; the results refer to the Olona River at Lozza, Italy, for the 100-year return period

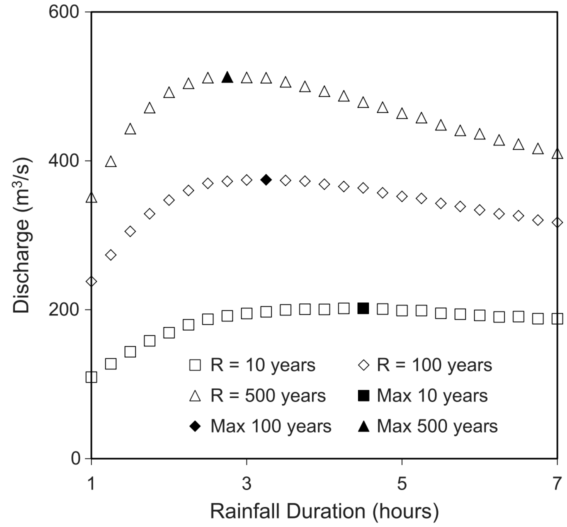

Fig. 5. Series of peak discharges obtained from different rainfall durations for different return periods, R, for the Olona River at Lozza, Italy; the design hydrograph for a given return period is the one characterized by the maximum value of peak discharge (solid black marks)

for a given return period is the one related to that rainstorm duration (d) that causes the hydrograph with the maximum peak discharge

$$\text{PMPDF: } \max[Q(d,t)] \tag{20}$$

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

In Fig. 5, the series of peak discharges for 10-, 100-, and 500-year return periods are reported for the Olona River at Lozza, Italy, for AMC III. The maximum value, marked with solid black, is the one associated to the PMPDF. Critical duration is defined as the one precipitation event that caused the maximum peak hydrograph.

Direct Design Discharge Assessment

Available discharge measurements are not sufficient to perform reliable statistical inference for direct assessment of design flood. In ungauged or poorly gauged catchments, regional analysis of flood peak discharges is used for more accurate estimates of flood quantiles. This is based on the identification of homogeneous zones, where the probability distribution of annual maximum peak flows is invariant, except for a scale factor represented by an index flood, μ_Q (Dalrymple 1960; Bocchiola et al. 2003). The index flood method is based on the estimation of the regional growth curve of the dimensionless quantile x_T; accordingly, the T-year flood flow q_T is estimated as (De Michele and Rosso 2002; Brath et al. 1997)

$$q_T = x_T \mu_Q \tag{21}$$

Direct assessment of index flood could be performed from annual flood series (AFS). If at a given river site, an n-year maximum annual flood peak series of measurements is available, the index flood can be estimated as the mean of sample data $q_1, \ldots, q_n$ [Eq. (22)]

$$\mu_Q = \frac{1}{n}\sum_{i=1}^{n} q_i \tag{22}$$

If n'-year data are available, index flood could be estimated from the mean of the flood peaks over a threshold series,

$q'_1, \ldots, q'_{n'}$, also referred to as the partial duration series (PDS). One computes

$$q_{\mathrm{PDS}} = \frac{1}{n'} \sum_{i=1}^{n'} q'_i \quad (23)$$

The index flood is associated with the mean of flood peaks of Eq. (22) through the rate of occurrence, λ, of the peaks over the threshold and the parameters of the PDS growth curve. For the case of GEV distribution for the maximum annual flood peaks, the index flood is given by (Bocchiola et al. 2003; Kjeldsen and Jones 2007)

$$\mu_Q = \frac{1}{\varepsilon + \frac{\alpha}{k}\left(1 - \frac{\lambda^k}{1+k}\right)} q_{\mathrm{PDS}} \quad (24)$$

where ε, α, and k = GEV parameters.

In ungauged sites, a data transfer scheme can be employed. In fact, maximum annual flood peaks in homogeneous regions are characterized by statistical scale invariant properties with respect to drainage areas (Gupta et al. 1994; Robinson and Sivapalan 1997). If A_g is the drainage area of the gauged section and A that of the ungauged one, this approach yields (Bocchiola et al. 2003)

$$\mu_Q(A) = \mu_Q(A_g)\left(\frac{A}{A_g}\right)^m \quad (25)$$

where m = regional scaling exponent.

Results and Discussion

Design hydrographs obtained with the indirect method for the 100-year return period for the Olona River at Lozza, Italy, are presented in Fig. 6. Hydrograph base time and time to peak decrease significantly as the return period increases. Precipitation intensity, in fact, increases with return period, causing an increase of runoff and flood celerity that, in turn, reduces flood travel time, as expected from the Muskingum-Cunge method with variable celerity that can describe the nonlinear relationship between water level and flow celerity.

Design discharge and critical duration for 10-, 100-, and 500-year return periods computed according to the indirect method are reported for all sections in Table 3. As expected, design discharge and critical duration increase with area of the basin. Critical duration, in agreement with behavior shown in Fig. 6, decreases as the return period increases for a given section.

Direct assessment of design flood can be performed in the framework of the flood evaluation (VAPI) project carried out by the National Group for Prevention from Hydrological Disasters (GNDCI) supported by the National Research Council (CNR) of Italy (De Michele and Rosso 2001). This project involves studies based on the statistical analysis of the frequency of annual maximums of extreme rainfall and observed discharges as documented by the Italian Hydrographic Services (S.I.I.). For this study, the specific work performed for northwestern Italy, including Lombardy, Piedmont, the Aosta Valley, Liguria, and Emilia Romagna regions, was employed (De Michele and Rosso 2002). The regionalization procedure is based on the generalized extreme value distribution, which proved to be accurate to explain observed peak flows. The Olona River is included in the homogeneous region A, Central Alps and Prealps, for the Po River subbasins from Chiese to Sesia, with range of validity 40–2,500 km^2.

Index flood computed using the partial duration series method is 59.7 m^3/s. Design flood at Lozza, Italy, for 10-, 100-, and 500-year return periods was obtained by multiplying index flood by the quantile of normalized flood flows in the region, equal to 1.68, 2.93, and 4, respectively. Design discharge for ungauged sections can be obtained using data transfer scheme by applying Eq. (25) with $m = 0.799$, which is valid for region A. Results of the direct procedure and data transfer for design discharge assessment are reported in Table 4.

In Fig. 7, the 100-year return period specific discharges per unit area obtained with the indirect method is compared with the ones from the direct method. The indirect method provides discharge significantly greater than the direct method. This is in agreement with the method for index flood estimation based on annual

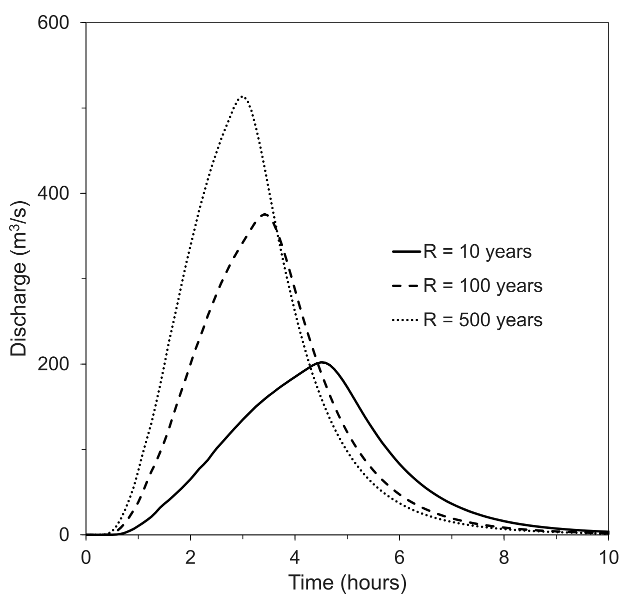

Fig. 6. Design hydrographs obtained with the indirect method for the Olona River at Lozza, Italy, for three return periods, R: 10, 100, and 500 years

Table 3. Design Discharge Q and Critical Duration D for 10-, 100-, and 500-Year Return Periods Computed according to the Indirect Method

Section	Q_{TR10} (m^3/s)	Q_{TR100} (m^3/s)	Q_{TR500} (m^3/s)	D_{TR10} (h)	D_{TR100} (h)	D_{TR500} (h)
Ol-1	17.02	35.52	51.15	3.33	2.25	1.92
Ol-2	14.16	27.87	38.79	2.75	1.92	1.75
Ol-3	53.46	101.85	139.87	2.50	2.17	2.00
Ol-4	63.5	119.71	162.33	2.67	1.92	1.92
Ol-5	82.59	154.98	210.21	2.58	2.00	2.00
Ol-6	114.69	215.1	293.39	3.42	2.42	2.33
Ol-7	149.77	279.09	377.49	3.33	2.58	2.25
Ol-8	191.07	356.49	488.54	4.00	3.33	3.08
Ol-9	196.08	366.91	502.14	3.92	3.33	2.83
Lozza, Italy	202.04	375.66	514.08	4.42	3.33	2.92

Table 4. Design Discharge Q for 10-, 100-, and 500-Year Return Periods Computed according to the Direct Regional Method

Section	Q_{TR10} (m^3/s)	Q_{TR100} (m^3/s)	Q_{TR500} (m^3/s)
Ol-6	45.25	78.92	107.74
Ol-7	49.18	85.78	117.10
Ol-8	77.69	135.50	184.99
Ol-9	79.36	138.40	188.94
Lozza, Italy	81.14	141.52	193.20

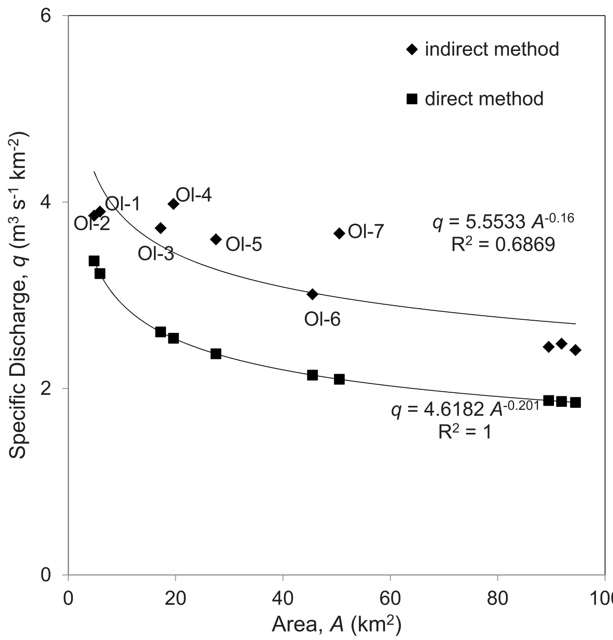

Fig. 7. Specific discharge per unit area obtained with direct and indirect methods

maximum peak flow measurements that are strongly biased by upstream river overflows that reduce discharge. Specific discharges by the indirect method show a negative trend with the basin area. However, local increase of specific discharge with basin area is reported for pairs of sections Ol-2 Ol-1, Ol-3 Ol-4, and Ol-6 Ol-7, which is in contrast with expected result. This is due to a significant increment of percentage of urban area, which compensates the increment of the basin area (Table 1).

Specific discharges obtained with the direct method exhibit a negative trend with area, perfectly reproduced by the power law, as expected from the application of Eq. (25).

Conclusions

In highly urbanized river basins, annual maximum peak flow measurements may be strongly biased by upstream river overflows, especially when rivers have been channelled into artificial drainage. In such a situation, in fact, overflow of the watercourses and subsequent inundation can be frequently observed during storm events. In these cases, direct estimation of design flood can lead to significant underestimation of peak flow with negative impacts on downstream river reach when subsequent modification of water course would increase river conveyance, with catastrophic consequences in terms of loss of life and damage to property.

A procedure for indirect estimation of design flood is presented. The proposed approach has been tested on the case of the Olona River north of Milan, Italy, most of which flows through densely developed areas causing damage to the population, especially during the rainy seasons.

It was shown that it is possible to obtain an hydrograph for any duration at a given frequency from a family of depth duration frequency curves by transforming rainfall into runoff, and finally, a series of hydrographs for each return period. According to the critical flood design criterion, the design flood for a given return period is the one related to that rainstorm duration (critical duration) that causes the hydrograph with the maximum peak discharge. For rainfall-runoff transformation, a spatially distributed hydrological model was employed. This allows taking into account the heterogeneity that generally characterizes river basins with a high degree of urbanization. In the Olona River case study, for example, it was shown that differences in percentage of urban area can explain an increase of specific design discharge per unit area with basin area.

The proposed approach can also provide design hydrograph. This is useful in cases when design flood only is not sufficient for planning or designing purposes such as retention pool design or flood map assessment (Ravazzani et al. 2009).

Despite the single case study presented in this paper, it could be assumed to be representative of the response of densely urbanized river basins. In these cases, differences between direct and indirect methods for design flood assessment are an indicator that direct methods can lead to underestimation of design flood with catastrophic consequences in terms of loss of life and damage to property. The methodology described in this paper can be extended to all those case studies in which water courses have been significantly altered by human activities.

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