Assessing Downstream Impacts of Detention Basins in Urbanized River Basins Using a Distributed Hydrological Model

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Received: 24 June 2013 / Accepted: 20 January 2014 /
Published online: 1 February 2014

1 Introduction

It is widely recognized that conversion of the landscape from natural vegetated area to urban areas is responsible for significant changes in hydrological runoff characteristics (Viessman and Lewis 2003; Su et al. 2010). As urban development occurs, the increase of impervious surface areas increases the volume of surface runoff and decreases infiltration volumes (Guo 2001). Detention basins can be used in an attempt to compensate reduction of natural infiltration, storage, and the attenuation of flow that is lost through urbanization. They are

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designed to reduce the flood peak by temporarily storing the excess storm water and then releasing the water volume at allowable rates over an extended period. Detention basin storage facilities play an important role in the control of pollution caused by combined urban stormwater and sewer overflow (Tung 1988). Sometimes they can be adapted to act as water storage during irrigation periods in order to reduce agricultural water shortages (Camnasio and Becciu 2011).

Detention basins are designed to reduce peak flow rate and they are not generally designed to attenuate runoff volume (Emerson et al. 2005). This implies possible extended runoff rates and increased volume in downstream locations that can cause problems to existing detention storages (Goff and Gentry 2006).

A detention basin may operate as an on-stream or off-stream facility (Guo 2012). An on-stream detention basin acts as a constriction in a stream, only allowing a certain amount of water through at a time. When the capacity of the outlet structure is exceeded, a portion of the stream's flow is temporarily stored. The stored water is released over an extended period of time, thus preventing flooding downstream by delaying discharge of runoff. The inflow channel associated with an off-stream basin is allowed to carry its base flow not to exceed the downstream capacity. When the inflow channel is full, the excess flood water is diverted into the off-stream basin through the side weir. Stored water can be released back to the channel after the flood recession.

Storage facilities in a drainage network should be placed at strategic locations in order to effectively attenuate the peak flood flows (Guo 2004). Although an individual detention basin can meet the design objectives, it is easier to localize a number of smaller detention ponds in urban developed area. The combined effect of a network of small detention ponds will tend to offset the time of concentration and reduce the peak flow of the contributing watershed; however, the resulting outlet hydrograph can combine with the other watershed flows in the region to produce flows that are more damaging than the predeveloped condition (Ormsbee et al. 1984).

Spatially distributed hydrological models are valuable tools to predict floods in heterogeneous urbanized area (Ravazzani et al. 2012). Distributed models can help to assess the effectiveness of a network of stormwater detention basins in controlling watershed peak runoff rates. Moreover, this approach can correctly model extreme storm events that exceed the capacity of individual detention basins.

This study presents the distributed hydrological model FEST as an effective tool for the assessment of effectiveness of a network of detention basins in the Olona river basin, a small watershed in northern Italy. The hydrological model is used to quantify the impact of the network of detention ponds on the design hydrograph and thus how increased runoff volume can affect an existing downstream large detention basin.

2 The Olona 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 intense rainfall events. 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, usually contained in narrow artificial banks.

The area at the considered closing section (Lozza) is 94.5 km^2 (Fig. 1). Its elevation ranges from 271 m a.s.l. at the outlet to approximately 1,070 m a.s.l. at the Tre Croci crest. The average elevation is 455 m a.s.l.. 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 about 1,600 mm.

Olona river experienced significant floods since 1584, year of the first reported event. Five major floods were reported in the last 20 years.

In 2010 an on-stream detention basin (Ponte Gurone dam) was built just upstream Lozza section to preserve downstream locations from flooding (Fig. 1). The basin maximum storage capacity is 1,520,000 m^3 over a 383,000 m^2 total area. The Ponte Gurone dam is regulated through three automatic gates to keep release rate under 36 m^3/s considered as maximum allowable discharge for downstream locations. A 114 m long sill free weir with Creager profile is expected to come into operation when water stage in the reservoir reaches 10.8 m.

Nine river sections along Olona river were considered in this analysis, including Lozza basin outlet as shown in Fig. 1. Basin area, average slope and average elevation are reported in Table 1. Meteorological and hydrologic data as well as depth duration frequency curves used in this study are described in Ravazzani et al. (2012).

3 Description of Distributed Hydrological Model FEST

Here, for the rainfall-runoff transformation, the FEST (flash-Flood Event-based Spatially distributed rainfall-runoff Transformation, including reservoirs system) model was employed (Montaldo et al. 2007; Rabuffetti et al. 2008; Pianosi and Ravazzani 2010; Corbari et al. 2011).

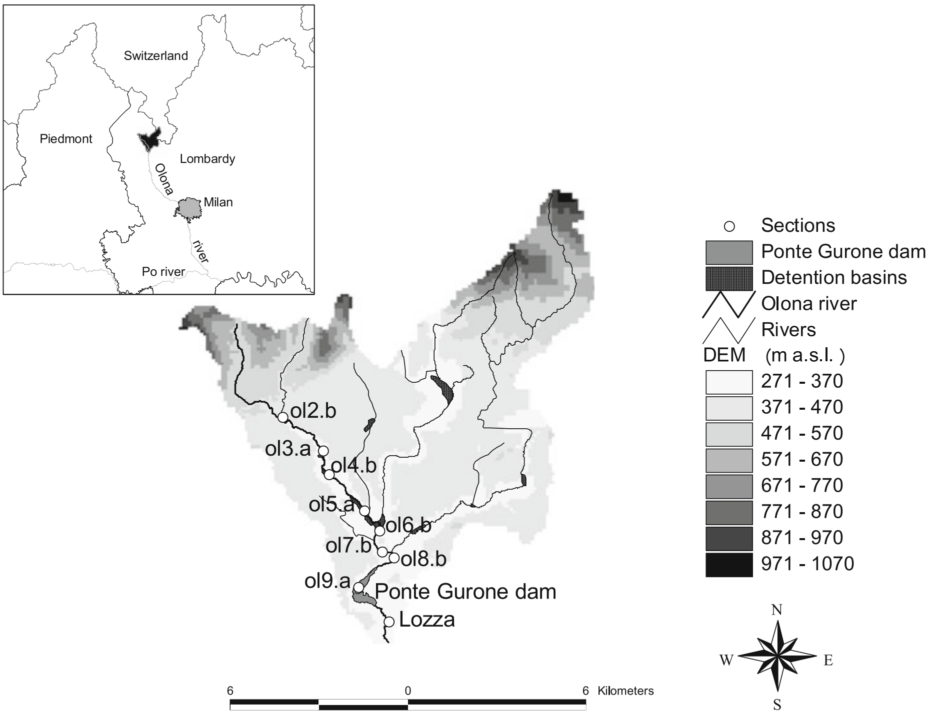

Fig. 1 The Olona river basin showing locations of cross sections and detention basins considered in the analysis

Table 1 Cross sections considered in the design flood assessment: code of cross section, extent of basin area in km^2, average slope (%), and average elevation (m a.s.l.)

Section	Area (km^2)	Slope (%)	Elevation (m a.s.l.)
ol2.b	11.7	23.1	552.2
ol3.a	17.2	19.5	508.5
ol4.b	18.3	18.9	500.3
ol5.a	19.9	18.3	488.2
ol6.b	45.5	15.9	446.3
ol7.b	56.8	15.5	450.6
ol8.b	89.9	15.5	464.5
ol9.a	92.1	15.4	461.9
Lozza	94.5	15.3	458.4

FEST is a distributed, raster based hydrologic model developed at the Politecnico di Milano focusing on flash flood event simulation. As a distributed model, FEST can manage spatial distribution of meteorological forcings, and heterogeneity in hillslope and drainage network morphology (slope, roughness, etc..) and land use. Spatial resolution is 100 m in the present application.

The FEST model has three principal components. In the first component, the flow path network is automatically derived from the digital elevation model using a least-cost path algorithm (Ehlschlaeger 1989). In the second component, the surface runoff is computed for each elementary cell using the SCS-CN method (Soil Conservation Service 1986; Miliani et al. 2011) in its differential form (Mancini and Rosso 1989). 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 via a diffusion wave scheme based on the Muskingum-Cunge method in its non-linear form with the time variable celerity (Ponce and Yevjevich 1978; Ponce and Chaganti 1994).

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:

$$\frac{\mathrm{dS_r}}{\mathrm{dt}} = I(t) - Q(t, H) \tag{1}$$

where S_r is water storage in the reservoir, $I(t)$ is the reservoir inflow as a function of time t, and $Q(t,H)$ is the reservoir outflow as a function of time and water elevation H. Since 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:

$$\frac{\mathrm{dH}}{\mathrm{dt}} = \frac{I(t) - Q(t, H)}{A_r(H)} = f(t, H) \tag{2}$$

where t is the independent variable, and H is the dependent variable.

To solve Eq. (2) 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

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

where the three successive approximations are estimated as

$$\Delta H_1 = \frac{I(t_j) - Q(H_j)}{A_r(H_j)} \Delta t \tag{4}$$

$$\Delta H_2 = \frac{I\left(t_j + \frac{\Delta t}{3}\right) - Q\left(H_j + \frac{\Delta H_1}{3}\right)}{A_r\left(H_j + \frac{\Delta H_1}{3}\right)} \Delta t \tag{5}$$

$$\Delta H_3 = \frac{I\left(t_j + 2\frac{\Delta t}{3}\right) - Q\left(H_j + 2\frac{\Delta H_2}{3}\right)}{A_r\left(H_j + 2\frac{\Delta H_2}{3}\right)} \Delta t \tag{6}$$

In case of on-stream 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 off-stream 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.

The model was subjected to a process of calibration and validation by comparison of simulated and observed discharge. Two flood events were included in calibration, the one occurred between 4/26/2009 and 4/30/2009, and the one occurred between 6/6/2009 and 6/9/2009. The flood event occurred between 3/28/2009 and 3/31/2009 was used for validation. For more details the reader is referred to Ravazzani et al. (2012).

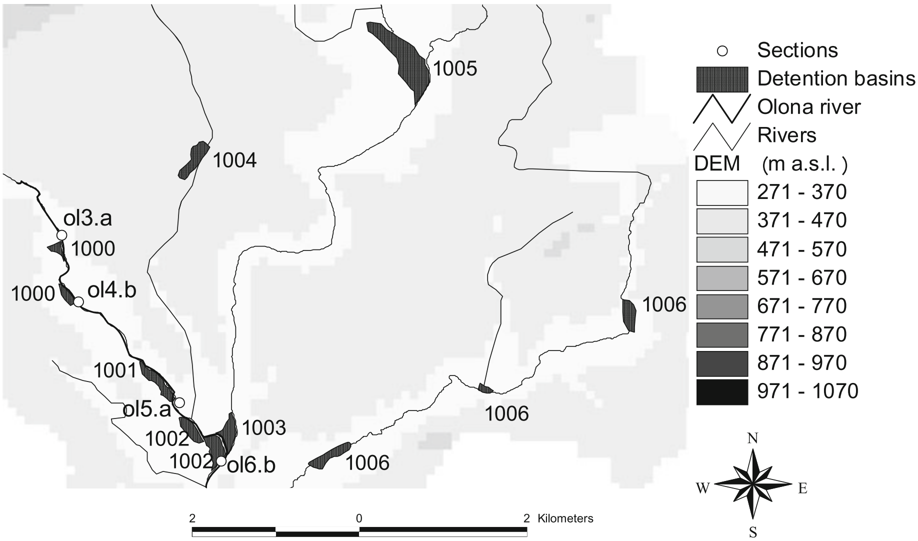

Fig. 2 Location of the designing detention basins

Table 2 Detention basins characteristics: identification code, number of elements, area and available storage volume

Id	N° elements	Area (m^2)	Volume (m^3)
1000	2	39,000	97,500
1001	1	55,000	137,500
1002	2	75,000	187,500
1003	1	55,000	137,500
1004	1	60,000	150,000
1005	1	230,000	575,000
1006	3	100,000	250,000
Total	–	614,000	1,535,000

4 Detention Basins Design

Urban development in Olona river basin limits the possibility to locate detention basins to small portion of flood plain adjacent to the water course. Seven sites were identified to locate detention basins (Fig. 2). Single detention basin is supposed to occupy all available area with volume linearly increasing till a maximum water depth of 2.5 m (Table 2). Total storage volume is comparable to the one of the downstream large Ponte Gurone dam. Detention basins 1000, 1002 and 1003 are composed of more than one in-line elements considered as a whole in terms of impact on discharge hydrograph. In order to test the impacts of detention basins on downstream locations, the seven detention basins were assumed to operate alternatively as on-stream as well as off-stream structures. Designing of lateral spillway and outlet structures of every detention basin was accomplished through an iterative procedure starting from the furthest upstream basin. The underlying principle is the assessment of design discharge hydrograph with indirect method (Ravazzani et al. 2009, 2012). According to this, from a family of depth duration frequency curves, by transforming rainfall into runoff, a series of hydrographs for each return period is obtained for different durations. The Probable Maximum Peak Design Flood (PMPDF) for a given return period, is the one related to that rainstorm duration that causes the hydrograph with the maximum peak discharge. Critical duration is the one of the precipitation event that causes the maximum peak hydrograph.

After design of one detention basin is achieved by optimization of lateral spillway and outlet structures with the objective of reducing discharge at allowable rates, the design

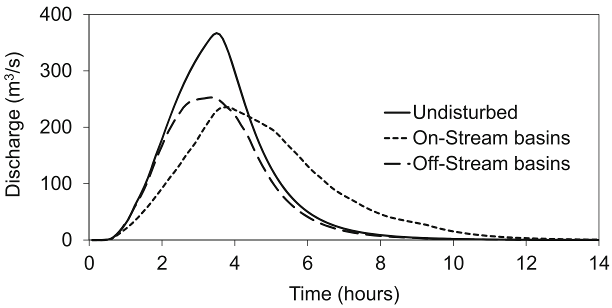

Fig. 3 Undisturbed design hydrograph for 100-year return period at section ol9.a and hydrographs caused by the same rainfall and affected by on-stream and off-stream detention basins

Table 3 Critical duration, d_c, peak discharge, Q, and hydrograph volume, V, at the nine sections in case of undisturbed river basin, on-stream detention basins, and off-stream detention basins completion

Section	Undisturbed			On-stream			Off-stream		
	d_c (min)	Q (m^3/s)	V (m^3)	d_c (min)	Q (m^3/s)	V (m^3)	d_c (min)	Q (m^3/s)	V (m^3)
ol2.b	140	64.97	446,484	140	64.97	446,484	140	64.97	446,484
ol3.a	130	101.85	779,121	130	101.85	779,121	130	101.85	779,121
ol4.b	125	110.07	830,073	200	91.29	1,039,932	125	88.65	773,646
ol5.a	130	111.81	866,127	110	77.65	687,960	110	77.65	687,960
ol6.b	145	215.1	1,767,174	200	108.53	3,197,181	160	100.12	1,150,737
ol7.b	155	279.09	2,416,818	215	164.21	2,844,762	100	169.6	1,356,810
ol8.b	200	356.49	3,687,585	310	243.21	4,867,662	125	246.56	2,210,463
ol9.a	200	366.91	3,791,589	310	253.21	5,005,851	130	258.91	2,336,685

hydrograph for downstream locations is recomputed including the effect of upstream detention basins on discharge routing. Recomputed hydrograph is then used for designing downstream detention basin. Iterative procedure stops when all detention basins are properly designed.

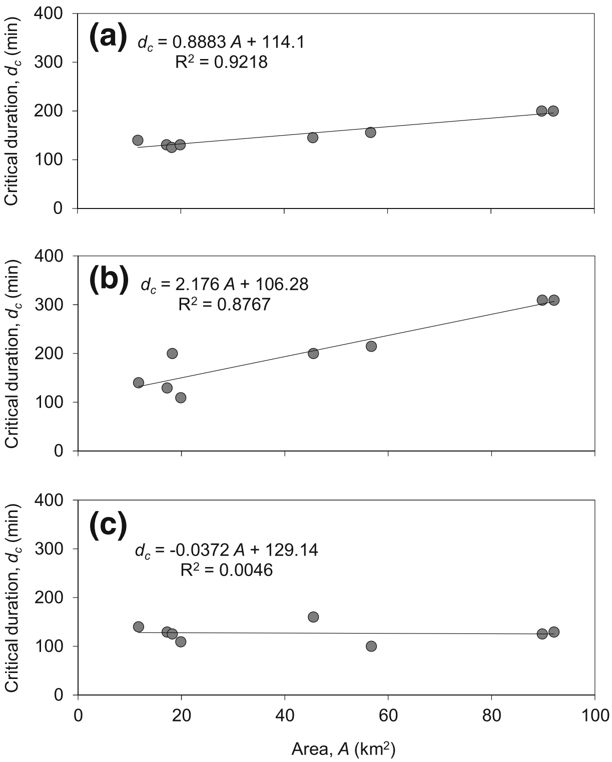

Fig. 4 Critical duration for 100-year return period as a function of basin area under three configurations: **a** undisturbed, **b** on-stream detention basins, **c** off-stream detention basins. Each point refers to a section out of the eigth included in this analysis

5 Impact Analysis

Impact of designing detention basins on undisturbed design hydrograph is shown in Fig. 3. Hydrographs affected by detention basins were computed by introducing the same rainfall, in terms of both depth and duration, that caused the undisturbed design hydrograph in the distributed hydrological model. Analysis was undertaken considering all on-stream detention basins together and then doing the same but by considering all detention basins off-stream. The two type of detention basins lead to nearly the same peak flow reduction (31 % and 36 % in case of off-stream and on-stream detention basins, respectively). On-stream facilities lead to shift ahead the time to peak, while flood peak affected by off-stream facilities is in phase with undisturbed hydrograph. Hydrograph volume obtained with off-stream detention basins is lower. This is due to simulation assumption to release stored volume in the reservoir after the end of the flood, in agreement with how off-stream detention basins are usually operated.

Operation of detention basins may modify river basin response characteristic to rainfall events. Generally, effect of on-line detention basins is to delay the peak flow hydrograph while preserving flood volume, effect of off-stream detention basins is to preserve the time to peak while reducing peak and flood volume. So while operation of detention basins may reduce the peak flow of design hydrograph computed in undisturbed condition, the effect of the change of river basin characteristics may lead to finding a design hydrograph different from the one obtained in undisturbed condition that may impact existing downstream facilities.

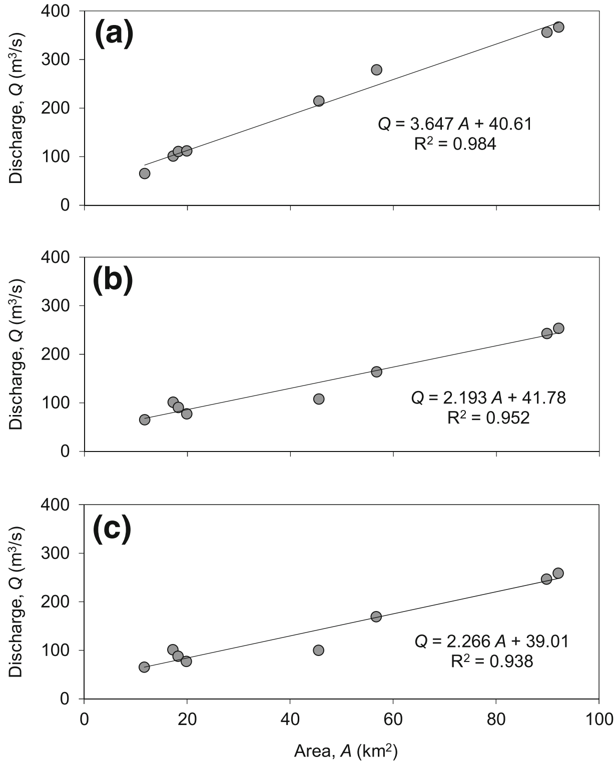

Fig. 5 Peak discharge for 100-year return period as a function of basin area under three configurations: **a** undisturbed, **b** on-stream detention basins, **c** off-stream detention basins. Each point refers to a section out of the eigth included in this analysis

In order to assess the impact of the designing detention basins on the existing downstream Ponte Gurone dam, the probable maximum peak design flood for 100 year return period was computed according to the above mentioned procedure under further two configurations: the case of considering the presence of seven on-stream detention basins, and the case of considering the presence of seven off-stream detention basins. Critical duration, peak discharge and flood hydrograph volume for 100-year return period as a function of basin area are presented in Table 3 and shown in Figs. 4, 5, and 6, respectively. Three configurations are shown: (a) undisturbed, (b) on-stream detention basins, (c) off-stream detention basins. Each point refers to a section out of the seven included in this analysis.

It is seen that both on-stream and off-stream detention basins show the same reduction in undisturbed peak flow since their operation has been optimized for the specific purpose of maintaining the flow rate within the range of the maximum allowable river discharge. Values for sections ol2.b and ol3.b remain unchanged in all the three configurations since there are no detention basins upstream of those sections.

Under undisturbed configuration, critical duration increases linearly with the area of the river basin. Under on-stream detention basins configuration, the critical duration increases linearly with the area of the basin more than in the previous case. This is due to the effect of diffusion on the flood hydrograph propagation. In this case in fact, the procedure for the assessment of the critical event identifies as critical the hydrograph caused by a precipitation with higher duration. Under off-stream detention basins configuration, the critical duration is

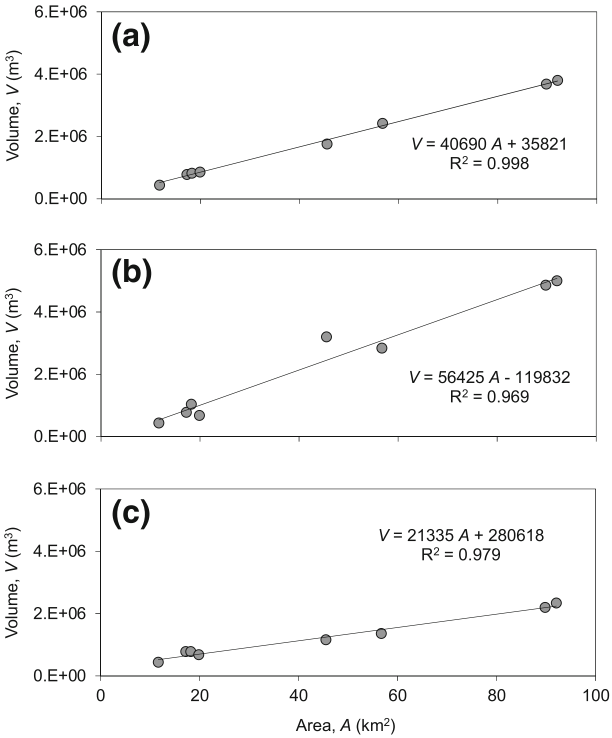

Fig. 6 Flood hydrograph volume for 100-year return period as a function of basin area under three configurations: **a** undisturbed, **b** on-stream detention basins, **c** off-stream detention basins. Each point refers to a section out of the eigth included in this analysis

approximately constant with increasing of area. This is explained by the fact that off-stream detention structures do not alter the time to peak so that outflow hydrograph remains in phase with the undisturbed hydrograph but with a reduced peak flow. Stored volume is released only subsequently to the transit of the flood.

The different behavior of the two types of detention basins is reflected in the volumes of the design flood hydrograph. In the case of undisturbed condition the volume increases linearly with the area of the river basin. Under on-stream detention basins configuration, the volume increases linearly with the area of the basin more than undisturbed condition. In case of off-stream detention basins the volume grows linearly with the area of the river basin but with a lower rate than the other two cases.

The three flood hydrographs obtained in the three considered configurations have different impacts on the existing Ponte Gurone detention basin (Fig. 7). It is well known, in fact, that detention basins are susceptible to flood volume and, for this reason, not necessarily a flood hydrograph with a reduced peak flow guarantees optimal operation of downstream detention basin. The undisturbed hydrograph is the one with the greatest impact on Ponte Gurone dam.

The peak of outflow hydrograph reaches 276 m^3/s, thus significantly exceeding the maximum allowable discharge (36 m^3/s) (Fig. 7a). The Ponte Gurone detention basin is therefore not able to sufficiently reduce the peak flow hydrograph with a return period of 100 years under undisturbed condition. The Ponte Gurone detention basin is not able to reduce the peak flow in the case of on-

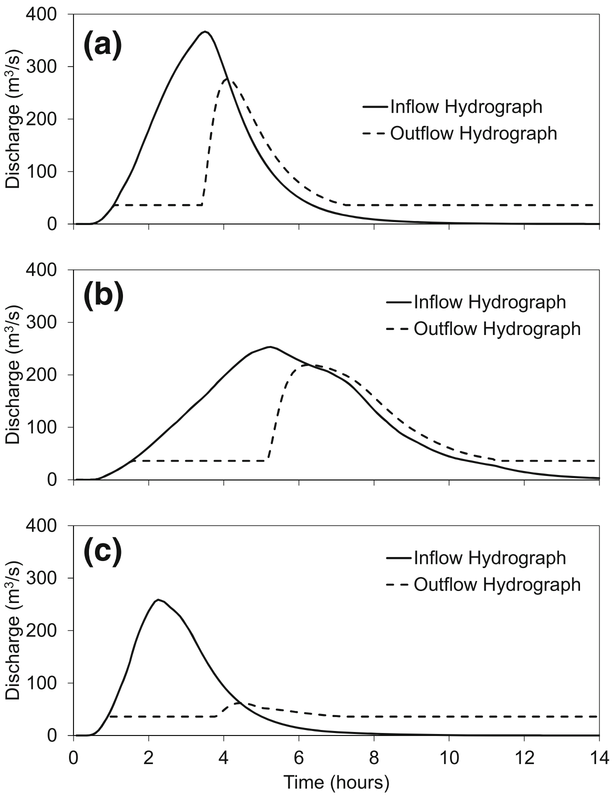

Fig. 7 Ponte Gurone dam inflow and outflow hydrograph for 100-year return period under three configurations: **a** undisturbed, **b** on-stream detention basins, **c** off-stream detention basins

stream detention basins, too. The design discharge is lower than the one obtained for undisturbed case, but the peak of outflow discharge reaches (219 m^3/s) (Fig. 7b). Assumption of operating the seven detention basins as off-stream structures, leads to the most favorable result: outflow discharge reaches 62 m^3/s correspondent to 76 % of peak flow reduction (Fig. 7c).

6 Conclusions

A procedure for the assessment of the impacts of detention basins on downstream locations is presented. The proposed approach has been tested on the case of river Olona in the north of Milan, Italy, where an on-stream detention basin was built in 2010 to preserve downstream locations from flood inundation. The distributed hydrological model FEST was used to assess design hydrograph and, in parallel to design the seven detention basins optimized for the specific purpose of maintaining the flow rate within the range of the maximum allowable discharge. In order to test the impacts of detention facilities on downstream locations, the seven detention basins were assumed to operate alternatively as on-stream as well as off-stream structures.

Assessment of the effect of detention basins on undisturbed design hydrograph showed that the two types of facilities lead to practically the same peak flood reduction.

In order to assess the impact of the designing detention basins on the existing downstream Ponte Gurone dam, the probable maximum peak design flood for 100 year return period was computed under further two configurations: the case of considering the seven on-stream detention basins, and the case of considering the seven off-stream detention basins. It was shown that under on-stream detention basins configuration, the critical duration increases linearly with the area of the basin more than in the undisturbed case, while under off-stream detention basins configuration, the critical duration is approximately constant with increasing of river basin area. As a consequence, in case of off-stream detention basins the hydrograph volume grows linearly with the area of the river basin with a lower rate than both undisturbed and on-stream facilities conditions.

The shape of the three possible hydrographs affect differently the existing Ponte Gurone detention basin. The undisturbed hydrograph is the one with the greatest impact on Ponte Gurone dam. The Ponte Gurone detention basin is not able to reduce the peak flow in the case of on-stream detention basins, too. Assumption of operating the seven detention basins as off-stream structures, leads to the most favorable result. The methodology described in the paper can be extended to all those case studies where the choice of technology for locally mitigating flood damage may affect existing downstream facilities.

Acknowledgments This work was supported by *Comune di Varese*, *Agenzia Interregionale per il fiume Po* (*AIPO*), and *Autorità di Bacino del Fiume Po*. We thank Regional Agency for Environmental Protection (*ARPA Lombardia*), *Centro Geofisico Prealpino* and *Meteonetwork* for providing hydrological and meteorological data. We thank the anonymous reviewers for their comments and suggestions which contributed to improve the quality of the manuscript.

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