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A methodology to derive Synthetic Design Hydrographs for river flood management

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ABSTRACT

The design of flood protection measures requires in many cases not only the estimation of the peak discharges, but also of the volume of the floods and its time distribution. A typical solution to this kind of problems is the formulation of Synthetic Design Hydrographs (SDHs). In this paper a methodology to derive SDHs is proposed on the basis of the estimation of the Flow Duration Frequency (FDF) reduction curve and of a Peak-Duration (PD) relationship furnishing respectively the quantiles of the maximum average discharge and the average peak position in each duration. The methodology is intended to synthetize the main features of the historical floods in a unique SDH for each return period. The shape of the SDH is not selected a priori but is a result of the behaviour of FDF and PD curves, allowing to account in a very convenient way for the variability of the shapes of the observed hydrographs at local time scale. The validation of the methodology is performed with reference to flood routing problems in reservoirs, lakes and rivers. The results obtained demonstrate the capability of the SDHs to describe the effects of different hydraulic systems on the statistical regime of floods, even in presence of strong modifications induced on the probability distribution of peak flows.

1. Introduction

In many problems of flood management, the elements of interest in the definition of hydrological risk are not only the peak discharge but also the flood volume and the shape of the hydrograph. Typical cases include design of control reservoirs and flood plain analysis concerning the definition of the inundation maps and the optimization of flood plain management in view of risk mitigation (Alfieri et al., 2016). A practical solution to these problems can be achieved through the estimation of Synthetic Design Hydrographs (SDHs).

Yue et al. (2002) distinguished four approaches to obtain SDHs: Traditional Unit Hydrograph (TUH), Synthetic Unit Hydrograph (SUH), Typical Hydrograph (TH) and Statistical Method (SM). Apart from the first two categories, that are based on rainfall-runoff models, TH and SM techniques try to obtain SDHs via frequency analysis of recorded flood hydrographs.

The TH hydrograph (Sokolov et al., 1976) is constructed by selecting the most representative historical flood hydrograph and rescaling its abscissa (duration) and ordinate (discharge values Q) to obtain the flood peak Q_{pT} and/or flood volume V_T corresponding to a given return period T. This approach suffers from some arbitrariness and cannot be considered satisfactory from a statistical viewpoint, since no attempt is made to include the shape variability of the observed flood hydrographs. To overcome this limitation, Sauquet et al. (2008) introduced a 'representative hydrograph', obtained by averaging the dimensionless hydrographs $Q(t)/Q_p$ centred around the peak position.

The SM approach (Yue et al., 2002; Pramanik et al., 2010; Serinaldi and Grimaldi, 2011) employs some Probability Density Functions (PDFs) to describe each recorded flood hydrograph, after baseflow separation. To account for the randomness of the hydrograph shape, Yue et al. (2002) introduced two variables, namely *shape mean* and *shape variance*, to estimate the two parameters of the beta PDF representing the hydrograph. Using this approach, one can derive a SDH with flood peak Q_{pT}, volume V_T, duration Θ_T and shape parameters corresponding to a given return period T, although only two characteristics among Q_p, V and Θ can be directly incorporated in the procedure, while the third one has to be eventually adjusted after the SDH construction. Pramanik et al. (2010) developed the idea proposed by Yue et al. considering other two-parameter PDFs (Weibull, Gamma and Lognormal). Serinaldi and Grimaldi (2011) further extended this approach selecting also a three-parameter PDF with finite support. Moreover, to partly account for mutual relationships among the variables, these authors analysed the results obtained by expected values conditioned to the quantile of the 'driving variable' to be chosen according to the scope of the study (Q_{pT}, V_T or Θ_T).

The statistical relation among the main hydrological variables of flood hydrographs has been studied also in a multivariate framework, mostly focusing on flood peak and flood volume in a bivariate approach. To this purpose, Sackl and Bergmann (1987) and Bergmann and Sackl (1989) used a bivariate normal distribution after a suitable normalization of the variables. Other studies involving the bivariate normal distribution were carried out by Goel et al. (1998) and Yue

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(1999). More appropriate bivariate distributions to handle the above mentioned variables were considered by the latter author (Yue et al., 1999; Yue, 2000, 2001a, 2001b), while a comprehensive discussion about bivariate return periods was given by Yue and Rasmussen (2002). More recently, the same problem has been faced by means of copula functions (Salvadori and De Michele, 2004). With respect to the bivariate distribution approach, the main advantage is the possibility to model the dependence structure of the variables regardless the type of the marginals, relaxing also the condition that the marginal distributions should come from the same family. A discussion about the different return period approaches in a multivariate framework is presented by Gräler et al. (2013).

The multivariate approach can also be used in conjunction with Monte Carlo simulation strategy as shown by De Michele et al. (2005) and Requena et al. (2013) dealing with the design flood for dam spillways. The latter authors also introduced an additional ('routed') return period to properly characterize the risk of dam overtopping on the basis of the maximum water level in the reservoir. A similar analysis was performed by Volpi and Fiori (2014), who used a simplified description of the system to obtain a structure-based approach in which the bivariate environment of hydrological loads (Q_p and V) is linked to the design discharge of the spillway by means of an analytic procedure that replaces the Monte Carlo technique. The last two papers indicate that the results obtained by design events based on theoretical multivariate return periods can differ from those obtained by structure-based procedures to a degree depending on the system under investigation (see also Serinaldi, 2015). On the other hand, the approaches involving the analysis of global flood volume V and/or duration Θ require suitable criteria for a previous identification of independent flood events (e.g. Balistrocchi et al., 2013) and additional hypotheses to assign the time distribution of V (i.e. the shape of the SDHs).

In this paper we propose a different methodology to derive SDHs, based on the statistical analysis of the historical maximum average discharges Q_D in given durations D through the estimation of the Flow Duration Frequency (FDF) reduction curves Q_{DT} (NERC, 1975; Bacchi et al., 1992; Franchini and Galeati, 2000) and of the time location r_D of the peak in each duration. The latter is necessary to define the shape of rising and falling limbs of the hydrograph, which is important in many practical applications, like flow routing in rivers with floodplains, river bank seepage (Butera and Tanda, 2006), etc.

Although similar to the SM approach, the proposed methodology does not require to fit each recorded flood hydrograph with the same PDF, which sometimes could prove unsuited, but derives the shape of the SDH afterward on the basis of the behavior of Q_{DT} and r_D curves, synthesizing all the observed hydrograph shapes. Moreover, the procedure does not require to choose a 'driving variable' depending on the problem under investigation since the main characteristics of the historical floods are summarized in a single synthetic hydrograph for each return period, thus making easier and faster the use of SDH in practical problems. Finally, the proposed methodology allows to perform a unique simulation on the basis of the SDH corresponding to the design return period, whereas the alternative multivariate approach requires structure-based Monte Carlo techniques for a more reliable analysis, leading to a computational effort that in some cases can become prohibitive (for instance in 2D flood propagation problems).

After the presentation of the SDHs construction procedure, the validation of the methodology is performed with reference to flood routing problems in reservoirs, lakes and rivers by comparing the frequency distribution of peak outflow/downstream discharges deduced from the historical flood hydrographs with that obtained from the routing of the SDHs.

2. Material and methods

The proposed procedure for SDH construction is based on the estimation of the Flow Duration Frequency (FDF) reduction curves and on the Peak-Duration (PD) curve.

Both curves are obtained through the statistical analysis of historical hydrographs. The data sampling modality is illustrated in Fig. 1.

For each duration D ranging from 0 (corresponding to an instantaneous discharge) to a sufficiently large value, say, D_f representing the typical total duration of flood events for the considered river station the maximum average discharge

$$Q_D = \max\left(\frac{1}{D}\int_{t-D}^{t} Q(\tau)d\tau\right) \tag{1}$$

is extracted from the historical floods together with the ratio r_D ($0 \le r_D \le 1$) between the time prior to the peak and the duration D. Then a statistical analysis on each sample of Q_D is performed to derive the FDF curve furnishing the relation $Q_D(T)$-D for a given return period T. This statistical analysis can be performed on Annual Maxima (AMS) or Peak over a Threshold (PoT) series of Q_D. Once the Q_D samples have been selected, the corresponding r_D values are averaged for each duration to obtain the Peak-Duration (PD) curve which gives the average position $\bar{r}_D$ of the peak discharge in each duration D. The steps for the construction of the SDH are presented in the following Sections 2.1–2.3, while an example is given in Section 2.4.

2.1. FDF reduction curves estimation

The estimation of the FDF reduction curves can be obtained (NERC, 1975) relating the quantiles $Q_D(T)$ to $Q_0(T)$ by means of the reduction ratio $\varepsilon_D(T)$:

$$Q_D(T) = \varepsilon_D(T)Q_0(T), \quad \varepsilon_D(T) = \frac{Q_D(T)}{Q_0(T)}. \tag{2}$$

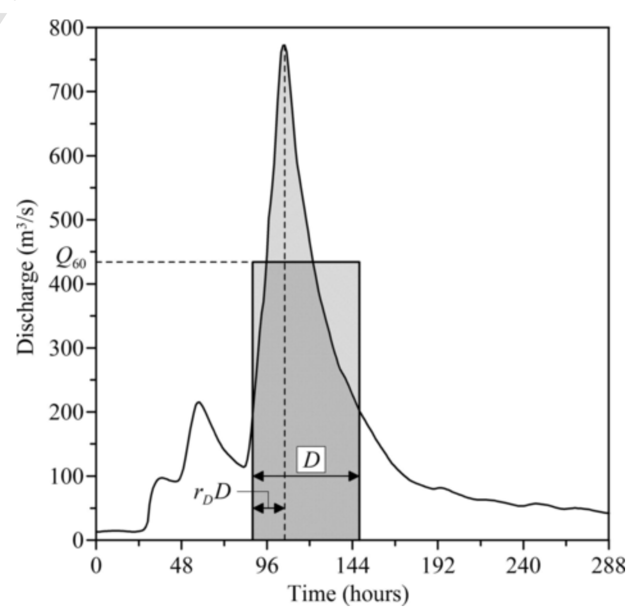

Fig. 1. Data sampling of Q_D and r_D from an historical hydrograph (D=60 h).

The quantiles $Q_D(T)$ can be estimated once the function $\varepsilon_D(T)$ and the probability distribution of peak flood discharges Q_0 have been identified and calibrated. Although in the general case the reduction ratio ε_D is a function of duration D and return period T, in many practical cases it can be assumed independent of the latter. This is strictly true only if – neglecting the influence of the statistical moments higher than the second – the coefficient of variation $CV(Q_D)$ and the probability distribution type of Q_D can be considered independent of D. Under these assumptions, usually well verified for medium-large watersheds, ε_D becomes independent of T and reduces to the ratio of the averages of Q_D and Q_0:

$$\varepsilon_D = \frac{\mu(Q_D)}{\mu(Q_0)}. \tag{3}$$

The position (2) together with (3) introduces several advantages in the statistical analysis. Firstly, the dependence of Q_D on T is contained only in the quantile of Q_0, for which longer historical series are usually available. On the other hand, Eq. (3) involves only the mean of Q_D, for which the sample size is less critical. Moreover, since the frequency analysis is performed uniquely on Q_0, potential inconsistencies coming from an independent analysis for each duration are avoided. Under the above assumptions, the estimation of the FDF curves reduces to the estimation of peak flood discharge quantiles $Q_0(T)$ and of the reduction ratio ε_D (3). For the first issue a huge amount of literature and standard procedures are available in different countries (e.g. Castellarin et al., 2012). For the second, some different functional forms have been proposed to fit the experimental data (NERC, 1975; Bacchi et al., 1992). Following a pure inductive approach, NERC (1975) proposed the following equation:

$$\varepsilon_D = (1 + bD)^{-c}, \tag{4}$$

where b and c are positive parameters. It was also found that b is correlated to the response time of the catchment, while c can be considered independent of the scale and the geomorphologic characteristics of the catchment.

Based on the crossing properties of a given threshold value from continuous gaussian stationary stochastic processes, Bacchi et al. (1992) derived the following theoretical formulation:

$$\varepsilon_D \approx \sqrt{\Gamma(D)} = \sqrt{\frac{\theta}{2D}\left[2 + e^{-\frac{4D}{\theta}} - \frac{3\theta}{4D}\left(1 - e^{-\frac{4D}{\theta}}\right)\right]} \tag{5}$$

in which $\Gamma(D)$ is the variance function, i.e. the ratio of the variances of the instantaneous process and of the integrated one. The time parameter θ (>0) is the scale of fluctuation, i.e. the integral of the autocorrelation function of the instantaneous process. Besides the advantages coming from its theoretical basis and from the presence of a unique parameter, Eq. (5) was demonstrated particularly suitable to fit the empirical reduction ratios of medium-large catchments (Bacchi et al., 1992). Franchini and Galeati (2000) showed that θ can be strictly correlated to the lag time of the catchment.

2.2. PD curve estimation

The average value $\bar{r}_D$ of the ratio r_D is computed for each duration D. Usually $\bar{r}_D$ decreases with D, starting from values close to 0.5. An interpolating function $\bar{r}_D$ of the form

$$\bar{r}_D = a + \frac{b}{c + D^d}, \tag{6}$$

can be adopted, but any other, even simpler monotonic equation that fits reasonably well the data can be equally used.

2.3. SDH derivation

The construction of the SDH is performed imposing that the maximum average discharge Q_D for each duration D coincides with the value obtained from the FDF reduction curve; the corresponding volume $Q_D \cdot D$ is located around the peak accordingly to the $\bar{r}_D$ value.

The above conditions lead to the following equations:

$$\int_{-\bar{r}_D D}^{0} Q(\tau;T)d\tau = \bar{r}_D Q_D(T)D; \quad \int_{0}^{(1-\bar{r}_D)D} Q(\tau;T)d\tau = (1-\bar{r}_D)Q_D(T)D \tag{7}$$

The rising and the falling limbs of the SDH $Q(t;T)$ are obtained by differentiating Eq. (7) with respect to duration D as follows:

$$Q(t;T) = \frac{\frac{d}{dD}\left(\bar{r}_D Q_D(T)D\right)\Big|_{D=D(t)}}{\frac{d}{dD}(\bar{r}_D D)\Big|_{D=D(t)}}, \quad t = -\bar{r}_D D, \quad -\bar{r}_{D_f} D_f \leqslant t \leqslant 0 \tag{8}$$

$$Q(t;T) = \frac{\frac{d}{dD}((1-\bar{r}_D)Q_D(T)D)\Big|_{D=D(t)}}{\frac{d}{dD}((1-\bar{r}_D)D)\Big|_{D=D(t)}}, \quad t = (1-\bar{r}_D)D, \quad 0 \leqslant t \leqslant (1-\bar{r}_{D_f})D_f. \tag{9}$$

If both $Q_D(T)$ and $\bar{r}_D$ are fitted by differentiable curves, the rising and falling limbs of the SDH can be computed analytically from Eqs. (8) and (9); otherwise numerical methods must be applied to solve Eq. (7).

The procedure is someway similar to that employed for the Chicago Design Storm (Keifer and Chu, 1957). However, in the present case the coefficient $\bar{r}_D$ is a function of D, whereas in the Chicago Design Hyetograph the analogous parameter r is defined as the ratio between the time prior to peak intensity and the total rainfall duration and is made a constant for all durations.

2.4. Example of SDH derivation

As an example, the mentioned procedure was applied to the 45 years long historical series of floods recorded at the Ponte Bottego gauging station on the Parma river, a tributary of the main Italian river Po.

From 67 recorded flood waves, the 45 annual maximum average discharges for each duration ranging from 0 to 72 h have been extracted. In some years (10 out of 45) the annual maximum for the selected durations belongs to different flood events. As shown in

Fig. 2a, $CV(Q_D)$ does not exhibit any particular trend and can be assumed constant in the whole set of considered durations.

The estimation of ε_D has been performed by means of Eq. (5). The value of $\theta = 7.2$ h was obtained by a least squares estimation. The corresponding curve is plotted in Fig. 2b together with the empirical values. Considering that Eq. (5) is a one-parameter curve the agreement is quite good.

Lognormal, Gumbel (EV1) and General Extreme Value (GEV) probability distributions have been considered in order to describe $Q_0(T)$. Parameter estimation was based on the Method of Moments for the first two distributions, whereas for the third one the method of L-moments has been adopted (Hosking, 1990). Fig. 3 shows, on Gumbel probability plot, the data sample (Gringorten plotting position) together with the fitted probability distributions. The Hosking test (Hosking et al., 1985) on the $k = 0$ hypothesis with a significance level equal to 5%, k being the shape parameter of the GEV distribution, demonstrates that the choice of the GEV distribution is justified, so that $Q_0(T)$ has been expressed in the form

$$Q_0(T) = \xi + \frac{\alpha}{k}\left\{1 - \left[-\ln(1 - \frac{1}{T})\right]^k\right\} \quad (k \neq 0). \tag{10}$$

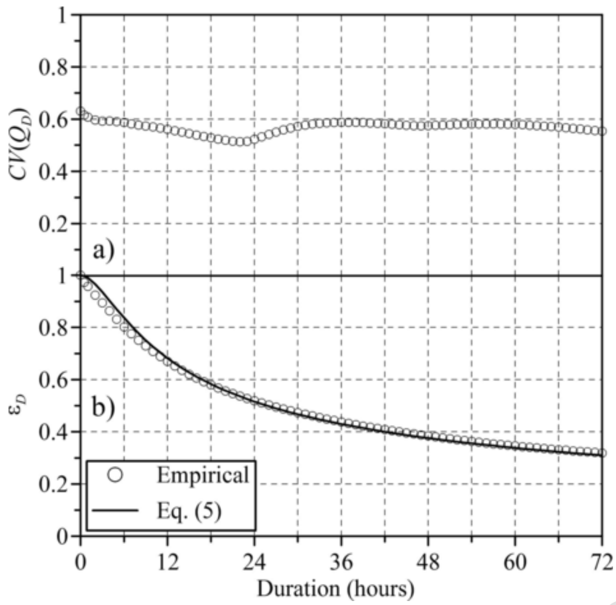

Fig. 2. (a) Coefficient of variation $CV(Q_D)$ versus D; (b) reduction ratio.

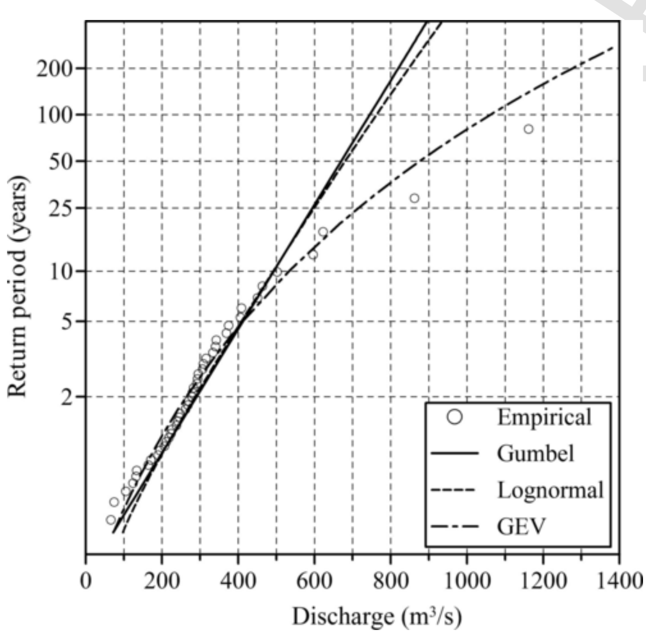

Fig. 3. Q_0 data sample together with the fitted probability distributions.

The FDF curves obtained according to Eq. (2) by multiplication of Eqs. (5) and (10) are plotted in Fig. 4a for return periods ranging from 2 to 200 years.

The equation of the PD curve has been obtained by interpolation of the empirical mean values $\bar{r}_D$. The interpolating function was chosen according to Eq. (6), with least squares estimates $a = 0.260$, $b = 0.635$, $c = 3.639$, $d = 3.495$. In Fig. 4b the interpolating curve is plotted together with the average values obtained from the historical series.

Once the estimates of FDF and PD curves have been obtained, the SDHs follow immediately by analytical derivation of Eqs. (8) and (9). Fig. 5 shows the SDHs obtained for return periods ranging from 2 to 200 years.

3. Case studies, results and discussion

In this section the previously outlined procedure was applied to natural and man-made hydraulic systems. The main goal is to verify whether the results derived from the SDHs compare favourably with

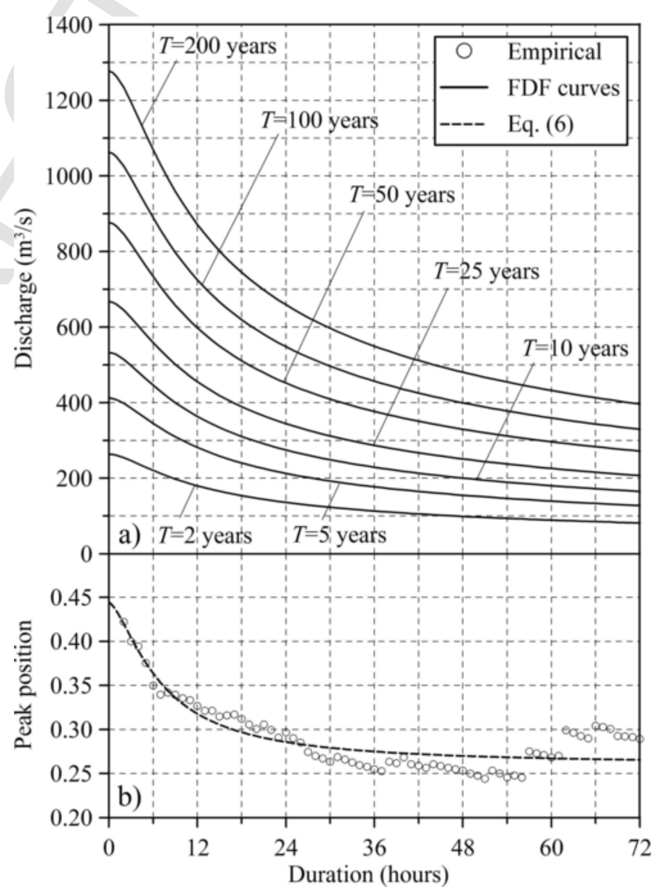

Fig. 4. (a) FDF reduction curves at Ponte Bottego gauging station; (b) PD values and interpolating curve.

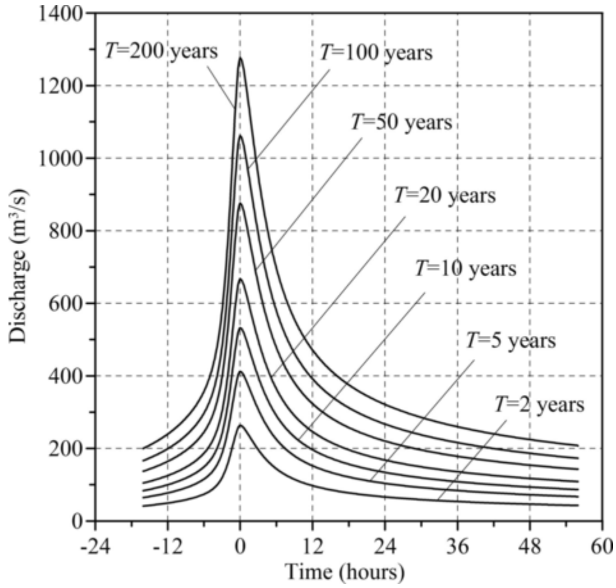

Fig. 5. SDHs for the Parma river at Ponte Bottego gauging station.

those obtained by means of the historical series of recorded hydrographs available for the hydraulic system under consideration.

3.1. Routing in a natural lake

This case study face the problem of evaluating the overall routing effect of a natural reservoir. The case of the Lake Maggiore (Fig. 6a), the second Italian one for extension ($220\,km^2$) and volume ($37\,km^3$) and the first for drained watershed area ($6600\,km^2$), was considered. This natural system possesses interesting peculiarities for the validation of the SDHs. First of all, the incremental volume that can be stored into the lake during floods ($\approx 0.5–1.0 \times 10^9\,m^3$) allows a considerable damping of the inflow peak discharges; moreover, the total inflow discharges to the lake often exhibit a complex multi-peak time pattern, due to the superimposition of the flood hydrographs coming from the four main tributaries (Toce, Maggia, Ticino, Tresa).

Firstly, on the basis of a 62-year long time series of outflow discharges and recorded water levels in the lake, the total inflow discharge time-series was reconstructed. Then, the inflow SDHs were obtained applying the procedure previously outlined. Finally, the SDHs were routed in the lake. The same return period of the inflow SDH was attributed to the corresponding outflow peak discharge obtained after the routing. Fig. 6b shows:

a) The peak outflow discharges derived by routing the inflow SDHs;
b) The frequency distribution of the historical AMS of the same variable;
c) The probability distribution (GEV) fitted to b);
d) The frequency distribution of the reconstructed AMS of peak inflow discharges;
e) The probability distribution (Gumbel), fitted to d).

As a) and b) are in very good agreement, it can be argued that SDHs are able to synthesize and reproduce, at least for the hydraulic system under consideration, the main characteristics of the historical floods. Moreover, it has to be remarked that a) is also very close to the probability distribution c) for the full range of return periods considered. This is not a trivial result, considering the strong routing effects induced by the lake, highlighted by the difference between the inflow (d) and outflow (b) frequency distributions.

3.2. Routing in a flood control reservoir

This case study concerns the routing in a flood control reservoir built in the 1980s on the Secchia river (a tributary of the Po river, Northern Italy) to provide flood protection for the downstream town of Modena.

The Rubiera reservoir is composed by an on-line storage of about $5 \times 10^6\ m^3$ and an off-line storage of about $13 \times 10^6\ m^3$ (Fig. 7a). The main regulating structure is a overtoppable concrete dam crossed by four unregulated culverts (Fig. 7b). An internal spillway connects the two storages. During ordinary floods, the off-line storage is not involved or only partly filled and then emptied by a bottom outlet of small capacity. During more severe floods, instead, off-line storage can be filled to an extent that the internal spillway is backwatered. The almost equalized levels in the two reservoirs can further increase and eventually overtop the dam crest, thus reducing the hydraulic efficiency of the whole structure.

The system can be simulated by the continuity equations of the two reservoirs:

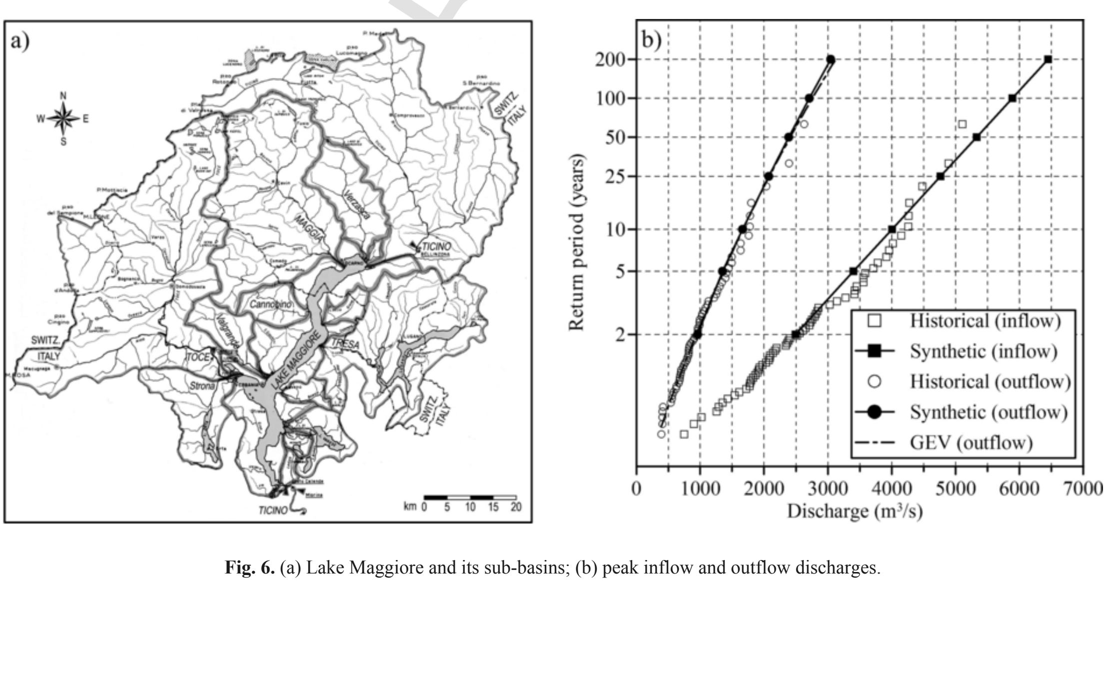

Fig. 6. (a) Lake Maggiore and its sub-basins; (b) peak inflow and outflow discharges.

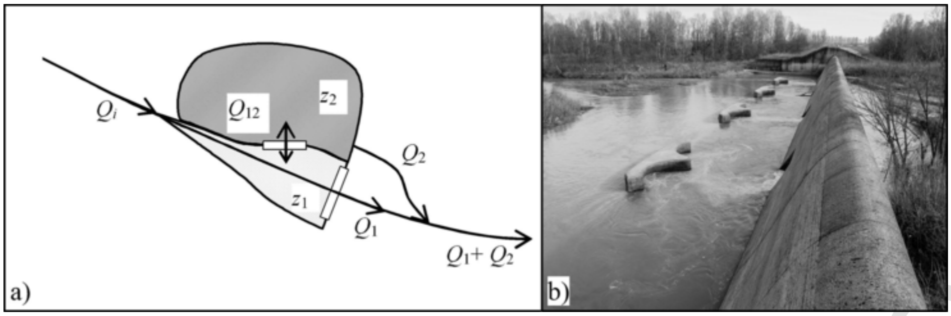

Fig. 7. (a) Sketch of the Rubiera flood control reservoir; (b) main regulating dam.

$$\begin{cases} \dfrac{dz_1(t)}{dt} = \dfrac{Q_i(t)-Q_1(t)-Q_{12}(t)}{S_1(z_1)} \\ \dfrac{dz_2(t)}{dt} = \dfrac{Q_{12}(t)-Q_2(t)}{S_2(z_2)} \end{cases} \tag{11}$$

together with the three outflow relationships of the main regulating structure, of the internal spillway and of the secondary bottom outlet:

$$Q_1(t) = f_1(z_1), \quad Q_{12}(t) = f_3(z_1, z_2), \quad Q_2(t) = f_2(z_2), \tag{12}$$

where S_1, S_2 are the values of the water surface area in the pools at elevations z_1, z_2 respectively, while discharges Q_i, Q_1, Q_2 and Q_{12} refer to inflow, outflow from the two pools and exchange flow through the internal spillway. The above equations have been solved by a fourth-order Runge-Kutta method.

A 38 years long series of continuous discharges, recorded prior to the reservoir construction at a nearby gauging station, was routed in the hydraulic system. From the results, the AMS of peak discharges were extracted. Then, the same discharge inflow time-series was used to derive the SDHs. The SDHs were finally routed in the same way in the hydraulic system. For each SDH, a steady-state initial condition based on the first discharge value was assumed in the on-line storage, whereas the off-line storage was assumed initially empty. As in the previous example, the same return period of the inflow SDH was attributed to the corresponding outflow peak discharge.

Fig. 8 compares the frequency distribution of the AMS of peak outflow discharges (Q_1+Q_2) obtained by routing the historical time series with those obtained by routing the SDHs. The probability distribution fitted on the peak inflow discharges is also shown. Despite the complex behavior of the flood detention reservoir, the results obtained by routing the SDHs well reproduce the attenuation induced on the historical floods, which exhibit a maximum around $T=25$ years. In fact, for low return periods the off-line reservoir does not come into operation at all, whereas for high return periods the entire volume of the two reservoirs is uselessly filled by the incoming hydrograph before the peak and the 150 m long spillway (Fig. 7b) does not allow to increase the stored volume significantly above the dam crest.

3.3. Routing in a river reach

This case study faces the flood routing effect in a river. For this purpose, the ending reach of the Parma river, from the homonymous town up to the confluence within the Po river, was selected (Fig. 9).

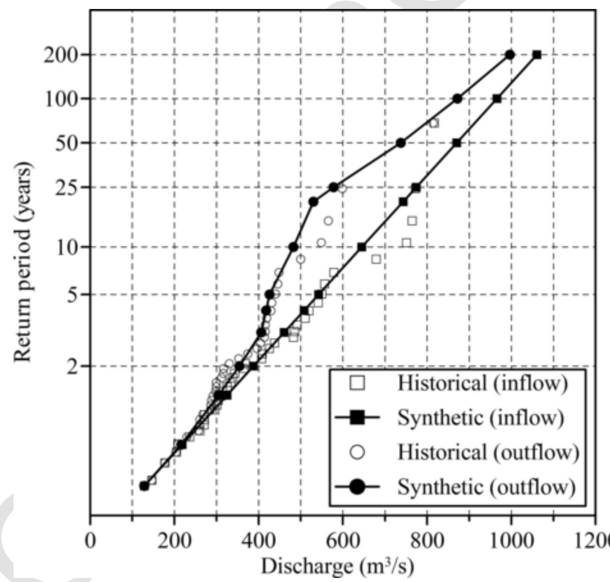

Fig. 8. Inflow and outflow peak discharges at Rubiera flood control reservoir (Secchia river).

The total length is about 38 km, with a very mild average slope of 0.06 %. The aim was to reconstruct the peak discharge distribution at the Colorno crossing, which is a critical node also for the presence of a bridge and a historical realm (Fig. 10).

The river is characterized by a deep main channel which meanders within two high earthen levees. Once flood waters leave the main channel, flood plains of significant width are inundated. In some places they are directly connected with the main channel while elsewhere one or more dikes must be overtopped to inundate the flood plain; this allows flooding only for high return periods and increases the routing efficiency.

The entire historical series of floods and the SDHs derived in Section 2.4 were routed in the river reach, by means of a previously calibrated unsteady quasi-2D mathematical model. An average constant water surface elevation for all (historical and synthetic) events was assumed as downstream boundary condition at the confluence with the Po river. This is justified by the (almost) independent floods of the two rivers, due to the completely different size of their watersheds (Parma river: $0.6\times10^3\,km^2$; Po river at the confluence: $52\times10^3\,km^2$).

More than one historical flood per year were considered in order to obtain the AMS of peak discharges at the Colorno section. In fact, for two comparable events belonging to the same year a swap in the ranking of peak discharges could occur between upstream and downstream sections, as the damping effect depends also on the shape and volume of the inflow hydrograph. Routing only one flood event per year (containing the annual maximum of peak discharge) could then

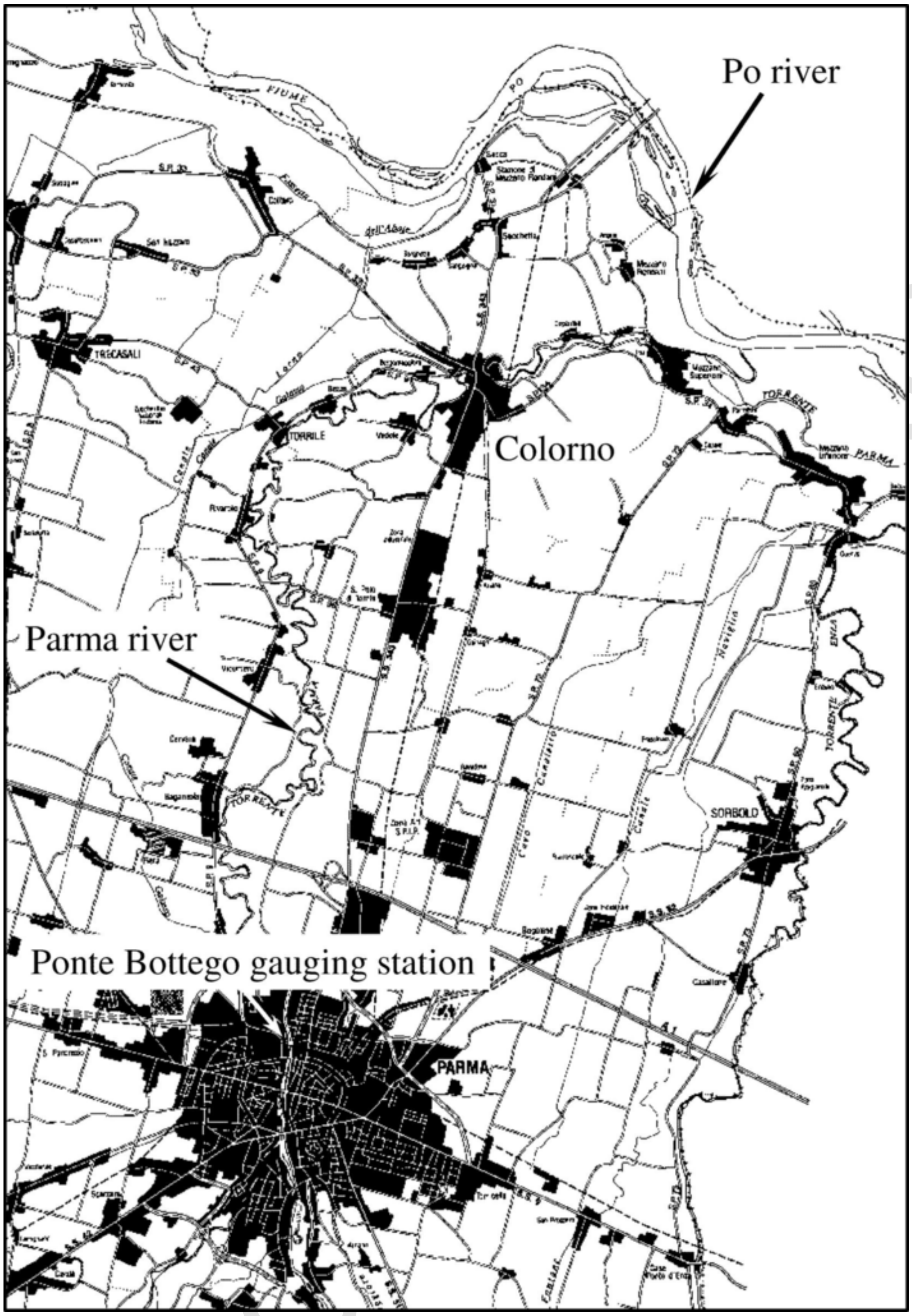

Fig. 9. Location of the considered river reach.

lead to an incorrect evaluation of the sample of annual maximum peak discharges at the selected downstream section.

Fig. 11a shows the inflow SDHs, together with the corresponding routed hydrographs at the downstream section (Colorno). Fig. 11b shows the comparison between the frequency distributions of annual maxima peak discharges obtained at the downstream section by routing the historical floods and the SDHs; for an immediate evaluation of the significant routing effect, the distribution of the annual maxima peak discharges at upstream section of the reach (Ponte Bottego) is also shown in the same plot.

The overall behavior of the routed SDHs peak discharges fits satisfactorily that of the routed historical floods. This confirms that the return periods attached to the routed SDHs have a statistical meaning.

4. Conclusions

A procedure for the estimation of Synthetic Design Hydrographs (SDHs) is proposed. The method is based on the construction of the Flow Duration Frequency reduction (FDF) curves, obtained from the statistical analysis of the maximum average flood discharges of given duration and on the determination of the Peak-Duration (PD) curve.

The statistical significance of the derived SDHs has been evaluated comparing the distribution of peak discharges obtained at a downstream section by routing a long series of historical floods and by routing the synthetic floods.

Fig. 10. The historical realm of Colorno and Parma river during a flood.

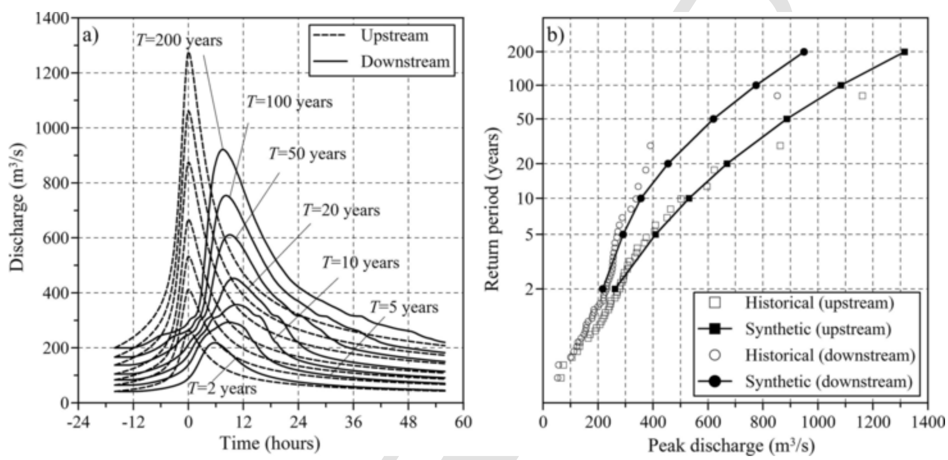

Fig. 11. (a) Routing of the SDHs in the considered reach of Parma river; (b) upstream and downstream peak discharges.

On the whole, the three examined case studies point out the capability of the SDHs to describe the effects of different natural /man made hydraulic systems on the statistical regime of floods, even in presence of strong modifications induced by the system on the probability distribution of peak flows.

The satisfactory results obtained suggest that SDHs construction procedure is reliable and the return periods attached to the routed synthetic floods have statistical meaning. The SDHs can then be applied with confidence to many river flood management problems.

The procedure is immediately and easily applicable for gauged river sections. For ungauged sites it is possible to perform a regional estimation of the main characteristics of the SDHs (Majone et al., 2003).

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