A semiprobabilistic approach for the design of a flood control reservoir

Une approche semi-probabiliste pour le projet d'un résevoir de prévention des inondations

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RÉSUMÉ

Depuis peu, les bassins de détention sont utilisés pour réduire les risques d'inondation dans les vallées et les plaines alluviales. Les bassins sans surveillance sont dimensionnés au moyen d'un hyétogramme de projet obtenu à partir d'un hyétogramme synthétique dérivé des courbes de probabilité de précipitation. Pour dépasser les limites de ces méthodes, cette communication décrit une première démarche pour appliquer l'approche semi-probabiliste aux bassins naturels et larges. On suppose alors que les précipitations sont un phénomène stochastique, c'est-à-dire un enchaînement d'orages indépendants, avec une durée et une hauteur de précipitations totale propre à chacun, et séparés par des périodes sèches. Les fonctions de distribution de probabilité des pics de débit en amont et en aval du réservoir de détention sont dérivées dans l'hypothèse que les hydrographes de l'entrée et de la sortie soient triangulaires. Pour valider cette démarche, un cas d'étude situé dans la région préalpine du Nord de l'Italie a été choisi : on a modélisé un bassin de rétention au niveau du torrent Garza, juste en amont de la ville de Brescia, bassin versant d'environ 53 km². Les résultats obtenus avec l'approche semi-probabiliste ont alors été interprétés et comparés aux résultats obtenus avec la méthode de la crue de projet et les simulations continues.

ABSTRACT

Recently, detention reservoirs have been largely adopted to decrease the flood risk in valleys and floodplains. For ungauged basins, the traditional sizing methods utilize a design hydrograph, obtained from a synthetic hyetograph, whose volume satisfies a representative depth-duration-frequency curve. In this work, to overcome the limits of such an approach, a proposal to apply the semiprobabilistic methodology to large natural catchments is described. In this framework the rainfall is assumed to be a stochastic process composed of a sequence of independent storms, each of which is described by the total depth and the wet weather duration and is separated by interevent dry periods. The probability distributions of the peak rates upstream and downstream the detention storage are then derived assuming a triangular shape for the inflow and the outflow hydrographs. For validating the model, a case study was selected in the alpine foothill area of northern Italy: a detention reservoir aimed at the wet weather discharge control of the Garza stream and located immediately upstream the city of Brescia, whose catchment area is about 53 km^2. The performances assessed by the semiprobabilistic approach were compared to those achieved by the design method and the continuous simulations.

KEYWORDS

Derived distributions, flood risk, storage reservoir, urban waterbodies

1 INTRODUCTION

In the last decades the expansion of the urban areas in Italy has strongly increased the flood vulnerability of the human activities. Such a growth has been especially intensive in the northern side of the Po Valley, the so called “Italian corridor”, where an uninterrupted anthropogenic environment can be actually observed. Currently the floodplains, formerly devoted to farming, are mostly interested by residential, industrial and commercial settlements. The anthropogenic activities affect the drainage network as well, diminishing its conveyance capacity. In fact, narrowing the cross sections or constraining the open canals into culverts are widely applied practices to broaden the urbanized areas.

In addition, the increase in the imperviousness and the decrease in the times of concentrations, that are both consequences of the land use change, locally enlarge the flood volumes and raise the peak flow rates (Ranzi et al., 2002). As a result, the inundation events become more and more severe and frequent, particularly in those areas, as the valley bottoms and the foothill plains, where the runoff discharge rapidly slows down and the natural floodplains are wider. Additional changes in the hydraulic regime are caused by the alteration of the sediment delivery balance and by the river morphology dynamics (Walesh, 1989).

Following the more advanced approach (ASCE and WEF, 1998; CIRIA, 1999; USEPA, 1993), that promotes the mitigation of the urbanization effects on the hydrological cycle, storage practices are encouraged in spite of other flood prevention techniques, as channel widening, pathways shortening by meander cutting, building of diversion canals or embankment elevation. Indeed, such structural remedies can only locally attenuate the risk, which is however increased downstream. On the contrary, the temporary storage of the wet weather discharge provides a restoration of the previous routing capacity of the whole catchment. Moreover, these devices, that can be constructed either as concrete basins or as open ponds by reshaping the natural landscape, own amenity and quality control potentialities (Urbonas and Stahre, 1993).

Flood detention devices can play a major role in the urban stormwater management, but they are quite expensive both in terms of the building costs and of the extension of the land area usually exploited. An efficient sizing method is therefore desirable. The traditional design event approach is based on strongly simplifying hypotheses (see for example Akan and Houghtalen, 2003). Leaving apart the assumptions strictly regarding the catchment hydrological modelling, the most critical concerns lie in assuming that the hydrograph return period is equal to the storm one and that the reservoir is completely empty at any flood event occurrence. In addition, the performance of the device is assessed only for a single severe hydrograph and not for any other possible major events. On the other hand, the continuous simulation of the hydraulic device could provide a tool to perform more realistic analysis, but it requires high computational times and many hydrological data which, in most cases, are not available.

Another promising technique is suggested by the semiprobabilistic methodology. In this approach the simplicity of the formulation is maintained and the analysis is performed not on one single event but on all the events, as in the continuous simulation (Eagleson, 1972; Marsalek, 1978; Díaz-Granados et al., 1984). By applying the derived probability distribution theory, the methodology is able to develop the probability functions of the design hydrological variables starting from those describing the rainfall stochastic process. So, the evaluation of the return period of the derived variables is conceptually correct and the analytical relationships can be written, if suitable distribution functions are adopted. The derivation procedure requires single event statistics, whose parameters are assessed by analyzing long continuous rainfall time series. Further improvements are then provided by the ability of the semiprobabilistic approach to take into account the entire range of the observed rainfall events and their temporal sequence.

The semiprobabilistic methodology has been already used in the context of small urban catchment (Guo and Adams, 1999; Balistrocchi et al., 2009b). Herein this method, based on a rainfall event probabilistic model that suits the Italian climate and a simplified equation for the hydrological loss estimate (Bacchi and others, 2008), is applied for the design of a flood detention facility aimed at the control of the Garza stream wet weather discharge (Brescia province, Lombardy). The Garza stream has its sources in the pre-alpine area and it encloses the medieval Brescia downtown with a semicircular shape channel. The reliability of the model was tested by comparing the semiprobabilistic outputs to those obtained from long term continuous simulations, outlining a quite satisfactory agreement.

2 METHODOLOGY

The semiprobabilistic model development requires rainfall event statistics, which must be derived by separating the rainfall time series into individual independent events. This analysis is usually performed by using an interevent time definition IETD and a rainfall threshold IA. The first one represents the minimum dry weather period between two independent rainfall events (Adams and Papa, 2000, pp. 55-59); the second one is the catchment initial abstraction of the hydrological losses (see for example Chow et al., 1988, pp. 147-155) and allows to remove the not producing runoff events from the sample. Therefore, the precipitation stochastic process can be represented by the probability distribution of three random variables: the event rainfall volume, the wet weather duration and the interevent dry weather period.

For deriving the distributions of the peak inflow rate and the peak outflow rate, that are the main hydrological variables involved in the detention reservoir design, the probability functions of the storm event volume v and the wet weather duration t have to be assessed: a couple of marginal distributions suitable for the Italian climate is given in equation (1). As previously shown by Balistrocchi (2007), the volume non exceedance probability $P_{V(v)}$ may be expressed by a three parameter Weibull function, where the threshold IA denotes the volume lower limit, while β and ζ are the calibration parameters and determine the shape and the scale features respectively. A simpler exponential function is instead satisfactory for the non exceedance probability of the rainfall duration $P_{T(t)}$, whose parameter λ equals the theoretical mean.

$$\begin{cases} P_{V(v)} = 1 - \exp\left[-\left(\dfrac{v - \mathrm{IA}}{\zeta} \right)^{\beta} \right] & \text{for } v \geq \mathrm{IA} \\ P_{T(t)} = 1 - \exp\left[-\left(\dfrac{t}{\lambda} \right) \right] & \text{for } t \geq 0 \end{cases} \tag{1}$$

It is well known that these two random variables are dependent on each other (Eagleson, 1970, p. 186). Such a behaviour has been recently observed and modelled also for some Italian precipitation series recorded in various Italian climatic regions (Balistrocchi et al., 2009a). Nevertheless, to guarantee the analytical integration of the derived probability distributions, according to other Authors (Eagleson, 1972; Díaz-Granados et al., 1984; Guo and Adams, 1998), the usual operative hypothesis of independence is herein adopted. So, the joint probability density function $p_{VT(v,t)}$ takes the form:

$$p_{VT(v,t)} = p_{V(v)}\ p_{T(t)} = \frac{\beta}{\lambda\,\zeta} \left(\frac{v - \mathrm{IA}}{\zeta} \right)^{\beta - 1} \exp\left[-\left(\frac{t}{\lambda} \right) - \left(\frac{v - \mathrm{IA}}{\zeta} \right)^{\beta} \right] \tag{2}$$

2.1 Peak inflow rate distribution

In the framework of a design method development, the hydrological losses can be estimated by using a simplified model. Hence, a runoff coefficient Φ can be applied to the rainfall portion that exceeds the initial abstraction. The flood volume v_r can be consequently evaluated, according to the expression (3).

$$v_r = \Phi \left(v - \mathrm{IA} \right) \tag{3}$$

As previously suggested (Wycoff and Singh, 1976), the inflow hydrograph for a detention facility can be approximated to a triangle, as shown in figure 1: the height equals the inflow peak rate Q_{pi} and the base is given by the sum of the storm duration t and the catchment time of concentration t_c, assumed to be a characteristic constant time of the basin. Enforcing a fixed shape for the hydrograph is obviously a strong limiting hypothesis, that is acceptable only if the objective is the estimation of the peak discharge value during a severe flood. On the contrary, the implementation of the catchment routing equations inside the derivation procedure leads to very complex analytical solutions, difficult to manage in practice. The Q_{pi} flow rate is related to the independent random variables t and v by the equation (4), a deterministic relationship suitable for the application of the derived distribution theory.

$$Q_{pi} = \frac{2\, v_r}{t + t_c} = \frac{2\, \Phi \left(v - \mathrm{IA} \right)}{t + t_c} \tag{4}$$

Hence, the probability distribution of the peak inflow rate P_{Qpi} can be derived as shown in (5) and expressed by the equation (6), if the β exponent in (2) is set equal to one.

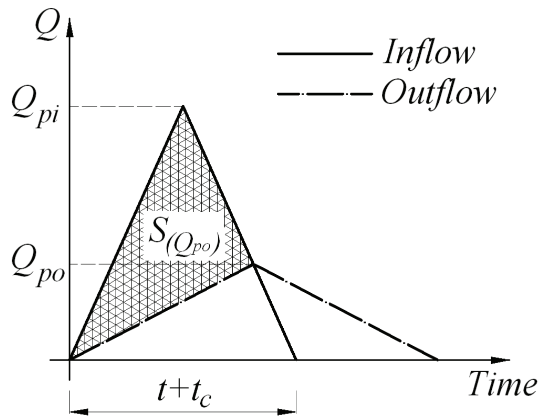

Figure 1: Simplified hydrographs associated with the routing process

This further simplification is justified by the close to one values found for the shape parameter of the volume distribution when the rainfall probabilistic model (1) is calibrated for this kind of structural device (see Table 1).

$$P_{Qpi}(q_{pi}) = \text{Prob}\left\{Q_{pi} \le q_{pi}\right\} = \text{Prob}\left\{\frac{2\,\Phi\,(v-\text{IA})}{t+t_c} \le q_{pi}\right\} = \text{Prob}\left\{v \le q_{pi}\,\frac{t+t_c}{2\,\Phi} + \text{IA}\right\} \tag{5}$$

$$P_{Qpi}(q_{pi}) = 1 - \frac{2\,\Phi\,\zeta}{q_{pi}\,\lambda + 2\,\Phi\,\zeta}\exp\left[-\left(\frac{q_{pi}\,t_c}{2\,\Phi\,\zeta}\right)\right] \tag{6}$$

2.2 Peak outflow rate distribution

The distribution function of the outflow peak flow rate Q_{po} is achievable by imposing a triangular hydrograph for the discharge routed by the detention basin, as in figure 1. The storage volume S required for decreasing the runoff peak from Q_{pi} to Q_{po} is then provided by the relationship (7).

$$S = \frac{1}{2}\left(Q_{pi} - Q_{po}\right)\left(t + t_c\right) \tag{7}$$

When this deterministic relationship is established, the equation (8) is derived, which finally results in the non exceedance probability function P_{Qpo}, when β is equal to one.

$$P_{Qpo}(q_{po}) = \text{Prob}\left\{Q_{po} \le q_{po}\right\} = \text{Prob}\left\{2\frac{\Phi\,(v-\text{IA})-S}{t+t_c} \le q_{po}\right\} = \text{Prob}\left\{v \le \frac{q_{po}\left(t+t_c\right)+2S}{2\,\Phi} + \text{IA}\right\} \tag{8}$$

$$P_{Qpo}(q_{po}) = 1 - \frac{2\,\Phi\,\zeta}{q_{po}\,\lambda + 2\,\Phi\,\zeta}\exp\left[-\left(\frac{q_{po}\,t_c}{2\,\Phi\,\zeta} + \frac{q_{po}\,S}{\Phi\,\zeta}\right)\right] \tag{9}$$

3 CASE STUDY

A case study for applying the semiprobabilistic method was identified: a detention basin was designed to protect the north-eastern outskirt of the Brescia city (Lombardy Region, Italy) against the flood risk associated with the Garza stream. This natural waterbody originates from the hills located immediately upstream the urban area, where the terrain is characterized by very steep slopes (up to 60°). In the foothill plain the stream flows immediately in proximity of many commercial, industrial and residential buildings and the riverbed is often constrained into culverts or very narrow open channels. In the city downtown the canal is mainly buried and becomes part of the ancient combined and separate sewer systems.

A suitable location for the detention basin was found in the valley bottom a few kilometres upstream the urban area boundary, as illustrated in figure 2: the river section hosting the detention device inlet drains a catchment area of about 53 km^2, where the stream is 18 km long. The steep slopes are responsible of high flow rates and a rapid hydrological response: following the formulation of Bacchi et al. (1999), who analyzed several flood events in the Brescia province, the time of concentration t_c was estimated in 3.6 h by means of the relationship (10), where the catchment area A is expressed in km^2, the river length L in km and the difference ΔH between the mean and the minimum elevations in m. The natural portion of the basin surface is moderately impervious and the mean Curve Number (CN) of the SCS method (Soil Conservation Service, 1972) was formerly estimated by Natale (1994) in 75.

$$t_c = \frac{3.3\sqrt{A} + 3.2\,L}{\sqrt{\Delta H}} \qquad (10)$$

The device was sized by using a traditional design method, in which the hyetograph was derived from the intensity-duration-frequency curves (11) corresponding to a return period of 50 years, defined by Bacchi et al. (1995) for the Brescia Pastori raingauge (located a few kilometres south of the watershed); the point precipitation was then transformed into an areal precipitation by using the area reduction factor proposed by Moisello and Papiri (1986). The rainfall time pattern was developed following the Chicago method (Akan and Houghtalen, 2003, pp. 27-29) setting a storm duration double than the time of concentration. The areal precipitation was subsequently routed by using a Nash conceptual scheme (Nash, 1957) with two linear reservoirs and assuming the average antecedent moisture condition (AMC II) of the CN for the calculation of the rainfall excess. The storage constant k of the reservoir system was set to 0.77 h by using the equation (12), where n denotes the number of reservoirs, obtained by equalling the instantaneous unitary hydrograph peak of the Nash method and the one characterizing the kinematic method assuming a triangular Instantaneous Unit Hydrograph (Bacchi et al., 1989).

$$v = 54.71\,d^{0.33} \qquad (11)$$

$$k = \frac{t_c\,\Gamma(n)}{2\,(n-1)^{n-1}e^{-(n-1)}} \qquad (12)$$

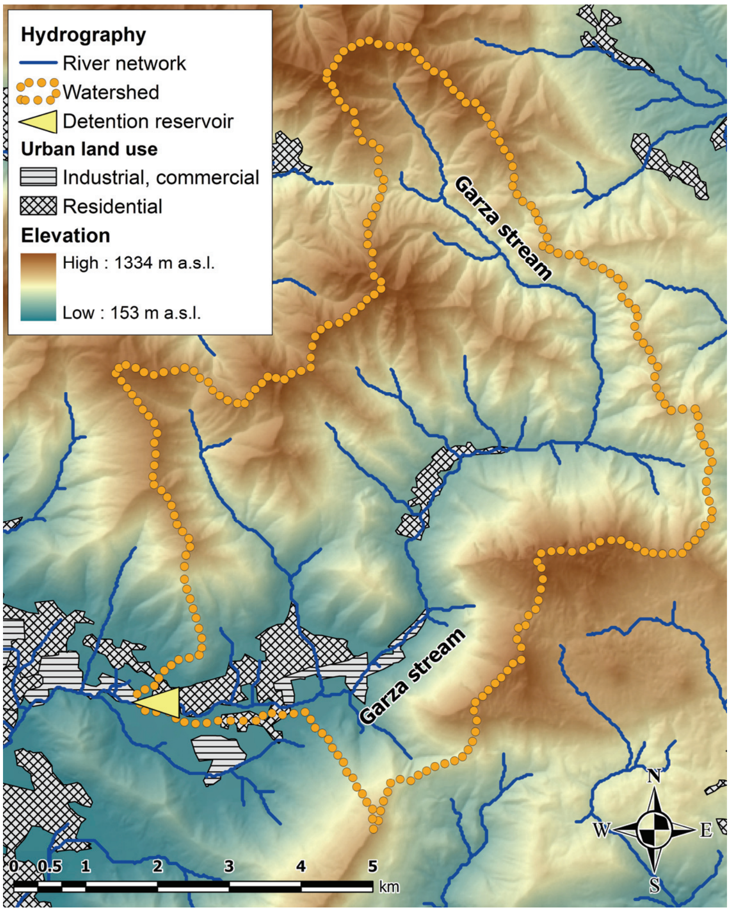

Figure 2: Contributing watershed of the detention basin

The storage capacity was designed to be an open pond, off-line connected to the stream bed as illustrated in figure 3a. The flood discharge is diverted into the device by a side weir combined with a gate and released downstream by a control orifice; a pumping system is provided for completing the tank emptying. Accounting for the local topography and geology and trying to reduce the visual impact of the embankments, the maximum detention volume was fixed in about 200000 m^3: this is expected to decrease the peak flow rate of the design hydrograph from 94 m^3/s to 72 m^3/s, as the simulation of the routing effect due to the presence of the designed basin shows (figure 3b) (Taccolini and Bacchi, 2001).

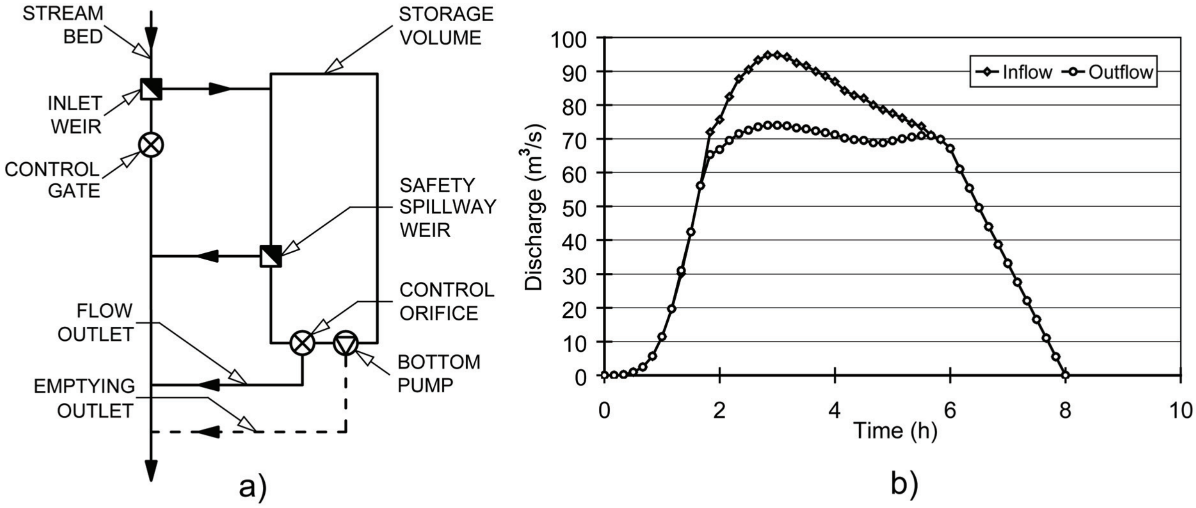

Figure 3: a) hydraulic scheme; b) design inflow and outflow hydrographs (Taccolini and Bacchi, 2001)

3.1 Calibration of the semiprobabilistic model

The 45 yearlong rainfall time series recorded at the Brescia Pastori raingauge was utilized for calibrating the derived distributions (6) and (9). As expected, the fitting parameters of the precipitation probabilistic model (1) are highly sensitive with regard to the thresholds IETD and IA (Bacchi et al., 2008). In practical applications, their values should be assessed in consideration of the involved hydrological processes, for enhancing the reliability of the overall semiprobabilistic modelling (Bacchi et al., 2008). While IA has a clear physical meaning, being an incipient runoff depth that may be assessed by some practical method, the IETD can be estimated like the minimum time between two subsequent storms in order to consider their flood discharges as belonging to two separate flood events. The minimum interevent time can be set equal to the catchment time of concentration, if the interest is addressed to the inflow peak discharge distribution (6), but it should include also the storage volume emptying time for the outflow peak discharge distribution (9), for avoiding the overlapping of the routed discharge hydrographs.

Table 1 reports the fitting values for the Brescia rainfall time series and the mean number of independent storm events per year θ. The IETD upper limit (48 h) accounts for the long period expected for completing the device emptying phase, while the highest IA values are related to the CN estimate. The scale parameters ζ and λ increase with both the thresholds, while the annual event number tends to decrease. In fact, as the two thresholds rise, the number of storms that are lumped together into larger events, or suppressed if too small, is greater. Finally, the close to one values of the β shape parameter justify the simplification adopted in the derivation of the probability distributions.

IETD (h)	**3**			**6**			**12**			**24**			**48**		
IA (mm)	**10**	**15**	**20**	**10**	**15**	**20**	**10**	**15**	**20**	**10**	**15**	**20**	**10**	**15**	**20**
β (/)	0.88	0.94	0.89	0.88	0.91	0.87	0.90	0.91	0.87	0.92	0.91	0.91	0.93	0.93	0.93
ζ (mm)	12.2	14.0	14.1	14.4	15.8	15.8	17.1	18.0	18.3	21.3	21.9	23.1	29.1	29.8	30.8
λ (h)	11.1	12.9	14.0	16.5	19.1	20.8	24.9	28.4	31.1	42.0	47.2	52.1	84.3	93.4	103
θ (n/y)	29	18	12	29	20	14	28	20	15	26	20	15	22	18	15

Table 1: Variability of the distribution parameters for the Brescia series with regard to the IETD and IA thresholds

3.2 Validation through continuous simulations

In order to validate the semiprobabilistic procedure, a continuous simulation of the rainfall-runoff transformation processes occurring in the watershed was performed by implementing the hydrological data previously described into a lumped simulation model. The semi-hourly rainfall time series recorded in Brescia (from January 1949 up to December 1993) was used to simulate the discharge time series at the reservoir inlet. Following the equation (3), at each time step the hydrological losses were computed by applying a runoff coefficient Φ at the rainfall depth exceeding the initial abstraction IA. The incipient runoff depth was fixed in 17 mm, by considering the CN method formulation, while the runoff coefficient was used as a calibration parameter: a value of 0.25 was assumed to achieve the design event inflow peak rate (94 m^3/s for a return period of 50 years). The flow rate series were obtained by applying the same Nash routing of the design procedure.

The simulation outputs were then statistically analyzed by separating the continuous discharge series into independent events assuming an IETD of 4 h. An event based statistics (Individual Event Statistics IES) was carried out for assessing the annual number of independent discharge events θ_Q and the corresponding peak rates Q_{pi}. Consistently with the rainfall probabilistic model calibration, a number of about 16 independent events was found (see Table 1 for IA=15÷20 mm and IETD=3÷6h). So, the experimental return period *T* was estimated by the expression (13), where F_{Qpi} is the plotting position of the individual event discharge peak rate.

$$T = \frac{1}{\theta_Q \left(1 - F_{Qpi}\right)} \tag{13}$$

A different frequency curve was obtained by a more traditional procedure: the calendar annual maximum peaks were extracted from the simulated flow series (Annual Maximum Statistics AMS). The corresponding return period was estimated like in (14), where F'_{Qpi} denotes the annual maximum plotting position.

$$T = \frac{1}{1 - F'_{Qpi}} \tag{14}$$

4 RESULTS AND DISCUSSION

The statistical analysis of the results of the continuous simulation provided the frequency distribution of the peak inflow rate reported in Figure 4, together with the probability distribution derived through the semiprobabilistic approach and the results obtained by the design event approach. The isolated dots (DM) correspond to the peak discharge values derived by the design event procedure (Taccolini and Bacchi, 2001), for the three classes of Antecedent Moisture Conditions AMC (with reference to the CN method); actually only for the return period of 50 years, that is the fixed design return period, all the three AMC classes were considered, while a few return periods (10, 20 and 100 years) were evaluated. The continuous line (SPM) plots the semiprobabilistic analytical curve calibrated for IETD and IA equal to 3.8 h and to 17 mm, respectively (β=1,0, ζ=14.4 mm, λ=15,8 h); the two continuous simulation frequency curves deduced by the IES and the AMS are finally shown by the dotted lines (CS IES and CS AMS, respectively).

As Figure 4 shows, the AMS and IES analyses lead to the same result when the return period increases over 10 years. When the return period is close to 1 year though, the two statistics supply quite different estimates: in such a condition the IES approach is considered to be the most reliable. The overall agreement between the continuous simulation IES output and the semiprobabilistic one is very satisfactory and it supports the estimate of the 50 year return period Q_{pi} in about 100 m^3/s. In this case the design event procedure overestimates the peak discharge at the reservoir inlet when the third class of AMC (wet catchment) is considered, but underestimates for the first AMC class (dry catchment).

Perfect agreement is attained for the second AMC class (average soil moisture condition) as a result of the calibration criterion. The performance of the second part of the semiprobabilistic model (equation 9) was evaluated by comparing the derived probability distribution of the outflow peak discharge to the routed peak discharge (a single dot) obtained through the design event procedure (Figure 5). Since the results of the semiprobabilistic approach is expected to be sensitive to the time threshold (IETD) value, the probability distribution of the outflow peak discharge was computed for three different values of IETD (12, 24 and 48 hours), according to three different hypotheses of the emptying time adopted for the detention basin management. In any case, for the design return period, the semiprobabilistic model provides estimates of the peak outflow rate which are lower than the one supplied by the design event approach. When the IETD is set to 48 h, which may be considered a reasonable time interval for completing the emptying phase of the analyzed storage, the SPM curve gives however a result close to the design method. Finally, the greater the IETD is, the lower the routing performance is: such an occurrence was expected, since the available storage volume is reduced when the detention times are extended. This behavior has already been evidenced by other kinds of storage device (Balistrocchi et al., 2009b).

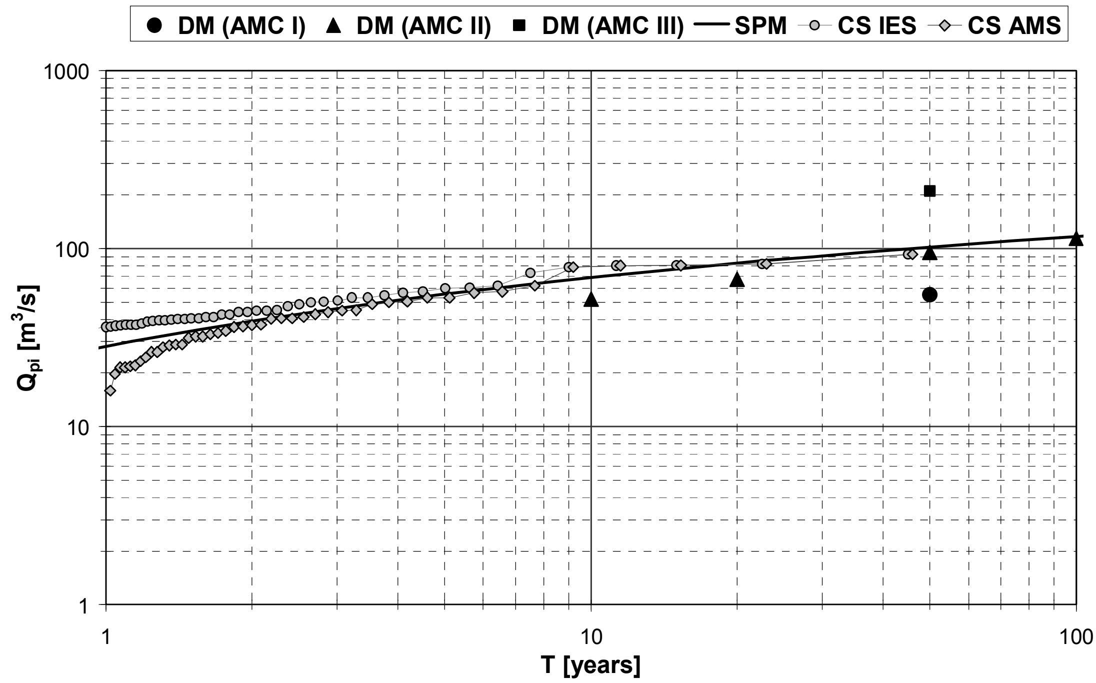

Figure 4: return period of the inflow peak discharge rate estimated through the different procedures

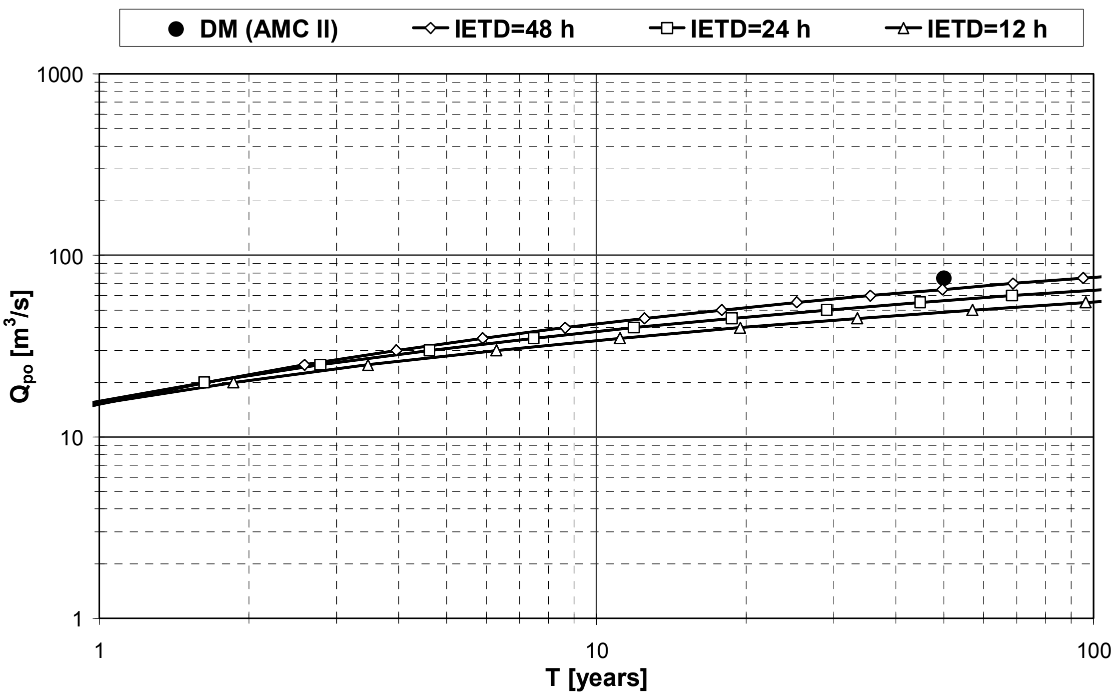

Figure 5: return periods of the outflow peak discharge rate estimated for different emptying times

5 CONCLUSIONS

A semiprobabilistic procedure was developed for assessing the performances of a flood routing reservoir, which was formerly designed by a traditional methodology. This approach, based on a stochastic representation of the rainfall process and on a simple rainfall-runoff transformation scheme, suggests an alternative solution which could provide a conceptually more correct procedure to estimate the whole probability distributions of the most meaningful variables (runoff volume, peak flood, outflow peak runoff). A continuous simulation of the river flow was performed using as input the long rainfall time series observed at the nearest available raingauge (a few kilometres south of the basin).

The comparison among the three different procedures used to estimate the peak flow rate corresponding to the design return period (50 years) shows a very good agreement between the output of the continuous simulation and the semiprobabilistic model. As expected, discrepancies are noticeable for short return periods (a few years), in this range of frequencies different behaviours are shown by event based statistics IES and the annual maximum statistics AMS. The results obtained by the design event approach show a strong variability when different soil moisture conditions are considered for the watershed. When the average antecedent soil moisture condition is selected, very good agreement is obtained between the continuous simulation and the event based procedure, as a result of the adopted calibration criterion.

Even if also in this approach strong simplifications have to be introduced in the representation of the hydrological processes occurring in the watershed, in the case of the inflow peak rate the results of the validation are supporting the efficiency of the developed model. At this step, only the first result of the semiprobabilistic model is satisfactorily tested. Nevertheless, the agreement between the routing performances estimated by the semiprobabilistic and the design method supports the reliability of the first methodology as a whole. This is a favourable position in the hydraulic device design. A continuous modelling of the storage hydraulic behaviour is obviously needed for a definitive validation.

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