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Deriving a practical analytical-probabilistic method to size flood routing reservoirs

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**TECHNICAL REPORT**  
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1 **Abstract**

2 In the engineering practice routing reservoir sizing is commonly performed by using the  
3 design storm method, although its effectiveness has been debated for a long time. Conversely,  
4 continuous simulations and direct statistical analyses of recorded hydrographs are considered  
5 more reliable and comprehensive, but are indeed complex or seldom practicable. A more  
6 handy tool is provided by the analytical-probabilistic approach, herein exploited to construct  
7 probability functions of peak discharges issuing from natural watersheds or routed through  
8 on-line and off-line reservoirs. A rainfall-runoff model based on a few essential hydrological  
9 parameters and a simplified routing scheme were implemented. To validate the proposed  
10 design methodology, on-line and off-line routing reservoirs were firstly sized by means of a  
11 conventional design storm method for a test watershed located in northern Italy. Their routing  
12 efficiencies were then estimated by both analytical-probabilistic models and benchmarking  
13 continuous simulations. Bearing in mind practical design purposes, satisfactory agreements  
14 were evidenced.  
15

1      **CE Subject headings**

2      1. Routing reservoir design

3      2. Derived distributions

4      3. Continuous simulations

5      4. Design storm methods

6      5. On-line storage

7      6. Off-line storage

8

9

# 1 TEXT

## 2 1 Introduction

3 The flood routing practice presently involves a very wide range of application frameworks  
4 (Urbonas and Doerfer, 2005). The most conventional one is the river flood control, in which  
5 extensive reservoirs must operate at a watershed scale. On the other hand, relatively small  
6 storages have progressively gained a greater importance in the planning of urban drainage  
7 systems, in order to compensate the increase in the wet weather runoff peaks that issues from  
8 the impervious surface and the canalization expansions (American Society of Civil Engineers  
9 and Water Environmental Federation, 1998; Urbonas and Stahre, 1993). Following the low  
10 impact development criteria (US Environmental Protection Agency, 2000), they can be  
11 supplied to perform on-site control, even as small multiple storages distributed throughout the  
12 catchment.

13 The range of possible design return periods therefore includes low values of 5:10 years, for  
14 the smallest application scale, and higher values of 100:200 years, normally adopted for the  
15 greatest scale. For instance, the Flood Risk Directive (Directive 2007/60/EC) has recently  
16 been approved in order to provide a homogenous framework for flood management and flood  
17 risk assessment throughout the European Union. The directive defines three hazard scenarios:  
18 low, medium and high probability, the second one referring to a return period equal to or  
19 greater than 100 years.

20 Regardless of the demand of great versatility, the most popular sizing methods are still based  
21 on the traditional design storm approach (Walesh, 1989; Urbonas and Stahre, 1993; Akan and  
22 Houghtalen, 2003), which had previously been developed for the design of drainage canals.  
23 The possibility to effectively extend this straightforward methodology to the routing volume

1 sizing has however undergone several critics so far, because it inevitably relies on  
2 conceptually erroneous hypotheses, which often lead to non-conservative criteria.

3 Leaving apart concerns strictly related to the catchment hydrological modelling, the most  
4 crucial ones consist in: i) the inflow hydrograph return period is equal to the storm one, ii) the  
5 storage volume is completely empty at the storm beginning , iii) device performances are  
6 evaluated with respect to a single event.

7 The first topic represents a general matter independently of the application type, which is  
8 however, particularly remarkable in this case. Actually, the hypothesis formulation itself is  
9 conceptually meaningless, because the return period expresses the frequency of occurrence of  
10 a univariate random process.

11 On the contrary, a hydrograph features a number of characteristics, which can be considered  
12 to be generated by a non-homogeneous multivariate random process. Focusing only on those  
13 which run the behaviour of a routing basin, the peak flow rate, the runoff volume and the  
14 runoff discharge duration must be mentioned (Krstranivic and Singh, 1987; Goel et al., 1998;  
15 Yue, 2000). In general, such variables, even though moderately or strongly associated (De  
16 Michele et al., 2005; Grimaldi and Serinaldi, 2006; Kao and Govindaraju, 2007), show  
17 different return periods within the same event.

18 Moreover, depth-duration-frequency (ddf) curves are constructed by assuming only the storm  
19 depth as a random variable: storms having equal depth yield very different runoff  
20 hydrographs, when the spatial distribution, the temporal pattern or the catchment initial  
21 conditions change. In particular, this last aspect strongly influences the runoff volume and the  
22 peak flow generation processes (Lynch et al., 2006), so that several authors argue that it  
23 represents the most important factor (Sivapalan et al., 1990; De Michele and Salvadori, 2002).

24 Nonetheless, the design storm approach cannot account for their natural variability and their  
25 setting is left to the arbitrary choice of the designer.

1 Indeed, hydrographs of various pattern can be derived from the same ddf curve or, conversely,  
2 lower return period ddf curves might produce more severe floods under various conditions  
3 (Adams and Howard, 1986). Therefore, estimates derived by means of the design storm  
4 method are always biased, even in the purely theoretical situation in which all its assumptions  
5 are satisfied (Bacchi et al., 1993). These deficiencies are more serious for the storage facility  
6 design than for the conveyance canal one, owing to its greater sensitivity to the runoff  
7 volume, which is chiefly influenced by the wet weather duration and the antecedent soil  
8 moisture condition.

9 The second matter is more specific of storage applications but it is still a consequence of the  
10 design storm approach incapability to account for the initial condition variability into the  
11 modelling. As it can be seen in Hong (2008), who briefly but comprehensively classified  
12 several sizing methods that have been developed by using the design storm approach, the  
13 storage volume is always assumed to be completely empty at the flood onset. This hypothesis  
14 is very non conservative for those routing basins whose characteristic emptying times are  
15 comparable to the dry weather periods, such as extended reservoirs for regional flood control  
16 or smaller urban devices having limited downstream conveyance capacity. Some studies  
17 conducted in Mediterranean climates highlighted its relevance in the systematic undersizing  
18 of storage volumes (Mambretti, 1991; Papiri et al., 1998) and stressed the need to evaluate the  
19 performance detriment due to the interaction with antecedent floods.

20 Finally, the third matter is directly connected to the environmental sustainability objectives  
21 introduced in the drainage system planning, by which the change in the natural hydrologic  
22 regime due to the urban development must be completely compensated (Walesh, 1989). The  
23 convenience of using multiple return periods, rather than a single return period, has been  
24 already underlined by Akan (1989): when a routing basin is sized according to a large return  
25 period, it generally performs poorly with respect to minor flood events, which pass through its

1 outlets without peak attenuation. On the contrary, if a little return period is employed the  
2 device is undersized for extreme events, so that the peak rate reduction is negligible.  
3 Nevertheless, efficient routing basins are expected to reduce discharge peak rates of floods  
4 having various size, temporal pattern and antecedent dry weather period, in order to  
5 accomplish the prescribed mitigation targets.

6 Deriving complete frequency distributions of the key hydrological variables is certainly the  
7 most feasible manner to cope with design problems (Marsalek, 1978; Adams et al., 1986). To  
8 this aim, continuous approaches, such as long term simulations and analytical probabilistic  
9 methods, have been strongly advocated, especially in the urban drainage framework (Adams  
10 and Papa, 2000) for which regional or direct statistical analyses cannot be exploited. In  
11 particular the latter appeared to be appealing in order to construct a sizing method primarily  
12 aimed at practical engineering applications. In fact, if the hydrological processes are  
13 represented by suitable functions, the derived distributions can be expressed by analytical  
14 relationships, as well as in the simplest design storm methods.

15 With reference to the design storm approach deficiencies previously examined, the  
16 advantages of the analytical-probabilistic approach are pointed out by the following topics: i)  
17 though the involved physical phenomena are represented by using simple models, the  
18 derivation procedure and the consequent estimation of the return period are conceptually  
19 correct; the simplifying hypotheses concerning the precipitation process are however less  
20 restrictive than those of the design storm approach; ii) the performance estimation is sensitive  
21 to by the real storm temporal sequence, since the interevent period is involved in the rainfall  
22 statistical analysis; thus, the flood overlapping probability can be properly evaluated or,  
23 otherwise, the initial emptying condition can be guaranteed on average through a suitable  
24 setting of the model parameters; iii) the distribution of the rainfall variables are fitted



1 according to the complete range of the observed events and the precipitation model relies on a  
2 variety of storms changing in volume, duration and antecedent dry weather period.  
3 In this work, distribution functions of catchment peak inflow rates and peak outflow rates,  
4 routed either by on-line and off-line storage capacities, are firstly developed. Approximate  
5 models for the precipitation process, the hydrological losses and the routing dynamics were  
6 implemented to achieve relatively simple closed analytical forms (chapter 2).  
7 Since the majority of the developed analytical-probabilistic models were employed to face  
8 urban drainage problems (Guo and Adams 1998, 1999; Quadder and Guo, 2006; Balistrocchi  
9 et al., 2009; André-Doménech et al., 2010), the possibility to extend the proposed methodology  
10 to a wider hydrological scale was herein tested by applying the proposed models to a small-  
11 medium size natural catchment, located in the province of Brescia (Lombardy, North Italy).  
12 For the study case, on-line and off-line routing basins were sized by using a multiple-return  
13 period design method and by changing design targets (chapter 3.1).  
14 Since the main hydrological characteristics and an extended time series of rainfall data are  
15 known but no flow rate record is available, long-term continuous simulations of the  
16 hydrologic catchment response and of the device hydraulic behaviour were adopted as a  
17 benchmark. Discrepancies among performances, assessed by way of the continuous  
18 approaches in chapter 3.2 and 3.3, were finally compared and critically discussed in chapter  
19 3.4.

## 20 **2 The analytical-probabilistic method**

21 In the analytical probabilistic method herein proposed the design procedure is based on the  
22 evaluation of the peak rate cumulative distribution function (cdf) featuring the outflow  
23 discharge of a routing basin. In accordance with the derived distribution theory, probability  
24 functions are constructed by assuming that the runoff variables depend on constituent rainfall  
25 variables through analytical relationships (Adams and Papa, 2000).

1 Their joint distribution function must be assessed with respect to samples of individual  
2 rainfalls, or storms, which are detected by separating the continuous time series of observed  
3 data into independent events. Although there is no definitive strategy (Bonta and Rao, 1988),  
4 such a preliminary discretization plays a crucial role in ensuring the overall method reliability,  
5 because if the independence prerequisite is not met, the derived models could lead to  
6 unacceptable estimation errors (Bacchi et al., 2008).

7 This is a consequence of the strong sensitivity that the statistical properties of the input  
8 random variables show towards this procedure. As underlined by Dunkerly (2008), reviewer  
9 of a certain number of recent studies which exploited event based statistics, reporting the  
10 applied discretization criterion is mandatory to provide comprehensive information on the  
11 performed analysis.

12 Hence, before illustrating the derivation of the design variable cdfs, the identification the  
13 independent storms is briefly discussed along with its implications on the event based  
14 statistics. After that, a precipitation probabilistic model is defined, evidencing its connections  
15 to the discretization procedure itself and justifying the adopted simplifications.

16 The peak discharge cdf of the runoff produced by uncontrolled watersheds is derived at first,  
17 in order to estimate the frequency of floods forcing the device. Finally, starting from this  
18 achievement, distinct peak discharge cdfs are obtained for outflows issuing from storage  
19 capacities supplied either on-line or off-line.

## 20 ***2.1 Identification of independent storms***

21 Various criteria have been established so far to identify independent storms. They can be  
22 classified in two basic categories: those regarding exclusively the rainfall process and those  
23 matching the rainfall characteristics to the watershed hydrological conditions (Bonta and Rao,  
24 1988). Owing to its better potential to attend practical design objectives, a discretization  
25 procedure belonging to the second group was utilized in this study by combining two

1 discretization parameters: an interevent time definition (IETD) and a rainfall depth threshold.  
2 The values of both of them were chosen according to key hydrological properties of the  
3 catchment-device system.

4 The former quantity was introduced in hydrology by the fundamental paper of Restrepo-  
5 Posada and Eagleson (1982) as an empirical, but convenient, statistical solution to avoid  
6 unfeasible physical analyses. It was defined as the minimum rainless time between storms that  
7 is necessary to achieve a prescribed degree of independence.

8 This time interval is directly applied to the continuous series: if two successive hyetographs  
9 are separated by a dry weather time shorter than the IETD, they are considered to be  
10 dependent. Therefore, they are aggregated in a greater event, whose duration extends from the  
11 beginning of the previous one to the end of the following one, and whose volume amounts to  
12 the total depth fallen within such a period. On the contrary, they are identified as independent  
13 storms and maintained separated. Any isolated event is defined by three quantities: the rainfall  
14 volume, the wet weather duration and the antecedent dry weather period.

15 The event sample is then filtered by using the rainfall depth threshold, which deletes the  
16 storms having a lower total volume. When this occurs, the corresponding wet weather  
17 duration is interpreted as a dry weather period and joined to the adjacent ones. The meaning  
18 of this additional parameter consists in eliminating the negligible rainfalls that do not affect  
19 the estimation of the derived variable cdfs, so that only actual storm events remain in the final  
20 sample.

21 An operative manner to assess the IETD arises from the possibility to recognize a storm as  
22 independent if the effects that it produces on the considered hydrological phenomenon do not  
23 interfere with those due to any other storm. When dealing with wet weather discharges, this  
24 general criterion may be translated into a search for the minimum dry weather period that  
25 avoids overlapping flood hydrographs. Consequently, the IETD value can be computed as the

1 maximum travel time that is required for the runoff to reach the system outlet. This obviously  
2 corresponds to the definition of the catchment time of concentration, which must account for  
3 the emptying time of relevant storage capacities, if they exist.

4 The main attraction of this choice consists in the possibility to effectively cope with the  
5 troublesome problem of the system initial condition, since the hypothesis that the drainage  
6 network and the routing basin are empty at the beginning of the flood can be overall satisfied.  
7 This significantly simplifies the derivation of the design variable distributions, because the  
8 input precipitation probabilistic model may only express the volume-duration joint  
9 distribution omitting that of the interevent dry period.

10 Moreover, in this framework actual events are those that produce runoff, while very small  
11 rainfalls do not affect neither the flood frequency, nor the estimation of the routing basin  
12 efficiency. When these insignificant events are purged out of the rainfall time series, the  
13 probability of the runoff null event is zero. Conversely, if maintained, they would complicate  
14 the next derivation steps, which should include an impulse probability mass in the origin of  
15 the runoff volume cdf.

16 Bearing in mind the SCS-CN model of the hydrological losses (Soil Conservation Service,  
17 1975), the rainfall depth threshold was identified with the average initial abstraction (IA) and  
18 estimated according to average soil and land cover characteristics of the watershed.

19 When dealing with large catchments or relevant storage volumes, extended IETDs are needed,  
20 while natural land covers imply high IAs. As noticed in very diverse climates (Adams et al.,  
21 1986; Dunkerly, 2008; Andrés-Domenéc, 2010), the means of the three rainfall event  
22 variables show a sensibly linear increasing trend when IETD raises over 1:3 h. Analogous  
23 proportional behaviours are evidenced according to IA, if it is larger than a few millimetres  
24 (Bacchi et al., 2008). Standard deviations also exhibit linear increasing behaviours with  
25 respect to both the discretization parameters.

1 In general none of these sensitivities can be neglected because of the size of their variation  
 2 ranges, which deeply affects the rainfall distribution calibration. Greater the discretization  
 3 parameters, larger the rainfall volumes and longer the dry weather and the wet weather  
 4 periods are. Consistently, the average annual event number diminishes. This explains the  
 5 previously mentioned possibility to adapt the rainfall probabilistic model to the specific  
 6 hydraulic application.

## 7 **2.2 Precipitation probabilistic model**

8 As demonstrated by Balistrocchi and Bacchi (2011) under various climatic conditions and for  
 9 a number of discretization parameter settings, a joint probability function for the rainfall  
 10 volume and duration may be established by means of the copula theory (Nelsen, 2006).

11 The dependence structure is well modelled by a bivariate Gumbel-Hougaard function, due to  
 12 the moderate-strong concordant association that has always been detected for Italian climate  
 13 regimes. Weibull distribution functions can be implemented in this copula to complete the  
 14 joint distribution, thanks to their satisfactory fit to the marginal frequencies in every case.

15 As a matter of fact, the analytical form of this probabilistic model is such that a numerical  
 16 integration of the dependent variable distributions is required. Therefore, despite such  
 17 findings, two important simplifications must be introduced to achieve an analytical  
 18 formulation of the derived distributions: i) the rainfall event variables are independent of each  
 19 other, ii) the marginal distributions are approximated by exponential functions.

20 The precipitation probabilistic model is herein expressed by the joint probability density  
 21 function  $p_{VT}(v,t)$  written below, as the product of the exponential marginal densities  $p_V(v)$  and  
 22  $p_T(t)$  of the storm volume  $v$  and duration  $t$ , respectively.

$$23 \quad p_{VT}(v,t) = \begin{cases} p_V(v) p_T(t) = \frac{1}{\lambda \zeta} \exp \left[ - \left( \frac{t}{\lambda} + \frac{v - IA}{\zeta} \right) \right] & \text{for } v \geq IA; t \geq 0 \\ 0 & \text{elsewhere} \end{cases} \quad (1)$$

1 In this equation  $\zeta$  and  $\lambda$  indicate the scale parameters that rule the distributions of  $v$  and  $t$ ,  
2 while the initial abstraction IA is introduced as the lower boundary of the runoff volume  
3 distribution, in accordance with the discretization criterion.

4 Neglecting the association between volume and duration should generally lead to improper  
5 increase in the estimate of the mean and of the variance of the dependent variables. However,  
6 owing to the upper tail dependence property of the Gumbel-Hougaard copula, the error is  
7 expected to be larger for the extreme storms. In fact, volume and duration prove to be more  
8 strongly associated in these events than in the ordinary ones.

9 The exponential distribution is a very popular model in hydrology, since it was applied in a  
10 huge number of works (an extensive list can be found in Salvadori and De Michele, 2006).  
11 Actually, it represents a particular case of the Weibull distribution, in which the variation  
12 coefficient is unitary.

13 Nevertheless, especially in Mediterranean precipitation regimes, sample distributions show  
14 variation coefficients significantly different from unity, so that the exponential distribution  
15 hypothesis is often rejected. The reason mainly lies in the predominance of short duration and  
16 high intensity, but low depth, storms. Therefore, both the rainfall volume and the wet weather  
17 period distributions have very high relative frequencies near the origin. The error involves the  
18 overall distribution, but it is more relevant for short events, whose frequency of occurrence is  
19 underestimated by the exponential model.

### 20 ***2.3 Peak inflow discharge distribution***

21 A considerable improvement in the analytical-probabilistic representation of flood  
22 frequencies has recently been achieved by Guo and Adams (1998), who implemented in the  
23 derivation procedure the schematization of the discharge hydrographs suggested by Wycoff  
24 and Singh (1976), in spite of rainfall-runoff routing models previously employed in many

1 flood frequency derivations (Eagleson, 1972; Cordova and Rodriguez-Iturbe, 1983; Diaz-  
2 Granados et al., 1984).

3 Following this solution, the peak discharge rates are directly expressed as a function of flood  
4 characteristics, such as the runoff volume and the hydrograph duration, that are related to the  
5 precipitation constituent variables by algebraic expressions. Closed-form analytical  
6 distributions were obtained both for the peak inflow discharge to a detention reservoir and for  
7 the outflow one (Guo and Adams, 1999).

8 In this derivation, the runoff volumes are estimated by applying a runoff coefficient  $\Phi$  to the  
9 rainfall depth exceeding the initial abstraction IA. As demonstrated in previous works for  
10 urban catchments (Balistocchi et al., 2009), although the model is extremely simple, it is able  
11 to mimic the natural reduction of the hydrological losses during a rainfall event. The runoff  
12 volume  $v_r$  is calculated by the expression (2).

$$13 \quad v_r = \Phi (v - IA) \quad (2)$$

14 The inflow hydrograph shape is then approximated to the triangle represented in figure 1: the  
15 height corresponds to the specific peak inflow rate  $q_{pi}$  and the base is given by the sum of the  
16 storm duration  $t$  and the catchment time of concentration  $t_c$ , assumed to be a characteristic  
17 watershed constant. This is indeed an additional simplifying hypothesis introduced in the  
18 derivation procedure. Nevertheless, investigating the effects of utilizing a variable time of  
19 concentration in analytical-probabilistic urban stormwater modelling rather than a constant  
20 one, Quadder and Guo (2006) concluded that the accuracy enhancement is trivial. In figure 1,  
21 the area bounded by the inflow hydrograph triangle represents the runoff volume (2), so that  
22 the  $q_{pi}$  peak rate is expressed with respect to the constituent rainfall variables as follows.

$$23 \quad q_{pi} = \frac{2 v_r}{t + t_c} = \frac{2 \Phi (v - IA)}{t + t_c} \quad (3)$$

24 The peak inflow rate cdf  $P_{Q_{pi}}$  may therefore be derived by using the non-exceedance  
25 probability definition (4).

$$P_{Q_{pi}(q_{pi})} = \text{Prob} \left\{ \frac{2 \Phi (v - \text{IA})}{t + t_c} \leq q_{pi} \right\} = \text{Prob} \left\{ v \leq q_{pi} \frac{t + t_c}{2 \Phi} + \text{IA} \right\} \quad (4)$$

By intersecting the region defined by the inequality in (4) with the domain of definition of the probability density function (1), the domain of integration for applying the derived distribution theory is set. As illustrated in figure 2, this is a semi-infinite subset in random variable space contoured by linear boundaries, whose trapezoidal shape is almost identical for all possible choices of the hydrological model parameters. The non exceedance probability (4) is accordingly computed by the integral (5):

$$P_{Q_{pi}(q_{pi})} = \int_0^\infty \int_{\text{IA}}^{q_{pi} \frac{t+t_c}{2\Phi} + \text{IA}} \frac{1}{\lambda \zeta} \exp \left[ - \left( \frac{\tau}{\lambda} + \frac{\nu - \text{IA}}{\zeta} \right) \right] d\nu d\tau. \quad (5)$$

The analytical cdf of the peak inflow discharge rates is finally established by the relationship (6):

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

Since in this formulation all storms produce runoff, the exceedance probability of the null event  $P_{Q_{pi}}(0)$  is zero. Consequently, the average annual storm number  $\theta_s$  exactly equals the flood one  $\theta_f$ .

#### 2.4 Peak outflow discharge distribution for on-line basins

Dealing with an on-line storage basins, the outflow discharge hydrograph may be still represented as a triangular hydrograph (figure 1), having height  $q_{po}$ . The stored volume  $S_{(Q_{po})}$ , which produces the routing of the runoff peak from  $q_{pi}$  to  $q_{po}$ , must satisfy the relationship (7):

$$S_{(q_{po})} = \frac{1}{2} (q_{pi} - q_{po}) (t + t_c). \quad (7)$$

The derived distribution theory makes the definition of the non exceedance probability function  $P_{Q_{po}}$  as shown in (8) possible. Expressions (2) and (7) are implemented in (8):



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

2 The stored volume  $S(q_{po})$  is itself a random variable depending on the event characteristics.

3 Assuming its constancy is a very restrictive assumption, which would lead to unacceptable

4 overestimations of the routing efficiencies especially for major devices, for which the storage

5 capacity is completely exploited only during extreme events. In general, more severe the

6 flood is, greater the stored volume and the downstream peak discharge. Such a relationship is

7 herein represented by using the conceptual model of the linear reservoir (9), in which  $S(q_{po})$  is

8 proportional to the outlet discharge  $q_{po}$  through a storage constant  $k_S$  (Chow et al., 1988).

$$S(q_{po}) = k_S q_{po} \quad (9)$$

10 This constant can be assessed with regard to the routing basin hydraulic characteristics and

11 has the important advantage to rely on both the storage volume and the outlet conveyance

12 capacity. Consistently with the inequality (8), the expression (9) and the domain of definition

13 in (1), the domain of integration of the peak routed discharge cdf is delimited by the

14 trapezoidal region illustrated in figure 3, while the integral that analytically defines the non

15 exceedance probability (8) is written in the terms of equation (10).

$$P_{Q_{pi}(q_{pi})} = \int_0^\infty \int_{\text{IA}}^{q_{po} \left( \frac{t+t_c+2k_S}{2\Phi} \right) + \text{IA}} \frac{1}{\lambda \zeta} \exp \left[ - \left( \frac{\tau}{\lambda} + \frac{\nu - \text{IA}}{\zeta} \right) \right] d\nu d\tau \quad (10)$$

17 The outflow peak discharge cdf  $P_{Q_{po}}$  is hence expressed by the probability distribution (11),

18 which maintains the same structure of the inflow one (6), except for the insertion of the

19 storage constant  $k_S$  of the routing basin.

$$P_{Q_{po}(q_{po})} = 1 - \frac{2\Phi\zeta}{q_{po}\lambda + 2\Phi\zeta} \exp \left[ - \left( \frac{t_c + 2k_S}{2\Phi\zeta} q_{po} \right) \right] \quad (11)$$

## 1 **2.5 Peak outflow discharge distribution for off-line basins**

2 When the storage capacity is supplied off-line, the routing scheme in figure 1 does not fit the  
 3 real facility behaviour, because the inflow discharge is not routed until the spill toward the  
 4 storage volume begins. The inflow threshold discharge beginning the spill towards the storage  
 5 volume  $q_s$  must be accounted for in the probabilistic model.

6 Floods with peak rates less than  $q_s$  are not attenuated at all and the same cdf of the inflow  
 7 discharge (6) must be employed for the routed one. Conversely, only the portion of the peak  
 8 inflow rates exceeding the spillway threshold discharge ( $q_{pi} - q_s$ ) attenuates down to ( $q_{po} - q_s$ ),  
 9 consistently with the former routing scheme, and sums to  $q_s$  downstream the device. Since the  
 10 cdf of ( $q_{po} - q_s$ ) does not exist for total routed discharges  $q_{po}$  less than  $q_s$ , its behaviour  
 11 formally matches that of a censored random variable cdf. In this case, the probability that the  
 12 random variable is included in the censored range corresponds to the non exceedance  
 13 probability of the threshold discharge  $q_s$ , which is easily estimated by the peak inflow  
 14 discharge cdf (6).

$$15 \quad P_{Q_s} = P_{Q_{pi}(q_s)} = 1 - \frac{2 \Phi \zeta}{q_s \lambda + 2 \Phi \zeta} \exp \left[ - \left( \frac{t_c}{2 \Phi \zeta} q_s \right) \right] \quad (12)$$

16 The peak outflow discharge cdf  $P_{Q_{po}}$  is thus summarised in the distribution (13), in which the  
 17 formulation of cdf (11) is exploited to represent the non exceedance probability of the not  
 18 routed portion of the total discharge.

$$19 \quad P_{Q_{po}(q_{po})} = \begin{cases} 1 - \frac{2 \Phi \zeta}{q_{po} \lambda + 2 \Phi \zeta} \exp \left[ - \left( \frac{q_{po} t_c}{2 \Phi \zeta} \right) \right] & \text{if } q_{po} \leq q_s \\ P_{Q_s} + (1 - P_{Q_s}) \left\{ 1 - \frac{2 \Phi \zeta}{(q_{po} - q_s) \lambda + 2 \Phi \zeta} \exp \left[ - \left( \frac{t_c + 2k_s}{2 \Phi \zeta} (q_{po} - q_s) \right) \right] \right\} & \text{if } q_{po} > q_s \end{cases} \quad (13)$$

### 1 **3 Model application and verification**

2 In order to test the possibility to apply the analytical-probabilistic methodology to spatial  
3 scales greater than those of urban catchments, the upstream Garza watershed was selected as a  
4 case study. This is a natural reach whose catchment area belongs to the southern edge of the  
5 central Alpine foothills (Lombardy, Italy) (lower-right box in figure 4).

6 After an upstream stretch with very steep slopes, the stream enters the higher Padan Plane  
7 where it immediately crosses the strongly urbanized northern outskirts of the town Brescia.  
8 Here its river bed is often constrained into culverts or narrow canals with insufficient  
9 conveyance capacity, while original floodplains are now covered by industrial and residential  
10 settlements. A routing basin could therefore be a viable solution to mitigate the unacceptably  
11 high flooding risk, since a suitable location can be identified in the valley bottom a few  
12 kilometres upstream the urban area.

13 The selected outlet identifies the possible basin location (figure 4) and delimitates the most  
14 upstream mountain catchment area which is mainly natural. Morphometric parameters,  
15 assessed through a terrain analysis of a 20 m resolution DEM, are listed in table 1.

16 As proposed by Giandotti (1934), who analysed a number of natural floods in Italy, the time  
17 of concentration  $t_c$  (h) can be estimated by equation (14) as a function of the catchment area  $A$   
18 ( $\text{km}^2$ ), the main reach length  $L$  (km) and the elevation drop, expressed as the difference  
19 between the average elevation  $H_{ave}$  (m a.s.l.) and the minimum elevation  $H_{min}$  (m a.s.l.).

$$20 \quad t_c = \frac{4\sqrt{A} + 1.5L}{0.8\sqrt{H_{ave} - H_{min}}} \quad (14)$$

21 The infiltration capacity is moderately low and soils can be averagely classified in hydrologic  
22 group C, according to the CN-SCS method (Soil Conservation Service, 1972). The land cover  
23 consists in woods in the upstream catchment which are progressively substituted by  
24 agricultural parcels included into residential and industrial areas in the valley bottom.

1 The CN value is reported in table 1, along with the soil moisture deficit at the time runoff  
2 begins  $S$  and the initial abstraction  $IA$ , evaluated by using standard CN-SCS criteria for the  
3 average moisture condition AMC II.

4 The watershed precipitation regime is sub-alpine with two maximums, spring and autumn,  
5 and two minimums, winter and summer, while the annual average rainfall depth is about 900  
6 mm. During summer, short duration and high intensity storms separated by long dry weather  
7 periods occur. On the contrary, longer but less intense precipitations feature the wet seasons.

8 The meteorological input to the different procedures was represented by a 45 year long time  
9 series of rainfall observations recorded every 30 minutes at the ITAS Pastori rain gauge (1949-  
10 1993), located in Brescia, a few kilometres south of the watershed.

11 Two routing basins, one on-line and one off-line, were hypothesized to obtain diverse peak  
12 attenuation efficiencies. Storage volumes and outlet control structures were preliminary sized  
13 by means of a conventional design storm method. Long term performances were then  
14 estimated by continuous approaches.

### 15 **3.1 Basin design**

16 The design storm method was firstly based on a lumped hydrological model to derive inflow  
17 hydrographs for varying return periods. In the Italian hydrologic practice, ddf curves are  
18 usually expressed as simple monomial relationships (15), whose parameters  $a$  (mm/h<sup>n</sup>) and  $n$   
19 generally depend on the return period  $T$  (years).

$$20 \quad v_{(t,T)} = a_{(T)} d^{n(r)} \quad (15)$$

21 Nevertheless, when the rainfall volume variability shows scaling properties (Burlando and  
22 Rosso, 1996) and is suitably represented by Gumbel distributions for any durations, as in this  
23 case, equation (15) can be written in the terms of equation (16) (Bacchi et al., 1995).

$$v_{(t,T)} = v_1 \left\{ 1 - \frac{\overline{CV}}{1.283} \left[ 0.5772 + \ln \left( \ln \left( \frac{T}{T-1} \right) \right) \right] \right\} t^n \quad (16)$$

In this formulation the exponent  $n$  is constant and the coefficient  $a$  is directly expressed as a function of the return period  $T$  by two sample statistics:  $\overline{CV}$  is the variation coefficient mean of the maximum annual rainfall volume samples recorded for different durations  $t$  (h) (usually 1 h, 3 h, 6 h, 12 h and 24 h), while  $v_1$  (mm) is the mean maximum annual hourly rainfall volume. Ddf parameter values estimated for the Brescia raingauge are: 0.33, 0.36 and 28.3 mm for  $n$ ,  $\overline{CV}$  and  $v_1$ , respectively.

The point precipitation was converted into areal precipitation by the area reduction factor (17) proposed by Moisello and Papiri (1986) for rain fields North Italy, when the catchment area  $A$  ( $\text{km}^2$ ) ranges between 5 and 800  $\text{km}^2$  and the duration  $t$  between 0.15 and 12 h.

$$r_{(A,T)} = 1 - \exp \left[ -2.472 A^{-0.242} t^{0.6 - \exp(-0.643A^{0.235})} \right] \quad (17)$$

The design rainfall pattern was thus developed according to the Chicago method (Akan and Houghtalen, 2003), by setting the storm duration equal to twice the catchment time of concentration. This choice was intended to match two opposite design targets: on one hand, to have a significant flood volume, on the other hand, to maintain high peak flow rates. Since the area reduction factor (17) depends on the rainfall duration and the intensity varies inside the wet weather period, rainfall volumes for partial durations were deduced from the aerial ddf curve, given by the product of (16) and (17), so that short duration intensities were reduced more than the lower ones.

Hydrological losses were computed by means of the SCS-CN procedure under average antecedent moisture conditions (AMC II), since extreme conditions are considered to yield unreasonable large, or low, runoff volumes when applied within a design storm procedure.

The rainfall excess was finally routed by a Nash conceptual scheme (Nash, 1957) consisting in a combination of two linear reservoirs to obtain the outlet discharge. The storage constant

1  $k_N$  was assessed through equation (18), that originates from a similitude between the  
 2 instantaneous unitary hydrograph peak of the Nash method and that of the kinematic method,  
 3 if a triangular shape is assumed (Bacchi et al., 1989). In such an equation,  $n_i$  is the number of  
 4 reservoirs,  $\Gamma(\cdot)$  the complete gamma function and  $t_c$  the catchment time of concentration given  
 5 by equation (14):

$$6 \quad k_N = \frac{(n_i - 1)^{n_i - 1} e^{-(n_i - 1)}}{2\Gamma(n_i)} t_c. \quad (18)$$

7 According to Akan (1989), various design hydrographs related to return periods rising from 5  
 8 years to 100 years were derived and utilized as inflow to simulate the basin behaviour. To do  
 9 so, hydraulic models of the routing reservoirs were developed for the connection scheme  
 10 illustrated in figure 5 a), for the on-line case, and in figure 5 b), for the off-line one.

11 In the former, an embankment crossing the stream bed and directly intercepting the natural  
 12 flow is employed to delimitate an open pond. Outlet orifices and a spillway were set to  
 13 provide increasing discharge capacity with the water stage. In the latter, the stream flow is  
 14 diverted into an analogous open pond by a side weir combined with a gate. The captured  
 15 runoff is discharged towards the downstream reach by orifices, a spillway and a pump system,  
 16 to facilitate the emptying process for very low stages. Storage volumes and outlet discharge  
 17 curves were defined by a trial process in which they were changed step by step to obtain a  
 18 desired routing efficiency  $\eta$  (19).

$$19 \quad \eta = \frac{q_{pi} - q_{po}}{q_{pi}} \quad (19)$$

20 The storage capacity was finally evaluated in 400000 m<sup>3</sup> for both the connection types, the  
 21 threshold discharge  $q_S$  in 45 m<sup>3</sup>/s, and the basin storage constants  $k_S$  were established in 1.1 h,  
 22 for the on-line basin, and in 3.1 h, for the off-line one. Storage constants were estimated as the  
 23 ratio between the stored volume and the outlet discharge corresponding to the filling  
 24 condition. With reference to the previously defined variables, table 2 reports a synthesis of the

1 hydrologic balances and the routing efficiencies assessed for the analysed return periods. The  
2 well-known greater effectiveness of the off-line basin allowed to maintain an efficiency value  
3 of about 31%, for peak rates significantly larger than the threshold  $q_S$ , while a minor value of  
4 about 20% was admitted for the on-line storage solution.

### 5 **3.2 Continuous simulations**

6 Continuous simulations of the rainfall-runoff transformation and routing processes occurring  
7 in the watershed-basin system were performed by means of the previously described model,  
8 in which the observed precipitation time series was employed as meteorological input. Some  
9 adaptations were however necessary to pass from the event based analysis to the continuous  
10 simulation.

11 Firstly, the reduction of the point precipitation to the area was carried out by using variable  
12 factors inside the same storm by considering greater durations centred on the peak intensities.  
13 Secondly, the SCS-CN method can be implemented in this kind of modelling if the AMC  
14 variability and the catchment moisture decay during dry weather periods are properly  
15 accounted for (Mishra and Singh, 2004). Hence, during interevent periods, CN values were  
16 updated with regard to the conventional antecedent precipitation index  $API_5$  and seasonal  
17 AMC thresholds, while the residual moisture at the rainfall end was completely depleted  
18 within five days by an exponential function.

19 The three simulation outputs were statistically analysed through individual event statistics  
20 (IES), that is, the continuous series were separated in independent floods, detecting their peak  
21 flow rates and counting their average annual number  $\theta_f$ . In order to ensure the consistency  
22 with the analytical-probabilistic model, the minimum time between floods utilized for  
23 separating the discharge series were equal to IETDs adopted for fitting the corresponding  
24 probability functions. Thus, the experimental return period  $T$  was estimated by the expression  
25 (20), where  $F_{Qp}$  is the plotting position of the peak rates.

$$T = \frac{1}{\theta_f (1 - F_{Qp})} \quad (20)$$

The more conventional formulation basing on the maximum annual statistics actually gives matching results when the return period is greater than 10:20 years. However, this estimate is conceptually more correct and should be more reliable for the lower return periods.

### **3.3 Analytical probabilistic model calibration**

The precipitation time series was separated in independent events by the discretization thresholds IETD and IA. Bearing in mind the discussion regarding their physical meaning, IETDs of 3 h, 4 h and 6 h were taken into consideration for the inflow discharge, the outflow routed by an on-line basin and the outflow routed by an off-line basin, respectively. The first value was assumed equal to the catchment time of concentration  $t_c$ , while the others were computed as the sum of this quantity and the storage constant  $k_S$ , considered to be an index of the average residence time of the stored runoff. By this choice, the initial condition of totally empty storage volume, implicit in the routing scheme of Whycoff and Singh (1976) implemented in the derivation procedure, is mostly satisfied when an independent event occurs.

In this formulation the volume threshold of the discretization procedure must be equal to the initial abstraction of the simplified hydrologic loss model (2). In addition, the analytical-probabilistic approach must operate in average conditions. Therefore, the threshold IA was set equal to 17 mm, corresponding to AMC II, table 1. Indeed, because of its large value, the latter parameter demonstrated to be dominating in the calibration of the derived cdfs, so that variations in the IETD setting were not particularly relevant.

The scale parameters of the joint probability density function  $p_{VT}$  (1) were then assessed through the maximum likelihood criterion from the independent event samples, obtaining 16.8 mm for  $\zeta$  and 19.8 h for  $\lambda$ . The average annual number of storms  $\theta_s$ , required to convert



1 non exceedance probabilities (6), (11) and (13) in return period by equations analogous to (19),  
2 was estimated as well.

3 However, as it was expected, the parameter with respect to which all derived distributions  
4 proved to be more sensitive was the average runoff coefficient  $\Phi$  of equation (2). Being the  
5 main objective of this work evaluating the discrepancies between continuous approaches,  
6 their maximum consistency had to be ensured. Thus, the average runoff coefficient was  
7 computed through hydrological simulations, comparing the simulated runoff volumes to the  
8 rainfall portions exceeding the actual initial abstraction: a value of 0.32 was assessed.

### 9 **3.4 Result and discussion**

10 The first proof was aimed at verifying that the average annual number of floods  $\theta_f$  and of  
11 storms  $\theta_s$  are equal. In fact, because of the adopted discretization criterion, all independent  
12 storms are runoff producing and consequently issue a flood. As auspicated, matching  
13 outcomes were obtained: a number of 4:6 floods per years were deduced for the defined IETD  
14 range. Such values were considered to be reasonable for the flow regime of this kind of  
15 natural catchment.

16 The frequency distributions of the peak inflow discharge  $q_{pi}$  developed through continuous  
17 approaches are represented in figure 6, along with the peak flow rates derived by the design  
18 storm method. An overall satisfactory agreement is evidenced between the analytical-  
19 probabilistic curve and the continuous simulation statistics, especially when the return period  
20  $T$  rises over 2 years. In the interval 1:2 years, which is however out of interest even for the  
21 smallest urban applications, discrepancies consist in an overestimation of the peak discharges.  
22 This behaviour may be partly explained by the incapability of a triangular hydrograph to suit  
23 the real flood patterns for very low flows, which should lead to a peak magnification. The  
24 observation of the distribution graphs in figure 7 and 8 yield analogous considerations for the  
25 peak outflow discharges routed through the on-line and the off-line basins.

1 The effectiveness both of the derived model formulation and of the calibration criteria is  
2 therefore supported. In particular, i) the implementation of a variable stored volume in spite of  
3 a constant storage capacity demonstrated to be extremely important to avoid performance  
4 overestimations of the on-line basin when the return period ranges between 5:20 years, ii) the  
5 introduction of the non exceedance probability  $P_{Q_S}$  constrain is crucial to catch the general  
6 behaviour of the off-line basin over the discharge  $q_S$ .  
7 However, in all the three cases, peak discharges derived by the design storm method show to  
8 be realistic as well and, for 2 years return periods, to better match continuous simulations than  
9 the analytical-probabilistic curves do. Nevertheless, continuous simulations evidenced that  
10 AMC II, which was assumed for the hydrologic loss computation, is satisfied by only the 18%  
11 of storms. This goodness must consequently be interpreted as an expected, but fortuitous,  
12 compensation of errors.

## 13 **4 Conclusions**

14 Although relevant simplifications in the representation of the probabilistic precipitation  
15 structure and of the hydrologic processes were exploited, a practical analytical-probabilistic  
16 method devoted to size routing basins has been herein derived. The study shows that the  
17 method yields outcomes in satisfactory agreement with those of benchmarking continuous  
18 simulations.

19 Such a model goodness has been achieved through some significant improvements in the  
20 schematization of the device hydraulic behaviour, when the storage volume is supplied either  
21 on-line or off-line. Moreover, since the model was tested with respect to a small-medium size  
22 natural watershed, the possibility to employ analytical-probabilistic methodologies for  
23 hydrologic applications concerning large spatial scales is supported.

24 When the discretization thresholds are chosen with regard to parameters featuring the  
25 considered hydrologic-hydraulic phenomena, the proposed strategy appears to be efficacious

1 in keeping the derived model as simple as possible and evenly isolating events actually  
2 independent for the device. In this particular context, the large value of the initial abstraction  
3 IA allowed to generate floods having significant runoff volumes, while the value of the  
4 interevent time definition IETD equal to the catchment-basin response time avoids the flood  
5 overlapping and ensures that the basin is completely empty for any floods.  
6

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12

1      **TABLES**

2      Table 1: main hydrological characteristics of the Garza watershed.

<i>A</i>	<i>L</i>	<i>H<sub>min</sub></i>	<i>H<sub>med</sub></i>	<i>t<sub>c</sub></i>	<i>CN</i>	<i>S</i>	<i>IA</i>
km <sup>2</sup>	km	m a.s.l.	m a.s.l.	h	-	mm	mm
44,6	15,5	619,9	195,0	3,00	75	84	17

3

4



1 Table 2: main hydrologic characteristics of the design storm events employed to size the on-  
 2 line<sup>†</sup> and the off-line<sup>\*</sup> basins.

T	$v$	$v_r$	IA	$\Phi$	$q_{pi}$	$q_{po}^{\dagger}$	$\eta^{\dagger}$	$q_{po}^*$	$\eta^*$
years	mm	mm	mm	/	m <sup>3</sup> /s	m <sup>3</sup> /s	%	m <sup>3</sup> /s	%
2	41.5	5.6	17	0.23	16.7	13.7	17.9	16,7	-
5	55.6	12.2	17	0.31	37.7	28.5	24.4	37,7	-
10	64.9	17.4	17	0.36	55.6	43.4	22.0	46,1	17.1
20	73.8	22.9	17	0.40	75.3	60.1	20.1	51,6	31.5
50	85.3	30.7	17	0.45	103.6	82.3	20.6	70,6	31.9
100	93.9	36.8	17	0.48	126.6	103.8	18.0	89,4	29.4

3

4

5

## 1        **FIGURE CAPTIONS**

2        Figure 1: Simplified hydrographs associated with the routing process (adapted from Wycoff  
3        & Singh, 1976).

4        Figure 2: integration domain for deriving peak inflow rate distribution.

5        Figure 3: integration domain for deriving peak outflow rate distribution for on line basins.

6        Figure 4: location and DEM of the Garza stream catchment.

7        Figure 5: connection scheme of the storage volume to the stream riverbed a) on-line b) off-  
8        line.

9        Figure 6: distributions of the peak inflow rates (CS IES: continuous simulation individual  
10        event statistics, DSM: design storm method; APM: analytical-probabilistic method).

11        Figure 7: distributions of the peak outflow rates for on line basins (CS IES: continuous  
12        simulation individual event statistics, DSM: design storm method; APM: analytical-  
13        probabilistic method).

14        Figure 8: distributions of the peak outflow rates for off line basins (CS IES: continuous  
15        simulation individual event statistics, DSM: design storm method; APM: analytical-  
16        probabilistic method).

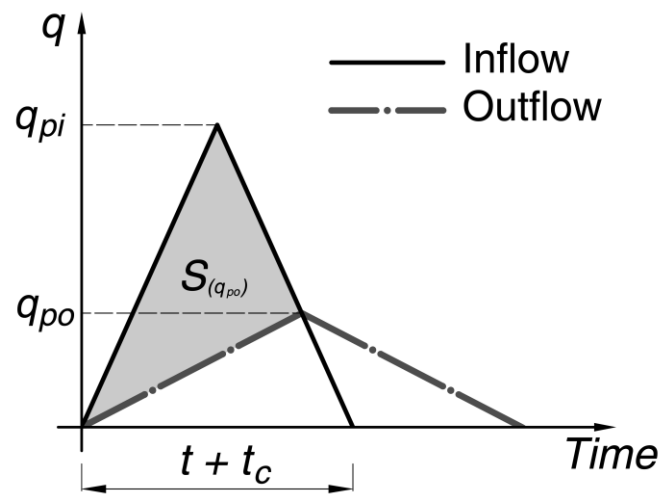
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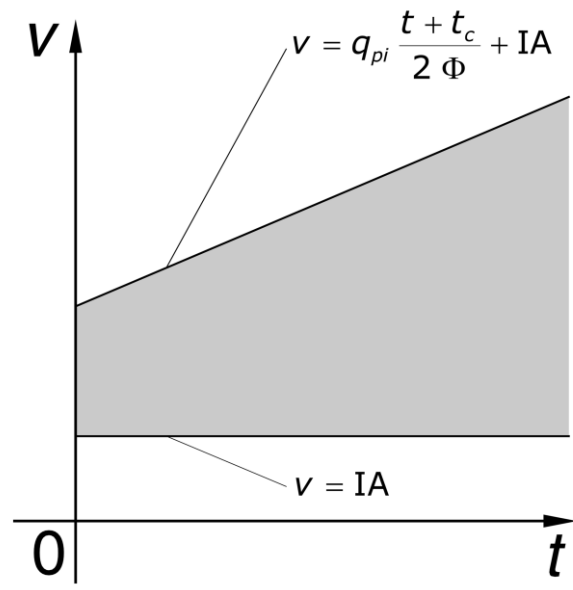
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1      **FIGURES**



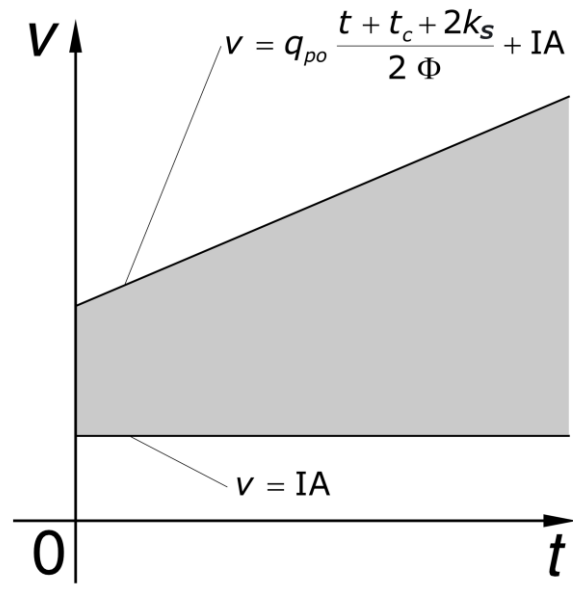
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Figure 1



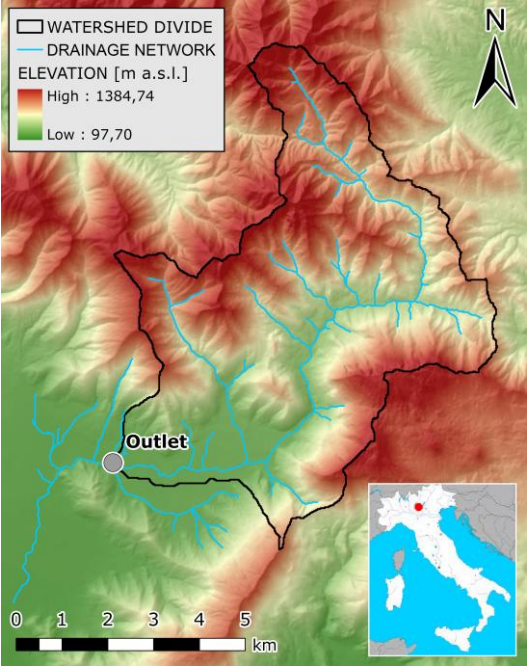
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Figure 2



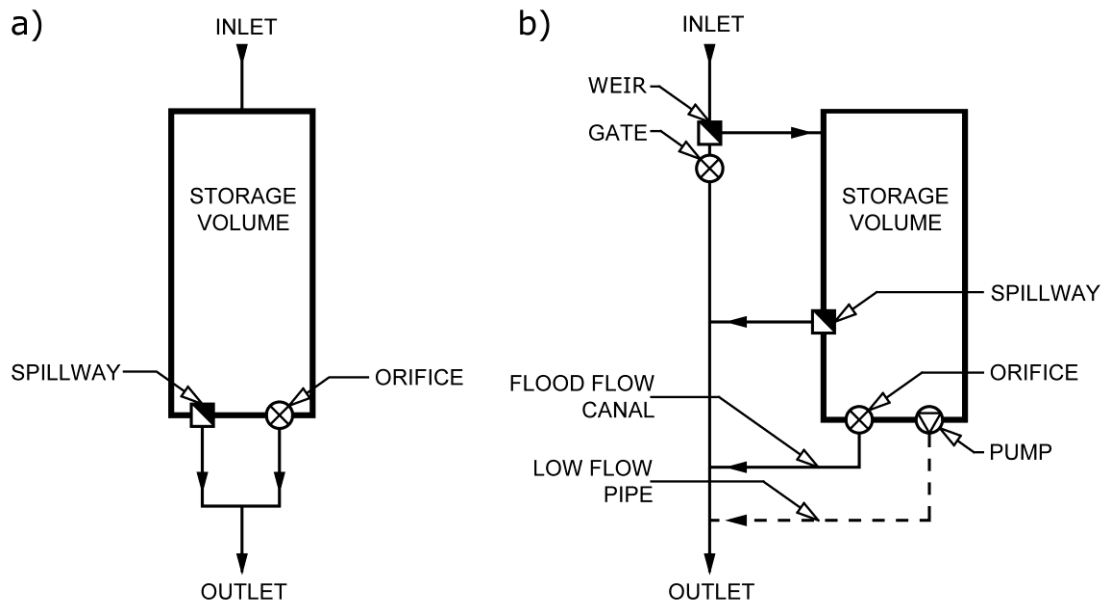
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Figure 3



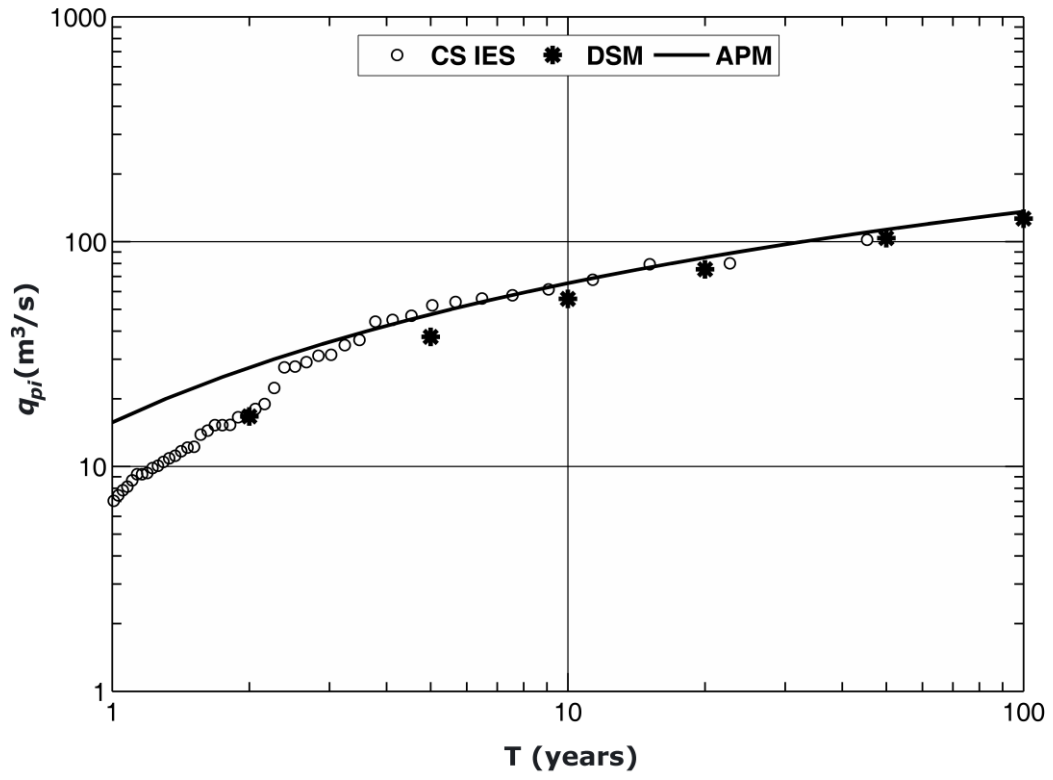
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Figure 4



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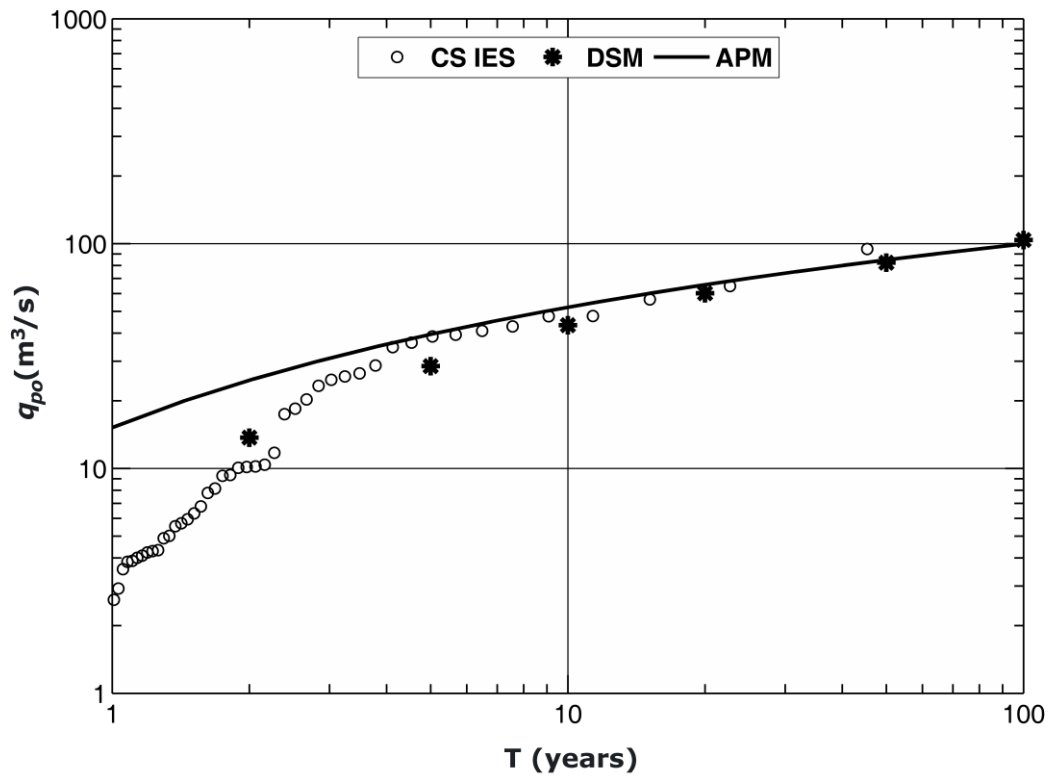
Figure 5



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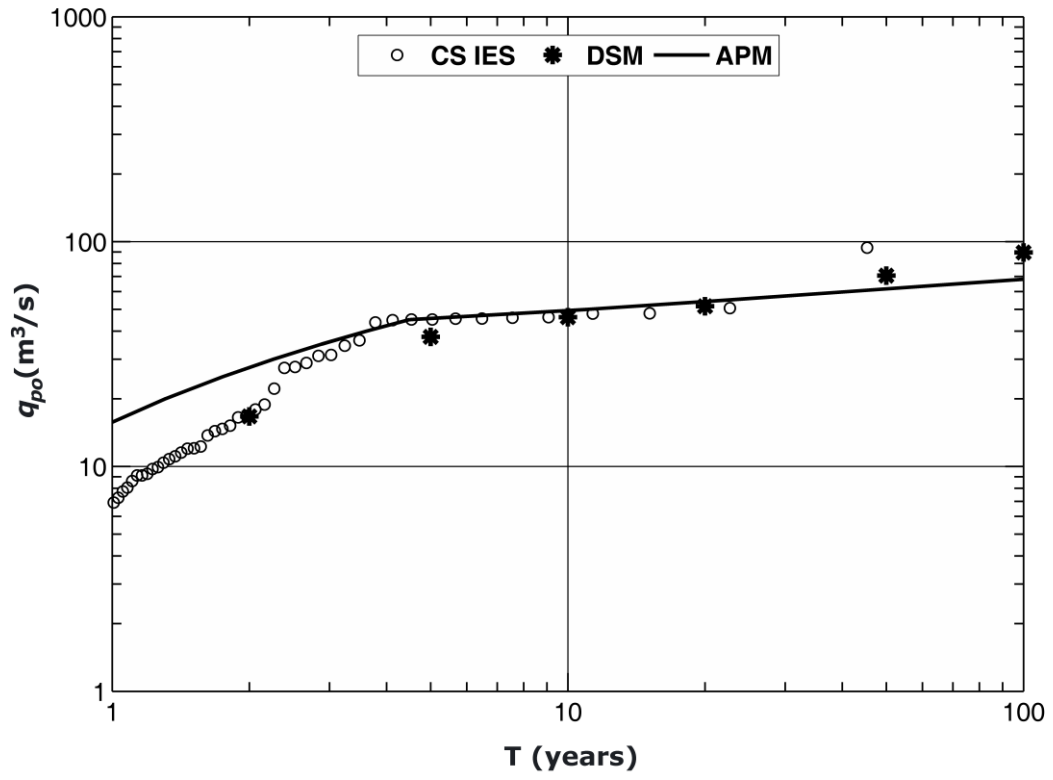
Figure 6





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Figure 7



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Figure 8