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Stochastic rainfall analysis for storm tank performance evaluation

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Abstract

Stormwater detention tanks are widely used for mitigating impacts of combined sewer overflows (CSO) from urban catchments into receiving water bodies. The optimal size of detention tanks depends on climate and sewer system behaviours and can be estimated by using derived distribution approaches. They are based on using a stochastic model to fit the statistical pattern of observed rainfall records and a urban hydrology model to transform rainfall in sewer discharge. A key issue is the identification of the optimal structure of the stochastic rainfall model. Point processes are frequently applied where rainfall events are schematised through the occurrence of rectangular pulses, which are governed by rainfall descriptors. In the model herein used these latter descriptors are the interevent time (duration of the dry period between consecutive storms), event rainfall depth and event rainfall duration. This paper focuses on the analytical derivation of the probability distribution of the number and volume of overflows from the storm tank to the receiving water body for different and non-standard shapes of the probability distribution for above mentioned descriptors. The proposed approach is applied to 2 different sites in Spain: Valencia and Santander located on the Mediterranean and northern Atlantic coastline, respectively. For both cases, it turned out that Pareto and Gamma-2 probability distributions for rainfall depth and duration provided better fit than the exponential model, widely used in previous studies. A comparison between the two climatic zones, humid and semiarid, respectively, proves the key role played by climatic conditions for storm detention tanks sizing.

1 Introduction

Stormwater detention tanks are widely used for mitigating impacts of combined sewer overflows into receiving water bodies. Even if a lot of methodologies for sizing these facilities have been developed in the last decades, there are still some open questions for the determination of the appropriate detention volume required to keep overflow pollutant concentration within acceptable limits (Deutsch et al., 2003). Some countries are making strong efforts to standardize these methods, always realizing that climatic conditions play a key role (Woods-Ballard et al., 2007). In general, techniques used to size such storage facilities and analyze their performance fall into two categories: analytical methods and simulation approaches. Analytical statistical methods estimate tank performances by analytically deriving the probability distribution of tank failure and overflow volume, depending on rainfall statistics and by using simple schemes to compute the sewer flow. Simulation approaches are carried out by generating synthetic long series of sewer discharges, from which statistics for tank performance can be derived. In this paper we focus on the former category, therefore estimating tank efficiency by means of analytical derivation, while we validate the results by performing continuous simulation.

Since DiToro and Small (1979) developed one of the first probabilistic methods the state-of-the-art has notoriously improved. Early developments by the US Environmental Protection Agency (EPA) (Driscoll et al., 1986) described the probability density function of the runoff process and their relationship with tank performance. Then, some quality aspects such as pollutant removal by sedimentation were considered (Walker et al., 1993; Papa and Adams, 1996).

All probabilistic approaches require a description of the rainfall input through a stochastic model. Point processes are frequently applied where rainfall events are schematised through the occurrence of rectangular pulses, which are governed by rainfall descriptors. In the model herein used these latter are the expected values of (1) interevent time, (2) event rainfall depth and (3) event rainfall duration. A key issue is the description with probability density functions (pdfs) of the frequency of occurrence for the above descriptors. Exponential functions have been usually adopted. In fact, most of the models used to date are based on this assumption (DiToro and Small, 1979; Adams et al., 1986; Guo and Adams, 1999; Guo and Urbonas, 2002). Actually, robust probabilistic methods, developed in the US and Canada, use the above assumption. One of the reasons why exponential probability distribution has been widely used is its simplicity which makes analytical derivation easier. On the other hand it has limited ability to describe rainfall statistics in some cases, including the rainfall series analysed in this paper. Therefore we looked for an alternative approach.

In this paper an analytical model for designing the storage volume of detention tanks is proposed, which uses alternative solutions to the exponential distribution for rainfall descriptors. A simple conceptual rainfall-runoff model is used to transform rainfall into sewer discharge, therefore allowing to analytically deriving the probability distribution of number and volume of overflows for the detention tank.

The suitability of the exponential model for rainfall descriptors is first discussed with reference to the rainfall series of two different sites in Spain: Valencia and Santander. As significant differences from the exponential model emerge, alternative probability distributions are considered. Then, results for rainfall characterisation in northern and eastern Spain are compared. Finally, analytical probabilistic expressions are derived in order to assess the frequency of occurrence of number and volume of tank overflows, allowing to estimate the storm tank efficiency. In detail we estimate volumetric efficiency and overflow reduction efficiency. The former is defined as the long term ratio between runoff volume produced in the urban catchment and volume detained by the tank. The latter is defined as the probability of an event to produce overflow, which can be estimated as the long term ratio of number of events completely detained over total number of events. Accuracy of the results achieved with this latter analytical approach is checked by comparison with the outcome of a continuous simulation performed in a urban catchment in Valencia.

2 Description of data set and case study

2.1 Meteorological data

High resolution rainfall data over an extended period are needed in order to reliably assess the suitability of different probability distributions for the rainfall descriptors.

Accordingly, rainfall series from Valencia (Eastern Spain) and Santander (Northern Spain) have been collected.

Valencia is located on the eastern Mediterranean coastline of Spain. Its climate is Mediterranean, mild, with some semi-arid features. Average temperature is around 18 °C, with oscillations between 11 °C in January and 26 °C in August. Average annual rainfall depth is close to 450 mm, with a very unequal distribution trough the year. Rainfall storms are usually concentrated in autumn, with typical very high peak intensities (torrential rain). The rainfall series was observed by the Júcar river basin hydrological service (SAIH) during the period 1990–2006 with 5-min resolution. Observations were checked and validated by a comparison with the Spanish Meteorological Agency (AEMET) daily observations.

Santander is located on the northern Atlantic coast of Spain. The city is under the influence of a humid oceanic climate; its main features are a mild and warm temperature regime and plenty of rainfall well distributed throughout the year. Thus, average temperatures are between 9 °C in February and 20 °C in August and average annual rainfall depth is over 1100 mm. The rainfall series was observed by AEMET with 5-min resolution during the period 1942–1951 and 1955–1983.

To further confirm reliability of the data, rainfall observations were aggregated into monthly and annual totals and then compared with those obtained in nearby raingauge stations. In the case of Santander the validation was fully satisfactory and, in the case of Valencia, it allowed us to identify and correct two observation errors occurred during the years 1990 and 2000. Monthly aggregated rainfall series for both locations are shown in Fig. 1.

2.2 Case study

The analytical model developed in this paper was applied to size and verify a detention tank located in an urban catchment in Valencia. The results are compared with those obtained with a continuous simulation approach. Environmental impacts of CSO to the receiving water bodies (Valencia beaches and the America's Cup leisure docks) are being increasingly taken into consideration. For this reason the local sewerage master plan is in charge of developing guidelines to size detention tanks.

The Pio XII urban catchment is located at the headwaters of one of the most important trunk sewer of the city which frequently overflows into the above referred docks. The catchment is 68.8 hectares large and the length of the considered sewer network is 13.4 km with 565 manholes, i.e. one manhole each 23.7 m on average (see Fig. 2).

The network topology and geometry, as well as historical hydraulic data, are provided by the Municipality of Valencia. Land use distribution, which is needed in order to estimate infiltration parameters for the rainfall-runoff model, is obtained from data provided by the Urban Master Plan and reclassified according to the local guidelines for sewer system design (Municipality of Valencia, 2004) which consider 4 land uses only: paved areas, high density building areas, low density building areas and green spaces. For each land use, a dimensionless area ratio, a_i, is defined as

$$a_i = \frac{A_i}{A}, \tag{1}$$

where A is the total tributary area of the catchment and A_i the total area of land use i in the catchment. In addition, each land use is characterized by an infiltration parameter P_{0i} (mm) which represents the runoff threshold, i.e. the amount of rainfall needed for runoff to begin. Table 1 summarizes all these parameters.

3 Framework of the analysis

3.1 Rainfall model

Rainfall characterisation is carried out by identifying and calibrating a suitable stochastic process for rainfall intensity along time. We consider a point process as a candidate model and assume rainfall events can be represented as rectangular pulses occurring accordingly to a Poisson process. This means that interevent time is exponentially distributed (Cox and Isham, 1980). We also assume that event rainfall depth and duration are outcomes from two different and independent stochastic processes. Therefore the rainfall descriptors are interevent time, $s(t)$, event rainfall depth, $v(t)$, and event duration, $d(t)$. These are supposed to be governed by stochastic processes indicated with the symbols S, V and D, respectively.

Main steps for identification and calibration of the above rainfall model are (1) identification of statistically independent storms, (2) study of temporal dependence and cross dependence between rainfall descriptors which are derived for each event and, finally, (3) fitting of probability density functions (pdfs) to the descriptors themselves.

As for step (1), the approach that is adopted here consists of selecting a critical value s_{crit} for the interevent time $s(t)$, so that events separated by a dry period greater than s_{crit} are considered to be independent (note that the interarrival time between events is also used with the same purpose. For a discussion see Bonta and Rao, 1988). A technique that is frequently used for selection of the optimal value of s_{crit} was proposed by Restrepo-Posada and Eagleson (1982). Accordingly, all events are first considered to be statistically independent therefore obtaining a sample of $s(t)$ values. Then, s_{crit} is identified so that the hypothesis that the $s(t)$ values greater than s_{crit} can be considered outcomes from the exponentially distributed stochastic process S cannot be rejected (see also Bonta and Rao, 1988). The obtained value of s_{crit} is used for identifying independent storms.

The original methodology as developed by Restrepo-Posada and Eagleson (1982) establishes the selection of s_{crit} by considering that the coefficient of variation (CV) of a Poisson process should be equal to unity. In fact, if the exponential probability distribution has mean and standard deviation equal to β^{-1} and therefore CV=1. In the operational practice, for a trial value of s_{crit}, statistical tests can be applied in order not to reject the hypothesis that CV=1 for an assigned confidence level.

In the present study we developed a modified statistical criteria for the selection of s_{crit}. In fact, we fitted to each realisation of S resulting from the corresponding trial s_{crit} value a bounded exponential distribution in order to take into account that the S series is limited from below by s_{crit}. The bounded exponential distribution is given by

$$F_S(s) = 1 - e^{-\beta(s - s_{crit})} \qquad s \geq s_{crit}. \tag{2}$$

Note that the CV of the bounded exponential distribution is different from unity. Then, we applied the Kolmogorov-Smirnov (KS) test (as done by Koutsoyiannis and Xanthopoulos, 1990) to compare the empirical and theoretical probability distribution given by (2) by using modified test statistics for the exponential model (Law and Kelton, 1991). In detail, Hazen plotting position was used in order to estimate the empirical frequency distribution of each s value, as recommended for highly skewed populations (an unbounded exponential population has a skewness coefficient equal to 2). Then, we estimated the parameter β by maximum likelihood and theoretical probabilities for each s were obtained by Eq. (2). Goodness of fit was also checked by computing the Nash-Sutcliffe (NS) index of empirical versus theoretical distributions and the progress of the mean value of S and the average number of events per year against s_{crit}, which should follow a linear relationship for a Poisson process (see Sect. 4.1). If a reasonable value for s_{crit} can be estimated, suitability of the exponential distribution for interevent time is confirmed, therefore providing support to the assumption that rainfall events occur accordingly to a Poisson process.

Once s_{crit} is selected, $v(t)$ and $d(t)$ values can be estimated for each independent event. Then, step (3) of the analysis can be carried out, which consists of checking the mutual independence of S, V and D. This check is necessary to provide further support to the assumption of independence among the identified rainfall events. In fact, independent events are characterised by the absence of temporal correlation for each of the stochastic processes S, V and D, as well the absence of mutual correlation between S, V and S, D. Besides, independence among S, V and D allows us to introduce simplifying assumptions for the analytical model of number and volume of tank overflows (see Sect. 4.1).

Dependence structure was analysed by estimating the linear autocorrelation coefficient, for increasing lag up to 10, of each stochastic process and the cross correlation coefficient, at lag 0, among them. The obtained coefficients were compared against Anderson limits of the null value at 98% confidence level.

Finally, step (3) of the analysis can be carried out by fitting pdfs to stochastic processes V and D. Different candidate models were considered, namely, the exponential distribution which is traditionally chosen in many studies, as well as the Weibull, Gamma-2, Lognormal and generalised Pareto distributions. This latter distribution turned out to be the most appropriate for $v(t)$ in the case studies considered here (see Sects. 4.1 and 4.2). In fact, this choice is supported by the maximum entropy principle applied to hydrological variables which implies that the appropriate distribution of certain variables, for a given coefficient of variation, should lead to the maximum entropy. The physical reason for this outcome would be that "nature behaves in a manner that makes uncertainty as high as possible" (Koutsoyiannis, 2005). Cumulative probability function for the generalised Pareto distribution is given by

$$F_V(v) = 1 - \left(1 + \kappa v/\alpha\right)^{-1/\kappa}, \quad (3)$$

where $\kappa > 0$ and $\alpha > 0$ are the shape and scale parameters, respectively. The generalised Pareto distribution performed satisfactorily for d as well, although for Valencia the Gamma-2 model provided a slightly better fit (see Sects. 4.1 and 4.2).

3.2 Rainfall-runoff model

A rainfall-runoff model was used in order to estimate the volume of sewer discharge generated in the catchment by each rainfall event. The SCS-CN model was adopted, which was previously recalibrated for the urban area of Valencia (González, 2001). Model basis is the continuity equation

$$v = r(v) + f(v) + P_0, \quad (4)$$

where P_0 is the initial abstraction and v, $r(v)$ and $f(v)$ are volumes of rainfall, runoff and infiltration of the event, respectively. Accordingly to the SCS-CN model $r(v)$ is given by

$$\begin{cases} r(v) = 0 & \text{if} \quad v \leq P_0 \\ r(v) = \dfrac{(v-P_0)^2}{v+4P_0} & \text{if} \quad v > P_0 \end{cases} \tag{5}$$

Equation (5) implies that no runoff occurs when event rainfall depth is smaller than the threshold value P_0. Thus, under the assumption that V is distributed according to the generalised Pareto probability distribution, we obtained that the cumulative probability of null flow is given by

$$F_R(0) = F_V(P_0) = \int_0^{P_0} f_V(v)dv = 1 - \left(1 + \kappa P_0/\alpha\right)^{-1/\kappa}. \tag{6}$$

In Eq. (6), R indicates the random variable whose outcome is the event runoff r. On the other hand, when the threshold value P_0 is exceeded then $R > 0$, and the expression for the cumulative probability distribution of runoff volume is

$$F_R(r) = \int_0^r f_R(r)dr = F_R(0) + \int_{P_0}^{v} f_V(v)dv = 1 - \left(1 + \kappa v/\alpha\right)^{-1/\kappa} \tag{7}$$

with an implicit expression for $v(r)$. Thus, probability density function for runoff volume is given by

$$f_R(r) = \frac{d}{dr}F_R(r) = \frac{1}{\alpha}\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} \cdot \frac{dv}{dr} = \frac{\frac{1}{\alpha}\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}}{\frac{d}{dv}r(v)} \tag{8}$$

with again an implicit expression for $v(r)$.

Total area A of the urban catchment was divided into 4 different types of land use, each one affected by a different initial abstraction P_{0i}. Thus, runoff volume generated by the rainfall event is:

$$r(v) = \sum_{i=1}^{4} a_i r^{(i)}(v) \tag{9}$$

where a_i is the land use area ratio defined by Eq. (1) and $r^{(i)}(v)$ is the runoff generated in area A_i computed by applying Eq. (5) to area i. Therefore, Eq. (8) can be rewritten as

$$f_R(r) = \frac{d}{dr}F_R(r) = \frac{1}{\alpha}\left(1+\kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}\cdot\frac{dv}{dr} = \frac{\frac{1}{\alpha}\left(1+\kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}}{\sum_{i=1}^{4} a_i \frac{d}{dv} r^{(i)}(v)}. \tag{10}$$

According to the minimum initial abstraction P_{01} which corresponds to paved areas (see Table 1), the impulse probability for $r=0$ is

$$F_R(0) = F_V(P_{01}) = \int_{v=0}^{P_{01}} f_V(v)dv = 1 - \left(1+\kappa\frac{P_{01}}{\alpha}\right)^{-1/\kappa}. \tag{11}$$

Finally, the expected value of event runoff volume in the catchment takes the form

$$E(R) = \int_0^{\infty} r f_R(r)dr = \frac{1}{\alpha}\int_0^{\infty}\left(1+\kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}\sum_{i=1}^{4} a_i r^{(i)}(v)dv. \tag{12}$$

3.3 Tank overflow model

The purpose of tank overflow model is to provide an analytical relationship for the number and volume of overflows from a CSO system controlled by a tank with volume V_D. We indicate with the symbol Q_V the maximum flow rate from the tank to waste water treatment plant (WWTP) (see Fig. 3).

To derive the above analytical expression, we need to first deduce the probability density function of the overflow volume which we assumed to be a random variable that will be indicated with the symbol W. Let us assume that runoff occurs as a rectangular pulse. Then, overflow volume during an assigned event is given by

$$\begin{cases} w = 0 & \text{if} \quad r(v) \leq V_D + Q_V \cdot (d(t) + t_C - t_R) \\ w = r(v) - V_D - Q_V \cdot (d(t) + t_C - t_R) & \text{if} \quad r(v) > V_D + Q_V \cdot (d(t) + t_C - t_R) \end{cases} \tag{13}$$

where $d(t)$is duration of rainfall event, t_C is concentration time of the catchment and t_R is lag time between storm and runoff origins. In order to obtain a precautionary estimation, we set $Q_V = 0$ during the event. Therefore the probability of no overflow is given by

$$F_W(0) = F_R(V_D), \tag{14}$$

and the runoff volume r is determined by event rainfall depth only, which we assumed to be distributed accordingly to a generalised Pareto distribution (see Sect. 3.1). Given that $Q_V = 0$, it follows that

$$F_W(0) = F_R(V_D) = F_V\left(V_D^*\right) = 1 - \left(1 + \kappa V_D^*/\alpha\right)^{-1/\kappa} \qquad w = 0 \Leftrightarrow v \leq V_D^* \tag{15}$$

where V_D^* is the rainfall depth generating a runoff volume equal to the tank volume, that is, $r(V_D^*) = V_D$. Thus, if rainfall volume is smaller or equal than this threshold value, that is, if $v \leq V_D^*$, then there is no overflow ($w = 0$). If $v > V_D^*$, then $w > 0$ and therefore,

$$F_W(w) - F_W(0) = F_R(r) - F_R(V_D) = \int_{V_D}^{r} f_R(r)dr \qquad w > 0 \Leftrightarrow v > V_D^*. \tag{16}$$

By considering that $F_R(r) = F_V(v)$ and $F_R(V_D) = F_V(V_D*)$, the latter probability could be written as

$$F_W(w) - F_W(0) = F_V(v) - F_V(V_D*) = \int_{V_D^*}^{v} f_V(v)dv = \left(1 + \kappa V_D^*/\alpha\right)^{-1/\kappa} - \left(1 + \kappa v/\alpha\right)^{-1/\kappa}. \tag{17}$$

Then, the distribution function assumes the following expression:

$$F_W(w) = \begin{cases} 1 - \left(1 + \kappa V_D^*/\alpha\right)^{-1/\kappa} & \text{if} \quad w = 0 \\ 1 - \left(1 + \kappa v/\alpha\right)^{-1/\kappa} & \text{if} \quad w > 0 \end{cases} \tag{18}$$

Therefore, expected value of spilled volume can be derived, given by the relationship

$$E(W) = \frac{1}{\alpha}\int_{V_D^*}^{\infty}\left(\sum_{i=1}^{N} a_i r^{(i)}(v) - V_D\right)\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} dv. \tag{19}$$

From Eqs. (12) and (19) volumetric efficiency of the tank, $\mu_v(V_D)$, can be derived, that is,

$$\mu_v(V_D) = 1 - \frac{E(W)}{E(R)} = 1 - \frac{\int_{V_D^*}^{\infty}\left(\sum_{i=1}^{N} a_i r^{(i)}(v) - V_D\right)\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} dv}{\int_0^{\infty}\sum_{i=1}^{N} a_i r^{(i)}(v)\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} dv}. \tag{20}$$

Volumetric efficiency is an important index of performance, allowing to assess the mean volume detained by the tank expressed as fraction of event runoff.

Finally, overflow reduction efficiency, $\mu_o(V_D)$, can be derived from Eq. (15) as it states the probability of no overflow. It is expressed by the relationship

$$\mu_o(V_D) = F_W(0) = 1 - \left(1 + \kappa V_D^*/\alpha\right)^{-1/\kappa} \tag{21}$$

and gives the probability of an event to not produce overflow depending on detention tank volume.

4 Results

4.1 Rainfall data analysis for Valencia

Different trial values of s_{crit} were considered in the range between 5 min (the series resolution) and 48 h. For each selected value of s_{crit} the resulting s series was extracted and maximum likelihood estimates for β parameter were obtained. Besides, for each case, the statistical tests mentioned in Sect. 3.1 were performed. Results are summarized in Fig. 4. Notice that, accordingly to the tests, s_{crit} seems to vary in the range between 18 and 30 h. Relationships of the average number of events per year and the mean value of S against s_{crit}, begin to exhibit linearity from an s_{crit} of 18–20 h. The NS index is above 0.95 for an s_{crit} greater than 18 h. Finally, the KS test is satisfactory for an s_{crit} value around 30 h (the p-value is 1.308 for a significance level α=0.01). Thus, a value of 22 h, which corresponds to β=0.0059 h^{-1}, was finally selected. It is close to the lower bound of the plausible values in order to increase the sample size of the rainfall events.

Selection of the optimal value of s_{crit} is also supported by the correlation analysis. In fact, it can be noticed that autocorrelation coefficients, $\rho_V(k)$ and $\rho_D(k)$, for event rainfall depth and event duration are always included within the 98% confidence limits for null value. For cross correlation analysis, cross correlations coefficients between s,v and s,d are close to 0 ($\rho_{S,V}(0) = -0.01$ and $\rho_{S,D}(0) = -0.03$) and so included within the same confidence limits. Fig. 5 shows scatterplots of v versus d and s. Notice that mutual independence between v and s is confirmed while the presence of significant correlation between v and d ($\rho_{V,D}(0)$ =0.667) is evident. This fact was also found by other authors (Adams and Papa, 2000). However, the above cross correlation is not significant enough to affect the probabilistic analysis later developed on Sect. 4.3. Therefore, the multivariate probability density function of S, D and V is readily obtained by multiplying the marginal distributions.

Table 2 shows the CV and γ values for the $v(t)$ and $d(t)$ time series. These values were also computed by censoring events whose rainfall depth is lower than 1 mm. Results show that the exponential model is quite unlikely in all cases, because CV and γ are not close to target values of 1 and 2, respectively, with the sole exception of the CV value for censored $d(t)$ series. Therefore a different pdf has to be selected for event rainfall depth and event duration. Competing formulations were identified by taking into account that the identified series are characterised by high skewness. Accordingly, Weibull, Gamma-2, Lognormal and Pareto distributions were considered by estimating their parameters with maximum likelihood procedure (MLE). Goodness-of-fit testing was performed by Kolmogorov-Smirnov (KS) test for exponential distribution, and Cramer-von-Mises (CVM) test (Choulakian and Stephens, 2001) for Pareto and Gamma-2 distributions. In all cases goodness of fit was better for censored series, whose results are summarised in Table 3. It turns out that $v(t)$ is better described by the Pareto model, while Gamma-2 model provides the better fit for $d(t)$, even though fitting provided by the exponential model is not much different in this latter case (see Fig. 6). Parameter values for best distributions are summarized in Table 4.

More accurate models than the exponential one for event rainfall depths were experienced by other authors. For instance, the Weibull model turned out to be more appropriate for some locations (Brescia, Milano, Palermo, Parma and Pavia) in Northern Italy (Balistrocchi et al., 2008).

4.2 Rainfall data analysis for Santander

Rainfall analysis was repeated for the Santander rainfall series. In this case, a s_{crit}=12 h was obtained. As for Valencia case study, $v(t)$ is well described by the Pareto distribution (see Table 5), which turns out to be the best model for $d(t)$ also. Fit provided by the exponential model is again significantly outperformed by the Pareto alternative (see Fig. 7).

It is interesting to compare the results obtained for Valencia and Santander, which are characterised by a semiarid and humid climate, respectively. Critical interevent time is half at Santander than at Valencia, highlighting that for maritime climate storms occur more frequently (Table 6). An important difference lies in event duration pdf. While at Valencia the Gamma-2 probability function provides good fit (being very close to the exponential model), Pareto model provides a better fit at Santander. These results confirm the significant variability of statistical behaviours for rainfall regime and justify the effort undertaken in this paper to develop a flexible approach for identify the most appropriate probability distributions for rainfall descriptors.

4.3 Application of the probabilistic model for detention tank design

The analytical model is finally applied to the selected case study of the Pio XII urban drainage catchment in Valencia. According to catchment land use (see Table 1) and estimated rainfall descriptors (see Table 4), expected values for runoff volume, $E(R)$, and overflow volume, $E(W)$, are obtained for different trial values of tank volume V_D. The above expected values have been estimated by means of numerical integration of $F_R(r)$ and $F_W(w)$. Thus, the tank efficiencies $\mu_v(V_D)$ and $\mu_o(V_D)$ were evaluated through Eqs. (20) and (21).

In order to validate results given by the analytical method, continuous simulation was performed by using observed 17 years rainfall record which includes a total of 464 independent rainfall events (identified by adopting s_{crit}=22 h). A complete model of sewer network was built with InfoWorks CS software (Wallingford Software, 2008) and simulations were thus performed for a set of 7 tank volumes defined by specific volumes equal to 5, 10, 36, 50, 75, 100 and 200 m^3/ha. The value 36 m^3/ha was simulated because it corresponds with the specific volume traditionnally recommended by the Municipality of Valencia.

For each tank volume, V_D, total volume spilled per event w_j was obtained. Depending on total runoff volume per event, r_j, volumetric efficiencies are evaluated by

$$\mu_{v,\text{sim}} = 1 - \frac{\sum_{j=1}^{464} w_j}{\sum_{j=1}^{464} r_j}. \tag{22}$$

Similarly, the number of simulations generating overflow were counted to evaluate overflow reduction efficiency,

$$\mu_{o,\text{sim}} = 1 - \frac{\sum_{j=1}^{464} \delta_j}{464} \tag{23}$$

where

$$\delta_j = \begin{cases} 1 & \text{if} \quad w_j > 0 \\ 0 & \text{if} \quad w_j = 0 \end{cases} \tag{24}$$

Figure 8 summarises volumetric efficiencies and overflow reduction efficiencies obtained with both probabilistic and continuous approaches. It can be seen that the probabilistic model provided satisfactory results.

5 Conclusions

Exponential models have been widely and successfully used in the US and Canada for rainfall regime characterization as a preliminary step for development of detention tank sizing procedures. Nevertheless, case studies prove that such modelling solution is simpler but often inefficient in many locations, including Valencia and Santander, considered in this study. As expected accordingly to entropy based considerations, the Pareto probability distribution provided a better fit for event rainfall depth, while event duration is better fitted by the Gamma-2 and Pareto distributions at Valencia and Santander respectively. These facts highlight the importance of local conditions for this issue.

An analytical approach was proposed to assess long term volumetric and overflow reduction efficiencies of storm detention tanks for sewer systems. Application of these probabilistic expressions at an urban catchment in Valencia shows satisfactory performance for a simple single tank system. Results presented here provide support to the design of storm detention tanks for limiting pollutant concentration into receiving water bodies.

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Table 1. Pio XII urban catchment parameters.

i	Area description	A_i (ha)	a_i	P_{0i} (mm)
1	Paved areas	26.73	0.388	1.0
2	High density buildings	20.92	0.304	4.4
3	Low density buildings	13.16	0.191	17.8
4	Green spaces	8.01	0.116	70.2
	TOTAL	68.82		

Table 2. Sample coefficients of variation (CV) and skewness (γ) of event rainfall depth and duration estimated for Valencia raingauge.

	Non censored series		Censored series (1 mm)	
	CV	γ	CV	γ
Depth	2.06	4.76	1.57	3.97
Duration	1.47	2.80	1.14	2.32

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Table 3. Goodness of fit statistic values (p-value) for original and censored rainfall series.

Rainfall descriptor	Probability distribution function	Goodness of fit test		
		Test	Series	p-value
Event rainfall depth	Pareto	CVM	Original	0.76
		CVM	Censored	0.62
Event duration	Gamma-2	CVM	Original	1.01
		CVM	Censored	0.11
	Exponential	KS	Original	6.93
		KS	Censored	2.39

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Table 4. Distribution functions and related MLE parameters for rainfall descriptors related to Valencia raingauge. Rainfall series was censored by excluding events whose rainfall depth is lower than 1 mm. F and f indicate cumulative probability and the probability density, respectively. Note that an explicit formulation of F can not be provided for the Gamma-2 distribution.

Rainfall descriptor	Probability function		MLE Parameters
Interevent time	Exponential	$F_S(s) = 1 - e^{-\beta(s - s_{\text{crit}})}$	$\beta = 0.0059$
Event rainfall depth	Pareto	$F_V(v) = 1 - \left(1 + \kappa\frac{v}{\alpha}\right)^{-1/\kappa}$	$\kappa = 0.4110$ $\alpha = 8.4605$
Event duration	Gamma-2	$f_D(d) = \frac{\lambda^\varepsilon}{\Gamma(\varepsilon)} d^{\varepsilon-1} e^{-\lambda d}$	$\varepsilon = 0.7401$ $\lambda = 0.0364$
	Exponential	$F_D(d) = 1 - e^{-\lambda d}$	$\lambda = 0.0492$

Table 5. Distribution functions and related MLE parameters for rainfall descriptors estimated for Santander raingauge.

Variable	Probability function		MLE Parameters
Interevent time	Exponential	$F_S(s) = 1 - e^{-\beta(s - s_{\text{crit}})}$	$\beta = 0.0158$
Event rainfall depth	Pareto	$F_V(v) = 1 - \left(1 + \kappa \frac{v}{\alpha}\right)^{-1/\kappa}$	$\kappa = 0.3435$ $\alpha = 9.7431$
Event duration	Pareto	$F_D(d) = 1 - \left(1 + \frac{\gamma d}{\mu}\right)^{-1/\gamma}$	$\gamma = 0.1000$ $\mu = 25.4573$

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Table 6. Comparison between Valencia (VLC) and Santander (STD) rainfall descriptors.

Rain-gauge	Critical IET (h)	E(S) (h)	Average number of events per year	Rainfall depth		Duration	
				PDF	E(V) (mm)	PDF	E(D) (h)
VLC	22	191	27.3	Pareto	14.4	Gamma-2	20.3
STD	12	75	68.9	Pareto	14.8	Pareto	28.3

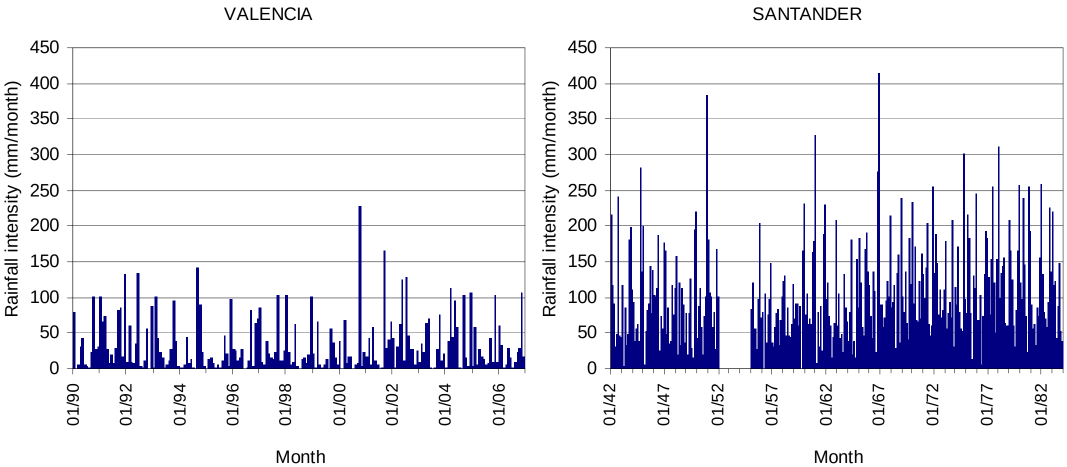

Fig. 1. Monthly rainfall time series for Valencia and Santander.

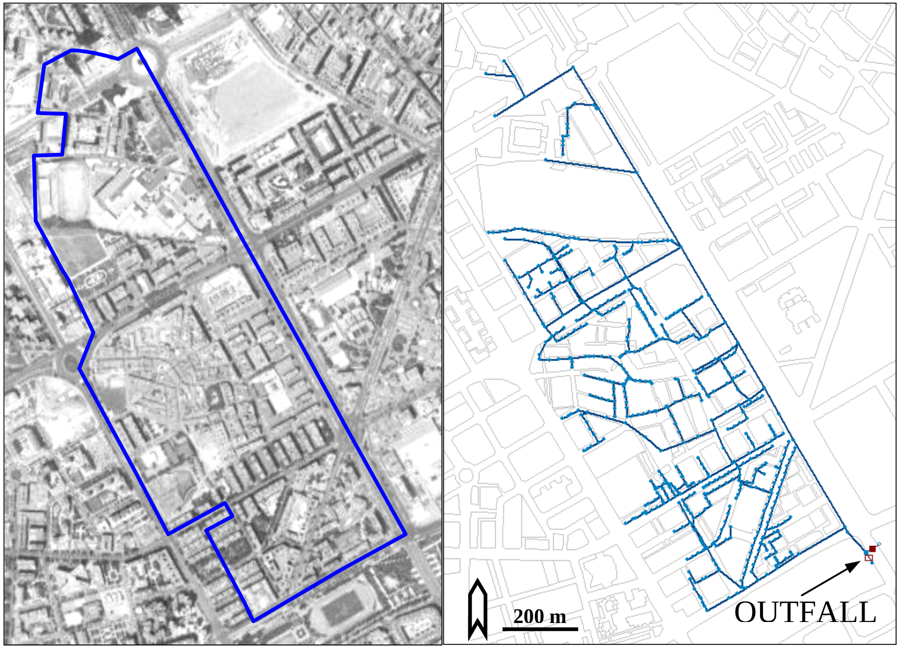

Fig. 2. The Pio XII urban catchment in Valencia: tributary area and topology.

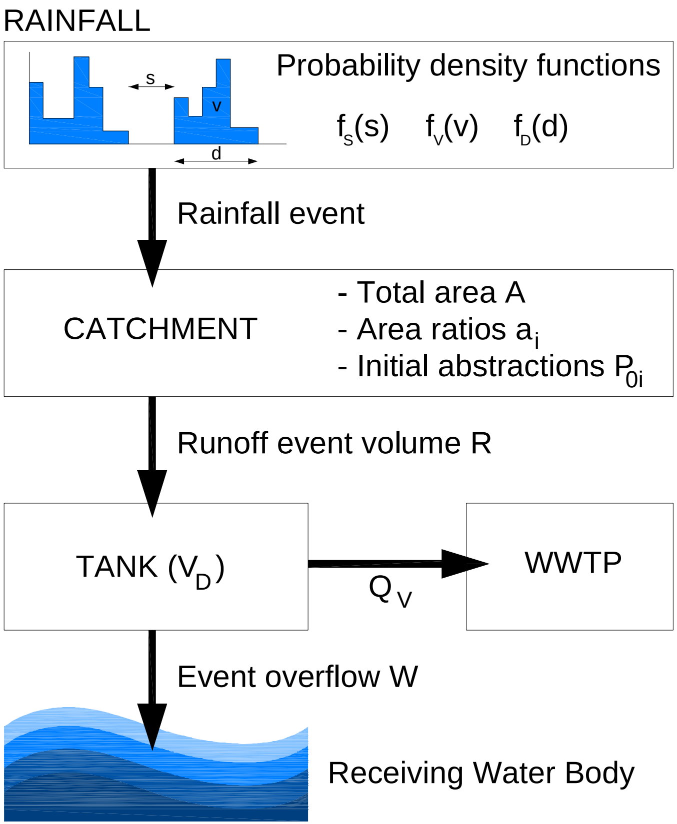

Fig. 3. System elements: rainfall, runoff and detention.

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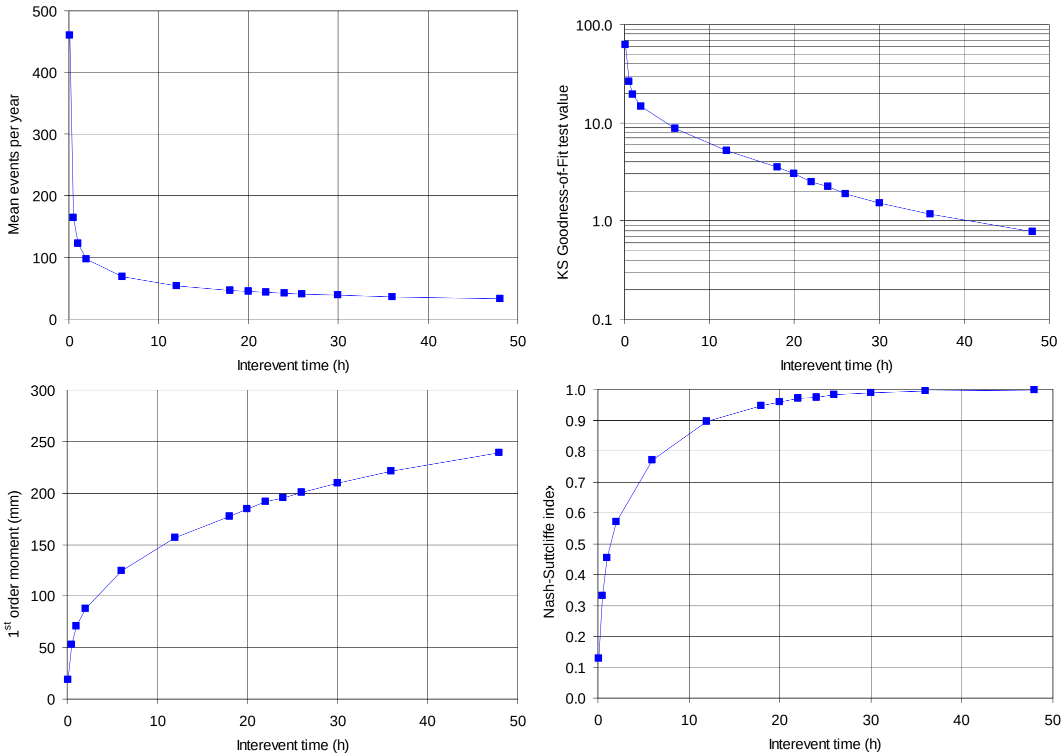

Fig. 4. Decision parameters evolution for critical interevent time selection.

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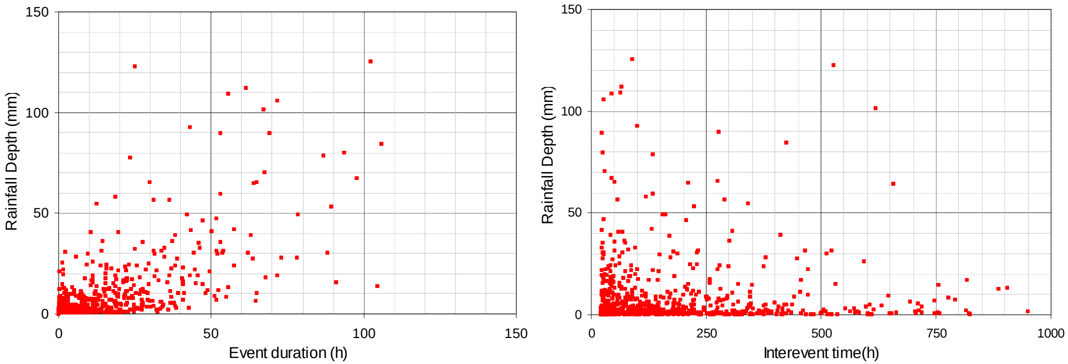

Fig. 5. Scatterplots of v versus d (left) and s (right).

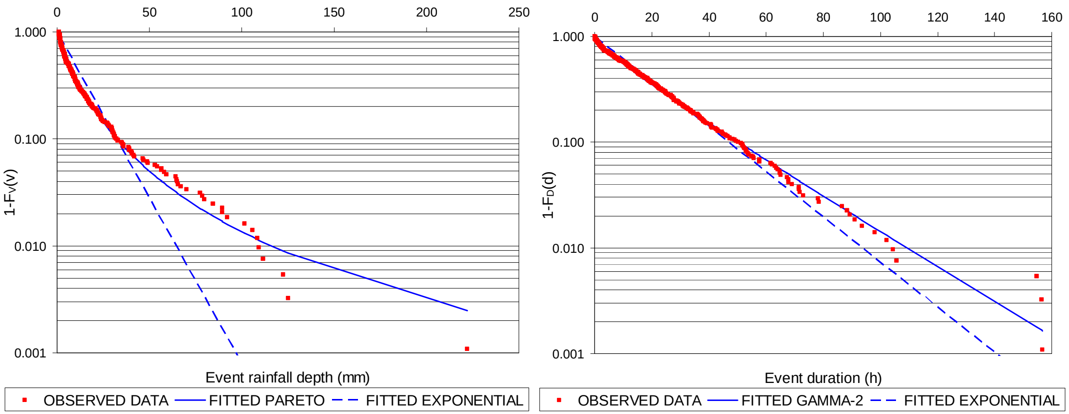

Fig. 6. Exponential and alternative probability distributions for rainfall event depth and duration (Valencia raingauge).

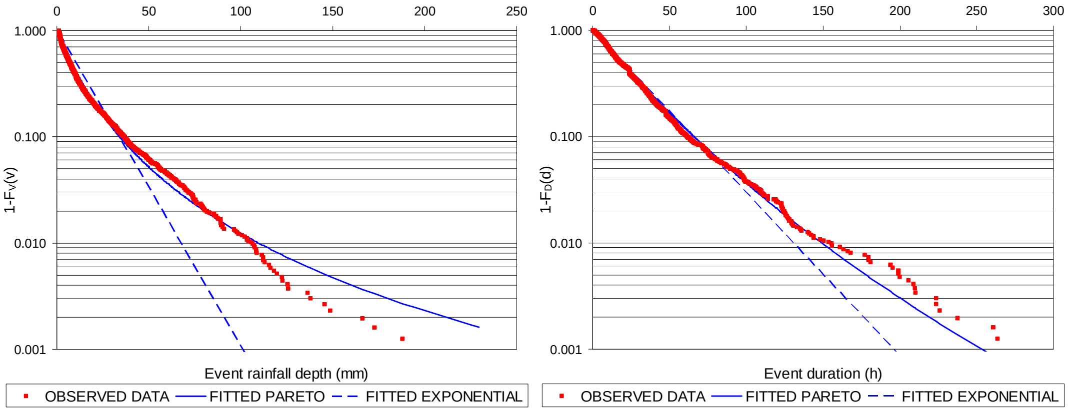

Fig. 7. Exponential and alternative probability distributions for rainfall event depth and duration (Santander raingauge).

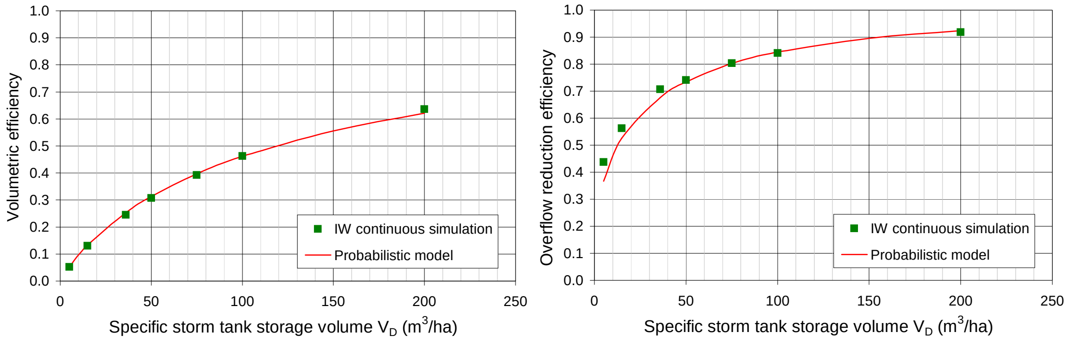

Fig. 8. Comparison between probabilistic model and continuous simulation results.