Efficiency of Storm Detention Tanks for Urban Drainage Systems under Climate Variability

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Abstract: Climate change effects on combined sewer systems efficiency is a great matter of concern. In fact, changes in rainfall regime could significantly affect combined sewer overflows (CSOs) into receiving water bodies. Given that CSOs are a significant source of pollution, a better understanding and modeling of climate change effects on urban drainage systems is a compelling requirement to support design of adaptation strategies. This paper aims at studying the resilience of storm water detention tanks efficiency with respect to changes in rainfall forcing. In detail, an analytical probabilistic model is proposed to assess overflow reduction efficiency and volumetric efficiency of detention tanks depending on behaviors of climate and urban catchment. Sensitivity of tank efficiencies is evaluated under assigned changes in rainfall forcing. Results show that resilience of storm tanks benefits from filtering of climate change effects operated by the urban catchment. **DOI: 10.1061/(ASCE)WR.1943-5452.0000144.** *© 2012 American Society of Civil Engineers.*

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Introduction

In many locations around the world, there is an increasing concern about the capability of existing water supply and sewer systems to adapt to environmental change. In fact, the preceding facilities were designed mostly on historical information. Changes in them could compromise the expected working conditions, therefore, inducing environmental problems. In particular, reasons for concern are mainly related to changes in water demand, land-use changes, and climate change (Willey and Palmer 2008). This paper focuses on the effects induced by the latter on combined sewer systems (CSSs). There is not a unanimous consensus among scientists about priorities that should be assigned to the previously mentioned environmental changes. Many scientists are convinced that climate change does not affect water supply and sewer systems as, for instance, pressures related to increasing population and water requirements (for a general discussion, see Koutsoyiannis et al. 2009). Moreover, part of the scientific community is convinced that climate change is the result of long-term climatic variability and, therefore, expected effects on the long range could be less concerning with respect to other irreversible environmental changes (for instance, Bloschl and Montanari 2010; Cohn and Lins 2005).

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However, despite these doubts, potential changes in climatic conditions are a great matter of concern for the efficiency of CSSs (Mailhot and Duchesne 2010). High-intensity rainfall events may exceed their capacity, resulting in the discharge of untreated storm water and wastewater directly into receiving water bodies. These combined sewer overflow (CSO) events can result in high concentrations of microbial pathogens, biochemical oxygen demand, suspended solids, and other pollutants in receiving waters.

Recent changes in rainfall regime have been discussed by many writers. Consistent trends in mean annual precipitation have been observed, e.g., an upward trend in the north and downward trend in southern Europe. However, for shorter time scales, the evidence is quite conflicting. Alexander et al. (2006) found a slightly increasing trend of extreme precipitation contributions to annual precipitation (0.41% per decade during 1979–2003) using a global data set, but other studies found increasing and decreasing trends depending on the region (Trenberth et al. 2007). According to the U.S. Environmental Protection Agency (USEPA), climate change is expected to increase the proportion of rainfall occurring in high-intensity events, resulting in increased storm water runoff and CSS overflows (USEPA 2008). The same concern is shared by the European Environment Agency (EEA 2007). The Spanish Ministry of Environment together with the Spanish Meteorological Agency (AEMET) published in 2009 a technical report on generation of regional scenarios for climate change in Spain (Brunet et al. 2009). A relevant uncertainty is affecting precipitation projections, although a trend is reported of annual rainfall reduction in southern part of the Iberian Peninsula with a south-north direction increasing gradient. For the region surrounding the city of Valencia whose urban drainage system case study will be presented in this paper worst predictions foresee a decrease in annual rainfall amounts of 20–30% in the next century, with a contemporary increase of torrential episodes.

Moreover, Article 5 of the European Water Framework Directive (establishing a framework for European community action in the field of water policy) acknowledges that emissions from CSO and separate systems are a significant source of river pollution,

e.g., heavy metals (BMU 2006). Therefore, the scientific community is stimulated to better investigate the effects of climate change on the efficiency of urban drainage systems (for instance, Ashley et al. 2005; Morgan et al. 2004).

This paper aims at studying the resilience of storm water detention tanks, located along CSSs, with respect to changes in event rainfall statistics. Storm water detention tanks are widely used for mitigating impacts of CSOs into receiving water bodies. Even if a lot of methodologies for sizing these facilities have been developed in the last decades and despite the previously mentioned growing call for adaptive management approaches, there is still limited literature on the effects of climate change on storm detention tank performances. Moreover, research which illustrates how adaptation processes can be initiated, facilitated, and influenced is still limited (Klein et al. 2005; Smit and Wandel 2006).

Our approach is driven by calls for development of infrastructure (e.g., sanitation facilities, wastewater treatment systems) designed to reduce vulnerability to climate variability and to enhance adaptive capacity (WHO 2003). Framework of the analysis is on the basis of deriving analytically the probability distribution of number and volume of tank overflows depending on rainfall descriptors, like average value, variability, and asymmetry of event rainfall. Then, we assess the sensitivity of tank performances by perturbating the rainfall descriptors according to hypothetical changes in climate. The results allow us to assess the resilience of sewer tanks and to study adaptation policies.

The next section describes meteorological data and the case study in which results are applied and discussed. The third section deals in detail with the theoretical framework of the analysis. The last section presents our results and concluding remarks are drawn.

Meteorological Data and Case Study

Analytical results are applied to a case study in the city of Valencia, Spain. High-resolution rainfall data over an extended period are needed to reliably assess statistical properties of meteorological forcing and suitability of different stochastic models for rainfall occurrence. Thus, the 1990–2006 observed series at Valencia, with a 5-min resolution, is used. To further confirm reliability of the data, rainfall observations were aggregated into monthly and annual totals and then compared with those obtained at nearby rain gauge stations.

The analytical model for storm tank performance assessment developed in this paper is applied to size and verify long-term efficiencies of a detention tank located in an urban catchment in Valencia, in which environmental impacts of CSOs to receiving water bodies (Valencia beaches and 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.

In detail, we refer to the Pío XII urban catchment that is located at the headwaters of an important trunk sewer of the city which frequently overflows into the previously mentioned docks. The catchment area A is 68.8 ha and length of its sewer network is 13.4 km with 565 manholes, i.e., 1 manhole each 23.7 m on the average. Its time of concentration (t_C) is about 0.5 h. Network topology and geometry, as well as historical hydraulic data, were provided by the Municipality of Valencia. Land-use distribution, which is needed to estimate infiltration parameters, was obtained from data provided by the urban master plan and classified according to local guidelines for sewer system design (Municipality of Valencia 2004) which considers four different land uses: 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 the ratio of the area, A_i, of land use i in the catchment over total catchment tributary area, A. In addition, each land use is characterized by an infiltration parameter, I_{ai}, (mm) representing its runoff threshold, i.e., the amount of rainfall needed for runoff to begin. Table 1 summarizes the catchment performance.

Table 1. Pío XII Urban Catchment Infiltration Parameters

i	Area description	A_i (ha)	a_i	I_{ai} (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	0.117	70.2

Framework of the Analysis

Rainfall Model

Rainfall characterization is one of the most important issues for designing urban drainage infrastructures. In fact, most hydraulic and hydrological designs are related to the determination of a design hyetograph. It is usually derived from intensity-duration-frequency curves for rainfall describing a critical event, in terms of rainfall volume, for a given return period. However, facilities like storm detention tanks need also to be tested with respect to frequent, low magnitude storms, which are small in volume but, nevertheless, induce relevant environmental problems. For instance, at Valencia nearly 70% of rainfall episodes are less than 5 mm in depth (Perales Momparler and Andrés-Doménech 2008) but convey the highest pollution concentrations in runoff. Therefore, it is important to reduce as much as possible polluted spills from these events into receiving water bodies. Besides, storm duration and interevent time between episodes are also important as they are related to runoff formation as well as wash off and transport of pollutants. Therefore, a continuous time description of rainfall is needed which can be obtained by means of stochastic processes.

Stochastic modeling of rainfall time series has been an up-to-date topic for many years, since first complete developments (Kavvas and Delleur 1981; Rodriguez-Iturbe et al. 1984, 1987). Among different approaches available, cluster-based point processes models, usually with rectangular pulses, have shown very good accuracy to model the rainfall process over a wide time scale range with relatively small number of parameters (Entekhabi et al. 1989; Islam et al. 1990). Bartlett-Lewis and Neyman-Scott models are probably the most known cluster models, in which rainfall events are described by means of clusters of cells, each of them representing a storm pulse with random duration and constant intensity (Rodriguez-Iturbe et al. 1987; Velghe et al. 1994; Calenda and Napolitano 1999).

To analyze long-term performance of storm tanks, a common approach is to apply stochastic models to simulate synthetic rainfall records that are subsequently routed through an urban rainfall-runoff model and then a hydraulic simulation model of the sewer system. Another widely considered approach is on the basis of derived probability distribution theory allowing to analytically estimate storm tank performances (Di Toro and Small 1979; Driscoll et al. 1986; Walker et al. 1993; Papa and Adams 1997; Adams and Papa 2000). By introducing appropriate simplifications, the latter approach leads to closed-form solutions that permit rapid

and efficient inspection of different design alternatives (Adams and Papa 2000).

In this paper, a point process model is used to obtain a stochastic description of rainfall intensity, and a derived distribution approach is used to analytically obtain probabilistic expressions for storm tank efficiency. Rainfall events are represented as rectangular pulses of duration $b(t)$ and depth $v(t)$, driven by a Poisson process which describes the interevent time between pulses, $s(t)$. It follows that interevent time is exponentially distributed (Cox and Isham 1980). It is also assumed that event rainfall depth $v(t)$ and duration $b(t)$ are outcomes from two different and independent stochastic processes. Therefore, rainfall descriptors are $s(t)$, $v(t)$, and $b(t)$ which are supposed to be outcomes from stochastic processes S, V, and B, respectively. Prior to inferring the statistical properties of rainfall descriptors, identification of statistically independent storms by selecting a critical value, s_{crit}, for the interevent time $s(t)$ is needed, so that events separated by a dry period greater than s_{crit} can be considered to be independent (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 selected so that the hypothesis that $s(t)$ values greater than s_{crit} are outcomes from the exponentially distributed stochastic process S cannot be rejected (Bonta and Rao 1988). The original methodology as developed by Restrepo-Posada and Eagleson (1982) selects s_{crit} by considering that the coefficient of variation (CV) of a Poisson process should be equal to unity. In the operational practice, for a trial value of s_{crit}, statistical tests can be applied not to reject the hypothesis that $CV = 1$ for an assigned confidence level. This approach does not take into account that S is bounded from below by s_{crit}.

Therefore, a modified statistical criterion for the selection of s_{crit} is adopted (Andrés-Doménech et al. 2010). In fact, for each realization of S resulting from the corresponding trial s_{crit} value, a lower bounded exponential distribution given by

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

can be fitted. Once β is estimated by using maximum likelihood, the Kolmogorov-Smirnov goodness of fit test is applied to check that (1) provides a good fit for S (for more details see Andrés-Doménech et al. 2010).

After independent storms are identified, $v(t)$ and $b(t)$ values are obtained and mutual independence of S, V, and B can be checked. In fact, independent events are characterized by lack of time correlation for each of the stochastic processes S, V, and B, as well as the absence of cross correlation among them.

Different alternative probability distributions were considered to fit the frequency of occurrence of stochastic processes V and B. In detail, we focused on exponential distribution, which is traditionally chosen in many studies, as well as Weibull, Gamma-two, lognormal, and generalized Pareto distributions.

For event duration $b(t)$, Gamma-two and exponential models adequately fit the rainfall data at Valencia. For the sake of parsimony, the exponential model is chosen:

$$f_B(b) = \lambda e^{-\lambda b} \qquad b \geq 0 \tag{2}$$

Maximum likelihood estimate for λ is the inverse value of sample mean, i.e., $[E(B)]^{-1}$.

Pareto distribution is the most appropriate model for $v(t)$ in the case study considered in this paper. Andrés-Doménech et al. (2010) also reported good accuracy of this model at some other locations in Spain. 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 CV, should lead to the maximum entropy (Koutsoyiannis 2005). Density function of the generalized Pareto model is given by

$$f_V(v) = \frac{1}{\alpha}\left(1 + \kappa\frac{v}{\alpha}\right)^{-1/\kappa - 1} \qquad v \geq 0 \tag{3}$$

where $\kappa > 0$ and $\alpha > 0$ = shape and scale parameters, respectively. Estimation of these parameters is not a trivial issue (Singh and Guo 1995; Askhar and Ouarda 1996; de Zea Bermudez and Amaral Turkman 2003; Öztekin 2005; Zhang 2007). If sample size is large enough (Valencia series has 464 events) and $-0.5 < \kappa < 0.5$ (this is also true for the rainfall series observed in Valencia), maximum likelihood offers satisfactory performances (Castillo and Hadi 1997). Mean μ_V, variance σ_V^2, skewness coefficient γ_V, and variation coefficient CV_V of the distribution are given by

$$\mu_V = \frac{\alpha}{1 - \kappa} \Leftrightarrow \kappa < 1 \tag{4}$$

$$\sigma_V^2 = \frac{\alpha^2}{(1-\kappa)^2(1-2\kappa)} \Leftrightarrow \kappa < \frac{1}{2} \tag{5}$$

$$\gamma_V = \frac{2(1+\kappa)(1-2\kappa)^{1/2}}{1 - 3\kappa} \Leftrightarrow \kappa < \frac{1}{3} \tag{6}$$

$$CV_V = \frac{1}{(1-2\kappa)^{1/2}} \Leftrightarrow \kappa < \frac{1}{2} \tag{7}$$

Rainfall-Runoff Model

The Soil Conservation Service Curve Number (SCS-CN) model was adopted to represent rainfall-runoff transformation which was previously calibrated and validated for the urban area of Valencia (González 2001). Accordingly, runoff volume, $r(v)$, is related to rainfall event volume, v, by the following relationship:

$$\begin{cases} r(v) = 0 & \text{if } v \leq I_a \\ r(v) = \frac{(v - I_a)^2}{v + 4I_a} & \text{if } v > I_a \end{cases} \tag{8}$$

Eq. (8) implies that no runoff would occur if event rainfall depth was smaller than threshold value I_a. Thus, if V is distributed according to generalized Pareto probability distribution, the cumulative probability of null runoff is given by

$$F_R(0) = F_V(I_a) = \int_0^{I_a} f_V(v)dv = 1 - (1 + \kappa I_a/\alpha)^{-1/\kappa} \tag{9}$$

where R = stochastic process whose outcome is the event runoff $r(t)$. Conversely, when threshold value I_a is exceeded, then $R > 0$, and the cumulative probability distribution of runoff volume is

$$\begin{aligned} F_R(r) &= \int_0^r f_R(r)dr = F_R(0) + \int_{I_a}^{v} f_V(v)dv \\ &= 1 - (1 + \kappa v/\alpha)^{-1/\kappa} \end{aligned} \tag{10}$$

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

$$f_R(r) = \frac{d}{dr}F_R(r) = \frac{1}{\alpha}\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}\frac{dv}{dr} = \frac{\frac{1}{\alpha}\left(1 + \kappa\frac{v}{\alpha}\right)^{-1-1/\kappa}}{\frac{d}{dv}r(v)} \tag{11}$$

Given that total area A of the urban catchment was divided into four different types of land use, each one is affected by a different initial abstraction I_{ai} (Table 1), runoff volume generated by each rainfall event can be written as

$$r(v) = \sum_{i=1}^{4} a_i r^{(i)}(v) \tag{12}$$

where a_i = land use area ratio previously defined and reported in Table 1 and $r^{(i)}(v)$ = runoff generated in area A_i computed by applying Eq. (8). Therefore, Eq. (11) can be rewritten as

$$f_R(r) = \frac{d}{dr}F_R(r) = \frac{1}{\alpha}\left(1+\kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} \cdot \frac{dv}{dr} = \frac{\frac{1}{\alpha}(1+\kappa\frac{v}{\alpha})^{-1-1/\kappa}}{\sum_{i=1}^{4} a_i \frac{d}{dv} r^{(i)}(v)} \tag{13}$$

According to the minimum initial abstraction I_{a1} corresponding to paved areas, discrete probability for $r = 0$ is

$$F_R(0) = F_V(I_{a1}) = \int_{v=0}^{I_{a1}} f_V(v)dv = 1 - \left(1+\kappa\frac{I_{a1}}{\alpha}\right)^{-1/\kappa} \tag{14}$$

Expected value of event runoff volume from the catchment takes the form

$$E(R) = \int_0^\infty r f_R(r)dr = \frac{1}{\alpha}\int_0^\infty \left(1+\kappa\frac{v}{\alpha}\right)^{-1-1/\kappa} \sum_{i=1}^{4} a_i r^{(i)}(v)dv \tag{15}$$

Storm Tank Model

The purpose of the tank model is to provide an analytical relationship for the number and volume of overflows from a CSO system controlled by a tank with a specific volume, V_D. We indicate with the symbol Q_V the maximum flow rate during the event from the tank to wastewater treatment plant (WWTP).

First, probability density function of the overflow volume must be previously derived. Assuming that runoff occurs as a rectangular pulse, then overflow volume for a given event is

$$\begin{cases} w = 0 & \text{if } r(v) \le V_D + Q_V \cdot [b(t)+t_C] \\ w = r(v) - V_D - Q_V \cdot [b(t)+t_C] & \text{if } r(v) > V_D + Q_V \cdot [b(t)+t_C] \end{cases} \tag{16}$$

where t_C = time of concentration of the catchment. Therefore, the probability of no overflow is given by

$$F_W(0) = F_R\{V_D + Q_V \cdot [b(t)+t_C]\} \tag{17}$$

and runoff volume $r(t)$ is determined by event rainfall depth only, which is assumed to be distributed according to a generalized Pareto distribution. Since $w = 0$ is equivalent to $r(v) \le V_D + Q_V \cdot [b(t)+t_C]$, with $b > 0$, it follows that:

$$F_W(0) = F_R(0) + \int_{b=0}^\infty \int_{r=0}^{V_D+Q_V(b+t_C)} f_R(r) f_B(b) dr db \tag{18}$$

By plugging in Eqs. (2), (13), and (14), the probability of no overflow is given by

$$F_W(0) = 1 - \left(1+\kappa\frac{I_{a1}}{\alpha}\right)^{-1/\kappa} + \int_{b=0}^\infty \int_{v=I_{a1}}^{v^*_{DQ}} \frac{1}{\alpha}\left(1+\kappa\frac{v}{\alpha}\right)^{-1/\kappa-1} \lambda e^{-\lambda b} dv db \tag{19}$$

where v^*_{DQ} = rainfall depth generating a runoff volume equal to the tank volume plus the volume conveyed to the WWTP during the runoff event, that is, $r(v^*_{DQ}) = V_D + Q_V(b+t_C)$. Thus if $v(t) \le v^*_{DQ}$, then there is no overflow ($w = 0$). In this case, by rearranging Eq. (19) we obtain

$$F_W(0) = 1 - \lambda \int_{b=0}^\infty e^{-\lambda b}\left(1+\kappa\frac{v^*_{DQ}(b)}{\alpha}\right)^{-1/\kappa} db \tag{20}$$

Furthermore, if $v > v^*_{DQ}$, then $w > 0$ and, therefore

$$F_W(w) = F_R\{w + V_D + Q_V \cdot [b(t)+t_C]\} = F_R(w + v^*_{DQ}) \tag{21}$$

and

$$F_W(w) = 1 - \frac{\lambda}{\alpha}\int_{b=0}^\infty \int_{v=v^*(b)}^\infty \left(1+\kappa\frac{v}{\alpha}\right)^{-1/\kappa-1} e^{-\lambda b} dv db \tag{22}$$

where $v^*(b)$ = rainfall depth generating a runoff volume $r[v^*(b)] = w + V_D + Q_V(b+t_C)$.

By rearranging Eq. (22)

$$F_W(w) = 1 - \lambda \int_{b=0}^\infty e^{-\lambda b}\left(1+\kappa\frac{v^*(b)}{\alpha}\right)^{-1/\kappa} db \tag{23}$$

which allows to estimate the expected value of spilled volume per event as

$$E(W) = \int_0^\infty w f_w(w) dw \tag{24}$$

From Eqs. (15) and (24) volumetric efficiency of the tank, $\varphi(V_D, Q_V)$, can be derived

$$\varphi(V_D, Q_V) = 1 - \frac{E(W)}{E(R)} \tag{25}$$

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

Overflow reduction efficiency, $\eta(V_D, Q_V)$, can be derived from Eq. (20)

$$\eta(V_D, Q_V) = F_W(0) = 1 - \lambda \int_{b=0}^\infty e^{-\lambda b}\left(1+\kappa\frac{v^*_{DQ}(b)}{\alpha}\right)^{-1/\kappa} db \tag{26}$$

This is the probability of an event not to produce overflow depending on detention tank volume and flow rate derived to WWTP. Denoting with θ_V the average number of rainfall events per year deducted from rainfall analysis, the number of overflows per year, θ_{OF}, is given by

$$\theta_{OF}(V_D, Q_V) = \theta_V[1 - \eta(V_D, Q_V)] \tag{27}$$

Storm Tank Efficiency Resilience to Change in Rainfall Statistics

Resilience of storm detention tanks with respect to changes in climate is investigated by assessing the sensitivity of volumetric efficiency and overflow reduction efficiency to rainfall statistics. According to classical sensitivity analysis, suitable changes are imposed to rainfall statistics to investigate the corresponding changes in efficiencies. From now on, modified values of rainfall parameters and statistics will be denoted with an asterisk superscript. Reference values obtained from the rainfall analysis section will be denoted with a zero subscript.

A first sensitivity analysis (A1) is performed by keeping the CV, CV_V, of event rainfall volume unchanged. Both the κ parameter of the Pareto distribution and the skewness γ_V remain unchanged as well [see Eqs. (6) and (7)]. First, sensitivity of efficiencies to mean event rainfall depth, with unchanged event duration parameter λ, is assessed (Case A1.a). We quantify the change in mean event rainfall depth through a coefficient $C_{\mu V}$, so that

$$\mu_V^* = (1 + C_{\mu V}) \cdot \mu_{V0} \qquad C_{\mu V} \in (-0.3, 0.3) \tag{28}$$

A variation of $\pm 30\%$ is adopted in view of the previously mentioned predictions by AEMET (Brunet et al. 2009). Such a change is intended to be an upper limit of reasonable scenarios for the next century.

To preserve variation coefficient, the same change should be applied to standard deviation

$$\sigma_V^* = (1 + C_{\mu V}) \cdot \sigma_{V0} \tag{29}$$

Adjusted values for the α-parameter of the Pareto distribution can be derived through Eq. (4)

$$\alpha^* = \mu_V^*(1 - \kappa^*) = \mu_{V0}(1 + C_{\mu V})(1 - \kappa_0) \tag{30}$$

Further investigation can be carried out by inspecting the effects of combined changes in event volume and event duration (Case A1.b). Event duration is varied according to the relationship

$$\mu_B^* = (1 + C_{\mu B}) \cdot \mu_{B0} \qquad C_{\mu B} \in (-0.3, 0.3) \tag{31}$$

in which a variation of $\pm 30\%$ is also adopted for consistency with Case (A1.a). The variation of μ_B affects parameter λ to be readjusted as

$$\lambda^* = \mu_B^{*-1} \tag{32}$$

Pareto distribution parameters are then derived again by preserving CV_V.

A second sensitivity analysis (Case A2) involves changes in CV, CV_V, of event rainfall depth and, therefore, in the κ parameter and skewness γ_V of the Pareto distribution. Therefore by varying CV_V, sensitivity to skewness can also be assessed. Skewness is defined for the Pareto distribution if $\kappa < 1/3$, whereas CV is defined for $\kappa < 1/2$ [see Eqs. (6) and (7)]. Maximum likelihood estimate of κ for Valencia rainfall series is $\kappa_0 = 0.411$ and, therefore, it is advisable to analyze the sensitivity to CV and not directly to skewness in this situation. Range of variation of CV_V is set from 1 (as if event rainfall was exponentially distributed) to 10 (corresponding to increasing left skewness). The preceding upper limit is deemed to be representative of an upper limit of reasonable climate change conditions. With modified CV value, CV_V^*, the modified κ^* parameter for the Pareto distribution is set as

$$\kappa^* = \frac{1}{2}\left(1 - \frac{1}{CV_V^{*2}}\right) \tag{33}$$

No changes in rainfall event volume are considered in Case A2 and, therefore, adjusted values for α^* are obtained as follows:

$$\alpha^* = \mu_{V0}(1 - \kappa^*) \tag{34}$$

For all previously described cases, once the modified Pareto parameters are known tank efficiencies can be derived through Eqs. (25) and (26).

Results and Discussion

Rainfall-Runoff-Tank Modeling Results

Different trial values of s_{crit} were considered in the range between 5 min (the series resolution) and 48 h. A value of $s_{\text{crit}} = 22$ h, which corresponds to $\beta = 0.0059\ \text{h}^{-1}$, was finally selected, resulting in an average number of events per year $\theta_V = 27.3$. Event volume $v(t)$ is best described by the Pareto probability distribution, whereas the exponential model provides best fit for $b(t)$. Parameter values for the previous distributions are estimated by maximum likelihood and summarized in Table 2. For more details, refer to Andrés-Doménech et al. (2010).

Storm tank efficiency analysis was then carried out for the Pío XII urban drainage catchment in Valencia. According to catchment land use (Table 1) and estimated rainfall descriptors (Table 2), expected values for runoff volume, $E(R)$, and overflow volume, $E(W)$, were obtained for different trial values of tank volume V_D and flow derived to WWTP, Q_V. The previously expected values have been estimated by numerical integration of $F_R(r)$ and $F_W(w)$. Thus, the tank efficiencies $\varphi(V_D, Q_V)$ and $\eta(V_D, Q_V)$ were evaluated through Eqs. (25) and (26).

To validate results, continuous simulation was performed for the 17 y-long observation period 1990–2006. Rainfall record, including 464 independent rainfall events, was routed through a hydraulic simulation model of the sewer network built within the InfoWorks CS software. Simulations were, thus, performed for a set of seven tank volumes defined by specific volumes of 5, 10, 36, 50, 75, 100, and 200 m^3/ha. The value 36 m^3/ha was simulated because it is the specific volume required by the Municipality of Valencia. These specific storage volumes were combined with flow rates derived from WWTP from 0 to 3 l/s/ha.

For each simulation in which the tank is defined by the couple (V_D, Q_V), total volume spilled per event, w_j, was obtained. Depending on total runoff volume per event, r_j, volumetric efficiencies were evaluated by

$$\varphi_{\text{sim}} = 1 - \frac{\sum_{j=1}^{464} w_j}{\sum_{j=1}^{464} r_j} \tag{35}$$

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

$$\eta_{\text{sim}} = 1 - \frac{\sum_{j=1}^{464} \delta_j}{464} \tag{36}$$

where

$$\delta_j = \begin{cases} 1 & \text{if } w_j > 0 \\ 0 & \text{if } w_j = 0 \end{cases} \tag{37}$$

Fig. 1 summarizes volumetric and overflow reduction efficiencies obtained with both analytical and continuous simulation

Table 2. Distribution Functions and Related MLE Parameters for Rainfall Descriptors Related to Valencia Rain gauge

Rainfall descriptor	Cumulative probability function	MLE parameters
S	$F_S(s) = 1 - e^{-\beta(s - s_{\text{crit}})}$	$\beta_0 = 0.0059\ \text{h}^{-1}$
V	$F_V(v) = 1 - (1 + \kappa \frac{v}{\alpha})^{-1/\kappa}$	$\kappa_0 = 0.4110$
		$\alpha_0 = 8.4605$ mm
B	$F_B(b) = 1 - e^{-\lambda b}$	$\lambda_0 = 0.0492\ \text{h}^{-1}$

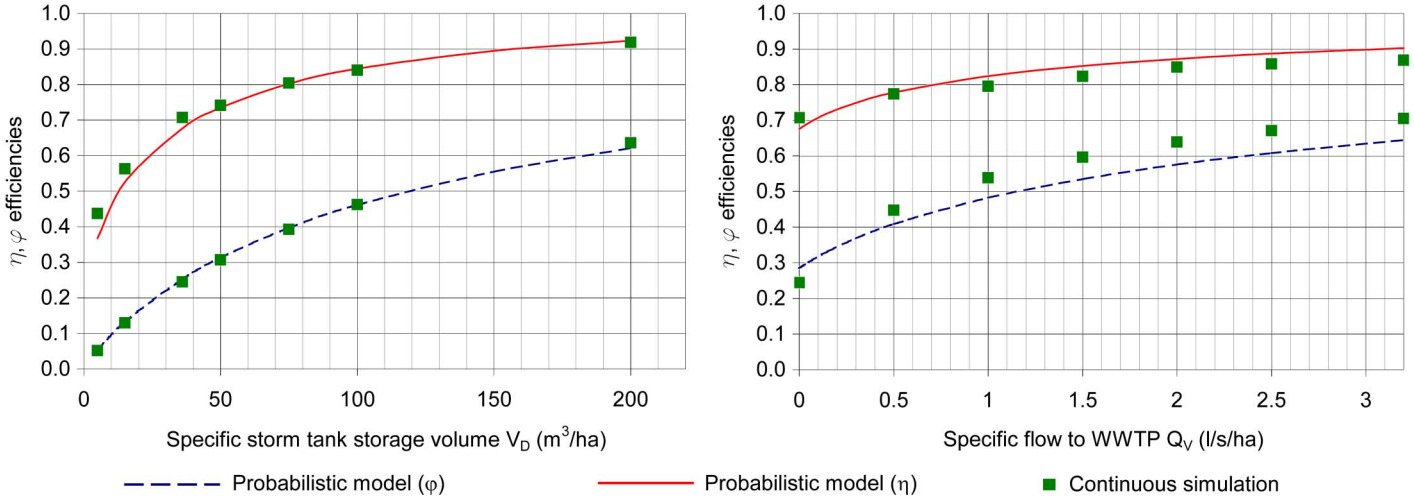

Fig. 1. Comparison between analytical model and continuous simulation for $Q_V = 0$ l/s/ha and specific tank volume ranging from 0 to 200 m^3/ha (a) and for $V_D = 36$ m^3/ha and Q_V ranging from 0 to 3 l/s/ha (b)

approaches. If there is no flow derived to WWTP (Fig. 1(a)), probabilistic model provided satisfactory results. Fig. 1(b) shows results when a positive Q_V is considered. In this case, results are also acceptable, although small discrepancy between analytical model and continuous simulation arises, because of simplified schematization of the rainfall process provided by the stochastic model. In fact, if a continuous emptying flow rate Q_V is allowed during an event, progress in time of the hyetograph becomes relevant to determining overflow spills and overflow volume. Actually by adopting a rectangular hyetograph, volumetric efficiency is underestimated. For the same reason, overflow reduction efficiency is slightly overestimated by the analytical model.

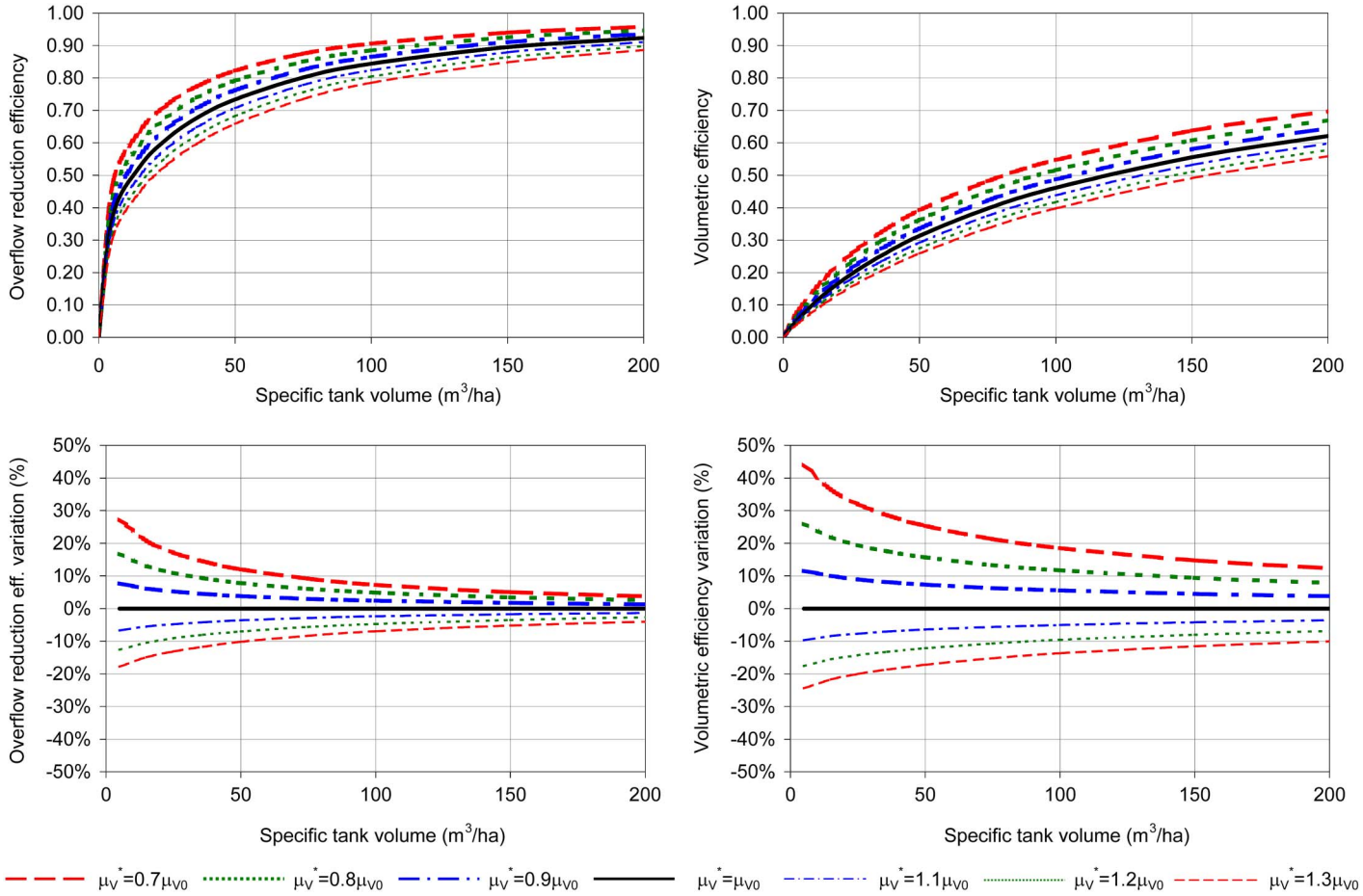

Fig. 2. Case A1.a; overflow reduction efficiency (η) and volumetric efficiency (φ) without flow derived to the WWTP during the event ($Q_V = 0$)

Storm Tank Resilience for Case (A1.a)

In Case A1.a, event rainfall depth is changed, whereas CV_V and event duration are unchanged. An immediate consequence is that, according to Eq. (7), $\kappa^* = \kappa_0 = 0.411$, in which κ_0 is the reference value estimated by maximum likelihood for Valencia rainfall series.

Fig. 2 shows volumetric and overflow reduction efficiencies for the simplest tank configuration, i.e., with $Q_V = 0$ l/s/ha. Besides η and φ efficiency plots, variations with respect to the reference situation ($\mu_V^* = \mu_{V0}$) are evaluated. As expected, a decrease in mean event rainfall depth induces an increase in tank efficiencies and vice versa. It is interesting to analyze the magnitude of efficiencies variation and its dependence on specific tank volume. Absolute variations of both efficiencies decrease rapidly as specific tank volume increase, since larger volume provides higher resilience. In fact, for tanks larger than 50 m^3/ha, 30% increase in mean event rainfall results in less than 10% reduction of the overflow efficiency. This result underlines that for tank size ranging from 50 to 100 m^3/ha, variations in overflow reduction efficiency are limited and will produce negligible changes on the number of overflows per year. Volumetric efficiency performance is similar but with higher relative variations.

Fig. 3 refers to the case when an emptying flow rate to the WWTP $Q_V = 1$ l/s/ha is considered. As a matter of fact, taking flow rate to WWTP into consideration increases the effective runoff volume that can be detained and/or treated per event. As a result, efficiencies are considerably higher; hence, relative variations are much more limited. As a result, overflow efficiency variations are always within the $\pm 10\%$ range for the entire analyzed tank volume range, whereas volumetric efficiency variations do not exceed $\pm 20\%$.

Storm Tank Resilience for Case (A1.b)

Combined variations of mean rainfall event volume and duration are now considered with constant CV CV_V. Resilience of overflow reduction efficiency in storm tanks with combined specific storage volumes of 50 and 100 m^3/ha and emptying flow rates of 1 and 2 l/s/ha are analyzed in Fig. 4. There is a reduction of η when μ_B decreases and μ_V increases. Such combined variations in rainfall regime may generate more intense rectangular rainfall pulses, increasing risk to produce overflow. Overflow reduction efficiency is much more sensitive to changes in μ_V than μ_B, proving that event rainfall depth is a much more influential factor.

Table 3 shows extreme variations achieved with this analysis, which correspond to combinations of increased rainfall depth and decreased duration (higher pulse intensity) and vice versa. In Case (A1.a), the higher the detention volume the lower the variations in η, since the system is more resilient.

Storm Tank Resilience for Case (A2)

The resilience analysis is completed introducing variations in the CV of event rainfall depth. Fig. 5 shows absolute values of efficiencies and their variations with respect to the reference situation (μ_{V0}, σ_{V0}, CV_{V0}) for a set of CV CV_V^*, without any change in any other rainfall statistic. Overflow reduction and volumetric efficiencies progress in opposite ways when the same CV_V variation is applied:

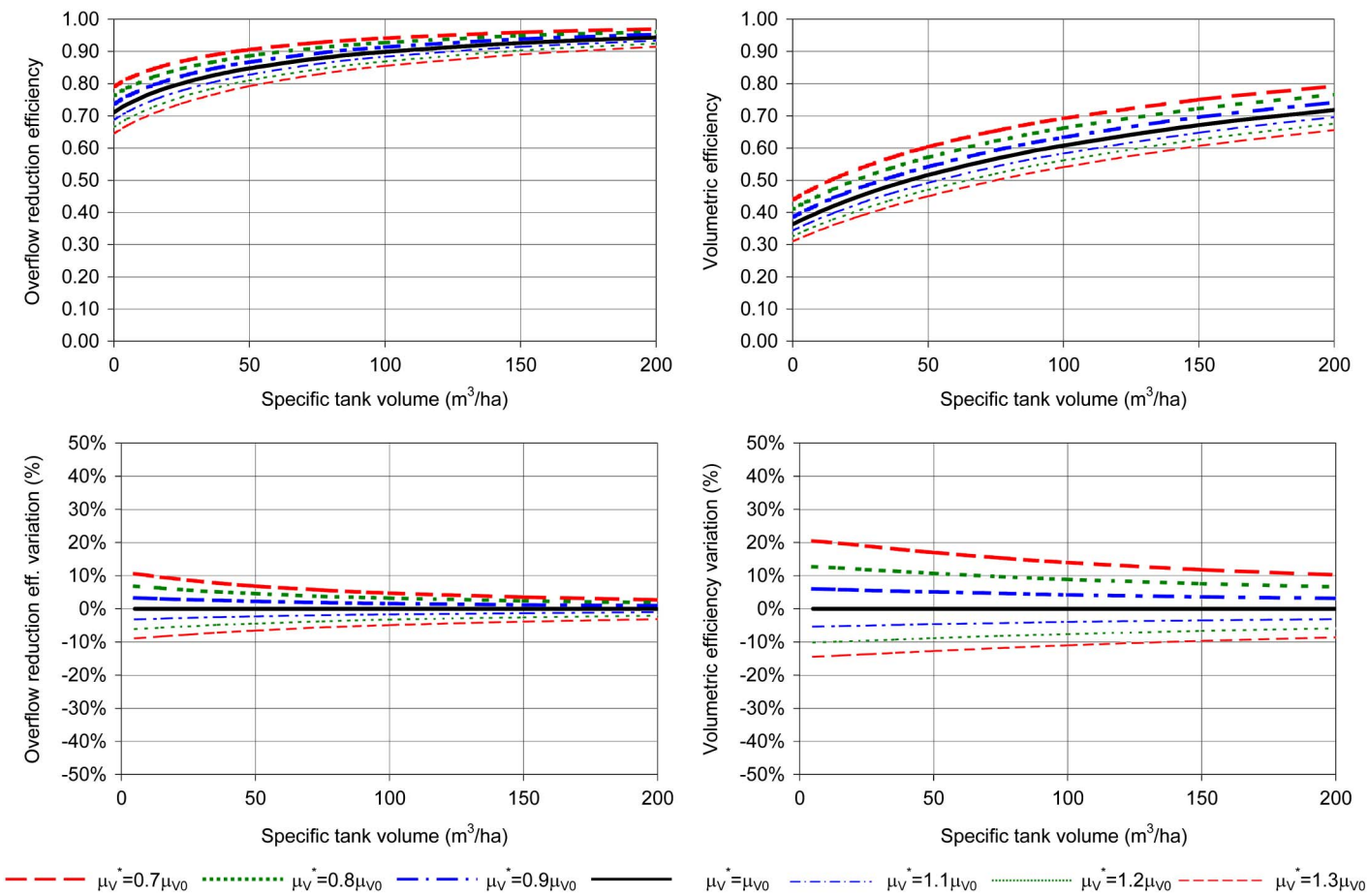

Fig. 3. Analysis A1.a; overflow reduction efficiency (η) and volumetric efficiency (φ) with flow derived to the WWTP during the event $Q_V = 1$ l/s/ha

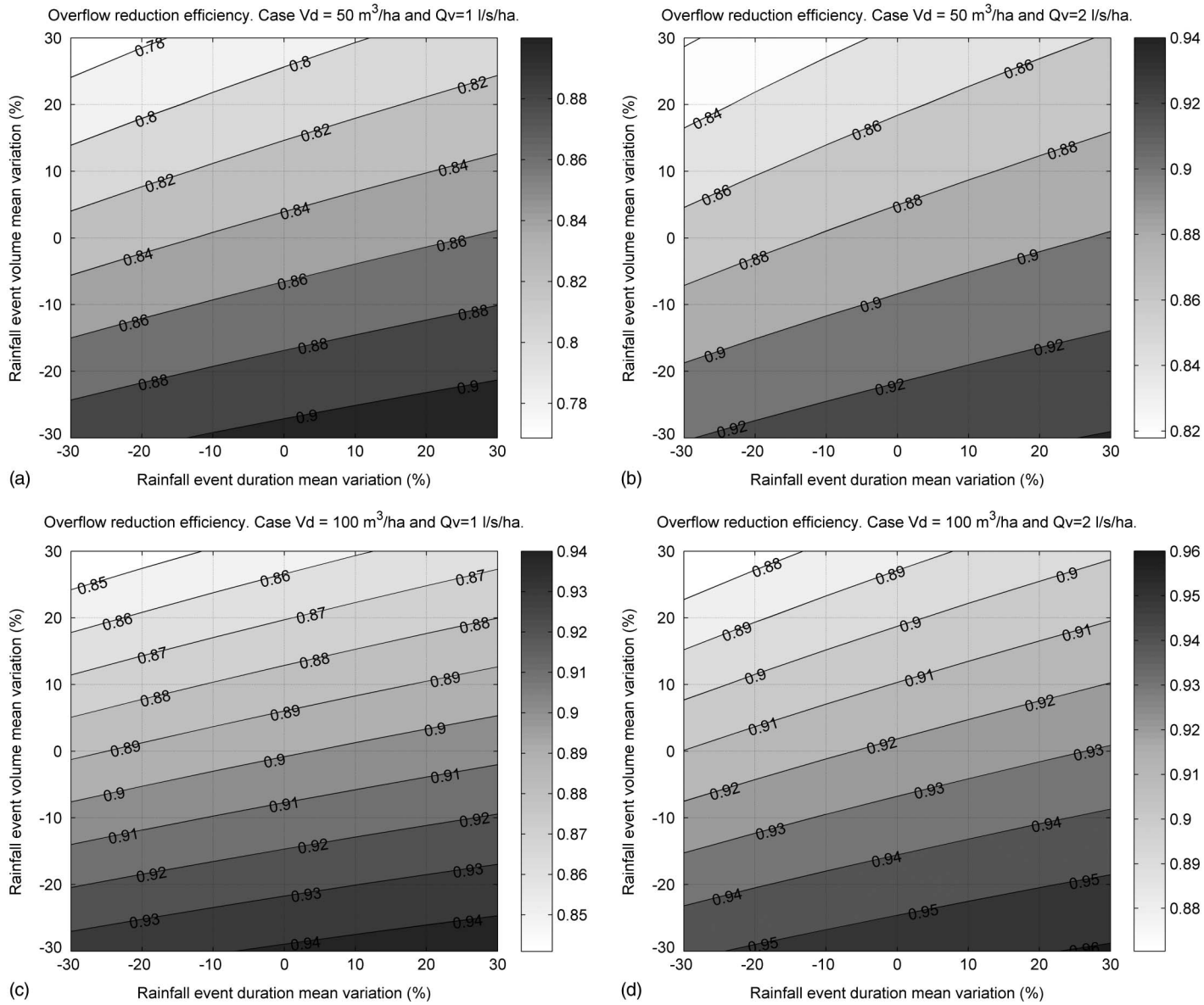

Fig. 4. Analysis A1.b; overflow reduction efficiency (η) for combinations of specific tank volume of 50 and 100 m^3/ha, flow derived to the WWTP during the event (Q_V) of 1 and 2 $l/s/ha$ and simultaneous variations of $\pm 30\%$ in average event rainfall volume and duration

Table 3. Case A1.b; Overflow Reduction Efficiency (Reference Values, Modified Values, and Variation) Depending on Selected Combination of Storm Tank Design Parameters

Storm tank parameters	Reference efficiency η_0	Rainfall variations	Modified efficiency η^*	Variation $\Delta\eta(\%)$
$V_D = 50\ m^3/ha\ Q_V = 1\ l/s/ha$	0.847	$\mu_V^* = 0.7\mu_{V0} - \mu_B^* = 1.3\mu_{B0}$	0.768	−9.3
	0.847	$\mu_V^* = 1.3\mu_{V0} - \mu_B^* = 0.7\mu_{B0}$	0.916	+8.1
$V_D = 50\ m^3/ha\ Q_V = 2\ l/s/ha$	0.887	$\mu_V^* = 0.7\mu_{V0} - \mu_B^* = 1.3\mu_{B0}$	0.818	−7.8
	0.887	$\mu_V^* = 1.3\mu_{V0} - \mu_B^* = 0.7\mu_{B0}$	0.941	+6.1
$V_D = 100\ m^3/ha\ Q_V = 1\ l/s/ha$	0.898	$\mu_V^* = 0.7\mu_{V0} - \mu_B^* = 1.3\mu_{B0}$	0.841	−5.7
	0.898	$\mu_V^* = 1.3\mu_{V0} - \mu_B^* = 0.7\mu_{B0}$	0.947	+5.5
$V_D = 100\ m^3/ha\ Q_V = 2\ l/s/ha$	0.922	$\mu_V^* = 0.7\mu_{V0} - \mu_B^* = 1.3\mu_{B0}$	0.870	−5.6
	0.922	$\mu_V^* = 1.3\mu_{V0} - \mu_B^* = 0.7\mu_{B0}$	0.961	+4.2

an increase in CV_V implies that overflow reduction efficiency increases and volumetric efficiency decreases. This fact could be explained by exploring the Pareto probability density function shape when changes in CV_V are introduced without changing the mean. If volume depth tends to be exponentially distributed ($CV_V^* = 1$), probability density decreases severely at the origin whereas slightly increasing for the medium-depth range. On the opposite, an increase in CV_V skews the density function toward the origin. Therefore, in this latter case the density of small and very small events increases rapidly, thus, inducing an increase

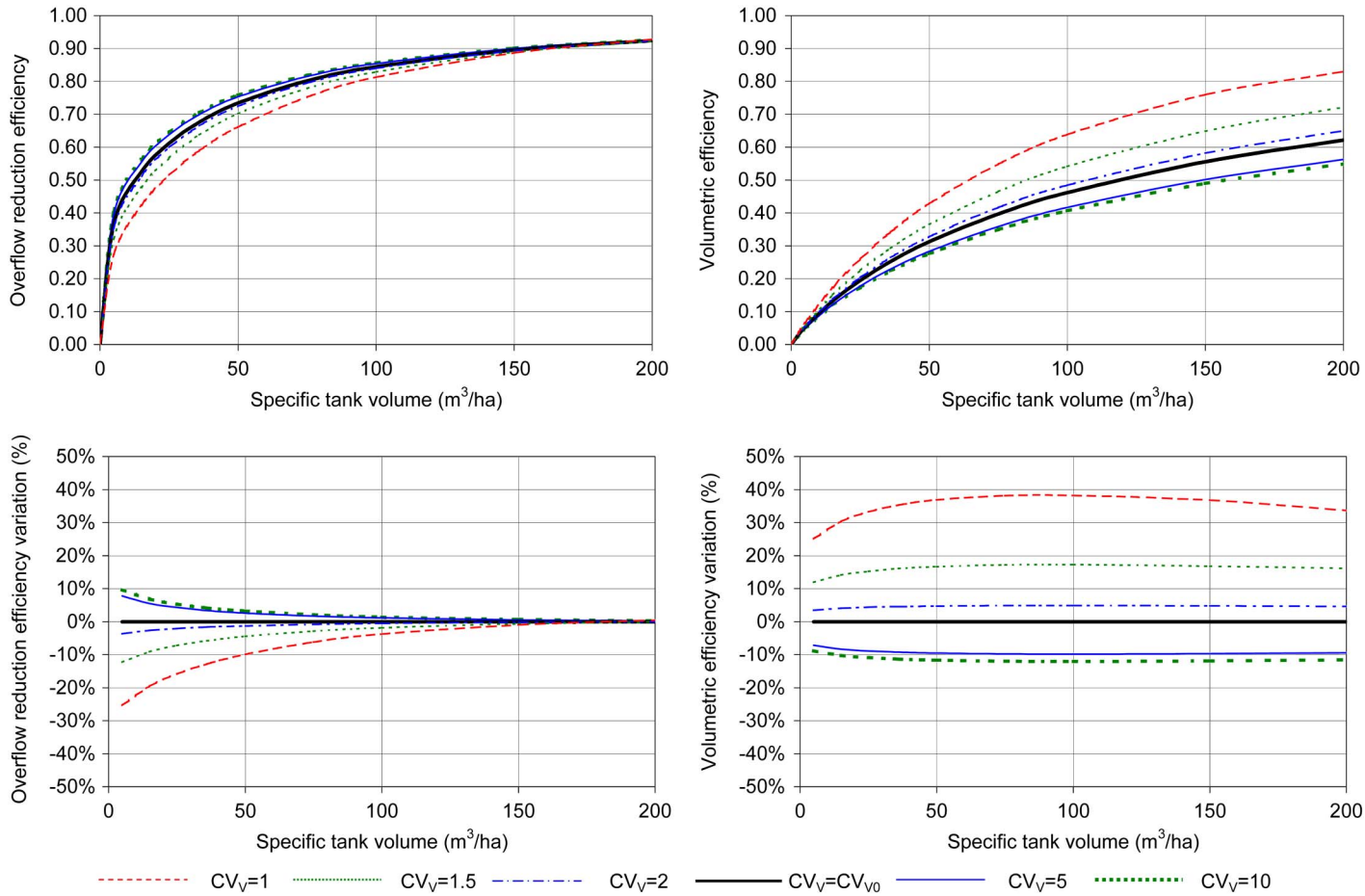

Fig. 5. Case A2; overflow reduction efficiency (η) and volumetric efficiency (φ) without flow derived to the WWTP during the event ($Q_V = 0$) and without changes in average event rainfall depth

of overflow reduction efficiency. Variations in η are never exceeding $\pm 10\%$, except when CV_V tends to unity, in which a maximum drop in η is obtained (nearly -20% for small specific tank volumes).

Evolution of volumetric efficiency is not as intuitive. The first consequence of changes in CV_V, even without changes in μ_V, is variation on runoff event expected values. As shown in Fig. 6,

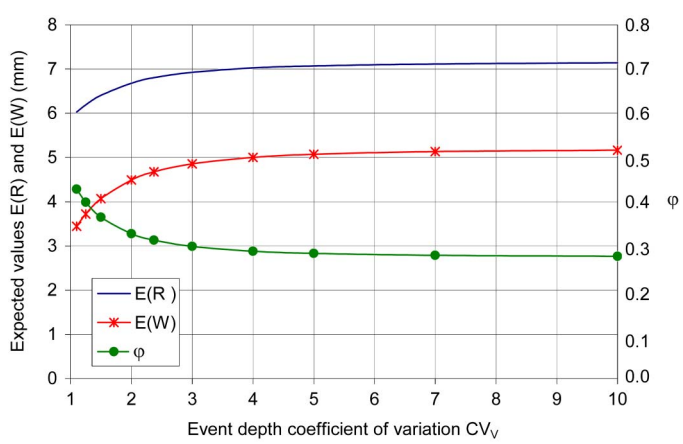

Fig. 6. Case A2; expected runoff and overflow values, and volumetric efficiency for changing coefficients of variation CV_V of event rainfall depth, in a tank with $V_D = 50\ \mathrm{m^3/ha}$ with null flow rate

expected value $E(R)$ increases with increasing CV as density of small magnitude rainfall events increases (population is more skewed toward the origin) and, therefore, remaining events will have a higher magnitude (as rainfall event mean is preserved). In the same way, expected values of spilled volumes $E(W)$ also increase when CV_V does, but much faster, with a consequently significant decrease in volumetric efficiency. Fig. 6 shows these results for a specific tank storage volume of $50\ \mathrm{m^3/ha}$. As the event rainfall depth is preserved, variations of volumetric efficiency do not change significantly when the specific storage volume is large.

Also, importantly, decreases in volumetric efficiency achieved with increases in CV_V are limited to -10%, whereas a decreasing CV_V induces more significant changes, up to $+40\%$ when event rainfall depth is exponentially distributed. This result highlights the importance of achieving good fit for rainfall descriptors. The use of an exponential distribution could lead to significant overestimation of volumetric efficiency.

By summarizing the previous results, changes in storm tank efficiencies are less relevant than changes in the considered rainfall statistics. Keeping overflow reduction efficiency unchanged implies relevant costs. In fact, if this target for Case (A1.a) is to be maintained assuming a storm tank with $V_D = 50\ \mathrm{m^3/ha}$ and $Q_V = 1\ \mathrm{l/s/ha}$, a 70% increase in storage volume or a 95% increase in emptying flow rate would be needed (see Fig. 3). Therefore, adaptation measures should consider alternative approaches, which may turn out to be less expensive.

Conclusions

Climate variability and its consequences on water systems are a topical issue. It is well known that changes in climatic conditions, and especially in rainfall regime, might heavily affect efficiency of urban drainage systems. This paper focuses on storm tanks resilience to changes in rainfall statistics.

An analytical approach was proposed to assess long-term reduction of volumetric and overflow efficiency of storm detention tanks for sewer systems. Application of these probabilistic expressions to a urban catchment in Valencia provided satisfactory performances when compared against the results provided by continuous simulation numerical models.

Our results for Valencia show that the effect of climate variability on storm tank efficiency is likely to be smoothed by the filtering effect operated by the urban catchment. As it happens for natural catchments, the rainfall-runoff transformation tends to act as a natural filter that smoothes rainfall variability in time. Consequently, efficiency change is generally lower than rainfall change in relative terms and, thus, one may conclude that storm tanks are expected to be resilient. Results show that the target of keeping storm tank efficiency unchanged under changing rainfall regime could not be feasible in Valencia. Sizeable changes in tank volume or emptying flow would be necessary. We conclude that storm tanks are expected to be resilient in Valencia, but adaptation may require substantial redesign.

Water quality of receiving water bodies is non linearly related to overflow volume from sewer systems. Therefore, combined analysis of water quality, in both sewer system and final receptor, and storm tank efficiency is needed to identify optimal adaptation policies.

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