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CATCHMENT & SEWER NETWORK SIMULATION MODEL TO BENCHMARK CONTROL STRATEGIES WITHIN URBAN WASTEWATER SYSTEMS

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RESEARCH HIGHLIGHTS

- The existing plant-wide Benchmark Simulation Model is spatially extended to include the catchment and sewer system.

- A catchment and sewer system model is developed to describe the dynamics in generation and transport of wastewater during dry and wet weather periods.

- Overflow quality based evaluation criteria are defined for benchmarking of control strategies and structural modifications.

- Case studies depicting the usefulness/application of the catchment and sewer network model are performed.
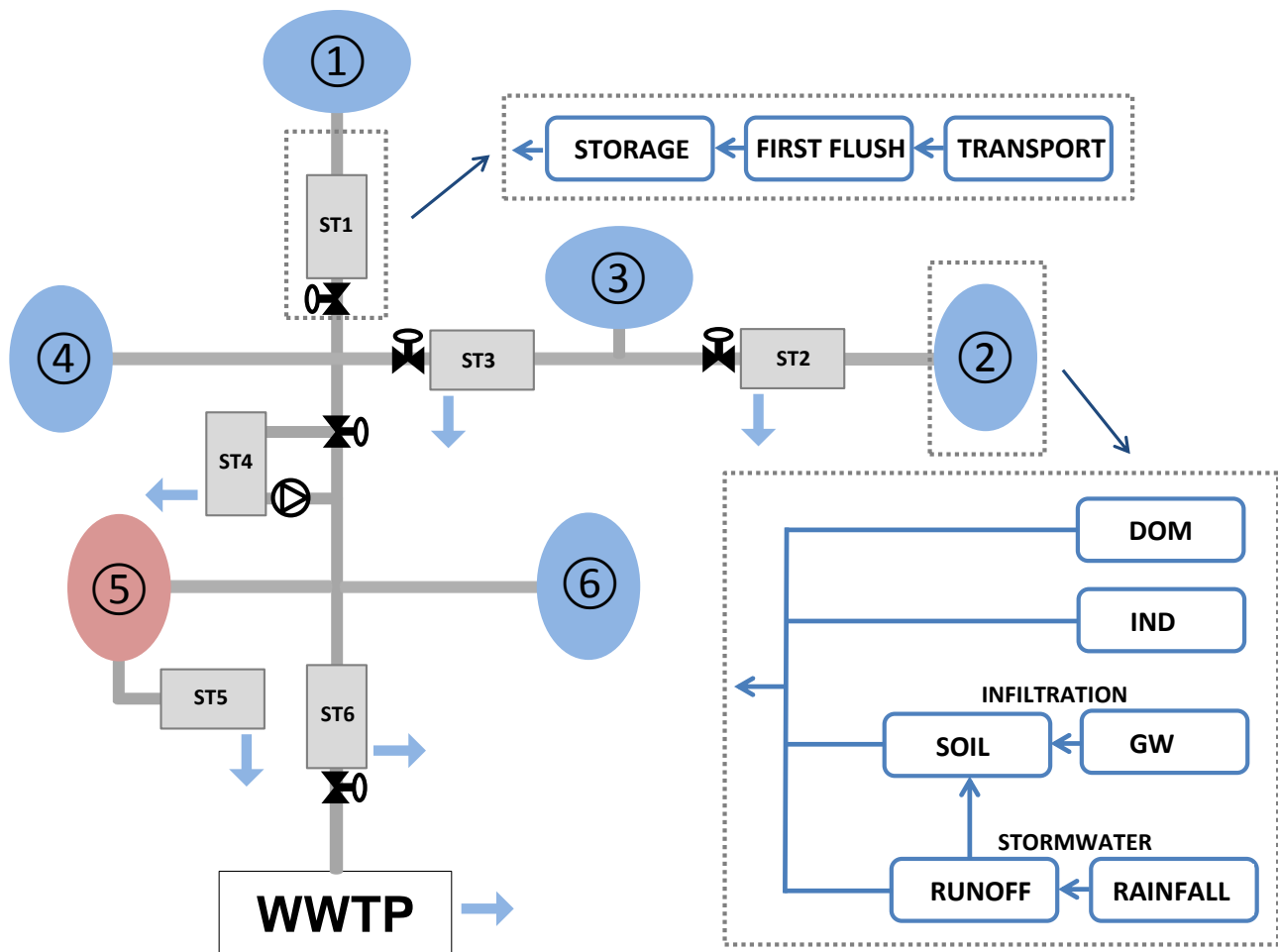
NOMENCLATURE

a	Rate of accumulation of pollutant (kg/ha.d) (runoff block)
A_c	Catchment area (m^2)
A_i	Area for the specific sub-catchment "i" (m^2)
A_{imp}	Impervious area of the catchment (m^2)
A_{soil}	Surface area of the tank (m^2) (soil block)
A_{st}	Area of the storage tank (m^2) (storage block)
b	Decay rate constant (1/d) (runoff block)
$C_{max(c)}$	Hourly maximum concentration for pollutant c (g/m^3)
COD	Chemical oxygen demand
COD$_{part}$	Particulate COD
COD$_{sol}$	Soluble COD
C_{st}	Constant for weir overflow (storage block)
EMC	Event mean concentration (g/m^3)
FF	Parameter to tune the strength of first flush effect (d^{-1}) (first flush block)
FFfraction	Fraction of particulate material that can settle in the sewers (first flush block)
GW_{in}	Annual mean groundwater inflow (m^3/d) (groundwater block)
$GW_{in,SCi}$	GW_{in} for each sub-catchment "i" (m^3/d) (groundwater block)
h_{inv}	Invert level of the tank (m) (soil block)
h_{max}	Maximum level in the tank (m) (soil block)
$h_{min,st}$	Minimum water level in the tank (m) (storage block)
$h_{o,st}$	Height in the storage tank when Q = Q$_{max,st}$/2 (m) (storage block)
$h_{ovf,st}$	Height of the overflow weir (m) (storage block)
h_{soil}	Height of the soil tank (m) (soil block)
h_{st}	Water level in the soil tank (m) (soil block)
i	Rainfall intensity (mm/h)
K_{down}	Gain for infiltration to groundwater aquifer (soil block)
K_{inf}	Gain for infiltration to sewers (soil block)
K_r	Residence time constant for the reservoir (d) (transport block)
K_{soil}	Soil permeability (m/d) (soil block)
$L_{weir,st}$	Length of the weir (m) (storage block)
M_{ff}	Mass of particulates accumulated in the sewer (kg) (first flush block)
$M_{i,st}$	Mass of pollutant "i" in the storage tank (kg) (storage block)
$M_{max,ff}$	Maximum particulate mass that can accumulate in the sewer system (kg) (first flush block)
M_r	Mass of pollutant in the reservoir (kg) (transport block)
M_s	Mass of particulate pollutant on the surface (kg) (runoff block)
n_{ff}	Parameter to tune the strength of first flush effect (first flush block)
NH$_4^+$	Ammonia
NO$_3^-$	Nitrate
N_{ovf}	Yearly overflow frequency (events/year)
n_r	Number of reservoirs in series (transport block)
OQI	Overflow quality index (kg pollution units/d)
PE_c	Population equivalents for the entire catchment
PE_i	Population equivalents in sub-catchment "i"
PO$_4^{3-}$	Phosphate
Q_{GW}	Infiltration to groundwater aquifer (m^3/d) (soil block)
$Q_{in,ff}$	Inflow to the first flush block (m^3/d) (first flush block)
$Q_{in,r}$	Inflow to the reservoir (m^3/d) (transport block)
$Q_{in,st}$	Inflow to the storage tank (m^3/d) (storage block)

Q_{inf}	Infiltration to sewers (m³/d) (soil block)	V_{soil}	Storage volume of the tank (m³) (soil block)
$Q_{\text{lim,ff}}$	Flow rate limit triggering the first flush effect (m³/d) (first flush block)	V_{st}	Volume of the storage tank (m³) (storage block)
$Q_{\text{max,st}}$	Maximum throttle flow (m³/d) (storage block)	w	Washoff constant (mm⁻¹) (runoff block)
$Q_{\text{out,r}}$	Outflow from the reservoir (m³/d) (transport block)	$X_{\text{i,st}}$	Inflow load for pollutant "i" in the storage tank (kg/d) (storage block)
$Q_{\text{out,st}}$	Outflow from the storage tank (m³/d) (storage block)	$X_{\text{in,ff}}$	Particulate flux entering the first flush block (kg/d) (first flush block)
$Q_{\text{ovf,st}}$	Overflow from the storage tank (m³/d) (storage block)	$X_{\text{in,r}}$	Input load to the reservoir (kg/d) (transport block)
$Q_{\text{pump,st}}$	Pumping rate at the storage tank (m³/d) (storage block)	$X_{\text{out,ff}}$	Particulate flux leaving the first flush block (kg/d) (first flush block)
$Q_{\text{throttle,st}}$	Throttle flow from the storage tank (m³/d) (storage block)	$X_{\text{out,r}}$	Output load from the reservoir (kg/d) (transport block)
RDI_{in}	Rainfall dependent inflow from pervious areas (m³/d) (soil block)	$X_{\text{ovf(c)}}$	Yearly overflow pollutant load for pollutant c (kg)
rrc	Rainfall runoff coefficient	α	Shape parameter for Gamma distribution (rainfall block)
$T_{\text{exc(c)}}$	Yearly exceedance duration for pollutant c (d)	β	Scale parameter for Gamma distribution (rainfall block)
T_{ovf}	Yearly overflow duration (d)		
V_{ovf}	Yearly overflow volume (m³)		
V_{r}	Volume of reservoir (m³) (transport block)		

ABSTRACT

This paper aims at developing a benchmark simulation model to evaluate control strategies for the urban catchment and sewer network. Various modules describing wastewater generation in the catchment, its subsequent transport and storage in the sewer system are presented. Global/local overflow based evaluation criteria describing the cumulative and acute effects are presented. Simulation results show that the proposed set of models is capable of generating daily, weekly and seasonal variations as well as describing the effect of rain events on wastewater characteristics. Two sets of case studies explaining possible applications of the proposed model for evaluation of: 1) Control strategies; and, 2) System modifications, are provided. The proposed framework is specifically designed to allow for easy development and comparison of multiple control possibilities and integration with existing/standard wastewater treatment models (Activated Sludge Models) to finally promote integrated assessment of urban wastewater systems.

1. INTRODUCTION

It has become increasingly clear that wastewater treatment plants (WWTPs) are strongly interconnected to other elements (sewer network, receiving media) within the urban wastewater system (UWS) and the evaluation of WWTP control strategies should be tackled in a more holistic manner (Rauch et al., 2002; Bach et al., 2014). For this reason, there is a need to move "outside the fence" of the WWTP and develop integrated tools for model-based evaluation and control of the UWS (Benedetti et al., 2013). This goal has inspired a large number of scientific contributions that attempt to investigate different aspects of integrated modelling. For example, Benedetti et al. (2004) and Vanrolleghem et al. (2005) tackled important issues such as model integration and model compatibility. Another important aspect has been model complexity reduction to allow for long term simulations (Erbe and Schütze, 2005; Fu et al., 2009a). The latter and the increase in computational power promoted the use of Monte Carlo simulations and the study of input uncertainty propagation through the model either during the model development process or during model use (e.g. Astaraie-Imani et al., 2012; Benedetti et al., 2008; Benedetti et al., 2010; Freni et al., 2011; Fu et al., 2009b). Long term simulations can be conducted as well, including the study of integrated control (e.g. Fu and Butler, 2012; Weijers et al., 2012). Finally, studies of the fate of particular compounds such as sulfur compounds (Jiang et al., 2010), greenhouse gas emissions (Guo et al., 2012) and micro-pollutants (Vezzaro et al., 2014; Snip et al., 2014) were also performed.

One of the major areas of application for integrated models is control. Integrated control has been studied for some years and the main benefits of using such an approach are demonstrated in several studies (e.g. Harremöes et al., 1994; Schütze et al., 2002; Vanrolleghem et al., 2005; Langeveld et al., 2013). With the future clearly pointing towards integrated management of the UWS, the need for development of efficient integrated control strategies is growing. In this context, we believe that a benchmarking tool can be extremely beneficial to develop and test control strategies in the UWS. Within sewer systems, Borsányi et al. (2008) conducted a benchmarking study using real-time control strategies applied to two virtual sewer systems. In the WWTP community, benchmarking control strategies has been very successful. Benchmark Simulation Models (BSM1, BSM1-LT and BSM 2) and associated spin-off products (influent generator, ADM1 implementation, sensor models, evaluation criteria etc.) have demonstrated to be valuable tools in the field of WWTP optimization and have been widely used in both industry and academia (Gernaey et al., 2014). Nevertheless, there is a lack of benchmarking tools that allow objective comparison of control strategies in urban catchments and sewer systems. Therefore: 1) Rigorous development/evaluation of control strategies in the WWTP (Gernaey et al., 2014) is based on influent generators (Gernaey et al., 2011; Flores-Alsina et al., 2014; Martin and Vanrolleghem, 2014), and such influent generators are not suitable for modelling control strategies upstream of the WWTP; and, 2) In many cases, integrated UWS control strategies cannot be developed and evaluated on a single simulation platform.

The objective of this paper is to develop a catchment and sewer network model to benchmark control strategies. The catchment model reproduces the generation of wastewater through the combination of four different sub-models (Domestic (*DOM*), Industrial (*IND*), Stormwater (*SW*) and Infiltration to sewers (*INF*)). The sewer model describes wastewater transport (TRANSPORT) as well as the sudden increase of particulates during the beginning of a rain event following a period of drought (FIRST FLUSH) and the retention of wastewater (especially during rain events) using storage tanks to avoid combined sewer overflows (STORAGE). A set of evaluation criteria are used to assess the overflow discharged into the receiving waters. The criteria can be applied for a specific overflow location (Local) or for the entire system (Global). The criteria can be further classified into those describing: 1) cumulative effects; and, 2) acute effects on the receiving water system. As a receiving water model is not used in this study, these criteria are only indirect indicators of the effect of overflow discharges on river systems. Additionally, case studies demonstrating the possible applications of the tool for analyzing the impact of: 1) local/global control strategies; and, 2) system modifications, are presented and discussed in detail. The proposed framework is specifically designed to allow for development and comparison of multiple control strategies, and allows easy interfacing with existing wastewater treatment (benchmark) models to finally promote integrated assessment of catchment, sewer network and WWTP performance.
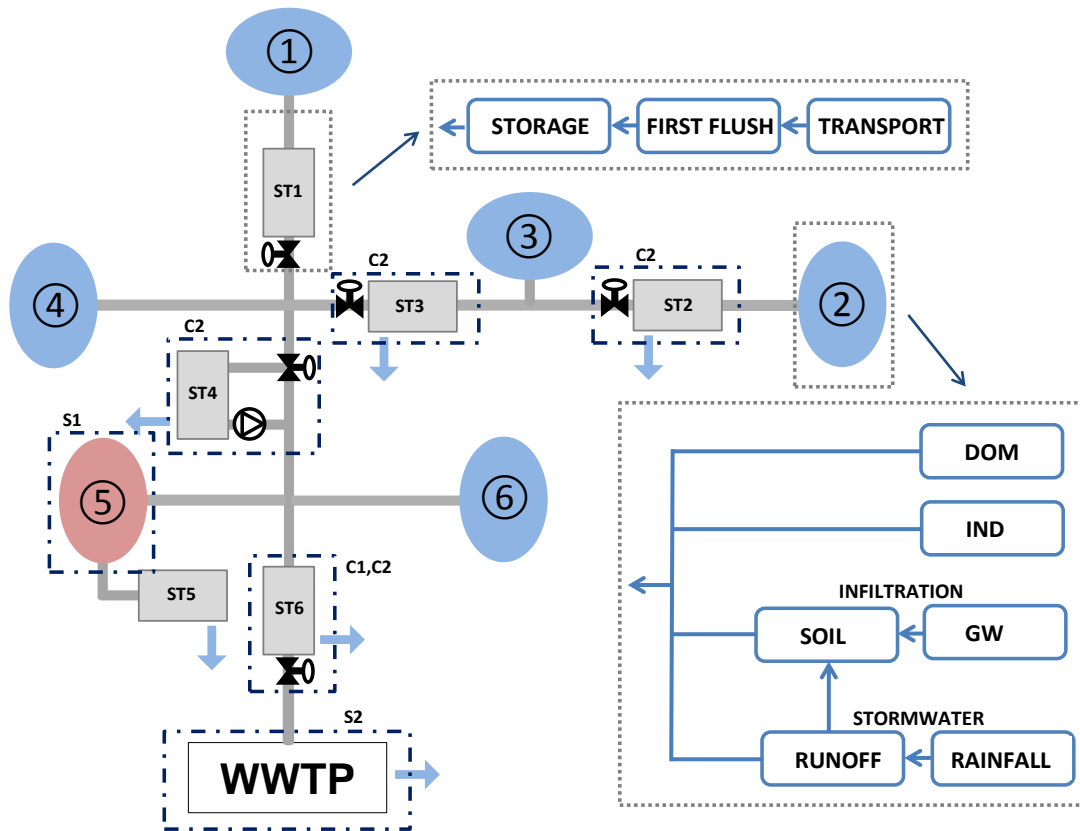
Figure 1: Catchment and sewer BSM layout indicating various sub-catchments (①, ②, ③, ④, ⑤, ⑥ represent SC1, SC2, SC3, SC4, SC5 and SC6 respectively), storage tanks (ST1,ST2, ST3, ST4, ST5, ST6) and control elements.

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Overflows are assumed to enter a receiving water (not modelled here). A snapshot of the underlying blocks for the CATCHMENT and SEWER models is presented. The locations for control strategies (C1, C2) and structural modifications (S1, S2) are highlighted. DOM, IND, GW stand for domestic, industrial and groundwater respectively.

2. SYSTEM CHARACTERISTICS AND GENERAL MODEL DESCRIPTION

A hypothetical system with a similar structure as the catchment described in ATV A 128 (ATV, 1992) is used as a case study. **Figure 1** illustrates the catchment configuration and its main characteristics. The total catchment area (A_c) is 540 hectares and comprises 80,000 population equivalents (PE_c). Dry weather flow is scaled up to be similar to the BSM2 influent characteristics (18,500 m³/d) (Gernaey et al., 2014). The three main contributors to dry weather flow are: 1) domestic sources with a daily average flow (DAF) of 12,000 m³/d; 2) industrial contribution with a DAF of 2,500 m³/d; and, 3) infiltration to sewers which corresponds to 25% of the dry weather flow.

Table 1: System characteristics for the catchment, storage tanks and sewer network.

Sub-catchment (SC)	Area (ha)	PE	DWF (m³/day)		Storage volume (m³)
			DOM	IND	
1	99	15,920	2,390		5500
2	21	3,920	590	2,500	1000
3	29	2,960	440		2000
4	71	9,600	1,440		4000
5	71	7,840	1,180		4000
6	249	39,760	5,960		15000
Total	**540**	**80,000**	**12,000**	**2,500**	**31,500**

The system under study is comprised of six sub-catchments (SC1,.., SC6) with different areas (A1,.., A6) and population densities (PE1,.., PE6) (see **Table 1**). All the defined SCs are considered to be domestic except SC2, which has both domestic and industrial contributions. SC1, SC2, SC3, SC4 and SC6 are connected to a combined sewer system whereas SC5 has a separate sewer system. The proposed catchment also has six storage structures (five on-line pass-through tanks and one off-line bypass tank) (see Figure 9 for additional details). Finally, it should be mentioned that the entire catchment is connected to a WWTP, which has the same layout/characteristics as the BSM2 plant-wide model (Jeppsson et al., 2007). Sewer overflows and WWTP effluents are discharged at various locations into the receiving waters as depicted in **Figure 1**. It should be noted that the current study does not include the river system.

3. CATCHMENT MODEL

The catchment model is largely inspired by the BSM2 dynamic influent pollutant disturbance scenario generator (DIPDSG) (Gernaey et al., 2011) and uses many of its salient model blocks for simulating the dynamics of flow rate and pollutant load generation. The generation of wastewater at each sub-catchment (SC_i) is achieved by combining the contributions from: 1) domestic (DOM_i); 2) industry (IND_i); 3) infiltration to sewers (INF_i); and, 4) stormwater (SW_i). The pollutants considered are chemical oxygen demand (COD), ammonia (NH_4^+), nitrate (NO_3^-) and phosphate (PO_4^{3-}). COD is further subdivided into

COD$_{sol}$ (soluble COD) and COD$_{part}$ (particulate COD). All pollutants are represented as loads (kg/d). The flow rate is expressed in m^3/d units. In the catchment model, the subscript "i" denotes various parameters and model state variables for each sub-catchment.

3.1. Domestic (DOM)

In the proposed approach, the domestic (*DOM*) sub-model contributes to the influent flow rate/pollutant dynamics by diurnal variations, a weekend effect and a holiday effect. This is achieved by combining three user-defined data files containing: 1) a *normalized daily* profile; 2) a *weekly* pattern including the weekend effect; and, 3) a *holiday* effect. The generated time series is then multiplied by the flow rate/pollution load per population equivalent (m^3/PE.day, kg/PE.day) and the number of person equivalents in the specific sub-catchment (*PE$_i$*) (for default values see Gernaey et al., 2011; Flores-Alsina et al., 2014; Snip et al., 2014).

- *Normalized daily profile*: The daily flow rate/pollution profile represents a general behaviour with a morning peak, an evening peak and a late night/mid-day minima (**Figure 2a**). It is important to notice that the particulate profile slightly lags behind that of the soluble pollutants. This effect is introduced to account for the slower transportation rate of particulates.

- *Weekly profile*: A drop in the flow-rate/pollutant generation during weekends is modelled using a uniform value during weekdays and a lower fraction during the weekends (**Figure 2b**). This corresponds to an 8% and 12% drop in flow rate on Saturdays and Sundays, respectively. For pollution loads, a higher reduction factor is applied (12% on Saturdays and 16% on Sundays).

- *Yearly profile (holiday effect)*: A similar approach as defined above is used to account for the yearly profile. The holiday period (3 week period during July-August) represents a 25% reduction of the flow rate/pollution load during the first two weeks and a 12% decrease during the third week (**Figure 2c**).

Zero-mean white noise can be added to these inputs. It is up to the model user to decide whether or not to include random noise. The purpose of including noise is two-fold: 1) To avoid having exactly the same profiles for pollutants/flow rates on different days of the week; and, 2) To avoid an exact correlation (correlation coefficient=1) between state variables in the catchment model and also ASM state variables (see Gernaey et al., 2011; Snip et al., 2014 for further information). This however does not remove the correlation completely (e.g. flow rate and soluble pollutant profiles are still correlated)..
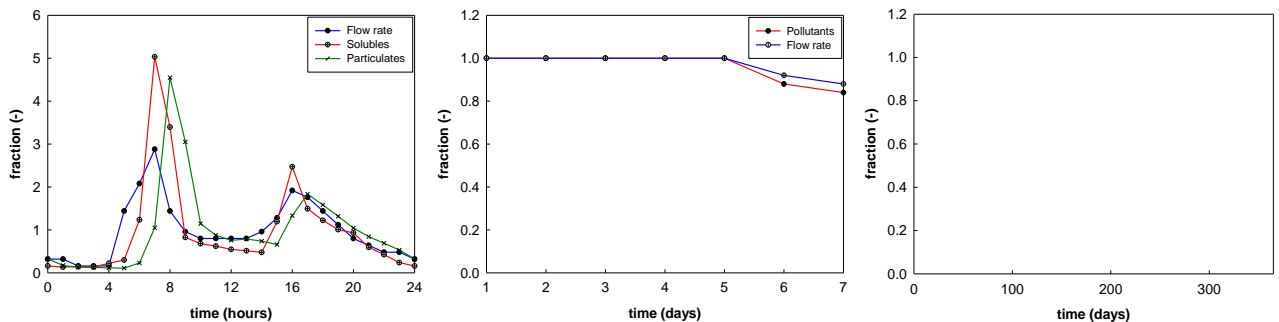
a   b   c

Figure 2: Diurnal variation in pollutant loads and flow rate (a). Weekly variation with two different profiles (red = pollutants, blue = flow rate) (b) and yearly profile (starting first week of July) with similar dynamics for pollutants and flow rate (c).

3.2. Industrial (IND)

The industrial (*IND*) contribution to the influent flow rate/pollutant load is generated similarly to the *DOM* sub-model. The industry model block is also based on user-defined files describing *weekly* and *yearly* effects. Again, the dynamic pattern is generated by sampling in a cyclic manner from source files and then multiplied by the average daily wastewater/pollution generation from the industry (m^3/day, kg/day) (see Gernaey et al. (2011) for additional information/default values). In the case demonstrated in this paper, these values only apply to SC_2 as it is the only sub-catchment with an industrial contribution. Adding zero mean white noise adds realism to the industrial wastewater profiles.

- *Weekly profile*: As can be seen in **Figure 3**, the variations in the industry pollutant fluxes are less extreme than the variations of the domestic pollutant fluxes. Also, when the industrial particulate pollutant flux is compared to the industrial wastewater flow rate, the particulate pollutant flux shows a four hour time delay to account for the slower transport of particulates. The Friday afternoon effect is also illustrated in **Figure 3a,** during which the pollutant fluxes are doubled, assuming to be the consequence of industrial cleaning. During the weekend, the industrial flow and pollutant fluxes are considerably lower compared to weekdays (60% decrease of the flux on Saturdays and 80% decrease on Sundays).

- *Yearly profile*: Two holiday periods marked with lower wastewater generation are modelled (**Figure 3b**). Hence, the industrial wastewater production is reduced by 70% during the summer holidays and 80% during the Christmas period to simulate the shutdown of industrial activities during these periods.
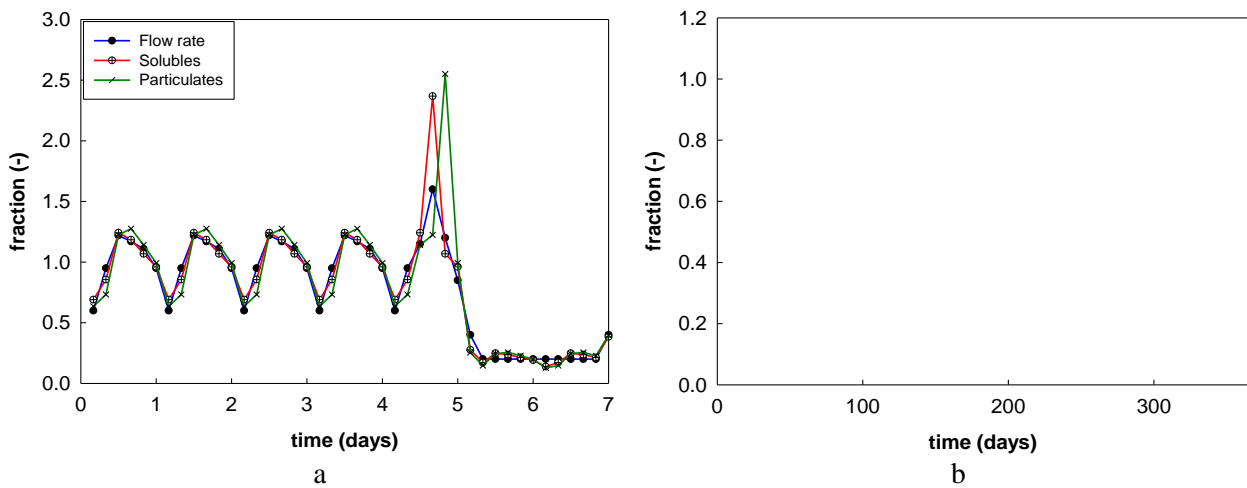
Figure 3: Dynamics of industrial dry weather pollutant and flow rate generation with weekly (a) and yearly (b) variations. The yearly profile begins in the first week of July.(For simplicity, we assume that the first day of july is a Monday).

3.3. Stormwater (SW)

The stormwater (SW) sub-model is comprised of two different elements: a rainfall generator block (*rainfall),* which characterizes the intensity and duration of precipitation and a runoff contribution block (*runoff)*, which generates the flow rate/pollution load corresponding to the rain events.

3.3.1. Rainfall generator block (*rainfall*)

The *rainfall* block can be used in two different ways. Firstly, rainfall data described as intensity (mm/h) can be used as a model input. A second option is based on a stochastic rainfall generation approach (Richardson. 1981). The latter approach is used in this paper. The implementation in this study is inspired by the rainfall generator proposed by Talebizadeh et al. (2016). The representation of rainfall is described mathematically using a two state Markov chain model. Two different states are defined representing dry (DRY) and wet (WET) weather periods. The transition between states is defined by a transition probability matrix (P) (see **Equation 1),** which is estimated from historic data. In the matrix P, the value $P_{d|w}$ represents the probability for the next period to be wet given that the current period is dry and vice-versa for $P_{w|d}$. The other probability values can also be interpreted in a similar fashion. Each period lasts for 15 minutes. These probabilities change on a monthly basis to better describe the seasonal variation in precipitation. A key property for the Markov chain is that it does not have any memory. Therefore, the state of a system for the next time step (t+1) is determined solely by its state in the current time step (t).

$$P = \begin{bmatrix} P_{d|d} & P_{d|w} \\ P_{w|d} & P_{w|w} \end{bmatrix}$$

Eq. 1

Finally, a gamma distribution (**Equations 2** & **3**) (Buishand, 1978) determines the rainfall intensity for the WET periods that are generated using the Markov chain. Parameters α and β, called the shape and scale parameters, are determined by fitting the historic rainfall data to a gamma distribution.

$$f(x) = \frac{\left(\frac{x}{\beta}\right)^{\alpha-1} e^{-\frac{x}{\beta}}}{\beta\Gamma(\alpha)}$$

Eq. 2

$$\Gamma(\alpha) = \int_0^\infty e^{-t} t^{\alpha-1} \mathrm{d}t$$

Eq. 3

Figure 4 presents the (synthetic) yearly rainfall data generated using the stochastic rainfall generator described above. The total annual rainfall from data and model is 721 mm and 738 mm, respectively. Simulation results show that the model produces similar monthly variations and annual rainfall but there is room for improvement when describing high intensity rainfall events. This is due to the fact that such high rainfall events are very rare and hence the probability of such an event being reproduced by the gamma distribution is low. It is important to highlight that the approach presented herein is an empirical one and is purely an engineering attempt. A detailed analysis to validate the rainfall generator in terms of its ability to reproduce the statistical properties of the historic rainfall time series is not performed (Ward and Robinson,

2000). Only visual inspection is used to validate the model. Also, the model has various limitations. Two of the main limitations are: 1) Transition between wet/dry states is only a function of the previous period (which can be less than a day). It does not consider the effect of previous days; and, 2) Rainfall intensity during each period is independent of the intensity in the previous periods. Owing to these limitations, users are suggested to exercise caution while using this model for their particular catchments. Nevertheless, we believe that the tool is useful to simulate various rainfall patterns for evaluating control strategies on an UWS scale. It can be easily adapted to simulate high/low intensity and long/short duration rainfalls by varying the transition probabilities and the parameters of the gamma distribution.
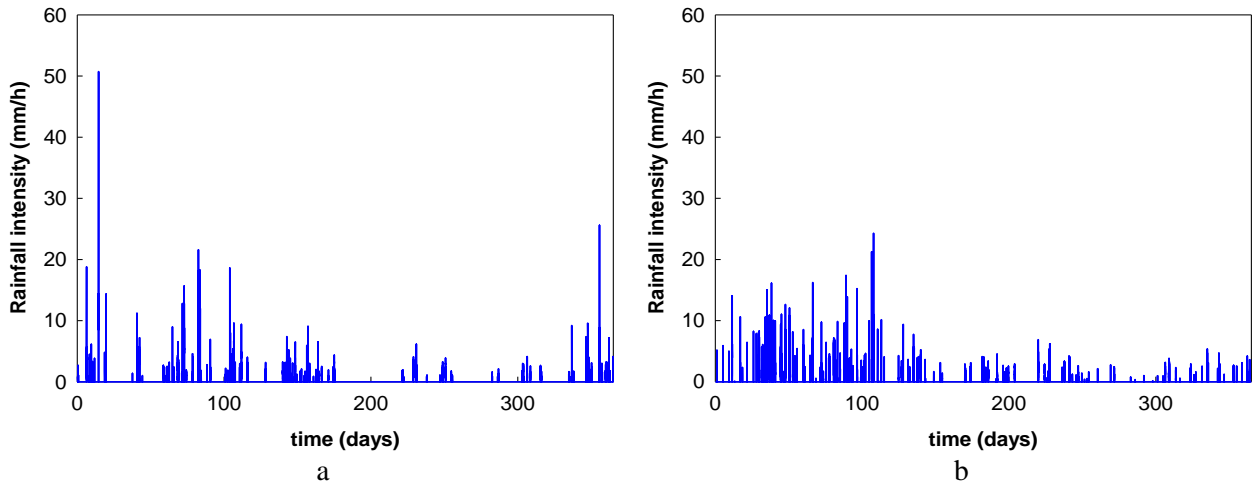
Figure 4: Rainfall intensity time series from data (a) and the one generated using the model (b) for a period of 1 year. The time series begins in the first week of July.

3.3.2. Runoff contribution block (*runoff)*

The *runoff* block is used to convert the rainfall intensities (mm/h) into surface runoff (m^3/d). It also accounts for the (soluble/particulate) pollution contribution from each sub-catchment surface to the sewer system.

- The *flow rate runoff* block uses a dimensionless rainfall runoff coefficient (rrc_i) to represent various continuous losses taking place within the sub-catchment. The impervious area ($A_{imp,i}$) is determined by the parameter φ_i representing the impervious fraction of the sub-catchment surface. Rain falling on impervious areas is multiplied by the rrc_i to generate the runoff which is then passed through a linear reservoir model to simulate the delay and attenuation typically observed in urban catchments. A similar approach is used in the sewer system (see Section 4).

- The soluble pollution contribution (*sol-poll runoff*) (**Figure 5a**) is calculated assuming a constant pollutant concentration during rain events. These values are also known as event mean concentrations (EMC) and may vary depending on the catchment characteristics and the rain event. EMC values for soluble COD (9 g/m^3) and ammonium (0.56 g/m^3) are based on Butler and Davies (2011). EMC values for nitrate and phosphate are assumed to be zero. These concentrations are then multiplied by the flow rate (m^3/d) obtained from the *flow rate runoff* block to generate pollutant

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loads (kg/d). Due to this simplified approach of assuming constant concentration for all rain events, the model cannot simulate the influence of antecedent dry days/rain on the soluble pollutant concentration.

- The last element is the particulate pollution contribution (*part-poll runoff*). This model block is based on an accumulation and washoff approach (Butler and Davies, 2011) (**Figure 5b**). There is an accumulation of particulate COD (COD_{part}) during dry weather periods until a maximum threshold is reached. During rain events, the accumulated pollutant is washed off depending on the intensity of the rain event and the amount of pollutant accumulated. **Equation 4** describes the variation of the mass of pollutant on the sub-catchment surface ($M_{s,i}$) (kg). The parameter (a_i) (kg/ha.d) defines the rate of accumulation of the pollutant and *(A_i)* is the sub-catchment area. In order to avoid pollutant mass reaching large values, a removal rate characterized by the parameter b_i (decay rate constant (1/d)) is used. Hence, during a long dry period, a maximum pollutant mass is reached and no further accumulation takes place. During a rain event, the pollutant is washed out at a rate determined by the washoff constant (w_i) (mm^{-1}) and rainfall intensity (i_{rain}) (mm/h) and the available mass on the catchment surface ($M_{s,i}$). A conversion factor (24) is used to convert the resulting washoff load from kg/h to kg/d. From Figure 5b, it can be seen that the parameters are aggressively tuned leading to consecutive washoff of particulates during day 516. The results presented correspond to the output of the accumulation and washoff block. There are a series of reservoirs (sewer network) that attenuate the peak values before the pollutant load reaches CSOs/WWTP. In the absence of such tuning, the increase in particulate load is not noticeable at CSOs/WWTP.

$$\frac{\mathrm{d}M_{s,i}}{\mathrm{d}t} = a_i A_i - b_i M_{s,i} - 24 w_i i_{rain} M_{s,i} \qquad \textbf{Eq. 4}$$
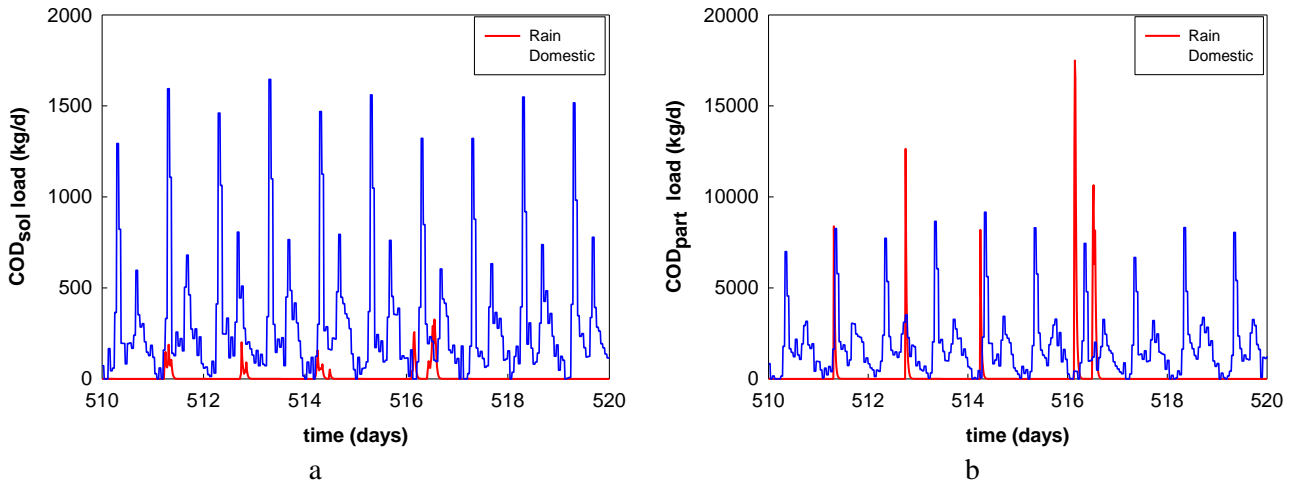
Figure 5: Effect of EMC during rainfall on soluble pollutant (COD_{sol}) (a), and effect of the accumulation and washoff model on COD_{part} (b).

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3.4. Infiltration to sewers (INF)

The infiltration to sewers (*INF*) sub-model is comprised of two main elements. Firstly, a groundwater block (*groundwater*) and secondly a soil block (*soil*) (Gernaey et al., 2011). The *groundwater block* describes changes in the amount of infiltration attributed to variations in the groundwater level over the year (**Figure 6**). Seasonal groundwater inflow is modelled as a sine wave with a yearly frequency. The groundwater inflow to the model is at its lowest during the dry period and at its highest during the rainy period of the year. Additional details can be found in Gernaey et al. (2011). The (total) annual mean groundwater inflow (GW_{in}) for the entire catchment is 7,100 m^3/d and the amplitude of variation (*Infamp*) is 25%. Based on the area of each sub-catchment, a mean groundwater inflow is defined as a fraction of the annual average for the entire catchment ($GW_{in,i}$).

The *soil block* is described using a variable volume tank model for each sub-catchment. It is used to represent the assumed volume of water stored in the soil ($V_{soil,i}$). Parameters for the soil model are: $A_{soil,i}$ (the surface area of the variable volume tank) which is the pervious area of the sub-catchment ($\varphi \cdot A_i$) , $h_{max,i}$ (the maximum level in the tank), $h_{inv,i}$ (the invert level, i.e. the maximum water level in the groundwater storage tank that will not cause infiltration, corresponding to the bottom level of the sewer pipes). $RDI_{in,i}$ (rainfall dependent inflow) is the runoff generated due to rain from pervious areas (see Section 3.3). $K_{soil,i}$ is defined as the soil permeability. $RDI_{in,i}$ is limited by the permeability of the soil (maximum $RDI_{in,i}$ equals $K_{soil,i} \cdot A_{soil,i}$). Any excess rainfall dependent inflow reaches the sewer system. Infiltration to sewers ($Q_{inf,i}$) from the soil (*soil*) block is modelled by the parameter $K_{inf,i}$ (a measure of the quality of sewer pipes). Similarly, infiltration to groundwater ($Q_{GW,i}$) is determined using the parameter $K_{down,i}$ (parameter to adjust the flow rate to the downstream aquifers). **Equation 5** represents the volume balance for the soil model. **Equation 6** elaborates on the volume balance in the *soil* block based on the relationship between various outflows and the storage height ($h_{soil,i}$). In order to keep the model simple, the case where wastewater from the sewer system reaches the groundwater (exfiltration) (Rutsch et al., 2006) is not considered here.

$$\frac{dV_{soil,i}}{dt} = GW_{in,i} + RDI_{in,i} - Q_{inf,i} - Q_{GW,i} \qquad \textbf{Eq. 5}$$

$$\frac{A_{soil,i} dh_{soil,i}}{dt} = GW_{in,i} + RDI_{in,i} - K_{inf,i}\sqrt{h_{soil,i} - h_{inv,i}} - K_{down,i} h_{soil,i} \qquad \textbf{Eq. 6}$$
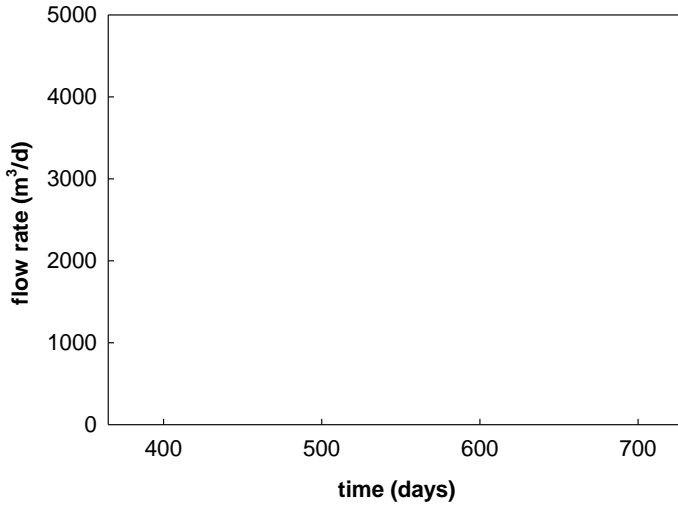
Figure 6: Infiltration to sewers from SC_1 depicting the annual variations and also rainfall dependent variations.

4. SEWER NETWORK MODEL

The sewer model is comprised of three different elements: 1) a transport sub-model (*TRANSPORT*) to describe the effect of the sewer system on both flow rate and pollutants; 2) a first flush sub-model (*FIRST FLUSH*) mimicking the sudden increase of particulates at the beginning of rain events following a period of drought; and, 3) different types of storage tank sub-models (STORAGE) acting as buffers to prevent discharge of rainwater into rivers during rain events. These three sub-models are used repetitively at various locations. Biological transformations within the sewer system (Huisman, 2001; Snip et al., 2014) are not considered in the model.

4.1. Sewer transport (TRANSPORT)

Flow and pollution transport within the sewer system is modelled using completely mixed tanks with varying volumes (Viessman et al., 1989). **Equation 7** represents the mass balance for volume (V_r) (m^3) of the reservoir where $Q_{in,r}$ and $Q_{out,r}$ are input and output flow rates (m^3/d), respectively, for each reservoir block. The outflow is related to the volume based on a residence time constant (K_r) (d).

$$\frac{dV_r}{dt} = Q_{in,r} - Q_{out,r};\ Q_{out,r} = \frac{1}{K_r}V_r \qquad \text{Eq. 7}$$

$$\frac{dM_r}{dt} = X_{in,r} - X_{out,r};\ X_{out,r} = \frac{1}{K_r}M_r \qquad \text{Eq. 8}$$

Similarly, in **Equation 8**, M_r is the pollutant mass (kg). $X_{in,r}$, $X_{out,r}$ are the input and output loads (kg/d). **Figure 7** shows the effect of the parameter K_r on the outflow. With longer residence time, a larger sewer system is simulated. Longer sewer lengths can also be simulated by connecting a number of such reservoirs in series. The number of reservoirs in series (n_r) depends on the length of the sewer system. The larger the catchment, the higher is the number of reservoirs in series. In this particular study, K_r and n_r values are

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estimated assuming a total sewer length of 1 km per 15 ha of catchment area. These values are in the same range as some Scandinavian cities (2 km per 15 ha) (VASYO 2015a;b).
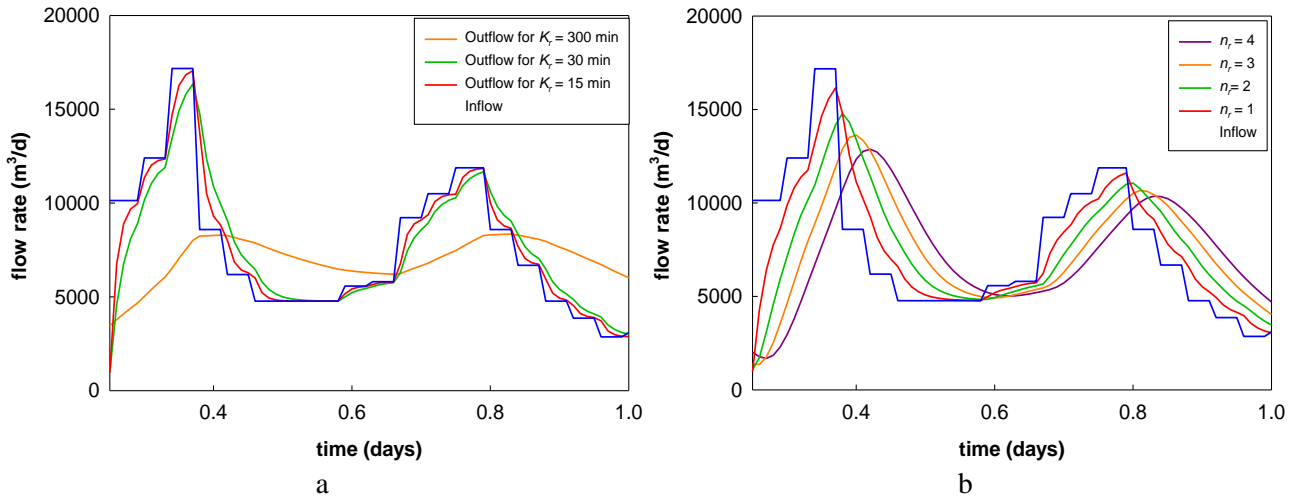
Figure 7: Reservoir model used for the sewer network. Effect of different residencence time constants (K_r) for a given inflow (a). Variations in the outflow based on the number of such reservoirs (n_r) in series (b).

4.2. First flush of particulates (FIRST FLUSH)

The *FIRST FLUSH* sub-model mimics the sudden increase of particulates that have been accumulated within the sewer during dry weather periods. The model relies on the assumption that only a part of the particulate material can settle in the sewer system (*FFfraction*) and be accumulated until a flow rate threshold is reached. The accumulated particulates are washed out during rain events. The extent of washoff depends on the intensity of the flow rate. **Equation 9** describes the accumulation of particulates (COD_{part}) (M_{ff}) in the sewer as a function of the flux of solids entering ($X_{in,ff}$) and leaving ($X_{out,ff}$) the system. $Q_{in,ff}$ represents the influent flow rate (m^3/d). $M_{max,ff}$ (kg) is the maximum amount of particulates that can be stored in the sewer system. $Q_{lim,ff}$ (m^3/d) is the flow rate limit triggering the first flush effect. FF (d^{-1}) and n_{ff} (-) are adjustable parameters to tune the desired strength of the first flush effect. The first term in the equation represents accumulation of particulates. Particulates accumulate until a maximum mass $M_{max,ff}$ is reached. The second term is a Hill function representing the washoff during rain events. At very low $Q_{in,ff}$ values (dry weather flows), the washoff is negligible. As the inflow increases and reaches $Q_{lim,ff}$, the particulate washoff increases rapidly.

$$\frac{dM_{ff}}{dt} = X_{in,ff}\left(1 - \frac{M_{ff}}{M_{max,ff}}\right) - \frac{Q_{in,ff}^{n_{ff}}}{Q_{lim,ff}^{n_{ff}} + Q_{in,ff}^{n_{ff}}}M_{ff}FF \qquad \textbf{Eq. 9}$$

Figure 8 depicts the influence of the *FIRST FLUSH* model on the particulate pollutant behaviour for the sewer system connected to SC_6. When the influent flow rate is higher than the triggering flow rate ($Q_{lim,ff}$ = 29,820 m^3/d) and the sewer is full of sediments ($M_{max,ff}$ = 2,490 kg) there is a sudden increase of COD_{part} load in the influent to the WWTP (FF = 2,500, n_{ff} = 15 and *FFfraction* = 0.25). A similar over-tuning of parameters (as noticed in accumulation and washoff model (section 3.3.2)) can be noticed in Figure 8. Due to

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the presence of the sewer network, the pollutant peaks get reduced considerably before reaching the CSOs/WWTP. Over-tuning is necessary to compensate for this behaviour.
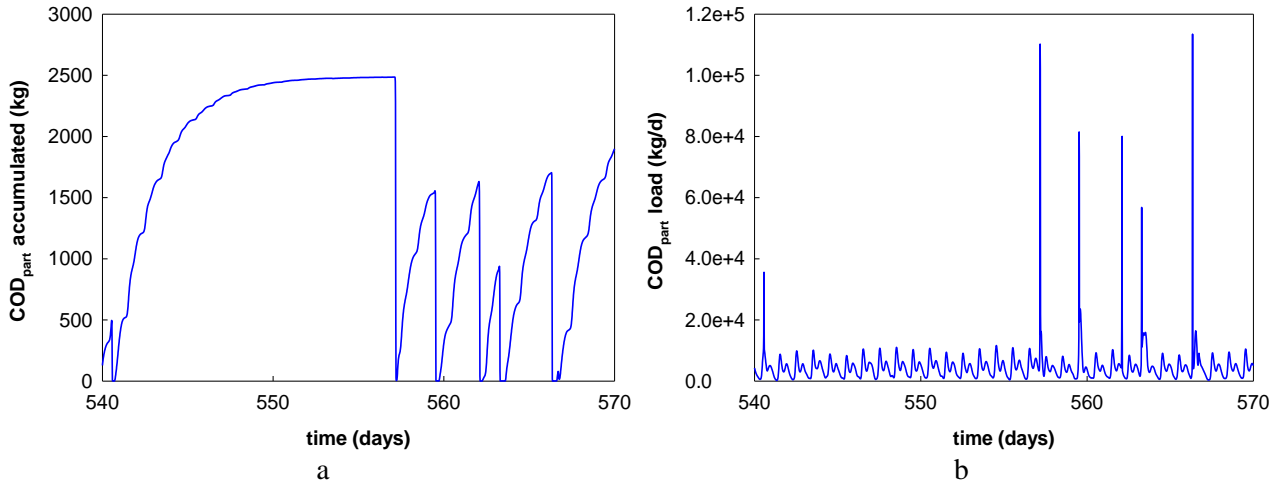
Figure 8: Accumulation of COD_{part} in the sewer system (a), and the sewer particulate load (blue) (b) .

4.3. Storage tanks (STORAGE)

Storage tanks (STs) are the main control elements to regulate the incoming flow to the WWTP and sewer overflows to rivers. The volume of each of these tanks is approximately 60 m³/ha of catchment area. In Europe, storage volumes range from 30 m³/ha to 200 m³/ha (Schutze et al., 2002). There are four different configurations of the tanks which are mainly classified into on-line and off-line modes (**Figure 9**).

1. *On-line tanks*: These tanks are in-line with the sewer network and the storage volume is in use during dry weather as well. The entire dry weather flow passes through the tank and reaches the WWTP. Valves can be used to limit the throttle flow. A valve model with a linear relationship between valve opening and flow rate variation is included.

2. *Off-line tanks:* These storage tanks are not directly in-line with the sewer network. The sewer pipes have a maximum capacity and any excess flow is directed to the storage tank. In the case of off-line tanks, typically pumps are used to send the stored wastewater back to the sewer system. Therefore, the outflow from the tanks is governed by the pumping rate. Pump flow can either be supplied as an input or as an actuator setting from a controller.

In addition, pass-through and bypass configurations are modelled for both on-line and off-line storage tanks.

1. *Pass-through tanks:* The overflow weir is located at the end of the storage tank. All the inflow to the storage tank passes through the tank before reaching the outlet or overflowing into the river.

2. *Bypass tanks:* These are tanks with overflow at the beginning of the storage tank. This is advantageous especially in systems with high first flush effects. For *on-line tanks*, this highly polluted stormwater reaches the WWTP. Similarly, for *off-line tanks,* the stored stormwater can later be pumped back to the trunk sewer and from there to the WWTP.

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Only two of the four available configurations are used in the current layout (**Figure 1**). ST1, ST2, ST3 and ST6 are on-line pass-through tanks while ST4 is an off-line bypass tank.
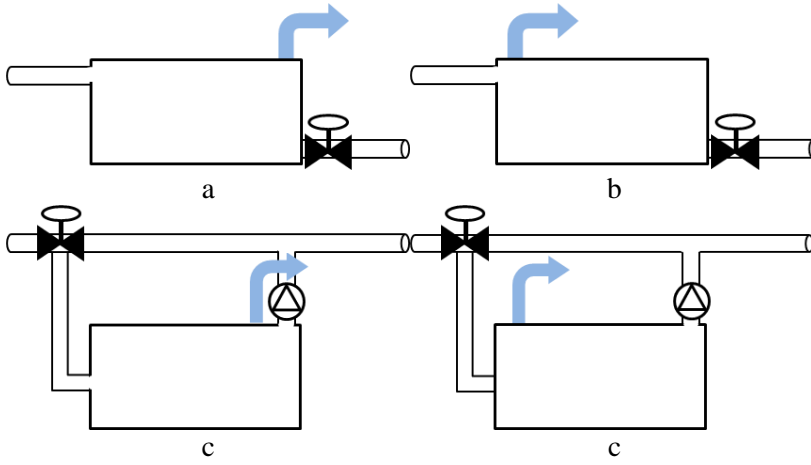
Figure 9: Different configurations of storage tanks: a) on-line pass-through tank; b) on-line bypass tank; c) off-line pass-through tank; d) off-line bypass tank. Pumps and valves are used as flow control elements in off-line and on-line tanks, respectively.

Table 2 summarizes the mass balance and equations used for the previously described storage tanks. V_{st} is the volume of the tank filled with water and A_{st} denotes the surface area of the tank. $Q_{in,st}$ and $Q_{out,st}$ represent the inflow and outflow from the storage tanks. In the case of *on-line tanks*, $Q_{out,st}$ represents the throttle flow from the tank (Vallet, 2011). For *off-line tanks,* it is the pumping rate $Q_{pump,st}$. Overflows are denoted by $Q_{ovf,st}$ (Hager, 2010). $M_{c,st}$ denotes the mass of each pollutant (c) and $X_{c,in,st}$ and $X_{c,out,st}$ represent the corresponding inflow and outflow loads for each pollutant respectively. $Q_{max,st}$ is the maximum outflow for on-line tanks (m³/d). $h_{o,st}$ is the water level in the storage tank (m) when $Q = Q_{max,st}/2$. $h_{min,st}$ is the minimum water level in the tank (m). h_{st} is the water level in the tank (m). C_{st} is a constant for weir overflow. $L_{weir,st}$ is the length of the weir (m) and $h_{ovf,st}$ is the height of the overflow weir measured from the bottom of the tank (m).

Table 2: Summary of modelling details for various storage tank models used in the system-wide BSM.

	ON-LINE		OFF-LINE	
	pass-through	bypass	pass-through	bypass
V_{st}	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$
$M_{c,st}$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}(Q_{out,st} + Q_{ovf,st})$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}} Q_{out,st} - X_{c,in}\frac{Q_{ovf,st}}{Q_{in,st}}$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}(Q_{out,st} + Q_{ovf,st})$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}} Q_{out,st} - X_{c,in}\frac{Q_{ovf,st}}{Q_{in,st}}$
$Q_{out,st}$	$\frac{Q_{max,st}(h_{st} - h_{min,st})^{n_{st}}}{h_{o,st}^{n_{st}} + (h_{st} - h_{min,st})^{n_{st}}}$	$\frac{Q_{max,st}(h_{st} - h_{min,st})^{n_{st}}}{h_{o,st}^{n_{st}} + (h_{st} - h_{min,st})^{n_{st}}}$	$Q_{pump,st}$	$Q_{pump,st}$
$X_{c,out,st}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$
$Q_{ovf,st}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$
$X_{c,ovf,st}$	$M_{c,st}\frac{Q_{ovf,st}}{V_{st}}$	$X_{c,in,st}\frac{Q_{ovf,st}}{Q_{in,st}}$	$M_{c,st}\frac{Q_{ovf,st}}{V_{st}}$	$X_{c,in,st}\frac{Q_{ovf,st}}{Q_{in,st}}$

*Note that X stands for pollutant load

Figure 10 presents the behavior of an *on-line* (ST$_6$) (a) and an *off-line* (ST$_4$) (b) storage tank model. In the case of *on-line tanks* (**Figure 10a**) simulations show that the outflow ($Q_{out,st}$) varies based on the tank volume (V_{st}). Another possibility is restricting the outflow with valves. **Figure 10b** shows the dynamics of an *off-line tank.* In this particular case, V_{st} and $Q_{ovf,st}$ are determined by $Q_{pump,st.}$ and $Q_{in,st.}$. The pumps are modelled in such a way that they turn on only during periods when there is no inflow to the off-line storage tank.
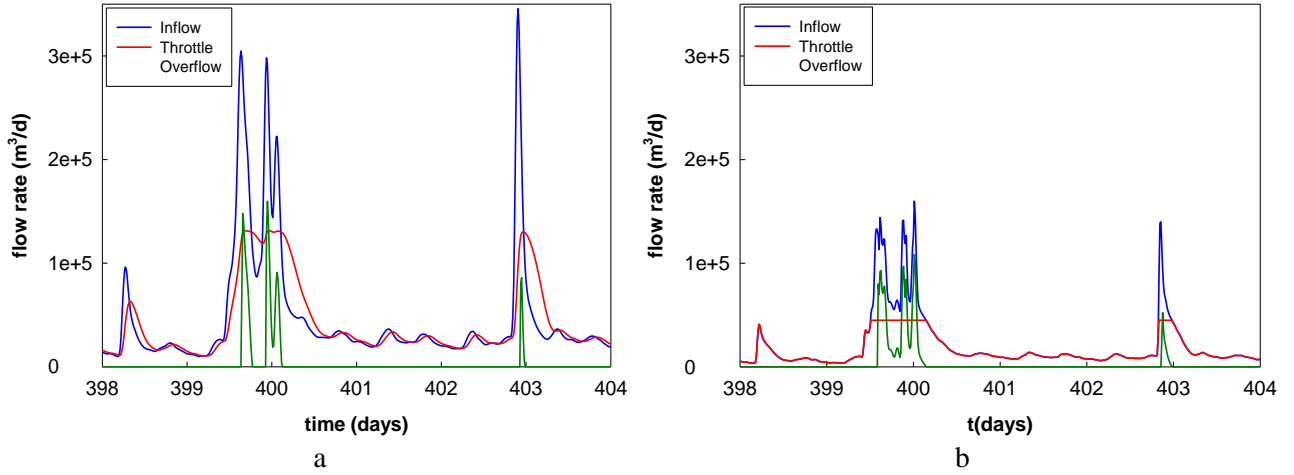
Figure 10: Effect of different configurations of the storage tanks on throttle (to the sewer) and overflow: 1) On-line (a); and, 2) Off-line (b).

5. EVALUATION CRITERIA

The following evaluation criteria are used for studying the behaviour of the system and the effects of various control strategies/system modifications on its performance. The evaluation considers various overflow locations in the sewer system and also the overflow at the WWTP bypass. Subscript "i" denotes the criteria for a specific overflow location.

1. Yearly overflow frequency ($N_{ovf,i}$) (events/year)*:* The total number of overflow events per year occurring at a given overflow location. Two overflow events that are separated by less than one hour duration are considered as a single event.

2. Yearly overflow duration ($T_{ovf,i}$) (days/year): This criterion represents the cumulative sum of overflow duration for all overflow events at one specific location (see **Equation 12**). Assuming that the simulation is run for *y* years, for *n* overflow events each with a time *t(n)*, the yearly overflow duration ($T_{ovf,i}$) is :

$$T_{ovf,i} = \frac{1}{y}\sum_{j=1}^{n} t(j)$$ **Eq. 12**

3. Yearly overflow volume ($V_{ovf,i}$) (m^3/year): The total volume of wastewater discharged into receiving waters from a particular overflow location (see **Equation 13**). Assuming that the simulation is run for *y* years, for *n* overflow events each with a duration *t(n)* (starting at time $t_o(n)$ and ending at time $t_e(n)$) and flow rate *Q(t)*, the total overflow volume (m^3) ($V_{ovf,i}$) is:

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$$V_{\mathrm{ovf,i}} = \frac{1}{y} \sum_{j=1}^{n} \int_{t_0(j)}^{t_e(j)} Q(t)\, dt \qquad\qquad \textbf{Eq. 13}$$

4. *Yearly overflow pollutant load* ($X_{\mathrm{ovf(c),i}}$) (kg/year): This represents the total load in the overflow for a given pollutant $X_c(t)$ at any given overflow location (see **Equation 14**). Assuming that the simulation is run for *y* years, for *n* overflow events each with a duration *t(n)* (starting at time $t_o(n)$ and ending at time $t_e(n)$).

$$X_{\mathrm{ovf(c),i}} = \frac{1}{y} \sum_{j=1}^{n} \int_{t_0(j)}^{t_e(j)} X_c(t)\, dt \qquad\qquad \textbf{Eq. 14}$$

5. *Overflow quality index* (*OQI*$_i$) (kg pollution units/day): It is an aggregated pollution index representing the daily total pollution arising from an overflow during a determined period of time (*t*). *OQI* gives an indication of the overall daily pollutant load by assigning weights to individual pollutant loads (see **Equation 15**). It is defined in a similar fashion as the effluent quality index (*EQI*) for BSM WWTPs. The influent fractionation proposed by Gernaey et al. (2011) converts the pollution load into ASM state variables in order to further calculate the different types of analytical variables (BOD, COD, TSS, TKN, NO_3^- and PO_4^{-3}). The weights for these compounds are w_{BOD}, w_{COD}, w_{TSS}, w_{TKN}, w_{NO3} and w_{PO4} respectively. The values for the weights are similar to those used in BSM2. Identical weights are used in order to be able to compare the effect of WWTP effluent discharges and CSOs.

$$OQI_{\mathrm{i}} = \frac{1}{t} \int_0^t [w_{\mathrm{BOD}} X_{\mathrm{ovf(BOD),i}}(t) + w_{\mathrm{COD}} X_{\mathrm{ovf(COD),i}}(t) + w_{\mathrm{TSS}} X_{\mathrm{ovf(TSS),i}}(t) +$$
$$w_{\mathrm{TKN}} X_{\mathrm{ovf(TKN),i}}(t) + w_{\mathrm{NO_3}} X_{\mathrm{ovf(NO_3),i}}(t) + w_{\mathrm{PO_4}} X_{\mathrm{ovf,(PO_4),i}}(t)]\, dt \qquad \textbf{Eq. 15}$$

6. *Yearly exceedance duration* ($T_{exc(c),i}$): It is the total duration per year for which a certain pollutant concentration exceeds a specified concentration threshold (C_{th}). Therefore, for a particular overflow event *n*, with concentration of a particular pollutant *C(t)*, the exceedance duration for the event *n* and the pollutant *c* ($t_{exe}(c,n)$) and for all the events occurring in *y* years at an overflow location, ($T_{exc(c),i}$) is defined as stated in **Equations 16** and **17**. The threshold concentrations (at various overflow locations) for TSS, TKN and PO_4 used in this study are 30 g/m^3, 5 g/m^3 and 0.5 g/m^3, respectively. It should be noted that these values are similar to the effluent discharge limits for BSM WWTPs. PO_4 is included although it is not toxic. It is due to the fact that the excess PO_4 can lead to eutrophication (especially in rivers with phosphorus limitation) and therefore depletion in oxygen concentration.

$$t_{\mathrm{exc,i}}(c,n) = \sum \{(t+1) - t\} \qquad when\ C(t) > C_{\mathrm{th}} \qquad\qquad \textbf{Eq. 16}$$

$$T_{\mathrm{exc(c),i}} = \frac{1}{y} \sum_{j=1}^{n} t_{\mathrm{exc,i}}(c,j) \qquad\qquad \textbf{Eq. 17}$$

7. *Hourly maximum concentration* ($C_{max(c),i}$): Maximum exceedance values for a certain concentration are defined for a specific time interval. In this study, 1-hour maximum exceedance is used. It is the highest

concentration that is continuously discharged for a period of at least 1 hour. Similarly, maximum concentrations for 2-hour, 6-hour time periods etc. can be defined.

The above criteria can be classified in two different ways based on: 1) location; and, 2) impact. In terms of location, the criteria can be defined on a: 1) local level (for each overflow location i); and, 2) global level (taking into account all the overflows and the bypass at the WWTP). From an impact perspective, the criteria are divided into those describing: 1) cumulative effects (N_{ovf}, T_{ovf}, V_{ovf}, X_{ovf} and OQI); and, 2) acute effects ($T_{exc(c)}$ and $C_{max(c)}$) on the receiving waters. These criteria are only an indirect representation of the effect of overflow discharges on receiving waters. They draw inspiration from similar criteria used in assessment of river water quality (Schutze et al., 2002; FWR, 1998). To consider pollutant quality in the sewer system evaluation, we used these additional criteria even though they are not commonly encountered in CSO evaluation literature. In this paper, the evaluation criteria $T_{exc(c)}$ and $C_{max(c)}$ are applied only to TKN in order to limit the number of evaluation criteria.

6. CASE STUDIES

This section presents simulation results from implementing different scenarios using the catchment and sewer network model (see **Table 3**). The evaluated control alternatives employ storage tanks as control handles. The control actuators are generally valves/gates/pumps that regulate the outflow from these storage tanks. Examples of the evaluation of both local and global (sewer & catchment system) control strategies are presented here. The strategies are:

- Reducing the bypass at the WWTP (C1);
- Reducing the total overflows from the system (C2).

Apart from evaluation of control strategies, the presented model can also be used to study the influence of structural modifications of the sewer network/catchment. To demonstrate this, two possibilities are implemented and their effects are analyzed:

- Modification of SC_5 from a separate sewer system to a combined sewer system (S1);
- Inclusion of an additional storage tank at the WWTP influent (S2).

The following section describes the effects of each of these evaluated alternatives from a global and local perspective with the set of criteria defined in Section 5.

Table 3. Summary of the global evaluation criteria for the different scenarios. No control (NC); C1 and C2 are the control strategies. S1 and S2 are the scenarios with structural modifications.

Criteria	NC	C1	C2	S1	S2
Cumulative effects					
N_{ovf} (events/year)	137	142	141	82	137
T_{ovf} (days/year)	71	71	71	21	71
V_{ovf} (m^3/year)	830,192	654,724	642,846	722,650	678,055
OQI (kg pollutant units/day)	3,110	2,118	2,068	2,937	2,076
Acute effects					

$T_{exc(TKN)}$ (days/year)	49.0	50.7	50.6	20.3	47.6
$C_{max(TKN)}$ (g/m^3)	51.1	51.1	51.1	48.8	51.1

6.1. Reducing the bypass at the WWTP (C1)

The existing configuration of the BSM2 layout includes a bypass at the inlet of the WWTP which redirects any excess inflow reaching the plant (inflow $>$ 60,000 m^3/d) to the effluent section where it is mixed with the treated wastewater (Gernaey et al., 2014). Storage tank 6 (ST_6) is located upstream of the WWTP. A rule based strategy (*control algorithm*) is developed to better utilize the available storage volume in ST_6. The sensor inputs (*measured variable*) to the control strategy are: 1) flow rate at ST_6 influent; and, 2) level measurement from ST_6 (max. level is 5 m). When the inflow to ST_6 exceeds 60,000 m^3/d and there is storage capacity available (level $<$ 4 m), the outflow from the tank is restricted using a valve (*control variable*). The valve opening is reduced to 65% under these conditions. In other situations, the valve is fully open. The reduced valve opening will lead to more storage and hence a better utilization of the tank capacity. As the tank is reaching its maximum capacity (h $>$ 4 m), the valve is fully opened so that the control will not lead to excess overflow at ST_6 while trying to reduce the bypass at the WWTP.

Table 4. Summary of the local evaluation criteria at ST_6 and bypass for the scenario C1.

Criteria	ST_6		Bypass	
	NC	C1	NC	C1
Cumulative effects				
N_{ovf} (events/year)	5	8	79	75
T_{ovf} (days/year)	0.6	0.9	18	21
V_{ovf} (m^3/year)	21,379	32,870	473,341	286,381
OQI (kg pollutant units/day)	32	67	2,072	1,045
Acute effects				
$T_{exc(TKN)}$ (days/year)	0.3	0.7	17.2	18.8
$C_{max(TKN)}$ (g/m^3)	8.2	12.2	47.8	47.5

Table 4 compares the evaluation criteria at ST_6 (overflow) and bypass (BP). Results show that the *yearly overflow frequency* ($N_{ovf, SC6}$) at ST_6 increased while it reduced at the bypass ($N_{ovf, bp}$). *Yearly overflow duration* shows an increase at both the locations ($T_{ovf,SC6}$, $T_{ovf,bp}$). The major outcome from the control is an improvement in both *yearly overflow volume* ($V_{ovf,bp}$) (39%) and *overflow quality index* (OQI_{bp}) (50%) at the bypass. The improvements at the bypass led to a drop in performance at ST_6. Thus, *yearly overflow volume* increased by 54% ($V_{ovf,SC6}$) and the *overflow quality index* (OQI_{SC6}) increased significantly by 110% at ST_6. The above criteria describing the cumulative effects indicate an improvement at the bypass at the cost of decreased performance at ST_6. Additionally, the effect of the control strategy is also analyzed using criteria that describe acute effects. *Yearly exceedance duration* for TKN ($T_{exc(TKN),bp}$, $T_{exc(TKN),ST6}$) at both locations increased due to the control strategy. *Hourly maximum concentration* for TKN remains almost similar at the bypass ($C_{max(TKN),bp}$) while increasing at ST_6 ($C_{max(TKN),ST6}$). From a global point of view, **Table 3** reveals that C1 has led to a decrease in the *yearly overflow volume* (V_{ovf}) discharged into the receiving water by 21%. Also, the *overflow quality index (OQI)* was reduced by 32%. The control strategy did not have any major

impact on the acute effects ($T_{\text{exc(TKN)}}$, $C_{\text{max(TKN)}}$). Summarizing, it can be said that C1 successfully decreased the cumulative pollutant load to the receiving water but was not effective in handling critical situations.

6.2. Reducing the total overflows from the system (C2)

In order to utilize the available storage capacity in a better way, several local control strategies similar to the one employed in Section 6.1 (C1) are implemented at all storage locations with overflow structures (see **Figure 1**). For ST_2, ST_3 and ST_6, the *measured variables* are water levels from the respective tanks. If the level is less than 4 m (max level = 5 m), the valve opening is reduced to 65%. It is otherwise fully opened (*control algorithm*). For ST_4, which is an off-line tank, the throttle flow to the main sewer (wastewater with flow rate in excess of this is directed to ST_4) is controlled based on the water level measurement (*control variable*). If the level in ST_4 is less than 4 m, the throttle flow is 40,000 m^3/d, which means that any flow in excess of 40,000 m^3/d reaches the storage tank. When the tank is filled above a level of 4 m, the throttle flow is increased to 55,000 m^3/d to allow passing more wastewater through the main sewer. Hence, the algorithm tries to send more water downstream than in the no control case (45,000 m^3/d). This is an example of various non-interacting local control strategies developed with an overall aim to reduce the cumulative overflow volume/load.
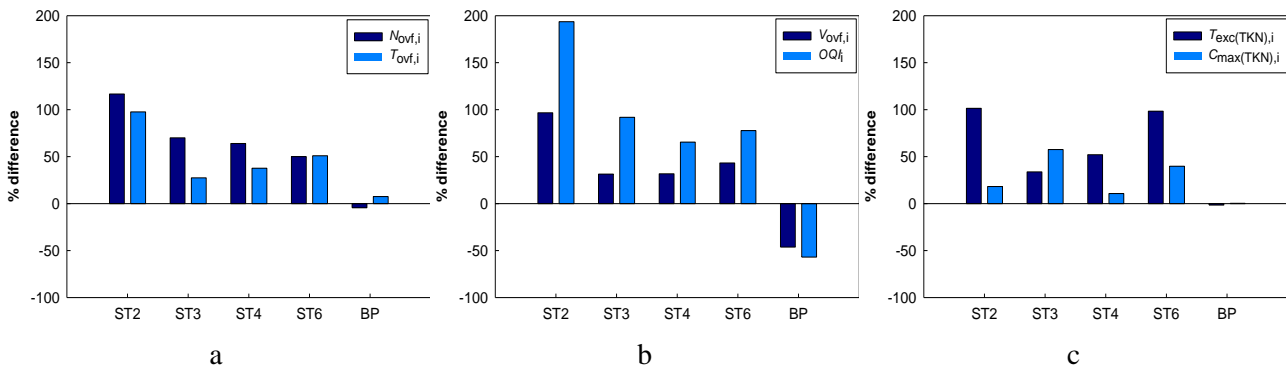
Figure 11: Evaluation of various performance criteria comparing the default case (NC) with the global control strategy (C2). The percentage difference in performance between NC and C2 is shown, evaluated for the criteria: a) $N_{\text{ovf,i}}$ and $T_{\text{ovf,i}}$; b) $V_{\text{ovf,i}}$ and OQI_i; and, c) $T_{\text{exc(TKN),i}}$ and $C_{\text{max(TKN),i}}$ in various storage tanks (ST2, ST3, ST4, ST6) and the bypass (BP).

The implementation of C2 has led to mixed results (**Figure 11**) at local level. The performance at ST_2, ST_3, ST_4 and ST_6 dropped for all evaluation criteria. The only location that showed improvement is the bypass (BP). At the bypass, criteria that showed major improvements are *yearly overflow volume* (46%) ($V_{\text{ovf,,bp}}$) and the OQI_{bp} (57%). The acute effects at the bypass did not change much due to the control. Looking at the entire system (see **Table 3**), with an improved utilization of the available storage (C2), a drop in *the yearly overflow volume* (23%) (V_{ovf}) and *overflow quality index* (34%) (OQI) is observed while there is no major change in the acute effects ($C_{\text{max;TKN}}$, $T_{\text{exc,TKN}}$). Although, the control led to lower overall quality in comparison to the default situation at many overflow locations, it had a net positive effect on the entire system. The results obtained from the global control strategy are very similar to those obtained from the

control strategy described in 6.1 (C1). This is due to the fact that overflow at ST_6 and the bypass are the major contributors to the total overflow from the system. In fact, it can be said that the improvement observed at the bypass lead to an overall improvement of the system performance even though the other overflow locations underperformed in comparison to the default case. Also, it should be noted that there are a large number of variables that are chosen by trial and error for this control strategy (e.g. valve opening for ST_2, ST_3, ST_4 and ST_6, throttle flow for ST_4 etc.). A more sophisticated optimization procedure can potentially lead to better results.

6.3. Modification of SC_5 from a separate sewer system to a combined sewer system (S1)

During the evaluation of C1 and C2, it was noticed that due to the existence of a separate sewer system at SC_5, any stormwater in SC_5 eventually reaches the river. This means that all rain events lead to an overflow at ST_5 as they cannot be redirected to the WWTP as in the case of a combined sewer system. A possible modification to the system is to convert SC_5 to a combined sewer system which will lead to reduction in the overflow volume/load from SC_5 and hence potentially improve the overall system behavior. It is assumed that the volume of the storage tank remains unchanged.

Table 5. Summary of the local evaluation criteria at ST_5 for scenario S1.

Criteria	NC	S1
Cumulative effects		
N_{ovf} (events/year)	134	2
T_{ovf} (days/year)	71	0
V_{ovf} (m^3/year)	268,821	2,132
OQI (kg pollutant units/day)	864	2
Acute effects		
$T_{exc(TKN)}$ (days/year)	40.7	0.0
$C_{max(TKN)}$ (g/m^3)	51.1	2.9

Table 5 shows that at ST_5, the improvements are very clearly visible. The change, as expected, led to orders of magnitude difference in all the evaluation criteria at the local level. Given that there are only two overflow events after the system modification is done, the *overflow quality index* (OQI_{ST5}) has also dropped significantly from 864 kg pollutant units/day to only 2 kg pollutant units/day. Also, as can be noticed, the acute effects improved significantly. The *yearly exceedance duration* ($T_{exc(TKN),ST5}$) and *hourly maximum concentration* ($C_{max(TKN),ST5}$) declined considerably (100% and 94% respectively). The results when looked at from a system-wide perspective show the influence of ST_5 on the overall performance (see **Table 3**). As expected, it led to a significant drop in the *yearly overflow frequency* ($N_{ovf,}$) (41%) and *yearly overflow duration* (T_{ovf}) (71%). This is expected as the separate sewer system (that discharges into the river for all rain events) is now modified into a combined sewerage where the discharges happen only if the storage capacity in ST5 is exceeded (2 events/year in this case). Although, there is a drop in *yearly overflow volume* (V_{ovf}) (13%), the discharges at ST5 and downstream are now more polluted due to mixing with the domestic wastewater from SC5. A significant drop in *OQI* is observed at ST5, but this does not lead to overall improvement in *OQI*. This is due to the fact that the discharges are now happening elsewhere (at ST6 and the

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bypass). Hence, the *overflow quality index (OQI)* has only improved by 6%. The changes also caused major improvements to the acute criteria. *Yearly exceedance duration* and *hourly maximum concentration* for TKN ($T_{exc(TKN)}$, $C_{max(TKN)}$) improved by 59% and 5% respectively. Hence, the system modification can be analyzed at various levels. In terms of its impact on the *local* overflow performance, the improvement is phenomenal. From an overall point of view, the changes did lead to major improvements but the improvements at SC_5 due to the change are masked by the overall system performance. Also it should be noted that such a change can be detrimental to the WWTP performance, especially if the WWTP is operating at its maximum capacity or the area is prone to heavy rainfall events.

6.4. Inclusion of an additional storage tank at the WWTP influent (S2)

The last evaluated scenario studies the impact of including an additional storage tank at the BSM2 WWTP influent. Hence, the system configuration is modified by including an on-line pass-through tank with pump at the WWTP inlet. The volume of the storage tank is 8,000 m^3. The additional storage tank is aimed at reducing the bypass at the WWTP.

Again, the storage tank has resulted in considerable improvements in all the evaluation criteria at the local level (see **Table 6**). At the bypass location, the effect of additional storage is clearly visible on the criteria for cumulative effects. Drops in *yearly overflow frequency* (56%) and *yearly overflow duration* (46%) are observed ($N_{ovf,bp}$, $T_{ovf;bp}$). The *yearly overflow volume* ($V_{ovf;bp}$) and the *overflow quality index (OQI$_{bp}$)* are reduced by 32% and 50%, respectively. The storage tank addition was also successful in decreasing the acute effects described by *yearly exceedance duration* and *hourly maximum concentration* for TKN ($T_{exc(TKN),bp}$, $C_{max(TKN),bp}$) as the tank helps in equalizing the incoming pollutant load and hence reduces the high concentration peaks. While comparing the changes in the performance of the entire system (see **Table 3**), the storage tank has not made any major changes to the *yearly overflow frequency* (N_{ovf}) and *yearly overflow duration* (T_{ovf}) as it is not the location with the highest duration and frequency in the default case. An 18% drop in the overall *yearly overflow volume* (V_{ovf}) and a 33% decrease in system-wide *overflow quality index (OQI)* are noticed. The modification also marginally decreases the *yearly exceedance duration* for TKN ($T_{exc(TKN)}$) by 3% indicating that the bypass location was one of the main contributors to the high concentration loads. In terms of *hourly maximum concentration* ($C_{max(TKN)}$), no changes are observed as the maximum concentration events are not occurring at the bypass. Finally, it can be said that the storage tank was useful in equalizing the incoming pollutants and acts as a buffer to store additional wastewater during rain events. In spite of the high costs involved in addition of a storage tank at the WWTP influent, the overall performance improvement from such a system modification is similar to that from the control modifications. This is due to the fact that the effect of C1, C2 and S2 is similar. They all lead to reduced overflows from the bypass. While the control strategies achieve this by modifying the operation of upstream storage tanks, the structural modification S2 does this by including additional storage. Also, C1, C2 and S2 were not successful in reducing the overall overflow frequency and duration (N_{ovf}, T_{ovf}). As SC5 is the major reason for high N_{ovf}

and T_{ovf} (as this is a separate sewer system and all rain events will lead to an overflow), only S1 is successful in reducing N_{ovf} and T_{ovf} whereas other strategies could reduce V_{ovf} and *OQI* as they try to reduce the total overflow volumes.

Table 6. Summary of the local evaluation criteria at the bypass for scenario S2.

Criteria	NC	S2
Cumulative effects		
N_{ovf} (events/year)	79	35
T_{ovf} (days/year)	18	10
V_{ovf} (m^3/year)	473,341	321,204
OQI (kg pollutant units/day)	2,072	1,037
Acute effects		
$T_{exc(TKN)}$ (days/year)	17.2	8.2
$C_{max(TKN)}$ (g/m^3)	47.8	31.7

7. DISCUSSION

The catchment and sewer extension to the BSM WWTP model has been described in detail in this paper. The model has successfully described the dynamics of wastewater generation from various sources (domestic, industrial) during dry weather and rain periods. Additionally, infiltration to the sewers is also included. A sewer network model that can simulate the transport of the generated wastewater has been implemented. The model can also describe the first flush of the particulate (sewer) pollutants during rain events. Models for different storage tank configurations together with control actuators, such as valves and pumps, are described. Overflow based evaluation criteria have been defined and are used to evaluate the performance of control strategies and structural modifications. Finally, the suitability of the catchment and sewer extension to describe the dynamics of wastewater generation and transport as well as objective evaluation of control strategies has been successfully demonstrated. These case studies are only illustrative and do not represent any possible strategies that can be replicated in real catchments. The focus has been on demonstrating the capabilities of the model.

In general, benchmarking tools are developed for the evaluation of control strategies for a defined system layout. In the case of WWTP benchmark models, these models are employed not only for control strategy evaluation but are also extensively used for other purposes like model development, diagnosis, monitoring etc. (Gernaey et al., 2014). In a similar fashion, the spatial extension of the benchmark system can also be employed to develop and evaluate control strategies and structural modifications as illustrated by the case studies. Additional scenarios like adapting the benchmarking tool to a particular catchment and evaluating scenarios specific to any individual urban catchment are also possible.

7.1 Benchmark system layout

The system layout presented here is an upscaled version of the ATV case study and very similar to the layout used in the studies carried out in Schutze et al. (2002). Through various discussions at different stages of the development of this model, it has been clear that the sewer system layouts vary considerably across different

urban catchments and in different countries. It is unlikely that any proposed sewer layout will closely resemble a majority of the sewer system layouts. Hence, the focus in this work has therefore been on choosing a reasonable system layout, with the purpose of providing a framework for the evaluation of control strategies. Although, the variation in layout will influence the performance of control strategies, the control schemes identified using the benchmark model can potentially be transferred to other layouts. Nevertheless, we plan to work in different directions to address this issue in the future: 1) Presenting more than one benchmark layout; and, 2) Comparing the performance of control strategies on the benchmark layout with that on actual catchment layouts. This will provide us with additional insight on the extent to which knowledge derived from the extended BSM layout can be used to address issues in other urban catchments.

7.2. Adaptation to other catchments

As in the case of BSM1 and BSM2, many users might be interested in adapting the extensions to their catchment layouts. It is for this purpose that the model building is performed in a block-wise manner making it easy for future users to adapt model blocks for any specific system layout. The first step in the process will be modifying the catchment layout. The major sections that will need modification (apart from modifying the layout) are influent dynamics, sewer reservoirs and storage tank characteristics. A list of key parameters required to be adapted are available in **Appendix 1**. Although users have the choice of using commercial softwares for this purpose, the main advantages of these extensions are that it is a complete toolbox (comprising of a system layout, underlying models and evaluation criteria) that is: 1) flexible for adaptations; 2) freely distributed; and, 3) open source (which means users can look into the code and even modify it, if required).

7.3. Model limitations

However, owing to the conceptual approach used for modelling the sewer network and other hydraulic elements, the model has some limitations. It is not suitable to evaluate scenarios where phenomena like pressurized flow, backwater effects and surface flooding are prominent. Also, biological reactions within the sewer system are not yet considered (Huisman, 2001). The transport and accumulation of particulate pollutants is dealt with in a simplified way. Additionally, the rainfall generator model is also limited in its ability to reproduce extreme rain events. Hence, the rainfall generator is more suitable for evaluating control applications rather than performing studies that are more specific to high intensity rainfall.

7.4. Future directions

The current paper mainly deals with sewer overflows.. It is well established that any integrated evaluation of the urban wastewater systems should be focused on improving the receiving water quality. Although the current evaluation criteria give an indirect indication on the impact of sewer overflows on river water quality, a direct river quality based evaluation will be a more preferable approach. For such an analysis, the benchmark system extension discussed here should be combined with a river water quality model (RWQM1) (Reichert et al., 2001) and also be integrated with the BSM family of WWTP models. River quality based

evaluation criteria should be developed. This paper is the first attempt at developing spatial extensions to the BSM platform, and more work is in progress in the direction of integrating the model with a WWTP and river system.

With respect to the control strategies and system modifications presented as case studies, it is essential to highlight the fact that the results also depend to a great extent on parameters like valve opening for on-line tanks, throttle flow for off-line tanks and the level and flow rate values that act as inputs to these rule-based control strategies. Mathematical optimization procedures can play a major rule in identifying the most suitable set points in such cases (Fu et al., 2008). Other options that are not evaluated in this case study are changes to the catchment characteristics. For example: 1) restricting industries not to have peak loads on Fridays; and, 2) addition of a seventh sub-catchment to the system etc. The effect of such changes on wastewater generation and its subsequent impact on sewer dynamics can be analyzed.

Last but not least is the interfacing between water quality models for different sub systems. Since, the catchment and the sewer models use the same variables, there is no need for interfacing between them. The interface between sewer and WWTP is performed using the elemental balancing approach proposed by Volcke et al. (2006) and Grau et al. (2007). As the elemental composition based approach was originally proposed in the RWQM1, future interfaces between sewer/river and WWTP/river will also use the same approach.

8. CONCLUSIONS

The presented model will enable practitioners/researchers to evaluate integrated control strategies/structural modifications (within catchment and sewer system) using overflow based evaluation criteria. The key findings of the presented study can be summarized in the following points:

1) The catchment model is capable of generating (dry/ wet weather) flow rate and pollution loads (soluble/particulate) through the combination of four different sub-models (*DOM*, *IND*, *INF*, *SW*). These sub-models contribute to the total wastewater profile with different types of dynamics.

2) The sewer model can mimic wastewater transport and storage using three different sub-models (*TRANSPORT*, *FIRST FLUSH* and *STORAGE*). These models account for sewer length, a sudden increase of particulates at the start of a rain event and wastewater storage to avoid combined sewer overflows.

3) A set of evaluation criteria are proposed to assess the (cumulative/acute) effects of different control strategies on both local and global level for different overflow locations. The cumulative effects are evaluated in terms of overflow frequency, duration, volume and loads. The acute effects are indicated using the criteria of exceedance duration and hourly maximum concentration for TKN.

4) Case studies highlighting the potential applications of the framework by implementing control strategies (local and global) and structural modifications (in both the catchment and sewer network) are presented. Varying levels of performance improvement are observed in these scenarios.

The model is an important contribution to the wastewater engineering field, especially in the direction of developing systematic procedures to evaluate "outside the fence" control strategies and potentially to be combined with existing and successful wastewater treatment plant evaluation models. Work is in progress to extend this model further to include a river system as well. This will in the future result in a complete system-wide UWS benchmark simulation model for analysis of integrated control strategies.

9. ACKNOWLEDGEMENTS

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