Benchmark Simulation Model for Integrated Urban Wastewater Systems

Model Development and Control Strategy Evaluation

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Integrated Urban Wastewater Systems

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LUND UNIVERSITY

Thesis for the degree of Doctor of Philosophy in Engineering

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To be presented, with the permission of the Faculty of Engineering of Lund University, for public criticism in the M:B lecture hall, Mechanical Engineering building, Ole Römers väg 1, Lund on Friday the 9th of June 2017 at 10:15 a.m.

Organization	Document name
LUND UNIVERSITY	**DOCTORAL DISSERTATION**
Faculty of Engineering Department of Biomedical Engineering Division of Industrial Electrical Engineering and Automation	Date of disputation 2017 June 09
Author Ramesh Saagi	Sponsoring organization EU SANITAS, Lund University

Title and subtitle

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Abstract

The integrated urban wastewater system (UWS) consists of different sections that are interconnected. These include: i) catchment; ii) sewer network; iii) wastewater treatment plant (WWTP); and finally, iv) receiving water system. Traditionally, these sections are operated and evaluated individually. However, it is now well-established that all the sections of an UWS should be operated in a holistic manner in order to improve the receiving water quality.

The thesis aims at developing an integrated model library that can be used to simulate the dynamics of flow rate and pollutant loads in all the sections of an UWS on a single simulation platform. It is further aimed at defining a hypothetical UWS using the model library, so that future users can use the pre-defined layout to study multiple control strategies and structural modifications. In order to facilitate an objective evaluation of the results, criteria for evaluating river water quality as well as the sewer network and WWTP performance are described.

Firstly, the suitability of existing model libraries is assessed. The building blocks from the Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) are used as the starting point for the catchment and sewer network model library. Additional model blocks that are missing in the DIPDSG are developed. A modified version of the Benchmark Simulation Model No. 2 (BSM2) is used to simulate the WWTP. Model blocks for river water quality assessment are developed.

Using the model library, a hypothetical UWS for an urban catchment with 80 000 population equivalents and an area of 540 hectares is described. The UWS layout is used to develop and evaluate different control strategies (local/integrated) and structural modifications. The case studies indicate that: i) the presented model library and the layout can be used to develop various control strategies and evaluate their impact on river water quality; and ii) improving the performance of an individual section does not necessarily lead to better river water quality. It is expected that the model library will be widely used as an open-source software toolbox for benchmarking purposes, integrated modelling studies as well as for modelling the individual sections.

Key words:

Benchmark Simulation Model, Integrated control strategies, Integrated modelling, River water quality

Classification system and/or index terms (if any)

Supplementary bibliographical information		Language: English
ISSN and key title		ISBN 978-91-88934-81-9 (print) 978-91-88934-80-2 (pdf)
Recipient's notes	Number of pages 208	Price
	Security classification	

Signature __________ Date 2017 May 03

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Cover illustration front: Eureka by Ramesh Saagi
Cover illustration back: Author's portrait

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Box 118
SE-22100 Lund
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ISBN: 978-91-88934-81-9 (print)
ISBN: 978-91-88934-80-2 (pdf)
CODEN: LUTEDX/(TEIE-1083)/1-208/(2017)

Printed in Sweden by Media-Tryck, Lund University, Lund 2017

Contents

Acknowledgements

I landed in this beautiful country on a nice summer day in June 2012. **Dr. Ulf Jeppsson** picks me up from my room on my first day to work, gives a tour of the campus before taking me to the department. The journey continued for five long years and it has been fantastic. **Ulf**, thank you for letting me be a part of the wonderful Water group at IEA. Your academic rigour, leadership qualities and immense kindness influenced me greatly. I enjoyed the many tours to DTU (and detours), fika meetings and history lessons. It was great fun and a rewarding learning experience working with you.

Dr. Xavier Flores-Alsina, thank you for the timely and close supervision of my Ph.D. If I learned anything about writing research papers, the biggest credit for that goes to you. The way you make a fantastic story out of boring numbers and data always amazes me. **Prof. Krist Gernaey**, you are the coolest and the most easy going professor that I have ever met. Thank you for taking time out of your busy schedule to give valuable inputs and read through all the research papers and the thesis.

I was fortunate to be part of an excellent research project (SANITAS) and I thank the coordinator **Dr. Joaquim Comas** and everyone else involved in it. I enjoyed all the SANITAS meetings and the evenings after the meetings. A special mention here goes to **Dr. Laura Snip**, my BSM buddy from DTU. We have progressed almost in parallel in our professional and personal lives. I cherish all the casual talks and also research discussions during our monthly meetings. **Prof. David Butler** and **Dr. Guangtao Fu** at the University of Exeter and **Dr. Stefan Weijers** at Waterboard de Dommel, thank you for hosting me for my secondments. **Dr. Fanlin Meng**, thanks for the nice time and all the help at Exeter. **Dr. Antonia Hadjimichael**, it was nice to have a SANITAS friend during my stay at Waterborad de Dommel. All the lunch times and train journeys will be cherished. **Mr. Stefan Kroll**, it was an excellent research collaboration that we have had and I will look forward to continue working together in the future. **Prof. Peter Vanrolleghem**, thank you for coming to Sweden during summer time and for all the nice interactions and inspiration.

Dr. Kimberly Solon (by the time this thesis is printed, you will be Dr.), we started almost at the same time and we are also finishing together (although, you finish a few weeks before). Thank you for the great company at the workplace, lunch table and also during all our conference trips and other travels. You also made my writing job easier. Remember that we still need to publish something together and please turn your handwriting into a font ☺. My other Water group colleague **Dr. Magnus Arnell**, it was nice to have you here at the department (although, rarely) and I enjoyed your company. Also, special thanks for the

excellent thesis that served as a guideline for my writing. **Prof. em. Gustaf Olsson**, the kindness and compassion that you showed towards me will always be remembered. I especially cherish the dinner we had at Exeter together with Fanlin. That was a great privilege. **Dr. Erik Lindblom**, I look forward to some fruitful collaborations and good times in the future. Thanks to **Dr. Gunnar Lindstedt** for helping with all the IT things and also for your support during the automation labs. **Mr. Getachew Darge**, thanks for being kind and helpful in fixing all the issues in the automation lab. I thank all my IEA colleagues for making this workplace fun to work and I promise to interact more with everyone else. Also, special thanks to **Dr. Lorenzo Benedetti** for the interactions about integrated modelling and KALLISTO project. Thanks to **Dr. Dines Thornberg** and **Mr. Carsten Thirsing** for giving us the data that is essential for the modelling work. **Dr. Bertrand Vallet**, it was nice to have you here in Lund for your secondment. I received some valuable modelling advice and also nice company and food.

A special thanks to all my teachers from schooling to post graduation. Even though I am not in touch with most of you now, I have immense gratitude to all my **teachers** at **ZINC School, ANITS** and **University of Stuttgart**.

Next comes friends. Here in Sweden, a small and nice Indian community has helped me stay sane outside work. Firstly, my roommates at different times, **Ravi, Pradeep, Vignesh** and **Prem**, I cannot lie that we all cooked great food, but we certainly had great company. Thanks **Gopi** for all the summer travels and tasty dinners. **Ramesh, Praveen** and **Ravi**, we keep bumping into each other every now and then mostly due to our kids and families. Thanks for your company and support. Special thanks to **Saritha, Sravani, Sunitha** and **Madhavi**, for your friendship with Swetha. **Bharat**, **Pradeep** and **Kavya**, having friends from your hometown is always nice. Thank you for all the help and your company for the trips. **Sunny**, **Avinash, Dinesh, Satish, Aaron** and **Karunakar**, I enjoyed playing badminton, poker and other skill games with you.

My friends from Germany, comrade **Abhinay**, **Krishna** and **Shelesh**. It feels silly to say thank you or praise you, so I will not do that ☺. My two year stay in Pune provided me with some cherished friendships (**Rajeev, Vinod, Rakesh, Preeth** and **Mousam**) and innumerable fun moments. I learned a lot during this phase and special thanks to everyone. My **college** and **school friends** are just too many to name here. I am in touch with most of you now and you will always remain special.

If I have to thank all my **family members**, that will turn into another thesis. In every success of mine, there is a lot of support from each one of you. **Swetha** and the new member of the family (**Shaurya**), you will have to bear with me for a very long time and will not get any thanks for that ☺.

List of Publications and Author's Contributions

This thesis is a summary of the following publications:

I. **Calibration and validation of a phenomenological influent pollutant disturbance scenario generator using full-scale data** (impact factor 2015 – 5.99)

   Flores-Alsina, X., **Saagi, R.**, Lindblom, E., Thirsing, C., Thornberg, D., Gernaey, K.V. and Jeppsson, U.

   Water Research, 2014, 51, 172-185.

II. **Benchmarking integrated control strategies using an extended BSM2 platform**

   Saagi, R., Flores-Alsina, X., Fu, G., Benedetti, L., Gernaey, K.V., Jeppsson, U. and Butler, D.

   13th International Conference on Urban Drainage, Kuching, Malaysia, 7-12 September 2014.

III. **Catchment & sewer network simulation model to benchmark control strategies within urban wastewater systems** (impact factor 2015 – 4.20)

   Saagi, R., Flores-Alsina, X., Fu, G., Butler, D., Gernaey, K.V. and Jeppsson, U.

   Environmental Modelling & Software, 2016, 78, 16-30.

IV. **A model library for simulation and benchmarking of integrated urban wastewater systems** (impact factor 2015 – 4.20)

   Saagi, R., Flores-Alsina, X., Kroll, S., Gernaey, K.V. and Jeppsson, U.

   Environmental Modelling & Software, 2017, 93, 282-295.

V. **Key control handles and design parameters for improving receiving water quality**

   Saagi, R., Kroll, S., Flores-Alsina, X., Gernaey, K.V. and Jeppsson, U.

   Manuscript.

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Author's Contributions

Paper I: Calibration and validation of a phenomenological influent pollutant disturbance scenario generator using full-scale data

I did the calibration and validation exercise together with Xavier Flores-Alsina. I was also involved in developing the case studies for the model, which were mainly conceptualized by Xavier Flores-Alsina. I also supported Xavier Flores-Alsina with the writing of the manuscript.

Paper II: Benchmarking integrated control strategies using an extended BSM2 platform

The conference paper was written during my secondment at the University of Exeter, U.K. The model development was mainly done by me. I was responsible for the writing with support from all co-authors.

Paper III: Catchment & sewer network simulation model to benchmark control strategies within urban wastewater systems

I did the entire model development for the catchment and sewer network. The sewer network model was developed mainly during my secondment at the University of Exeter, U.K. I was primarily responsible for writing the paper with support from Xavier Flores-Alsina and other co-authors.

Paper IV: A benchmark simulation model for evaluation of integrated control strategies in urban wastewater systems

The model development for the BSM-UWS was the major cumulative output of my Ph.D. work. The work was primarily carried out by me. Parts of the work were carried out in collaboration with the University of Exeter, U.K. and Aquafin, Belgium. I was mainly responsible for writing the paper with support from other co-authors.

Paper V: Key control handles and design parameters for improving receiving water quality

I devised the framing of the sensitivity analysis and the choice of the input and output parameters. Initial sensitivity runs were performed in collaboration with Stefan Kroll, Aquafin, Belgium. The final runs are performed by me after fine-tuning the sensitivity analysis parameter set and framing. I wrote the paper with support from other co-authors.

Subsidiary Publications

Peer reviewed conference publications:

VI. **Calibration and validation of a phenomenological influent pollutant disturbance scenario generator using full-scale data**

Flores-Alsina, X., **Saagi, R.**, Lindblom, E., Thirsing, C., Thornberg, D., Gernaey, K.V. and Jeppsson, U.

11th IWA Conference on Instrumentation, Control and Automation (ICA2013), Narbonne, France, 18-20 September 2013.

VII. **Generation of (synthetic) influent data for performing wastewater treatment modelling studies**

Flores-Alsina, X., Ort, C., Martin, C., Benedetti, L., Belia, E., Snip, L., **Saagi, R.**, Talebizadeh, M., Vanrolleghem, P.A., Jeppsson, U. and Gernaey, K.V.

4th IWA Wastewater Treatment Modelling Seminar (WWTmod2014), Spa, Belgium, 30 March-2 April 2014.

VIII. **System-wide Benchmark Simulation Model for integrated analysis of urban wastewater systems**

Saagi, R., Flores-Alsina, X., Kroll, S., Gernaey, K.V. and Jeppsson, U.

9th IWA Symposium on Systems Analysis and Integrated Assessment (Watermatex2015), Surfers Paradise, Australia, 14-17 June 2015.

IX. **Global sensitivity analysis of the system-wide Benchmark Simulation Model**

Saagi, R., Kroll, S., Flores-Alsina, X., Gernaey, K.V. and Jeppsson, U.

10th IWA World Water Congress and Exhibition (WWC&E2016), Brisbane, Australia, 9-13 October 2016.

Other publications by the author relevant to the Ph.D. thesis:

X. **Benchmark Simulation Model No. 1 – Modelling and control case study**

Saagi, R.

Technical Report, Division of Industrial Electrical Engineering and Automation, Lund University, Sweden. LUTEDX/(TEIE-7247)/1-22/(2014).

XI. **Modelling and control of urban wastewater systems – Literature review**

Saagi, R.

Technical Report, Division of Industrial Electrical Engineering and Automation, Lund University, Sweden. LUTEDX/(TEIE-7248)/1-24/(2014).

XII. **Modelling river water quality for system-wide benchmark simulation models**

Saagi, R.

Technical Report, Division of Industrial Electrical Engineering and Automation, Lund University, Sweden. LUTEDX/(TEIE-7249)/1-22/(2014).

XIII. **Towards a system-wide Benchmark Simulation Model – Catchment and sewer system modelling**

Saagi, R.

Technical Report, Division of Industrial Electrical Engineering and Automation, Lund University, Sweden. LUTEDX/(TEIE-7251)/1-22/(2015).

XIV. **Emerging challenges for control of the urban water system**

Paulo, L., **Saagi, R.**, Solon, K. and Vallet, B.

10th IWA Leading Edge Technology Conference (LET2013), Bordeaux, France, 2-6 June 2013.

XV. **Analysis of integrated urban water systems – System-wide benchmark models and control**

Vallet, B., Meng, F., Batista, J., Porro, J. and **Saagi, R.**

2nd IWA Specialized Conference in Ecotechnologies for Sewage Treatment Plants (ecoSTP2014), Verona, Italy, 23-26 June 2014.

Popular Summary

The generation of wastewater from different sources (domestic, industrial, rainfall etc.), its subsequent transport, treatment and discharge constitute the integrated urban wastewater system (UWS). While the urban catchment is the source of wastewater generation, the sewer network (transport) and wastewater treatment plant (WWTP) (treatment) are the engineering infrastructures that ensure safe disposal of the generated wastewater into the receiving waters (e.g. rivers, lakes, oceans). Improving public health and hygiene have been the drivers for building and operating the urban wastewater infrastructure (sewers, WWTPs). However, as we gain more knowledge on the implications of human activities on the environment, it is recognized that an additional driver (without compromising public health) for management of wastewater is the improvement of receiving water quality.

Modelling and simulation of the complex interconnected UWS can be a valuable tool to help understand and thereby efficiently operate such systems in order to achieve improved receiving water quality. Dynamic models enable us to study various possibilities (without actually implementing them in a real system) and choose the best solution for different conditions ranging from design, operation and system upgrade. When faced with the need to improve the performance of such systems, implementing control strategies is possibly the most cost effective solution (when compared to expanding the available capacity). In order to develop optimal control strategies, use of modelling tools is inevitable.

Currently, different software packages offer toolboxes to simulate the entire UWS and evaluate the effect of control strategies and structural modifications on the receiving water quality. However, since the solutions implemented are specific to each case study and also use different evaluation criteria, it is not possible to objectively evaluate the different alternatives.

The current thesis aims at developing a Benchmark Simulation Model for the integrated urban wastewater system (BSM-UWS) that consists of: i) an open-source model library that describes the operation of different sections of an UWS; and ii) a hypothetical UWS layout using the developed model library. The model toolbox and the pre-defined layout can be used for various purposes ranging from: i) evaluating different control strategies on the pre-defined layout using unbiased evaluation criteria; ii) using the model library as a software to develop integrated models for other real catchments as well as for the individual sections of the UWS; and iii) using the pre-defined layout as a basis to improve the underlying modelling principles for various sections. Hence, the thesis is primarily aimed at providing a Benchmark Simulation Model for UWSs but also contributes to the field of integrated modelling by: i) enhancing aspects of model

development for the different sections; and ii) providing a freely distributed, open-source model library for simulating UWSs. It is envisioned that the control and operational strategies evaluated using the BSM-UWS layout will serve as a repository of control ideas for future users and also enhance our understanding of the UWS in general.

The major milestones achieved in order to accomplish the above mentioned objectives are described below.

- Firstly, the Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG), which is primarily used to generate influent data for modelling WWTPs, is identified as a suitable starting point for the catchment model as well as for some aspects of sewer network modelling. The DIPDSG is calibrated for two real WWTPs in order to establish its predictive capabilities. The model blocks from the DIPDSG that are suitable for the BSM-UWS model library are identified.
- A comprehensive model library for BSM-UWS is defined for the different sections, namely: i) catchment; ii) sewer network; iii) WWTP; and iv) river water system. Evaluation criteria for river water quality as well as for the performance of the sewer network and WWTP are presented.
- A pre-defined layout for the BSM-UWS consisting of an urban catchment with an area of 540 hectares and 80 000 population equivalents is described. Different case studies are presented using: i) catchment and sewer extensions; and ii) the fully integrated BSM-UWS. Additionally, a global sensitivity analysis is performed to identify the most important control handles and design parameters that influence the river water quality.

Nomenclature

Acronyms

ADM1	Anaerobic Digestion Model No. 1
AER	Aeration tank. Suffixes 1,2 and 3 represent the first, second and third tanks, respectively (in BSM-UWS)
ANAER	Anaerobic tank. Suffixes 1 and 2 represent the first and second tanks, respectively (in BSM-UWS)
ANOX	Anoxic tank. Suffixes 1 and 2 represent the first and second tanks, respectively (in BSM-UWS)
ASM	Activated Sludge Model. Suffixes, 1, 2, 2d and 3 denote versions 1, 2, 2d and 3, respectively
ATS	Aeration tank settling
ATV	Abwassertechnische Vereinigung (German)
BP	Bypass (in BSM-UWS)
BSM	Benchmark Simulation Model platform. Suffixes 1, 1_LT, 2 and -UWS denote versions 1, 1 Long Term, 2 and Urban Wastewater System, respectively
CONTROL	Control elements model block (in BSM-UWS)
CSO	Combined sewer overflow
DHI	Dansk Hydraulisk Institut (Danish)
DIPDSG	Dynamic Influent Pollutant Disturbance Scenario Generator
DOM	Domestic model block (in BSM-UWS)
DORA	Dynamic Overflow Risk Assessment
DWF	Dry weather flow
EU	European Union
FIRST-FLUSH	Model block representing first flush effects in the sewer network (in BSM-UWS)
GSA	Global sensitivity analysis
HH	Households model block (in DIPDSG)
IFAK	Institut für Automation und Kommunikation (German)
IND	Industry model block (in BSM-UWS)
IndS	Industry model block (in DIPDSG)
INF	Infiltration to sewer model block (in BSM-UWS)
INTERURBA	Interactions between sewers, treatment plants and receiving waters in urban areas (conference)
IWA	International Water Association
PC	Primary clarifier (in BSM-UWS)
PE	Population equivalent
PI	Proportional-Integral controller

RAIN	Rainfall model block (in DIPDSG)
RST	Rainwater storage tank (in BSM-UWS)
RTC	Real-time control
RWQM1	River Water Quality Model No. 1
SBR	Sequencing batch reactor
SC	Sub-catchment (in BSM-UWS)
Sec.C	Secondary clarifier (in BSM-UWS)
SNOW	Snowmelt model block (in DIPDSG)
SOIL	Infiltration model block (in DIPDSG)
ST	Storage tank (in BSM-UWS)
STORAGE	Storage tank model block (in BSM-UWS)
SW	Stormwater model block (in BSM-UWS)
TRANSPORT	Sewer transport model block (in BSM-UWS)
USEPA	United States Environmental Protection Agency
UWS	Urban wastewater system
WWTP	Wastewater treatment plant

Chemical Formulae and Analysis Parameters

BOD_5	5-day biochemical oxygen demand	g BOD.m^{-3}
C	Carbon	
CO_2	Carbon dioxide	g C.m^{-3}
COD	Chemical oxygen demand. Suffixes sol and part denote soluble and particulate fractions, respectively	g COD.m^{-3}, kg COD.d^{-1}
DO	Dissolved oxygen	g.m^{-3}
H	Hydrogen	
H_2CO_3	Carbonic acid	g C.m^{-3}
H_2S	Hydrogen sulphide	g.m^{-3}
K	Potassium	
Mg	Magnesium	
N	Nitrogen	
NH_3	Un-ionized ammonia	g N.m^{-3}
NH_4^+	Ammonium	g N.m^{-3}, kg N.d^{-1}
NO_3^-	Nitrate	g N.m^{-3}, kg N.d^{-1}
O	Oxygen	
P	Phosphorus	
PO_4^{3-}	Phosphate	g P.m^{-3}, kg P.d^{-1}
TKN	Total Kjeldahl nitrogen	g N.m^{-3}, kg N.d^{-1}
TSS	Total suspended solids	g.m^{-3}

Model State Variables

S_{Ca}	Dissolved calcium ions	$g.m^{-3}$
S_{CO2}	Sum of dissolved CO_2 and H_2CO_3	$g\ C.m^{-3}$
S_{CO3}	Dissolved carbonate	$g\ C.m^{-3}$
S_H	Hydrogen ions	$g.m^{-3}$
S_{H2PO4}	Part of inorganic dissolved phosphorus	$g\ P.m^{-3}$
S_{HCO3}	Bicarbonate	$g\ C.m^{-3}$
S_{HPO4}	Part of inorganic dissolved phosphorus	$g\ P.m^{-3}$
S_I	Inert dissolved organic substrate	$g\ COD.m^{-3}$
S_{NH3}	Un-ionized ammonia	$g\ N.m^{-3}$
S_{NH4}	Ammonium	$g\ N.m^{-3}$
S_{NO2}	Nitrite	$g\ N.m^{-3}$
S_{NO3}	Nitrate	$g\ N.m^{-3}$
S_{O2}	Dissolved oxygen	$g.m^{-3}$
S_{OH}	Hydroxyl ions	$g\ H.m^{-3}$
S_S	Dissolved organic substrate	$g\ COD.m^{-3}$
X_{ALG}	Algae and macrophytes	$g\ COD.m^{-3}$
X_{CON}	Consumers	$g\ COD.m^{-3}$
X_H	Heterotrophic organisms	$g\ COD.m^{-3}$
X_I	Inert particulate organic material	$g\ COD.m^{-3}$
X_{II}	Particulate inorganic material	$g\ COD.m^{-3}$
X_{N1}	Organisms oxidizing ammonium to nitrite	$g\ COD.m^{-3}$
X_{N2}	Organisms oxidizing nitrite to nitrate	$g\ COD.m^{-3}$
X_P	Phosphate adsorbed to particles	$g\ P.m^{-3}$
X_S	Particulate organic material	$g\ COD.m^{-3}$

Other Symbols

a	Accumulation rate (in accumulation and washoff model)	$kg.ha^{-1}$
μ	Mean of elementary effects	
μ^*	Mean of absolute values of elementary effects	
A	Catchment surface area (in accumulation and washoff model)	ha
A	Cross sectional area of river stretch	m^2
A_{imp}	Percentage impervious area (in DIPDSG)	
A_{SC}	Area of secondary clarifier	m^2

A_{soil}	Hypothetical area for soil model (in DIPDSG)	m^2
b	Removal rate during dry weather (in accumulation and washoff model)	d^{-1}
C	Pollutant concentration in a tank. Suffixes in and out represent the inflow and outflow concentrations, respectively	$g.m^{-3}$
C_{max}	Hourly maximum concentration. Calculated for TSS, TKN and PO_4^{3-} from sewer overflows and DO, NH_3 from rivers	$g.m^{-3}$
C_r	Dimensionless runoff coefficient	
EMC	Event mean concentration	$g.m^{-3}$
EQI	Effluent quality index	kg poll. units.d^{-1}
$FFfraction$	Fraction of particulates that settle in the sewer network (in DIPDSG)	
G_{rain}	Effect of rainfall on temperature (in DIPDSG)	
G_{snow}	Effect of snow melting on temperature (in DIPDSG)	
i	Rainfall intensity. Suffixes e and n denote effective and net rainfall intensity, respectively	$mm.h^{-1}$
$Infbias$	Parameter affecting mean annual infiltration to sewers (in DIPDSG)	$m^3.d^{-1}$
IQI	Influent quality index	kg poll. units.d^{-1}
k	Residence time constant	d
K_{down}	Parameter determining flow to downstream aquifer (in DIPDSG)	$m^3.d^{-1}$
K_La	Oxygen transfer coefficient. Suffixes 1, 2 and 3 represent aeration tanks 1, 2 and 3, respectively	d^{-1}
k_r	Unit conversion factor for Manning's formula	
K_{inf}	Parameter determining infiltration flow to sewer (in DIPDSG)	$m^{2.5}.d^{-1}$
M_{max}	Maximum particulate load that can settle in the sewer network (in DIPDSG)	kg
M_s	Mass of solids accumulated in the catchment (in accumulation and washoff model)	kg

n	Manning's roughness coefficient for a river stretch	$s.m^{-1/3}$
N_{ovf}	Overflow frequency	$events.yr^{-1}$
OQI	Overflow quality index	kg poll. units.d^{-1}
$P_{i,hh}perPE$	Daily average pollutant load from households for the pollutant i (in DIPDSG)	$g.d^{-1}.PE^{-1}$
$P_{i,ind}$	Daily average pollutant load from industry for the pollutant i (in DIPDSG)	$g.d^{-1}$
Q	Flow rate from a tank. Suffixes in and out represent inflow and outflow, respectively	$m^3.d^{-1}$
Q_{BP}	Maximum flow rate after bypass location. Suffixes 1 and 2 denote bypass locations BP_1 and BP_2, respectively	$m^3.d^{-1}$
Q_{ind}	Daily average industrial wastewater flow rate (in DIPDSG)	$m^3.d^{-1}$
Q_{intr}	Internal recirculation rate	$m^3.d^{-1}$
Q_{lim}	Flow rate above which particulates are washed off from the sewer network (in DIPDSG)	$m^3.d^{-1}$
$Q_{max,RST}$	Maximum throttle flow rate for rainwater storage tank	$m^3.d^{-1}$
$Q_{max,ST}$	Maximum throttle flow rate from storage tank (online). Suffixes 2, 5 and 6 denote storage tanks 2, 5 and 6, respectively	$m^3.d^{-1}$
$Qpercm$	Volume per cm snow (in DIPDSG)	$m^3.cm^{-1}$
$Qpermm$	Volume per mm rain (in DIPDSG)	$m^3.mm^{-1}$
$QperPE$	Daily average flow rate per population equivalent (in DIPDSG)	$m^3.d^{-1}.PE^{-1}$
$Q_{pump,ST}$	Maximum pump capacity. Suffixes 1 and 4 denote storage tanks 1 and 4, respectively	$m^3.d^{-1}$
Q_r	Sludge recycle rate	$m^3.d^{-1}$
$Q_{throttle,ST4}$	Maximum throttle flow rate from storage tank 4 (offline)	$m^3.d^{-1}$
Q_w	Sludge wastage rate	$m^3.d^{-1}$
R_h	Hydraulic radius of a river stretch	m
S	Horizontal river slope	

subareas	Parameter representing the length of the sewer network (in DIPDSG)	
T_{amp}	Amplitude for the yearly temperature sinusoidal curve. Suffix d denotes daily sinusoidal curve (in DIPDSG)	°C
T_{bias}	Bias for the yearly temperature sinusoidal curve (in DIPDSG)	°C
T_{exc}	Exceedance duration. Calculated for TSS, TKN and PO_4^{3-} from sewer overflows and DO, NH_3 from rivers	d.yr^{-1}
T_{freq}	Frequency of the yearly temperature sinusoidal curve. Suffix d denotes daily sinusoidal curve (in DIPDSG)	rad.y^{-1}
T_{ovf}	Overflow duration	d.yr^{-1}
T_{phase}	Phase shift for the yearly temperature sinusoidal curve. Suffix d denotes daily sinusoidal curve (in DIPDSG)	rad
V	Volume of reservoir tank	m^3
V_{AER}	Volume of aeration tank. Suffixes 1, 2 and 3 represent aeration tanks 1, 2 and 3, respectively	m^3
V_{ANAER}	Volume of anaerobic tank. Suffixes 1 and 2 represent anaerobic tanks 1 and 2, respectively	m^3
V_{ANOX}	Volume of anoxic tank. Suffixes 1 and 2 represent anoxic tanks 1 and 2, respectively	m^3
V_{ovf}	Overflow volume	m^3.yr^{-1}
V_{PC}	Primary clarifier volume	m^3
V_{RST}	Volume of rainwater storage tank	m^3
V_{ST}	Volume of storage tank. Suffixes 1, 2, 4, 5 and 6 denote storage tanks 1, 2, 4, 5 and 6, respectively	m^3
w	Washoff rate for particulate pollutants during rain events (in accumulation and washoff model)	kg.mm^{-1}
x	Dimensionless factor for multi-linear reservoir model	
σ	Standard deviation of elementary effects	
ϕ_{soil}	Porosity of soil (in DIPDSG)	m.d^{-1}

Chapter 1

Introduction

Water, a major component for the survival of life on earth has intrigued human curiosity ever since the beginning of civilization. Being one of the most essential components in our daily life, it is perhaps not very hard to figure out various functions of water in our modern life style. Human consumption, agriculture, domestic activities (toilets, kitchen) and industrial activities are some of the common uses of water that we encounter every day. The wastewater from all these sources is a cause of pollution when left untreated, leading to detrimental effects on human health and the environment. The complex system of engineering infrastructures that deals with collecting this wastewater, transporting, treating and finally releasing it back into the environment (or even better re-using it) in a safe manner comprises the urban wastewater system (UWS). Initial attempts at building this infrastructure were driven by the need to protect humans from the ill-effects of coming in contact with wastewater. Hence, earlier wastewater infrastructure was mainly focussed on collecting and discharging all the wastewater away from human settlements (generally without any treatment). Now, with a better understanding of the environmental impacts of discharging untreated wastewater into receiving water bodies, the focus has been shifting towards protecting the quality of the receiving waters (rivers, lakes etc.).

The different sections of this wastewater system – sewer network, wastewater treatment plant (WWTP) and receiving water system – are generally managed by different organizations. However, in order to be able to assess the impact on receiving waters, the interactions between these systems should be considered in a holistic manner. Hence, there is a paradigm shift in research and industry – instead of focussing on one part of the UWS, one aims to evaluate the entire UWS in an integrated manner. Using modelling tools to perform such evaluations is becoming an increasingly important approach.

In simple terms, models are used to reproduce the behaviour of a system using mathematical tools that describe various phenomena occurring in the system. The more complex a system becomes, the more valuable it is to use modelling tools to study it. This makes the choice of modelling almost necessary, when assessing complex UWSs. The thesis deals with developing models for various sections of an UWS, integrating these models and defining metrics that make it possible to evaluate the impact of various changes on different sections of the UWS.

1.1. Aim and Outline

The aim of this thesis is to develop an urban wastewater system-wide Benchmark Simulation Model (BSM-UWS) that can be used to evaluate control strategies (local/integrated) and structural modifications in different sections of the UWS. With an integrated model, a further aim is to replace/append the traditional evaluation metrics (effluent quality based for WWTPs and overflow based for sewer networks) with river water quality based evaluation criteria. The toolbox developed can be used as a standard benchmarking tool to evaluate various control strategies on the pre-defined layout as well as be adapted to other UWSs using the building blocks provided in the model library. It can also be used to describe the individual sections of the UWS. The toolbox and the case studies presented in the thesis will enhance our understanding of the UWS and also provide inspiration for various integrated control strategies that can be implemented on a system-wide scale.

The thesis describes the spatial extensions to the WWTP benchmark simulation models. The extensions include model development for: i) catchment; ii) sewer network; and iii) receiving water system. In addition to the new model blocks, evaluation criteria for the three sections (sewer network, WWTP and river water system) are presented. The thesis also provides example case studies indicating the potential scenarios where the developed models can be used.

Chapter 2 presents an overview of the state-of-the-art in modelling the major sections of an UWS. It provides the background information on integrated modelling and the progress that has been achieved in this field. An overview of various integrated control strategies that are implemented in research/practise is provided.

Chapter 3 describes the modelling principles for the Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) that serves as the starting point for the catchment and sewer network extensions. The chapter provides two calibration exercises on full-scale WWTPs that are presented in Paper I as a support to the ability of the DIPDSG in simulating catchment and sewer phenomena.

Chapter 4 details the model library for various sections in the BSM-UWS, which include catchment, sewer network, WWTP and river water system. Evaluation criteria for sewer network, WWTP and river water system are presented. For the catchment and sewer extensions, Paper II describes the initial modelling results and Paper III provides a detailed description of all the model elements in an updated version of the model. Paper IV presents the entire model library for the BSM-UWS. Evaluation criteria are presented in Paper III and Paper IV.

Chapter 5 presents the UWS layout and characteristics for the BSM-UWS. Open loop results for the integrated model are presented. Case studies from Paper III (catchment and sewer BSM) and Paper IV (BSM-UWS) are summarized in this chapter. A global sensitivity analysis (GSA) study aimed at determining the major control handles and design parameters in the BSM-UWS is described (Paper V). Finally, potential applications of the platform and model limitations are discussed.

Chapter 6 summarizes the major conclusions from all the above chapters in the thesis. Future perspectives for the research are also discussed.

1.2. Key Contributions

The major contributions from the research are summarized below together with an overview of the papers published/written during the research period.

- An open-source, freely distributed integrated Benchmark Simulation Model (BSM-UWS) is developed, which includes description of flow rate and pollutant transformations in: i) catchments; ii) sewer networks; iii) wastewater treatment plants; and iv) river water systems. The models library can be used as a benchmarking tool as well as for developing integrated models for other real cases. The individual model blocks (e.g. storage tanks, river models etc.) can also be used as standalone models to simulate limited sections of the UWS.
- Evaluation criteria using traditional metrics for WWTP effluent and sewer overflows are defined. More importantly, holistic evaluation criteria based on the chemical quality of river water systems are developed.
- Various case studies highlighting control strategies (local/integrated) as well as structural modifications that can be evaluated using the BSM-UWS are presented and evaluated. This demonstrates the usefulness and applicability of BSM-UWS and integrated modelling studies.

The thesis is a summary of the five research papers mentioned in the list of publications. An overview of each of these papers is provided below.

Paper I: Journal paper published in Water Research (impact factor 2015 – 5.99). The paper presents calibration and validation of the Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) using data from two full-scale WWTPs. Case studies describing potential applications of the model are presented.

Paper II: A conference publication as oral presentation at the International Conference on Urban Drainage, Kuching, Malaysia, 2014. The paper describes the initial attempts to develop a catchment and sewer BSM. A simple control case study is also presented.

Paper III: Published in Environmental Modelling and Software (impact factor 2015 – 4.20) describes the first "outside-the-fence" extensions to the benchmark simulation models. The catchment and sewer BSM is developed and evaluation criteria for assessment of sewer overflows are presented. Various case studies presenting control strategies and structural modifications to the system are presented.

Paper IV: Journal paper published in Environmental Modelling and Software (impact factor 2015 – 4.20). It presents the complete urban wastewater system-wide BSM (BSM-UWS) together with evaluation criteria for sewer overflows, WWTP effluent quality and river water quality. Case studies illustrating various local and integrated control strategies are described.

Paper V: A manuscript describing the global sensitivity analysis (GSA) of the BSM-UWS in order to identify key control handles and design parameters that have a strong influence on the river water quality. The information can be used to develop various control strategies for the given layout in the future.

Source code from the research is open-source and freely distributed. The two model packages (catchment and sewer BSM, BSM-UWS) are described below.

- Catchment and sewer BSM includes the model library for various blocks in the catchment and sewer network. Evaluation criteria for sewer performance are also included.
- BSM-UWS consists of the model library for all sections in the UWS (catchment, sewer network, WWTP, river water system). The model library also includes evaluation criteria for sewer network, WWTP and river water system. A block-wise approach is used for the model development, which allows users to model either a specific section (or only a few components of a section) or the entire UWS.

1.3. Limitations

Simplifications are made in describing the different sections (e.g. hydrological processes in the catchment, flow phenomena in the sewer network, biological processes in WWTP and river system) considering the purpose of the study. Although the BSM-UWS layout can be used to evaluate various control strategies, the best control strategy thus obtained may not necessarily perform in a similar manner for another catchment due to differences in the layout and design capacities. However, the control principles demonstrated for the BSM-UWS can be transferable to other catchments. The model library mostly uses standard approaches that are well established. However, they are currently only used to describe a hypothetical UWS and not fully calibrated for a real case study.

Chapter 2

Background

This chapter details various elements that comprise the urban wastewater system. An overview of the commonly available modelling approaches for each of these elements is presented. Additionally, the state-of-the-art in integrated modelling and control of urban wastewater systems is described.

2.1. Urban Wastewater Systems

Various sections involved in the collection, transport, treatment and discharge of sewage and stormwater together comprise the urban wastewater system (UWS). In this chain of interlinked elements, the starting point for generation of wastewater/stormwater is the urban catchment. Sewage is generated from households and industries while stormwater is mainly the runoff from urban surfaces during rain events. The invisible underground sewer network transports the generated wastewater to the wastewater treatment plant (WWTP). As the name suggests, the WWTP is involved in removing the pollutants present in the raw sewage before discharging the treated effluent into receiving waters. Receiving waters form the final link in this chain (Figure 2.1). In many cases, this receiving water system is the starting point for the drinking water system for any downstream city (although this is outside the scope of this thesis). Historically, the objective of an UWS has been to convey the sewage away from the city in order to avoid health hazards to urban dwellers. However, owing to our increasing understanding of anthropological pressures on the natural ecosystem, the European Union has (re-)defined the objective of an UWS as to “protect the chemical and ecological status of a river” (Council of the European Communities, 2000).

In this context, it is essential to understand the interactions between different parts of an UWS in order to improve their performance individually as well as to protect the receiving waters in a holistic manner. Modelling can be a valuable tool not only for understanding the individual sections and their interactions but also for serving as an engineering tool to explore the potential for improvement in the performance using different approaches (e.g. process control, upgrading the existing infrastructure).

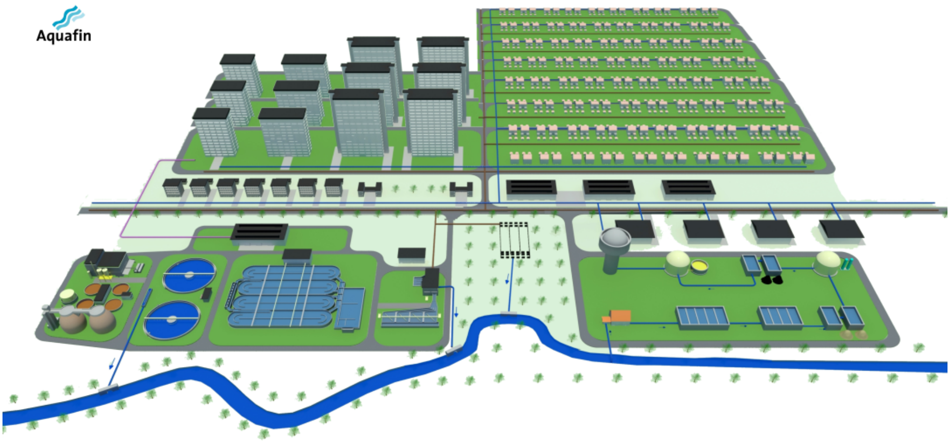

Figure 2.1: Various sections of an urban wastewater system which include: i) catchment (top); ii) sewer network (top and middle); iii) wastewater treatment plant (bottom left); and iv) receiving waters (bottom). (Copyright: Aquafin, Belgium. Reprinted with permission). Note that the drinking water system (bottom right) is outside the scope of this thesis.

The following sub-chapters present an overview of the state-of-the-art in the modelling of various sections of the UWS, namely:

i. catchment;

ii. sewer network;

iii. WWTP;

iv. receiving water system.

2.2. Modelling the Integrated Urban Wastewater System

2.2.1. Catchment

Generation of wastewater from urban catchments can be described using different model blocks for flow rate and pollutant loads during dry weather (sewage) as well as rain events (stormwater). A description of the commonly used approaches is provided in this section.

Sewage

A diurnal variation profile (can also be user-defined) (Figure 2.2) is one of the most common approaches for modelling the dynamics of dry weather wastewater generation. Such a profile, combined with daily average values for flow rate/pollutant loads, is available in commonly used simulation software (e.g. ifak, 2016; Gernaey et al., 2011). Weekly and seasonal variations can also be included in the profile. Different profiles and mean pollutant loads can be defined for domestic and industrial sources (Butler, 1993; Ainger et al., 1997). Another approach is to use a Fourier series to simulate the diurnal variation in dry weather flows (Langergraber et al., 2008). In cases where the dynamics of the daily variations are less important (e.g. urban drainage models), a constant value is generally used to represent the contribution of sewage to the total flow.

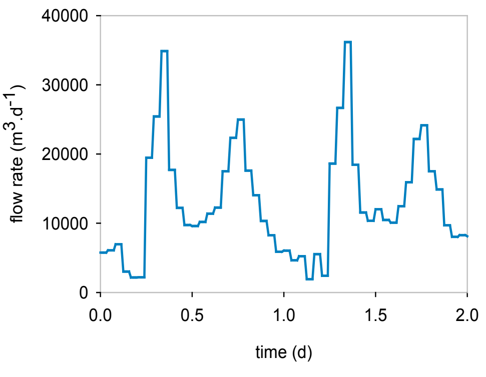

Figure 2.2: Diurnal variation in dry weather wastewater generation from domestic sources with distinct morning and evening peaks.

Stormwater

Modelling the generation of stormwater due to rain events requires describing the underlying processes for flow rate as well as pollutant loads.

Flow Rate Generation from Rainfall

The contribution of rainfall to urban runoff is modelled by a two-step approach: i) determining effective rainfall intensity by accounting for various losses; and ii) modelling the surface routing of the generated runoff to a sewer network (Butler & Davies, 2004).

Accounting for Losses in Rainfall: Detailed hydrological models take into account all the losses on the catchment surface. Major losses generally modelled are initial losses (interception, depression storage) and continuous losses (evapo-transpiration, infiltration to soil). As the name suggests, initial losses only affect the runoff in the beginning of the rain event while continuous losses are accounted for during the entire rainfall period.

A simple and commonly used approach is to lump all the continuous losses into a runoff coefficient. Firstly, net rainfall intensity is obtained after subtracting initial losses (generally assumed as a constant amount of rainfall, e.g. 2 mm). A runoff coefficient is then used to determine the effective rainfall intensity for the specific catchment,

$$i_{\mathrm{e}} = C_{\mathrm{r}} i_{\mathrm{n}} \qquad \text{(Eq. 2.1)}$$

where i_e is the effective rainfall intensity (mm.h^{-1}), i_n is the net rainfall intensity (mm.h^{-1}) (after subtracting initial losses) and C_r (-) is the dimensionless runoff coefficient.

Another approach is to compute the losses from impervious and pervious areas separately. While the impervious area runoff leads to overland flow and reaches the sewer network almost immediately, the runoff from pervious area leads to a delayed response from the catchment. An infiltration model is generally used to compute the pervious area runoff (e.g. Horton, 1940).

Surface Routing: After accounting for all the losses, the generated runoff is conveyed to the sewer network (e.g. into gully inlets, open stormwater drains) through surface routing. Figure 2.3 provides a graphical description for the conversion of the effective rainfall intensity (Figure 2.3a) into a response hydrograph from the catchment (Figure 2.3b).

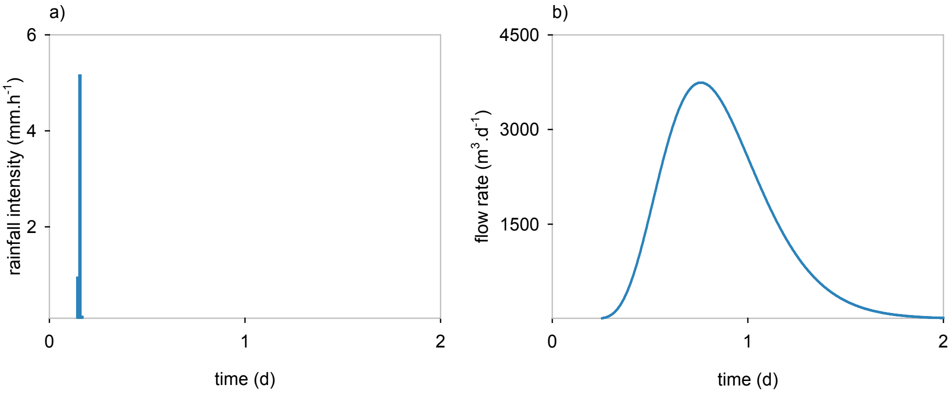

Figure 2.3: Effective rainfall (a) and the corresponding runoff from a catchment (b).

Commonly used methods for modelling surface routing are: i) synthetic unit hydrographs (Harms & Verworn, 1984); ii) time-area curves (Hall, 1984); and iii) reservoir models (e.g. Nash, 1957). In the context of integrated modelling, reservoir models are generally chosen. A description of various reservoir models is given in Chapter 2.2.2 – together with models for sewer transport.

Pollutant Loads During Rain Events

Pollutants accumulated on the roads, pavements, rooftops etc. reach the sewer network during rain events. Some of the common approaches used to model the generation of pollutant runoff from urban catchments are mentioned below.

Event Mean Concentrations (EMC): This is the simplest approach to describe pollutant loads during a rain event. It assumes that a uniform concentration of each pollutant reaches the sewer network during the entire rainfall duration. Values for *EMC* can be assumed based on literature (Ellis & Mitchell, 2006) (Table 2.1) or using experimental data from a specific catchment (Langeveld et al., 2013). The method is not suitable to predict concentration changes occurring during the rain event.

Table 2.1: Event mean concentrations (*EMC*) for different pollutants (reproduced from Ellis and Mitchell (2006)). Mean values are given within brackets ().

Quality parameter	*EMC* (g.m^{-3})
Suspended solids	21-2582 (90)
BOD_5	7-22 (9)
COD	20-365 (85)
Ammonium nitrogen	0.2-4.6 (0.56)
Total nitrogen	0.4-20 (3.2)
Total phosphorus	0.02-4.30 (0.24)

Regression Equations: Regression analysis of historic data is used to determine the relationship between the pollutant concentration and catchment characteristics, like surface type, land use etc. Such curves provide an accurate description of the pollutant loads for the catchments from using historic data and can also be used for other catchments with similar properties (Butler & Davies, 2004).

Accumulation and Washoff Models: A simplified description of pollutant buildup and washoff processes (Butler & Davies, 2004) is represented by accumulation and washoff models. Various factors such as land use, population, seasonal variations, street cleaning and surface conditions contribute to the accumulation of pollutants. The change in mass of particulate solids (M_s) (kg) on the catchment surface (A) (ha) due to accumulation and washoff processes is described as:

$$\frac{\mathrm{d}M_s(\mathrm{t})}{\mathrm{dt}} = aA - bM_s(\mathrm{t}) - wi(\mathrm{t})M_s(\mathrm{t}) \quad \text{(Eq. 2.2)}$$

where a (kg.ha^{-1}.d^{-1}) is the representative accumulation rate, b (d^{-1}) is the removal rate during dry weather introduced in order to limit the mass of solids accumulated and w (kg.mm^{-1}) represents the washoff rate during rain events. The extent of washoff is influenced by the rainfall intensity i (mm.d^{-1}).

Model parameters need to be calibrated using data from the catchment. A major advantage of such models is the ability to describe the variation in pollutant concentrations during the rain event as opposed to using constant values for the entire rainfall duration.

2.2.2. Sewer Network

Various phenomena affecting the flow rate and quality of wastewater (and/or stormwater) in the sewer network include: i) sewer hydraulics; and ii) pollutant transport.

Sewer Hydraulics

Hydraulic behaviour of wastewater in the sewer network can be described with either detailed hydrodynamic models or with simplified conceptual modelling approaches. The choice of the modelling approach mainly depends on the objective of the study.

Hydrodynamic Models

The Saint-Venant equation (Saint-Venant, 1870), consisting of a continuity equation and momentum equation, is used to provide a detailed description of wastewater transport in the sewer network. Although these equations are used widely in urban drainage engineering, they are not suitable in the context of integrated modelling. When using a single platform to describe all sections of an integrated UWS, these hydrodynamic models pose challenges in terms of model complexity and hence result in extensive simulation times. To overcome this problem, conceptual reservoir models are used.

Conceptual Reservoir Models

Different conceptual approaches can be broadly classified into:

i. linear reservoir models;

ii. multi-linear reservoir models;

iii. non-linear reservoir models.

A hypothetical reservoir (or tank) is modelled to mimic the behaviour of the sewer network. The input flow rate is the sewer flow from upstream pipes and/or catchment surface runoff. The output is described as a function of the storage volume. Different approaches can be used depending on the relationship between input, output and storage volume of the reservoir.

Linear Reservoir Models: This approach is based on the concept of Nash cascades used for hydrological routing models in catchments (Nash, 1957; Viessman et al., 1989). The volume balance and the relationship between volume and outflow for a single tank are described as:

$$\frac{\mathrm{d}V(\mathrm{t})}{\mathrm{dt}} = Q_{\mathrm{in}}(\mathrm{t}) - Q_{\mathrm{out}}(\mathrm{t}) \qquad \text{(Eq. 2.3)}$$

$$V(\mathrm{t}) = kQ_{\mathrm{out}}(\mathrm{t}) \qquad \text{(Eq. 2.4)}$$

where the inflow to the reservoir is Q_{in} ($m^3.d^{-1}$) and the outflow is Q_{out} ($m^3.d^{-1}$). V (m^3) is the storage volume. The parameter k (d) represents the residence time constant.

Multi-Linear Reservoir Models: Multi-linear models are a combination of different linear relationships between various components of sewer flow. One example of a multi-linear model is the Muskingum method (Cunge, 1969), which was originally developed to describe river flow. This approach has been adopted in some simplified sewer models (Achleitner et al., 2007). The storage volume as a function of both the inflow and outflow is given as:

$$\frac{dV(\mathrm{t})}{dt} = Q_{\mathrm{in}}(\mathrm{t}) - Q_{\mathrm{out}}(\mathrm{t}) \qquad \text{(Eq. 2.5)}$$

$$V(\mathrm{t}) = k(xQ_{\mathrm{in}}(\mathrm{t}) + (1-x)Q_{\mathrm{out}}(\mathrm{t})) \qquad \text{(Eq. 2.6)}$$

where a dimensionless factor x (-) describes the relation between inflow, outflow and total volume.

Non-Linear Reservoir Models: An improvement over linear reservoirs is to include the non linearity in the relationship between reservoir volume and outflow. This approach has been used in the modelling software SMUSI (Muschalla et al., 2007). The non-linear parameters are computed based on the pipe geometry (Mehler, 2000). An example of non-linear reservoir model as used in Gernaey et al. (2011) is:

$$\frac{\mathrm{d}V(\mathrm{t})}{\mathrm{dt}} = Q_{\mathrm{in}}(\mathrm{t}) - Q_{\mathrm{out}}(\mathrm{t}) \qquad \text{(Eq. 2.7)}$$

$$Q_{\mathrm{out}}(\mathrm{t}) = kV(\mathrm{t})^{1.5} \qquad \text{(Eq. 2.8)}$$

Pollutant Transport in Sewers

Soluble pollutant transport in sewers can either be described using advection-dispersion models or by using conceptual models representing completely mixed tanks in series. In addition, sediment transport is modelled by various empirical equations.

Advection-Dispersion Model

Advection represents the transport of pollutants at the mean velocity of flow while dispersion describes the spreading of the pollutants relative to the mean flow velocity. The full advection-dispersion equation is a partial differential equation describing the change in pollutant concentration with respect to time as well as distance. Owing to the partial differential nature of the equation, such models are seldom used in an integrated modelling framework.

Completely Mixed Tank Models

A conceptual approach is to represent the transport of pollutants using a series of completely mixed tanks. The mass balance for each of the tanks is defined as:

$$\frac{\mathrm{d}C_{\mathrm{out}}(\mathrm{t})}{\mathrm{dt}} = \frac{Q(\mathrm{t})}{V}(C_{\mathrm{in}}(\mathrm{t}) - C_{\mathrm{out}}(\mathrm{t})) \qquad \text{(Eq. 2.9)}$$

where Q ($m^3.d^{-1}$) is the flow rate and C_{in} ($g.m^{-3}$), C_{out} ($g.m^{-3}$) are inflow and outflow concentrations of the pollutant, respectively.

Sediment Transport Models

Modelling the complex phenomena involved in the sediment transport in sewers has been investigated mainly during the late 20th century. A comprehensive review is provided by Bertrand-Krajewski et al. (1993). Two major approaches are mentioned here.

Ackers-White Model: Initially developed to describe sediment transport in open alluvial channels (alluvial channels are characterized by self-formed morphology mainly due to sediment transport and deposition), the model was later adapted to circular pipes (Ackers & White, 1973; Ackers, 1991). Deterministic sewer software like MOSQITO (Payne et al., 1990) and MOUSE (Lindberg et al., 1989) use such approaches. These models are difficult to apply mainly due to lack of detailed sewer characteristics data for calibration.

Velikanov Approach: This is a simplified approach based on efficiency coefficients and threshold sediment concentrations that trigger different sediment transport phenomena (Bujon et al., 1992). Depending on the concentration of sediments in the sewer network, the sediment particles either: i) deposit in the sewers; ii) transport with the flow; or iii) erode from the existing deposits. This simplified approach is used in modelling software like FLUPOL (Bujon et al., 1992) and HORUS (Zug et al., 1999).

Nevertheless, it should be highlighted that sewer sediment transport has not been completely understood and further efforts need to be directed to address this problem.

Biological Transformations in Sewer Network

Depending on the characteristics of the sewer network (length, slope, gravity/pumped system etc.), various biological transformations can take place in this network of pipes (Nielsen et al., 1992). In principle, all the biological processes occurring in a WWTP can also occur in a sewer network. Nevertheless, the sewer network has considerable differences (Hvitved-Jacobsen et al., 1998). These include:

i. biological processes can occur in bulk phase, sediments as well as in biofilm;

ii. availability of biomass is limited in comparison to WWTP;

iii. soluble substrate is readily available in abundance;

iv. depending on the type of sewer network – gravity sewers or pumped system (rising mains) – predominance of aerobic or anaerobic processes is determined.

In brief, the major biological processes commonly modelled are:

i. degradation of organic matter in the presence of oxygen;

ii. anoxic degradation of organic matter, with nitrate as electron acceptor;

iii. conversion of organic matter to volatile fatty acids under anaerobic conditions;

iv. production of hydrogen sulphide and methane (anaerobic).

Additionally, other processes that are considered important include:

v. biofilm growth, attachment and detachment;

vi. surface aeration (for oxygen mass transfer);

vii. sediment deposition and resuspension.

Modelling of biological processes in sewer networks was initially developed to describe hydrogen sulphide (H_2S) production. The corrosion effects of H_2S led to empirical modelling of the underlying biological processes, mainly as an attempt to solve the corrosion problems (Nielsen et al., 1992). However, more mechanistic models are currently available to address this issue (Sharma et al., 2008). Hvitved-Jacobsen et al. (1998) described the aerobic processes in the bulk phase as well as a simplified biofilm model for a gravity sewer network. The model was later extended by Tanaka & Hvitved-Jacobsen (1998) to include anaerobic processes in a pumped sewer network. Huisman & Gujer (2002) developed a unified approach using Activated Sludge Model No. 3 (ASM3) (Henze et al., 2000), in order to facilitate the integration of a sewer model with WWTP and river models. Also, the biofilm processes were considered in detail. In addition, extensions to the sewer models to describe the transport and transformation of micropollutants have also been developed (Lindblom, 2009; Plósz et al., 2013; Snip et al., 2014).

2.2.3. Wastewater Treatment Plant

The key unit operations in a WWTP (Figure 2.4) can be broadly divided into: i) biochemical processes; and ii) physical processes. Other ancillary units include storage tanks and sludge handling units (thickeners, dewatering units etc.).

Biochemical Processes – Activated Sludge Models

At the heart of the WWTP is the activated sludge process. A series of biochemical reactors are used to remove organic matter and nutrients from the raw wastewater. A secondary settling tank after the biological reactors ensures that the sludge is recycled whereas the treated effluent is either released into the receiving water or sent for tertiary treatment. Various configurations of the biochemical reactors are used to achieve different treatment objectives (Tchobanoglous et al., 2004). The reactors are operated in anaerobic, anoxic and aerobic conditions. Additionally, for the treatment of excess sludge, anaerobic digesters are generally employed. State-of-the-art International Water Association (IWA) models (Henze et al., 2000) can be used to describe these unit operations. The major biological process models are briefly mentioned below.

- *Activated Sludge Model No. 1 (ASM1)*: This is the first and the most widely used activated sludge model from the IWA Task Group on Mathematical Modelling for Design and Operation of Biological Wastewater Treatment (Henze et al., 1987). The model includes key processes for biological removal of organic carbon and nitrogen. Apart from the ability to predict effluent concentrations, the model development is focused to be able to accurately describe concentrations of pollutants and solids in the biological reactors.
- *Activated Sludge Model No. 2 (ASM2)*: The ability to model biological as well as chemical removal (precipitation) of phosphorus is the major improvement in ASM2 compared to ASM1. The model not only includes new state variable for phosphorus accumulating organisms (PAO) but additionally includes internal storage components and polyphosphates (Henze et al., 1995).
- *Activated Sludge Model No. 2d (ASM2d)*: As a minor modification to ASM2, the model considers the denitrifying ability of phosphorus accumulating organisms in ASM2d (Henze et al., 1999). In addition, various extensions to ASM2d describing physico-chemical processes (acid-base reactions, ion pairing, precipitation etc.) are available (Flores-Alsina et al., 2015; Solon, 2017). This leads to a better description of phosphorus, iron, sulfur and nitrate dynamics in the system.
- *Activated Sludge Model No. 3 (ASM3)*: This model is considered as an update to ASM1 and is recommended to be used as a framework for addition of new processes and state variables (Gujer et al., 1999). One of the major changes is the decoupling of the death and regeneration processes and also the linkage between heterotrophs and nitrifiers as a result of this. In ASM3, the decay processes in heterotrophic and autotrophic bacteria are separated and also the death-regeneration concept is replaced with a death-endogenous respiration process. The second improvement in ASM3 is the modelling of cell internal storage compounds for all biomass state variables. As a replacement for ASM2d, an ASM3-BioP model (includes phosphorus processes) (Rieger et al., 2001), was later developed which uses the same principles as ASM3.

- *Anaerobic Digestion Model No. 1 (ADM1)*: The IWA Task Group on Mathematical Modelling of Anaerobic Digestion Processes developed a model framework for describing the biological and physico-chemical processes in anaerobic digesters (Batstone et al., 2002). The biological processes include: i) fermentation, anaerobic oxidation and methanogenesis as cellular processes; and ii) disintegration (partly non-biological) and hydrolysis as extra-cellular processes. The physico-chemical model includes different aspects, such as ion association/dissociation, gas transfer and precipitation, in a simplified manner. The model forms a generic framework and various modifications and additional processes have been implemented to enhance the ADM1 model (Batstone et al., 2006; Solon et al., 2015; Flores-Alsina et al., 2016).

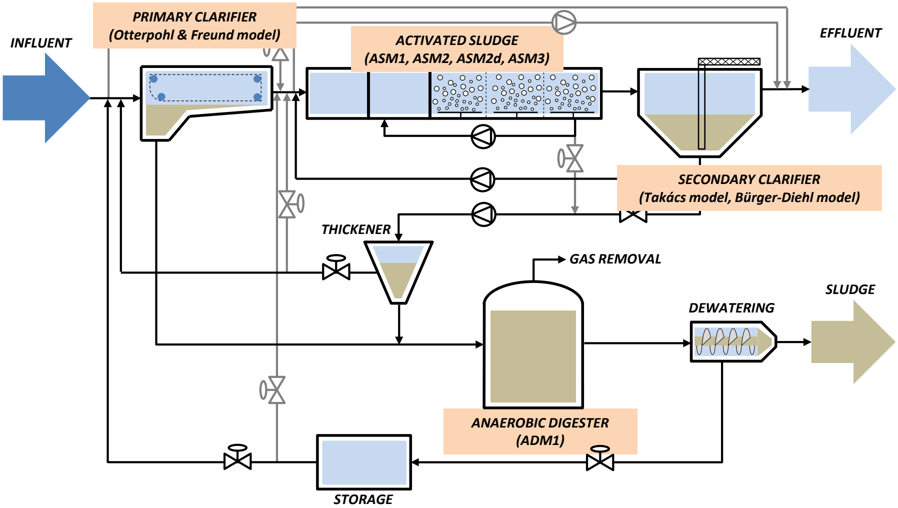

Figure 2.4: Major unit processes in a WWTP and commonly used models to describe the biochemical transformations in these processes (from Gernaey et al. (2014)).

Physical Processes

The two major physical processes in a WWTP are: i) primary settling; and ii) secondary settling.

Primary Settler

The major objective of primary settlers is removal of particulate organics as well as inorganic pollutants thereby facilitating improved biological removal in the later stages. The model developed by Otterpohl & Freund (1992) is commonly used to describe primary clarifiers. It makes use of efficiency factors for different soluble and particulate state variables in order to estimate their effluent concentrations. A more mechanistic approach using different settling velocities based on particle size distribution of the particulates is developed by Bachis et al. (2015). Although not widespread, such detailed models can improve the prediction capability of primary settler operation under different flow conditions.

Secondary Settler

Separation of the treated effluent from the sludge is achieved using secondary clarifiers. The treated effluent is either discharged as an overflow or sent to further tertiary treatment. The settled sludge is recycled back to the biological reactors. The Takács secondary settler model (Takács et al., 1991) is commonly employed for the majority of the WWTP models. It uses a modified Vesilind settling function (Vesilind, 1968) to describe hindered settling. Although widely used, the approach has issues related to numerical robustness and also limitations in its ability to predict wet weather operation of the settler. The Bürger-Diehl settler model (Bürger et al., 2011; 2012; 2013) overcomes these limitations. Three principal processes included in the model are: i) bulk flow; ii) hindered settling; and iii) compression. Without any significant increase in simulation time, the model has been able to improve the description of the secondary settler behaviour (Arnell, 2015).

The above section only covers the standard approaches for the configurations that are commonly found in municipal WWTPs. Nevertheless, models for other configurations and processes, like membrane bioreactors, high rate activated sludge processess, biofilm, filtration, granular sludge systems etc., are available (Fenu et al., 2010; Nogaj et al., 2013; Wanner & Reichert, 1996). Also, the model complexity of the existing models is ever increasing in order to be able to describe physico-chemical processes and nutrient recovery unit operations (Batstone et al., 2012; Flores-Alsina et al., 2016). In conclusion, it can be said that WWTP modelling is a fairly mature field of engineering with continuous model improvement efforts and also a quick adaptation of the developed models by the modelling industry.

2.2.4. River Water System

The first and the most simplified description of river water quality is the Streeter and Phelps model (Streeter & Phelps, 1925). It describes the effect of organic matter on the dissolved oxygen concentration in the river. Since then, more processes and state variables have been included to simulate the biochemical variations in the river water quality. The QUAL family of models (Brown & Barnwell, 1987) and WASP (Di Toro et al., 1983) by United States Environmental Protection Agency (USEPA) are commonly used in river quality simulations. However, for integrated modelling purposes, lack of compatibility of river water quality state variables with those from other sections of the UWS and the difficulty of information exchange limits the direct application of such models. The Duflow water quality model developed by Lijklema et al. (1996) is one of the models currently being used in UWS analysis. It is available in commercial software like WEST and SIMBA#. Another model that is currently available is the River Water Quality Model No. 1 (RWQM1) (Reichert et al., 2001b; Shanahan et al., 2001; Vanrolleghem et al., 2001). This model is the result of an IWA task group, set up to identify the missing gaps in river quality models and to build a river model that can be linked to the activated sludge family of models. A simplified version of RWQM1 is used to describe the river system in this thesis. A brief description of the RWQM1 framework is provided below.

River Water Quality Model No. 1

Taking into consideration the need for integrated modelling, a river water quality modelling framework that can be easily interfaced with the ASM family of models for WWTPs was developed by the IWA Task Group on River Water Quality Modelling (Reichert et al., 2001a). The task group addressed some of the major limitations in the existing water quality models for rivers.

- Biomass as a state variable: Earlier river models do not explicitly consider biomass as a state variable. It is either assumed that biomass is always available or a constant biomass concentration is assumed. RWQM1 includes different bacterial populations as state variables to address this limitation.
- Elemental composition of state variables: The elemental constituents of all the state variables are defined (for carbon (C), hydrogen (H), nitrogen (N), oxygen (O), and phosphorus (P)). The mass fractions can be easily altered by users. Also the chemical oxygen demand (COD) equivalent for each state variable is determined based on a balanced mineralization reaction using these mass fractions.

The task group did not present a single model but provided a framework on which models of varying complexity can be built depending on the purpose of the study. Commonly used state variables and processes are described here.

Table 2.2: List of state variables used in RWQM1.

Definition	Notation
Dissolved organic substrate	S_S
Inert dissolved organic substrate	S_I
Ammonium	S_{NH4}
Un-ionized ammonia	S_{NH3}
Nitrite	S_{NO2}
Nitrate	S_{NO3}
Part of inorganic dissolved phosphorus	S_{HPO4}
Part of inorganic dissolved phosphorus	S_{H2PO4}
Dissolved oxygen	S_{O2}
Sum of dissolved CO_2 and H_2CO_3	S_{CO2}
Bicarbonate	S_{HCO3}
Dissolved carbonate	S_{CO3}
Hydrogen ions	S_H
Hydroxyl ions	S_{OH}
Dissolved calcium ions	S_{Ca}
Heterotrophic organisms	X_H
Organisms oxidizing ammonium to nitrite	X_{N1}
Organisms oxidizing nitrite to nitrate	X_{N2}
Algae and macrophytes	X_{ALG}
Consumers	X_{CON}
Particulate organic material	X_S
Inert particulate organic material	X_I
Phosphate adsorbed to particles	X_P
Particulate inorganic material	X_{II}

State Variables

State variables described in RWQM1 can be broadly classified into organic matter, biomass, nutrients (nitrogen and phosphorous) and oxygen (Table 2.2).

1. Organic matter: Dissolved (S_S, S_I) and particulate state variables (X_S, X_I and X_{II}) are defined. While S_S represents the dissolved organic substrate that is readily available for biodegradation, S_I is the inert dissolved organic substance, which is non-biodegradable. X_S can be biologically degraded (after hydrolysis to S_S), while X_I and X_{II} are particulate organic and inorganic inerts, respectively. X_I can be produced during biological processes whereas X_{II} is a conserved state variable and is neither consumed nor produced by the processes modelled in RWQM1.
2. Biomass: X_H (heterotrophic organisms), X_{N1} and X_{N2} (nitrifiers), X_{ALG} (algae and macrophytes) and X_{CON} (consumers – higher order organisms) are included as biomass state variables.

3. Nitrogen: Nitrogen state variables are S_{NH4} (ammonium), S_{NH3} (un-ionized ammonia), S_{NO2} (nitrite), S_{NO3} (nitrate) and S_{N2} (elemental nitrogen). S_{NH3} is only involved in the equilibrium reactions between S_{NH4} and S_{NH3}.

4. Phosphorus: Phosphorus state variables used in the model are S_{HPO4}, S_{H2PO4} (inorganic dissolved phosphorus) and X_P (adsorbed phosphorus). Distribution of inorganic dissolved phosphorus into S_{HPO4} and S_{H2PO4} depends on the pH.

5. Oxygen: Dissolved oxygen is modelled as S_{O2}. It is affected by various biological processes as well as surface reaeration.

Other state variables included are S_{CO2} (dissolved carbon dioxide (CO_2) and carbonic acid (H_2CO_3)), S_{HCO3} (bicarbonate), S_{CO3} (carbonate), S_H (hydrogen ions), S_{OH} (hydroxyl ions) and S_{Ca} (dissolved calcium ions). These state variables are mainly used for pH calculations.

Processes

1. Aerobic growth of heterotrophs: In the presence of oxygen, heterotrophic organisms (X_H) consume dissolved organic substrate, oxygen and nutrients for growth.

2. Aerobic endogenous respiration: Endogenous respiration for all biomass state variables (X_H, X_{N1}, X_{N2}, X_{ALG}, X_{CON}) is described. It is an oxygen consuming process.

3. Anoxic growth of heterotrophs: Under anoxic conditions, a two-step denitrification process is modelled. Nitrate is first converted to nitrite and then to molecular nitrogen. Nitrate and nitrite are used as the electron acceptors, respectively.

4. Anoxic endogenous respiration of heterotrophs: In the absence of oxygen, a one-step process is used to describe the respiration of heterotrophic biomass using nitrate.

5. Growth of nitrifiers: Two different autotrophic organisms are modelled (X_{N1}, X_{N2}). While X_{N1} oxidizes ammonium to nitrite, X_{N2} oxidizes nitrite to nitrate.

6. Growth of algae: In the presence of sunlight, the growth of algae (X_{ALG}) is modelled with either ammonium or nitrate as the nutrient source.

7. Growth of consumers: Consumers (X_{CON}) are higher order organisms that depend on algae, particulate organic matter and bacteria for growth.

8. Death of algae/consumers: Algae and consumers are converted to slowly biodegradable particulates and inert organic matter in this process.

9. Hydrolysis: Heterotrophic conversion of slowly biodegradable particulates to dissolved substrate is modelled as hydrolysis.

10. Phosphate processes: These include adsorption of inorganic phosphate to particulate matter and desorption of bound phosphate as inorganic dissolved phosphorus.
11. Chemical equilibria: Various chemical equilibria processes for the carbonate system, hydrogen system, phosphate etc. are included.

2.3. Development of Integrated Urban Wastewater System Models

Integrated models combine different sections of an UWS. A model can be called integrated if it includes the interactions between at least two sections (Rauch et al., 2002). The primary driver for integrated modelling is to enhance the performance of an UWS in a holistic manner. Moving from the traditional evaluation measures (e.g. WWTP effluent quality, number and duration of sewer overflows etc.) towards integrated assessment (based on receiving water quality) is possible only with the use of integrated modelling approaches.

2.3.1. Brief History of Integrated Modelling

The idea of integrated modelling was proposed around 40 years ago by Beck (1976). One of the early integrated models was developed by Beck & Finney (1987). The model was mainly used to study operational strategies to reduce the stress on downstream rivers. Many authors (e.g. Harremoës et al., 1993; House et al., 1993) identified the interactions between the different sections in an UWS and argued for the development of holistic approaches to improve the quality of receiving waters. The INTERURBA (Interactions between sewers, treatment plants and receiving waters in urban areas) 1992 workshop highlighted the state-of-the-art and also identified the problems involved in UWS modelling (Lijklema et al., 1993). The workshop strongly emphasized the need to design and operate sewers as well as WWTPs based on receiving water quality impacts. Meanwhile, an important development in the WWTP modelling community had taken place. The efforts to standardize activated sludge models by the IWA Task Group on Mathematical Modelling for Design and Operation of Biological Wastewater Treatment led to the technical report on Activated Sludge Model No. 1 (ASM1) (Henze et al., 1987). This has not only helped the WWTP modelling community but also paved the way towards improving other models (sewer and river systems). With the research community recognizing the need for integrated approaches in managing UWSs, there has been a renewed interest in integrated modelling of

UWSs during the late 90s. Various factors contributing to this include the availability of technical know-how and also increased computational power. The enforcement of the European Union (EU) Water Framework Directive to have a good chemical and ecological status of the receiving water necessitated a shift from traditional emission based strategies to receiving water quality based approaches for the management of UWSs (Vanrolleghem et al., 2005a) and hence generated a new interest in the field of integrated modelling and control. The INTERURBA II conference in 2001 is an indication that there is great and continued interest in the idea of integrated modelling (Rauch et al., 2002). Another major advancement in the parallel field of river water quality modelling is the development of a standardized and consistent framework for modelling river water quality. One of the main features of RWQM1 is the ease of integration with WWTP models (Reichert et al., 2001b; Shanahan et al., 2001; Vanrolleghem et al., 2001). With the availability of standard models for all the sections, efforts towards integration strengthened further. Integrated models were developed by several academic groups as well as commercial software developers (e.g. Schütze et al., 1999; Mannina et al., 2006; Achleitner et al., 2007; Vezzaro et al., 2014a; DHI, 2016; ifak, 2016). Armed with integrated modelling toolboxes, the next step was to apply them to improve/prioritize efforts towards holistic analysis of UWS performance. Integrated modelling studies were undertaken for different urban catchments (e.g. Odenthal, Germany (Erbe et al., 2002); Copenhagen, Denmark (Harremeös et al., 2002; Vezzaro et al., 2014b); Lambro river, Italy (Benedetti et al., 2007); Dommel, The Netherlands (Weijers et al., 2012)). Also, integrated models were used to analyse future scenarios and mitigation strategies (Doglioni et al., 2009; Fu et al., 2009; Astaraie-Imani et al., 2012). Multi-objective optimization studies were used to support decision making and improve the river water quality (Muschalla, 2008). As the drivers for adopting integrated models keep increasing (in terms of software availability, potential environmental and economic benefits, legislative requirements), it is expected that undertaking such an exercise will only become more commonplace and academics, practitioners as well as society at large will be able to reap the benefits of the approach.

A more detailed review on integrated models and state-of-the-art is provided in Rauch et al. (2002), Schütze et al. (2011), Benedetti et al. (2013) and Bach et al. (2014).

2.3.2. Challenges to Integrated Modelling

Although detailed models for all the sections exist, the idea of integrating them is far more complex and challenging than simply plugging them together. The major challenges and some approaches to handle them are described below.

- Adapting to the purpose – simplifying the model: The models for sewers, WWTPs and rivers are made for different purposes and cannot be integrated without modifications/simplifications to the model. For example, in the case of sewer models, 2D continuity and momentum equations (Saint-Venant equation) are replaced by conceptual models (Meirlaen et al., 2002; Solvi, 2007). This not only allows for the integration of sewer models with other elements in the UWS, but also significantly reduces the simulation times. Other simplifications in terms of spatial and time boundaries are also commonly employed (Vanrolleghem et al., 2005a).
- Adapting to the purpose – adding complexity to the model: While some aspects of the models are simplified, other areas need an improved description of the underlying processes. The difference in the level of detail and complexity between sewer models and WWTPs in terms of their ability to describe pollutants needs to be reconciled (Fronteau et al., 1997). A major step towards reconciliation between WWTP and river quality models is the RWQM1 (Reichert et al., 2001a). The modelling framework for RWQM1 draws inspiration from ASM models for WWTPs and also facilitates interfacing with WWTP models.
- Model calibration and validation: Integrated modelling aggravates the identifiability and model calibration issues that already exist in the models for the individual sections (Reichert & Vanrolleghem, 2001; Sin et al., 2005). Coupled with this is the need for extensive data collection campaigns needed to satisfactorily calibrate the UWS models. Detailed frameworks are developed to address this issue and provide guidelines on the best practices for applying integrated models to real urban catchments (Breinholt et al., 2008; Muschalla et al., 2009). Nevertheless, one should recognize the fact that inherent identifiability issues and their effect on model calibration cannot be completely removed.

2.3.3. Integrated UWS – Commonly Used Software

The WEST modelling software offers WESTforIUWS (DHI, 2016), which can simulate the catchment, sewer, WWTP and river water system of an integrated UWS. It offers possibilities to evaluate water quality based objectives for both long term and short term evaluation periods. Additionally, uncertainty and sensitivity analysis of the models can also be performed.

SIMBA# water is developed by Institut für Automation und Kommunikation (ifak), Germany (ifak, 2016) and is used for simulation of the integrated UWS. The software consists of a model library to simulate processes in sewers, WWTPs and rivers. Simplified hydrological models as well as hydrodynamic models are available for the sewer network. Various modules for biochemical and physical processes in the WWTP and also different possibilities to simulate biochemical processes in the river are included. Additionally, it facilitates easy implementation of control studies. There is a possibility to program the controllers using industry standard languages such as structured text, petri nets etc.

CITY DRAIN is an open-source Matlab based toolbox for integrated UWS evaluations (Achleitner et al., 2007). It has hydrological models for sewers and river systems as well as a simplified WWTP model. It gives the users a possibility to create their own user defined blocks in addition to the existing model library. It allows for fast simulation of the UWS owing to its simplified nature.

2.4. Integrated Control of Urban Wastewater Systems

Availability of modelling tools and an increasing need to optimize/control UWSs in a holistic manner led to the study of integrated control in an UWS context (Rauch & Harremoës, 1999; Meirlaen et al., 2002; Butler & Davies, 2004; Schütze et al., 2011). Integrated modelling is clearly the pre-cursor to integrated control and hence the advances in the field of integrated control closely follow those from integrated modelling.

Schütze et al. (1999) have characterized integrated control in the context of UWSs based on the following two aspects and presented a methodology for implementation of integrated control strategies.

- Integration of objectives: Control objectives in one section (sewer network, WWTP or receiving water) may be based on criteria measured in other sections (e.g. minimize peak loads to the WWTP based on water levels in the sewer network).

- Integration of information: Control decisions in one section may be based on the information about states in another section (e.g. control of aeration rate in the WWTP based on flow rate in the upstream sewer network).

The majority of the studies on integrated control are theoretical where a system-wide model is used to demonstrate the benefits of integrated control for a particular urban catchment. Nevertheless, there are a few case studies where integrated control is implemented in a real catchment or at least tested in pilot scale.

Various studies on integrated control can be broadly classified into three categories namely:

i. heuristic control;

ii. offline optimization;

iii. online optimization.

2.4.1. Heuristic Control

Expert knowledge about the system can be used to devise control strategies that can enhance the performance of the system. Very often, such controls are achieved through: i) detailed understanding of the process dynamics either through experiments or existing knowledge (e.g. from operators); and ii) development of control strategies through several trial-and-error runs before coming up with the final set of rules and set points for the control algorithm. In many cases, the information about present/future situation can be obtained from a simulation model. A simple IF-ELSE control rule can be:

```
IF (inflow to WWTP greater than a defined flow rate)
  Reduce pumping at an upstream pumping station
ELSE
  Continue to pump at the current rate
```

Wiese et al. (2005) developed and tested a rule-based integrated control strategy for the operation of a sequencing batch reactor (SBR) at WWTP Messel, Germany. A simulation model determines the future state of the sewer network based on a sewer model and weather radar forecast information. The operation of the SBR plant was modified from a dry weather cycle (6h) to a wet weather cycle (8h) based on the information from the model. This facilitated improved operation of the WWTP due to early switching to the wet weather mode using rule-based control with a prediction model.

Meirlaen (2002) developed an integrated model for the Tielt river catchment in Belgium. The model was further used to demonstrate integrated control strategies. Rule-based control strategies to improve river ammonium as well as dissolved oxygen concentrations were implemented.

Sewer and WWTP control considering the interactions between these sections was also developed and implemented in Wilhelmshaven, Germany (Seggelke et al., 2013). It describes: i) a sewer real-time control (RTC) strategy that aims to reduce combined sewer overflow (CSO) frequency and volume; and ii) an integrated control strategy, which manipulates the pumping station operation (in the sewer network) to limit the maximum inflow to the WWTP. The control decision was based on the current operation state of the WWTP (e.g. suspended solids in the aeration tanks, sludge volume in the clarifiers etc.).

A simulation study of the Odenthal catchment in Germany was performed by Erbe et al. (2002). The study included sewer RTC as well as integrated control of sewer and WWTP. The integrated control strategy modifies the bypass (at the WWTP) and inflow to the WWTP based on secondary settler effluent solids concentration.

An impact-based RTC of the sewer network was developed at Dommel, The Netherlands (Benedetti et al., 2013; Langeveld et al., 2013). The control strategy manipulates the operation of pumping stations mainly in the sewer network, although an integrated model was used to evaluate the impact on the WWTP and more importantly on the river water quality.

Aeration tank settling (ATS) was implemented to avoid significant loss of biological capacity during wet weather in the WWTP at Aalborg, Denmark (Nielsen et al., 1996; 2000; Nielsen & Nielsen, 2005). The operation of the WWTP changes to ATS mode based on the influent flow rate. An improvement to this approach was also suggested that uses flow prediction to determine the future inflow to the WWTP. This information can be used to effectively change from normal operation to ATS in advance.

2.4.2. Offline Optimization

Optimization techniques can be used to determine optimum set points and actuator responses for various control strategies developed using rule-based/heuristic methods. An integrated model with the chosen control strategy together with: i) operational constraints for the actuators (e.g. upper and lower limits for the pumping rates); ii) a range of possible sensor set points to trigger controls (e.g. various possible river ammonium concentrations at which a particular control should be triggered); and iii) an objective function (or multiple objective functions) that can be used to determine the best combination, are generally needed. The optimization algorithm efficiently determines the best possible settings for various elements in the control algorithm.

Rauch & Harremoës (1999) used genetic algorithms to optimize RTC for a hypothetical urban catchment based on the Copenhagen wastewater system. A genetic algorithm was used to increase the minimum oxygen concentration in the river stretch predicted using an integrated model. The study highlighted the potential of integrated control versus sewer RTC (to minimize overflow volume) with respect to the river quality. It was shown that a reduction in overflow volume does not necessarily lead to the best river water quality. Also, a water quality based real time integrated control strategy was assessed on the Copenhagen UWS. The Dynamic Overflow Risk Assessment (DORA) (Vezzaro & Grum, 2014) strategy was applied taking into consideration water quality at the WWTP inlet and at a sensitive bathing location in the receiving water system (Vezzaro et al., 2014b).

Fu et al. (2008) demonstrated the case of using genetic algorithms to optimize control strategies with multiple objectives. An offline optimization procedure was used. Objective functions for optimization (in terms of river ammonium, dissolved oxygen concentration and pumping costs) were defined and the desired range of operation of the actuators was used as a constraint for the optimization. Finally, using the Pareto optimal front obtained from the study, control strategies could be chosen by understanding the trade-offs between different objectives. The study was carried out on a hypothetical urban catchment (Schütze et al., 2011) with the WWTP inspired from the Norwich WWTP and the sewer network inspired from the ATV case study (ATV, 1992) and upscaled.

Schütze et al. (2011) developed the system-wide simulation toolbox SYNOPSIS and used it to demonstrate the potential of integrated control strategies. Various offline optimization procedures were used to determine optimum set points for the operation of control handles in sewer and WWTP.

2.4.3. Online Optimization

In online optimization, a set of available actuators and their range of response are chosen; and an optimization algorithm is used to determine the ideal actuator response up to a predefined time period in the future. The optimization algorithm uses a model of the integrated system to minimize an objective function (e.g. minimize the overflow volume, reduce ammonium concentration in the river) by varying the actuators signals. The optimization procedure determines the best setting for each actuator variable that will be chosen for the prediction horizon. The process repeats itself at a chosen frequency.

The sewer network and WWTP in Copenhagen are controlled by an integrated control framework which consists of: i) online optimization of the sewer operation considering both the current state of the WWTP and also rainfall forecasts; and ii) switching the WWTP into wet weather mode (aeration tank settling) using radar based forecasts of inflow to the WWTP (Grum et al., 2011; Sharma et al., 2013; Mollerup et al., 2016).

Seggelke et al. (2005) described a prognosis tool that determines the maximum hydraulic capacity of the WWTP dynamically. An integrated model with WWTP and river was used for the online optimization. At each time-step, the optimizer determines the optimum inflow to the WWTP (based on several future scenarios) that will lead to the lowest ammonium concentration in the river.

2.5. Benchmark Simulation Models

The International Water Association (IWA) Benchmark Simulation Models (BSM1, BSM1_LT, BSM2) consist of a predefined plant layout, process models, sensor and actuator models, influent wastewater characteristics and evaluation criteria (Jeppsson et al., 2013). They were originally developed with the objective to evaluate control strategies in wastewater treatment plants (WWTPs) and have been widely used in both industry and academia. Up to date, more than 500 publications are related to the BSM family products (Gernaey et al., 2014). The major BSM models are described briefly below.

- *BSM1*: The plant configuration for BSM1 consists of activated sludge reactors (with two anoxic and three aerobic tanks in series) followed by a secondary settler. Various influent profiles (dry, rain, storm) are used as inputs and the evaluation is performed for a period of 7 days (Copp, 2000). The evaluation criteria are focused on the effluent quality and operating costs. Although, the layout is useful to evaluate short term effects of control/operation, it was not suitable to evaluate changes in sludge age etc. which have time constants that are longer than the 7-day evaluation period.
- *BSM1_LT*: In order to support long term evaluations, BSM1 is further extended temporally to BSM1_LT (LT stands for long term) (Rosén et al., 2004). A 1-year evaluation period is now used. Although, the plant configuration remains the same, the influent characteristics and evaluation criteria are improved. A dynamic influent generator (DIPDSG) (Gernaey et al., 2011) is used to generate influent files with different characteristics

(includes daily, weekly and yearly variations, medium and high intensity rainfall, temperature variations etc.).

- *BSM2*: In order to evaluate control strategies not only in the activated sludge reactors but also in other unit operations of a WWTP, BSM1_LT is extended with new unit operations. Primary settlers are included in the water line. Also, anaerobic reactors are added to the activated sludge tanks. A sludge line consisting of sludge dewatering units, thickeners and anaerobic digester is included. In terms of evaluation criteria, costs related to sludge handling and also the energy gains due to biogas production are considered (Jeppsson et al., 2007; Nopens et al., 2010).

Although, there has been a lot of progress in the BSMs, all of them are focussed on inside-the-fence evaluation of the WWTP performance. Life cycle analysis tools have been used to extend the scope of evaluation to include external factors like origin, production and transport of chemicals and electricity and also usage of the produced energy outside the WWTP (Arnell et al., 2017). The current project is the first attempt at spatially extending the BSMs outside-the-fence of a WWTP and thereby allowing for objective evaluation of integrated control strategies (and their impact on different sections of the UWS). More importantly, with an integrated BSM, evaluation of the control strategies can be directly linked to the river water quality instead of using traditional (and mostly indirect) metrics based on sewer overflow volumes and WWTP effluent quality.

Chapter 3

Influent Generator for WWTPs – A Pre-Cursor to System-Wide Modelling

This chapter summarizes the work presented in Paper I on an influent wastewater generator model for BSMs. An overview of the model is presented, calibration and validation of the model using full-scale data from two WWTPs and case studies using the influent generator are provided. The suitability of the model as the starting point for spatial extensions to the existing BSMs is discussed.

3.1. Introduction

Influent wastewater quality and flow rate are the most dynamic inputs to a WWTP and hence high frequency data for a variety of biochemical constituents is generally required for a detailed WWTP model calibration exercise. However, owing to large costs (in terms of time as well as money), it is not always possible to gather all the necessary data with the required level of detail. The Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) is a phenomenological model that is developed to address this problem. The approach is based on three important aspects namely: i) model parsimony – limiting the number of parameters to the smallest extent possible; ii) model transparency – using model parameters that have a physical meaning when possible; and iii) model flexibility – the proposed influent model can be easily modified/extended for other applications where long influent wastewater time series is needed.

Many of the essential elements available in DIPDSG can be re-used to develop the catchment and sewer models for the BSM-UWS. The work described in Paper I demonstrates the capabilities of the DIPDSG to generate influent profiles for real WWTPs. Hence, the suitability of the underlying model blocks from DIPDSG to describe the generation of wastewater from the urban catchment is strongly established. In this chapter, a summary of the DIPDSG model and the calibration/validation of the model to produce influent profiles for two WWTPs in Sweden and Denmark are presented. Furthermore, different scenarios where the model can be used are elaborated.

3.2. Model Description

The DIPDSG (Figure 3.1) is used to generate: i) flow rate; ii) pollutant concentrations; and iii) temperature dynamics at the WWTP inlet. The model structure includes source blocks for: i) flow rate; and ii) pollutant load generation from the catchment; and subsequently, iii) a transport block to describe their transport in the sewer network. Additionally, a temperature model block to simulate the temperature of the influent wastewater is also developed.

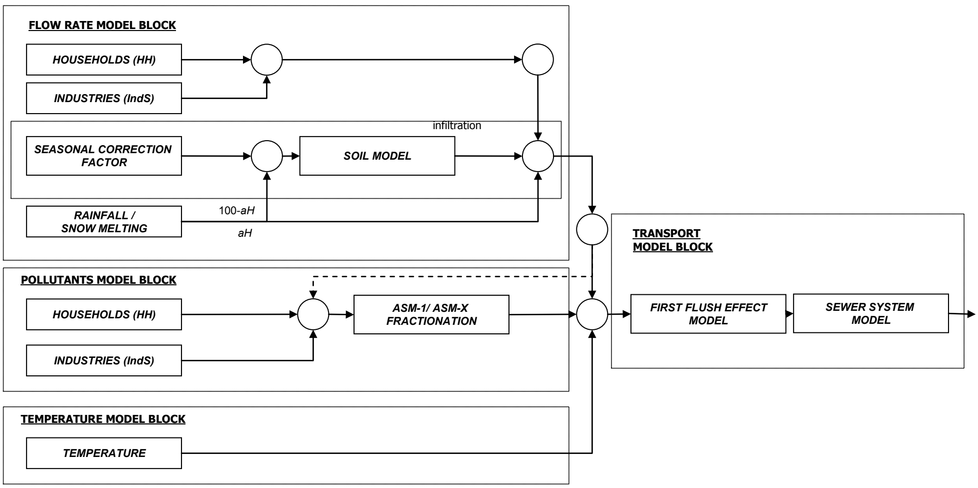

Figure 3.1: DIPDSG model illustrating various model blocks for generation of flow rate, pollutants and temperature at the WWTP inlet (from Paper I).

3.2.1. Flow Rate

In order to describe the influent flow rate, contributions from various sources, namely: i) households (HH); ii) industry (IndS); iii) rainfall (RAIN)/snow melting (SNOW); and iv) infiltration (SOIL), are modelled. Pre-defined flow rate profiles are used to simulate the daily, weekly and seasonal variations. These profiles are multiplied with the daily mean flow rate to generate the contributions from domestic and industrial sources during dry weather periods. RAIN and SNOW blocks describe the contribution of precipitation to stormwater flow. Precipitation on impervious area reaches the sewer network directly whereas that on pervious area reaches the infiltration (SOIL) model block. The SOIL block mimics the behaviour of soil with inputs from the precipitation on pervious areas and yearly groundwater profile (with seasonal variations in groundwater level). The output from the SOIL block determines the extent of infiltration to the sewer network as well as groundwater flow to the downstream aquifer. The net flow rate generation is the total sum of all the above contributions (HH+Inds+RAIN/SNOW+SOIL).

3.2.2. Pollutant Loads

Pollutant load generation from the catchment uses the same principles as the flow rate generation. Pollutants considered are soluble and particulate COD (COD_{sol}, COD_{part}), ammonium (NH_4^+) and total Kjeldahl nitrogen (TKN). Two sources of pollutant generation are considered (domestic and industrial). Source profiles for pollutant load variations on daily, weekly and yearly basis are defined. These profiles are multiplied with the mean pollutant loads. Only dry weather pollutant generation is modelled. Hence, it cannot simulate the dynamics of pollutant generation from the catchment due to rain events. However, first flush effects in the sewer network are considered. The pollutant loads generated are further transformed into ASM state variables depending on the type of activated sludge model used for the WWTP model (ASM1, ASM2, ASM2d and ASM3).

3.2.3. Temperature

The temperature block describes the variation in temperature at different time scales. The daily changes in temperature during day and night times are considered. Also, seasonal effects leading to variation in temperature are included. Both these phenomena are modelled using sinusoidal wave forms with daily and yearly frequencies. In addition, the change in temperature due to precipitation events is considered. A temperature drop proportional to the intensity of rainfall/snowmelt is modelled.

3.2.4. Transport

A tanks-in-series approach is used to represent the sewer network. A non-linear reservoir model is used to represent the volume balance in each tank. Additionally, a first-flush block describes the high pollutant loads to the WWTP during the beginning of a rain event. The higher loads are due to washoff of particulate pollutants, which accumulate in the sewer network during a preceding dry weather period. The first-flush model assumes that a constant fraction of the particulates are accumulated in the sewer network during dry weather periods. During rain events, a Hill function (Hill, 1910) is used to represent the washoff of the accumulated solids depending on the flow rate and the available solids in the sewer.

3.3. WWTPs Under Study

Data from two large WWTPs at Bromma, Sweden (WWTP1) and Lynetten, Denmark (WWTP2) are used for the calibration of the DIPDSG model (Table 3.1). Carbon (COD), nitrogen and phosphorus removal are achieved in both WWTPs using mechanical, biological and chemical unit operations. Data for WWTP1 is available for 2009/2010 while WWTP2 data is available for the period between 2010 and 2011. Daily average values are used for flow rate and pollutant loads.

Table 3.1: Major inflow characteristics for WWTP1 and WWTP2.

Parameter	WWTP1	WWTP2
Catchment area	25 000 ha	76 000 ha
Population equivalents	300 000	750 000
Treatment capacity	10 800 $m^3.h^{-1}$	23 000 $m^3.h^{-1}$
Influent flow rate	120 000 $m^3.d^{-1}$	170 000 $m^3.d^{-1}$
COD load	35 000 $kg.d^{-1}$	95 000 $kg.d^{-1}$
N load	3 100 $kg.d^{-1}$	7 300 $kg.d^{-1}$
P load	380 $kg.d^{-1}$	1 200 $kg.d^{-1}$

Flow and temperature data are available at high frequency (1 sample every 6 mins) at both locations. Rainfall data with a frequency of 3 samples per day is available for the catchment of WWTP1. Two to three daily averaged flow proportional samples per week are available for pollutant concentrations at WWTP2. Due to the limited data availability, WWTP1 data is used for the calibration of influent flow rate and temperature while WWTP2 data is used to calibrate the influent pollutant loads.

3.4. Results

3.4.1. Flow Rate at WWTP1

Global sensitivity analysis (GSA) results from Flores-Alsina et al. (2012b) are used to perform a step-wise calibration and validation exercise. Key parameters affecting long term (monthly, yearly trends), daily and hourly dynamics are determined from the GSA study. The first step is to calibrate the model for yearly and monthly average flow rate data. It is assumed that 56 % of the influent flow to the WWTP originates from domestic and industrial sources, where domestic sources contribute to 82 % of the flow and the industrial sources contribution is 18 %. The remaining 44 % arises from infiltration to the sewers. The parameters calibrated (Table 3.2) are: i) *PE* (300 000 PE) (population equivalents) and *QperPE* (0.15 $m^3.d^{-1}.PE^{-1}$) (daily average flow rate per population equivalent) from the HH block; ii) Q_{ind} (10 000 $m^3.d^{-1}$) (daily average industrial flow rate) from the IndS block; and iii) SOIL block parameters *Infbias* (80 000 $m^3.d^{-1}$) (effects mean annual infiltration flow rate to sewer), K_{down} (30 000 $m^2.d^{-1}$) (determines flow to downstream aquifer) and K_{inf} (450 000 $m^{2.5}.d^{-1}$) (determines infiltration flow to sewer). In order to calibrate the daily dynamics, parameters representing the wet weather phenomena are calibrated. These include: i) RAIN/SNOW block parameters *Qpermm* (25 000 $m^3.mm^{-1}$) (volume per mm rain), *Qpercm* (35 000 $m^3.cm^{-1}$) (volume per cm of snow) and A_{imp} (75 %) (percentage of impervious area); and ii) SOIL block parameters A_{soil} (25 000 m^2) (hypothetical area of the soil model) and ϕ_{soil} (2.0 $m.d^{-1}$) (porosity of soil). The hourly dynamics are calibrated using the number of *subareas* (8) parameter. It represents the length of the sewer network. Each subarea includes three completely mixed tanks in series.

Table 3.2: Major parameters used for calibration of DIPDSG in different time scales.

Calibration variable	Monthly	Daily	Hourly
Flow rate	*PE*	*Qpermm*	*subareas*
	QperPE	*Qpercm*	
	Q_{ind}	A_{imp}	
	Infbias	A_{soil}	
	K_{down}	ϕ_{soil}	
	K_{inf}		
Temperature	T_{bias}	$T_{d,amp}$	G_{rain}
	T_{amp}	$T_{d,freq}$	G_{snow}
	T_{freq}	$T_{d,phase}$	
	T_{phase}		
Pollutants	*PE*	*FFfraction*	
	$P_{i,hh}perPE$	M_{max}	
	$P_{i,ind}$	Q_{lim}	

The calibration (Figure 3.2a) (year 2009) and validation (Figure 3.2b) (year 2010) results for the flow rate indicate that the model is successful in reproducing the yearly flow rate trends. From Figures 3.2c, e and Figures 3.2d, f, the diurnal variations in daily flow rate due to changes in water usage patterns throughout the day can be noticed. Morning and evening peaks, followed by a minima during night can be observed. Both the calibration (e.g. Figure 3.2c – day 164) and validation (e.g. Figure 3.2d – day 161) indicate that the model can successfully reproduce the high flow rates during rain events. It is also interesting to note the flow rate “tails” (delayed high flows) after these rain events. Such an effect is observed due to the delayed flow rate (e.g. from infiltration to sewers) that reaches the WWTP even after the end of a rain event. The effect of snow melting on flow rate, leading to increased flow rate at the WWTP, can be noticed in Figure 3.2e (e.g. day 86) and Figure 3.2f (e.g. day 81). However, it can be seen that the model has some inaccuracies as well. This can be due to non-availability of high frequency rain data and/or inaccuracies in the rainfall and snowmelt models.

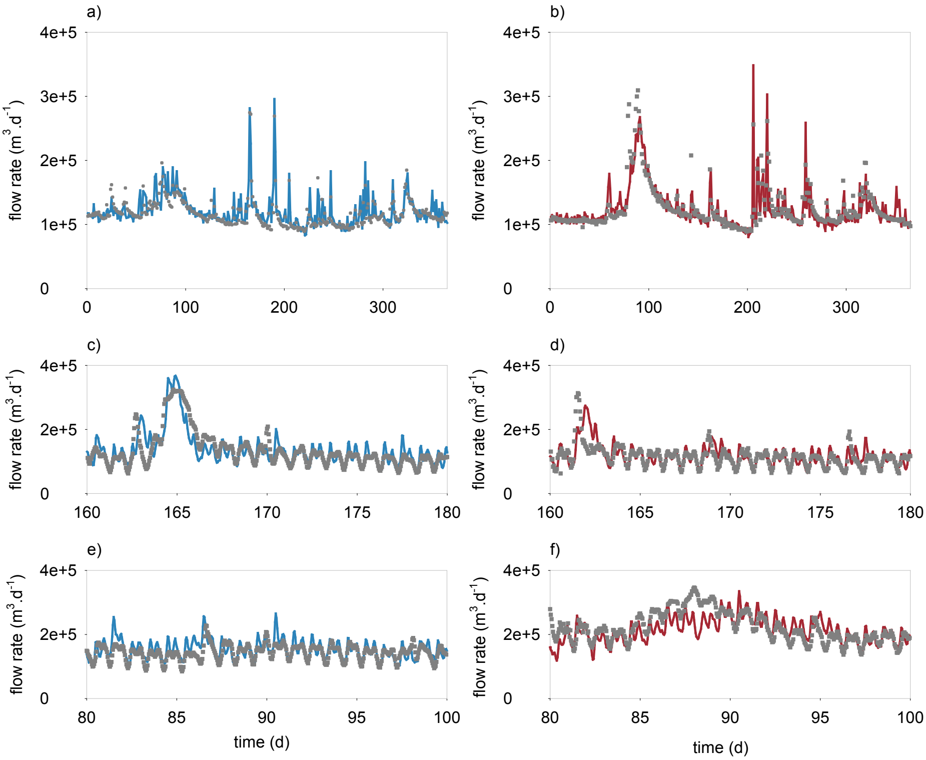

Figure 3.2: Calibration (blue) and validation (red) results of DIPDSG for flow rate data (grey) from WWTP1. Daily average flow rate for the entire year (a, b); and snapshots from summer (c, d) and winter time periods (e, f) representing higher flows due to rainfall and snow melting, respectively. Day 0 is 1st January, day 80 is 21st March and day 160 is 9th June (modified from Paper I).

3.4.2. Temperature at WWTP1

A similar approach is followed to calibrate the temperature at WWTP1. Firstly, the long term dynamics are calibrated using the parameters representing bias (T_{bias}) (15 °C), amplitude (T_{amp}) (5 °C), frequency (T_{freq}) (2π rad.y^{-1}) and phase shift (T_{phase}) (0.8π rad) for the yearly sinusoidal curve. For the daily dynamics, the variatios are mainly caused by day/night changes in temperature. The daily variation in temperature is described using the parameters $T_{d,amp}$ (0.5 °C), $T_{d,freq}$ (2π rad.d^{-1}) and $T_{d,phase}$ (1.5π rad) representing the amplitude, frequency and phase shift for the daily sinusoidal curve. During rain events and snow fall, a drop in temperature can be observed. Precipitation effects on temperature are calibrated using the proportional gain parameters G_{rain} (0.4) and G_{snow} (0.4). The parameters lead to a decrease in temperature (in comparison to the dry weather values) proportional to the precipitation intensity. All the calibrated parameters are listed in Table 3.2.

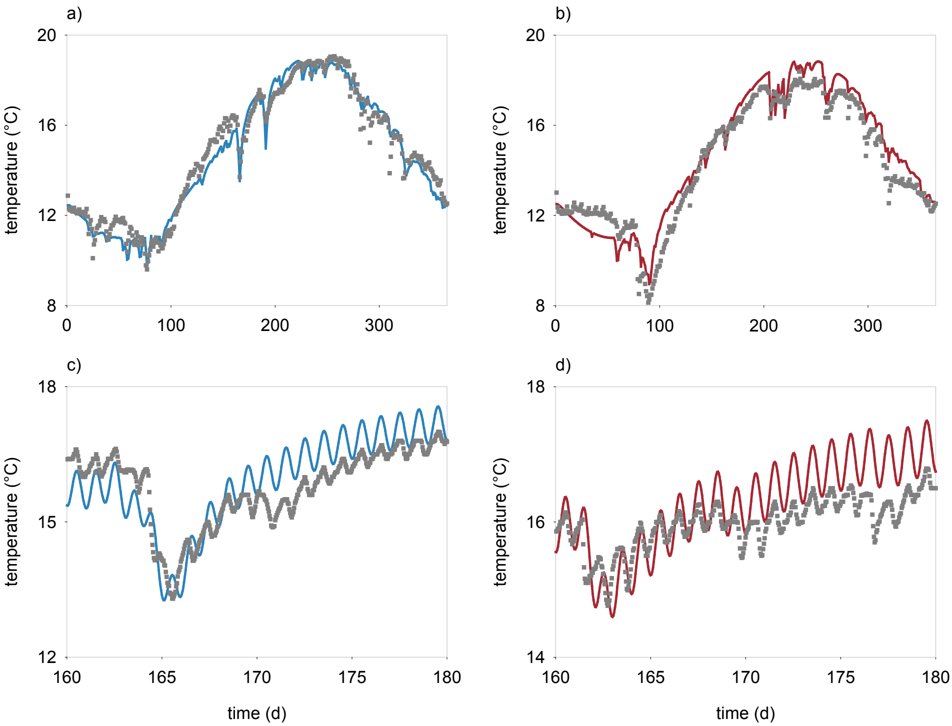

Figure 3.3: Calibration (blue) and validation (red) results from DIPDSG temperature module for WWTP1 temperature data (grey). Yearly dynamics (a, b) and daily variations (c, d) are presented. Day 0 is 1st January and day 160 is 9th June (modified from Paper I).

Although the temperature model is simple, it is successful in reproducing the daily and seasonal trends in temperature. The calibration and validation of daily and monthly temperature changes are presented in Figures 3.3a, b. Data from year 2009 is used for calibration while that from 2010 is used for validation. A significant change in temperature between cold (e.g. 0 – 100 days) and warm months (e.g. 200 – 300 days) can be noticed in the data and is reproduced successfully by the model. The variation in temperature on a daily basis (day and night differences) is also clearly reproduced both during calibration (Figure 3.3c) and validation (Figure 3.3d). Additionally, the model is able to describe the drop in temperature observed due to precipitation events as observed in Figure 3.3c (day 165) and Figure 3.3d (day 163). However, in some cases, a bias or delay in the results is noticed (e.g. days 170 – 180 in Figure 3.3c), especially with respect to daily variations and the influence of precipitation events. A more accurate description would hence require improvements to the modelling of daily variations and precipitation induced temperature differences in the model.

3.4.3. Influent Pollutant Loads at WWTP2

Due to the simplified approach, pollutant load generation from the catchment is only represented by the HH and IndS blocks. No wet weather pollutant load from the HH and IndS blocks is assumed. Nevertheless, the transport block can be used to represent the additional particulate pollution arising from the sewer network during the beginning of rain events. During the calibration exercise, COD and TKN pollutant loads at WWTP2 are described by the model using the parameters given in Table 3.2. In the HH blocks, *PE* (750 000 PE) (population equivalents) and daily average pollutant loads per PE ($P_{i,hh}perPE$) (COD_{sol}: 16.42 g.d^{-1}.PE^{-1}; COD_{part}: 97.82 g.d^{-1}.PE^{-1}; NH_4^+: 4.82 g N.d^{-1}.PE^{-1}; TKN: 7.23 g N.d^{-1}.PE^{-1}) are used for calibration. Similarly, the daily average loads ($P_{i,ind}$) (COD: 18 000 kg.d^{-1}; TKN: 2 700 kg N.d^{-1}) from the industry block (IndS) are calibrated. The contribution of industrial loads to the total pollutant load is determined based on the data. Also, the ratio between particulate and soluble COD is determined from the data. For the particulate pollutant variation during rain events, the first-flush model is calibrated. The major parameters are: i) *FFfraction* (0.75) – determines the fraction of particulates that settle in the sewer network during dry weather periods; ii) M_{max} (120 000 kg) – maximum particulate load that can settle in the sewer network (this parameter limits the amount of solids available in the sewer network, especially during long dry weather spells); and iii) Q_{lim} (560 000 m^3.d^{-1}) – flow rate limit above which washoff of particulates intensifies. It should be noted that the model does not use absolute flow rate limit values to trigger the washoff. Instead, a Hill function is used to represent a smooth transition in washoff with increasing flow rate values.

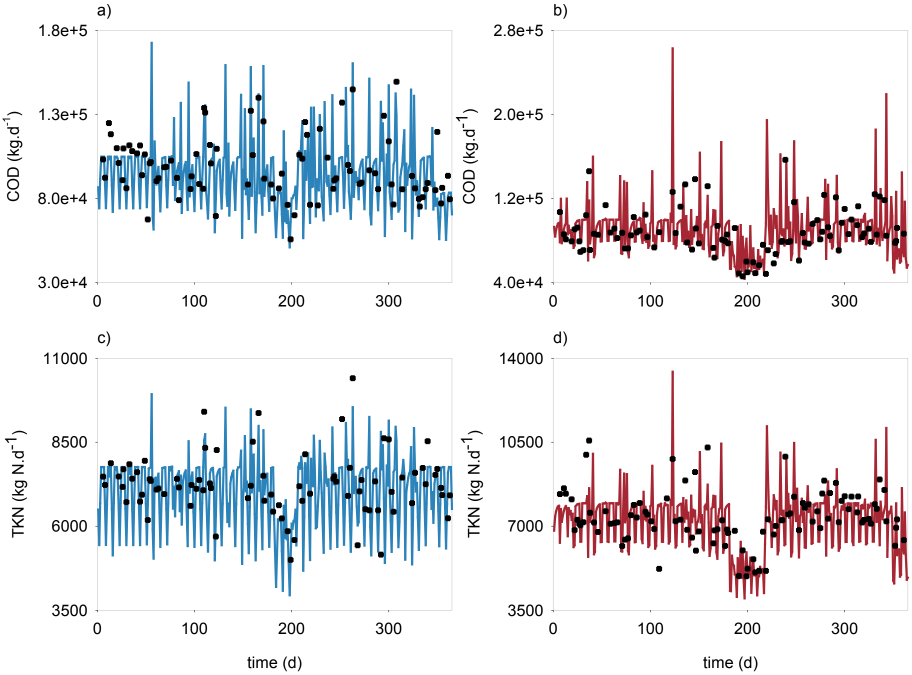

Figure 3.4: Pollutant loads (kg.d^{-1}) for COD (calibration (a), validation (b)) and TKN (calibration (c), validation (d)) at WWTP2. Calibration (blue) and validation (red) results from the model compared with data (black). Day 0 is 1st January (from Paper I).

Data from year 2010 is used for calibration while that from 2011 is used for validation. For both COD (Figures 3.4a, b) and TKN (Figures 3.4c, d), it is noticed that the model predictions (calibration and validation) are well within the range of the observed values. Additionally, it is clear that the monthly trends are well described by the model, especially the drop in pollutant loads during the holiday period (month 7 – around day 200). This is observed during both the calibration and validation periods for COD as well as TKN. Another effect is the increased pollutant loads at the beginning of rain events due to first flush of pollutants (e.g. COD: Figure 3.4a – day 166; TKN: Figure 3.4d – day 37). Although, the model fits reasonably to the daily pollutant loads, the accuracy of the model in describing diurnal variations could not be validated due to lack of high frequency data. An area of potential improvement for the model is the description of pollutant loads from rainfall runoff. The current model assumes that no pollution reaches the WWTP from surface runoff of stormwater. It can be improved in a simple manner by assuming constant pollutant concentrations in the surface runoff during rain events.

3.5. Scenario Analysis

A calibrated and validated influent generator model can be put to many other uses apart from using it for the designed purpose of generating long term influent data for WWTPs. In order to present the capabilities of the model, three example scenarios are evaluated:

i. effect of rainfall on influent flow rate;

ii. determining uncertainty in influent loads;

iii. generating high frequency data.

3.5.1. Effect of Rainfall on Influent Flow Rate

The model considers the effect of rainfall on the influent dynamics through two different model blocks: i) RAIN block simulates the runoff generated immediately during the rainfall; and ii) SOIL model describes the delayed response of the catchment to rainfall. The impact of two different types of hypothetical rainfall time series (generated using the rainfall model presented in Gernaey et al. (2011)): i) low intensity and high frequency – many small rain events; and ii) high intensity and low frequency – a few heavy rain events, is evaluated (Figure 3.5). The results clearly indicate a strong difference in the influent flow rate profile for the two scenarios. Figure 3.5a represents frequently repeated low intensity events leading to a lot of peak flows to the WWTP although the peaks are only around 2 – 3 times the dry weather values. In Figure 3.5b, a much stronger effect is observed with higher peak values.

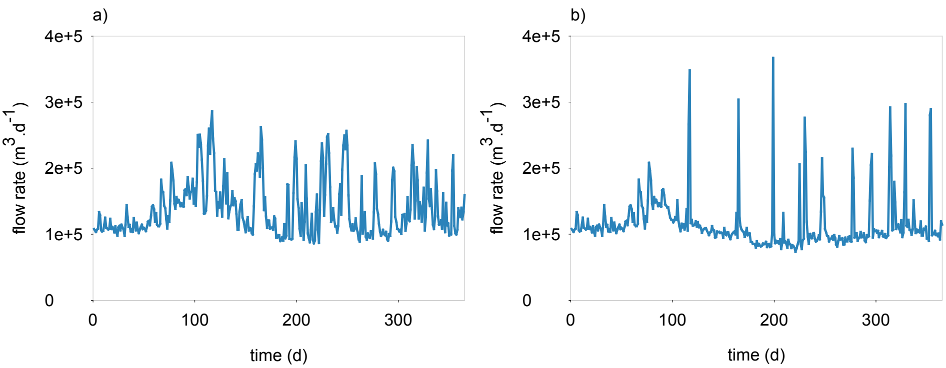

Figure 3.5: Scenario analysis for flow rate at WWTP1 using two different rainfall patterns. Low intensity and high frequency rainfall (a) and high intensity and low frequency rainfall (b) patterns. Day 0 is 1st January (from Paper I).

Such an analysis can be valuable for WWTP modelling to evaluate various control and structural modification possibilities. The effect of rainfall profiles on the control strategy performance can be evaluated and also optimized. Wet weather operational procedures can be identified to mitigate any potential issues during rain events. This can be simple strategies like activating a bypass control, modifying recycle and wastage rates to advanced operation including aeration tank settling etc.

3.5.2. Determining Uncertainty in Influent Loads

Although the model has been successful in describing the pollutant loads at WWTP inlets, it is well known that ASM fractionation parameters have a strong influence on the organic matter biodegradability (Henze et al., 2000). Most modelling studies either use the default fractionation parameters or use limited data to calibrate the fractionation parameters. The ASM fractionation block in the DIPDSG model provides the opportunity to evaluate the uncertainty arising from the values chosen for the fractionation parameters. 250 Monte Carlo simulations are performed by varying the ASM fractionation parameters in a range of 50 % from the default values. Figure 3.6 shows that all the data points can be represented within the 5th and 95th percentiles. Additionally, it also generates high frequency data, which is missing in the measurements (e.g. peak values). The uncertainty thus produced can later be used to simulate WWTP dynamics by varying the influent loads within the range of values obtained using the DIPDSG model.

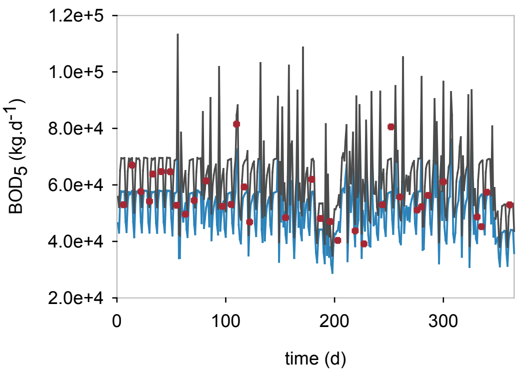

Figure 3.6: Monte Carlo simulations representing the 5th (blue) and 95th (grey) percentile for influent 5-day biochemical oxygen demand (BOD_5) loads (kg.d^{-1}) at WWTP2. Data is represented as red dots. Day 0 is 1st January (from Paper I).

An advantage of such an analysis is that it provides a clear idea on the range of variability in organic matter arriving at the WWTP. This information can be used to design and operate the WWTPs without using large safety factors (Tchobanoglous et al., 2004). It can lead to reduction in construction costs due to reduced design volumes (Belia et al., 2009).

3.5.3. Generating High Frequency Data

Another potential application of the DIPDSG model is to generate high frequency data for detailed WWTP process analysis with limited measured influent data for calibration. Such data is especially important for WWTP modelling studies that are aimed at analysing control strategies or evaluating peak effluent concentrations etc. Also, the high costs involved in data collection efforts can be reduced using such an approach. Figure 3.7 represents the match between data and model results for daily pollutant loads. Furthermore, high frequency (hourly) diurnal profiles for the COD load to a WWTP representing morning and evening peaks are produced by the model. It is noticed that the profiles for consecutive days are different in spite of having similar source profiles for the HH and IndS blocks. This is due to the addition of white noise. Additionally, the diurnal profiles can be varied by using the *subareas* parameter representing the length of the sewer network. The longer the sewer network, the more delayed and attenuated are the daily peak values.

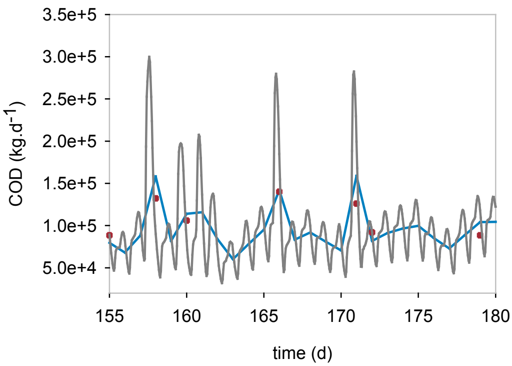

Figure 3.7: Generating high frequency data (grey) from the DIPDSG model calibrated with daily average influent data (red) for COD loads (kg.d^{-1}). Daily average values from the simulations (blue) are also presented. Day 155 is 4th June (from Paper I).

3.6. DIPDSG as a Pre-Cursor to Catchment and Sewer Models

The calibration and validation efforts confirm the capability of the DIPDSG toolbox to accurately model short term and long term influent profiles for WWTPs. Various scenarios that can be simulated using the model were also presented. It is clear that the phenomenological approach adopted in the model development allows modelling some of the underlying processes involved in catchment and sewer network (albeit in a simplistic manner) in order to describe the influent profiles. Hence, the suitability of the building blocks from the DIPDSG model as components in the system-wide benchmarking exercise can be examined.

Several of the model blocks can be directly used for the system-wide model, especially to describe the catchment and sewer network dynamics. In order to represent the dry weather wastewater generation from the catchment: i) HH and IndS blocks describing the flow rate generation from catchment; and ii) pollutants block that includes domestic and industrial sources are suitable. However, they need to be integrated to represent the generation of wastewater (which includes both pollutants and flow rate). The temperature block describing seasonal, daily and short term (hourly) variations is directly transferable without any further modifications. The output from the model is useful to describe temperature state variable in the sewer network, WWTP as well as the river. Another model block that is suitable for the system-wide model is the SOIL block. It can be used to represent the contribution of infiltration flow to the sewers during dry weather as well as to describe the delayed response from the catchment to rainfall. The first-flush model can also be used directly. It might be necessary to use multiple first-flush blocks (e.g. for each sub-catchment).

However, the RAIN and SNOW blocks can be further improved by using a more physically-based approach. The calibration parameters *Qpermm* and *Qpercm* do not have a direct physical meaning. Hence, an approach using runoff coefficients and catchment area can be considered. The transport block representing the sewer network can be used as a starting point to describe the transport of wastewater in the sewer network. However, instead of using a single transport block, the underlying sewer models should be used at different locations in the catchment.

Nevertheless, the DIPDSG model lacks some key features for it to be used directly as a model for describing catchment and sewer network. Although the transport block can describe the effect of a sewer network, it does not have the possibility to model storage tanks and overflow structures. Without these features, control and operational strategies cannot be evaluated (which is an important feature for an integrated model). Another aspect is the description of pollutant load generation during wet weather events. The DIPDSG assumes that no pollutants are generated from the catchment surface during wet weather events. While this assumption may be justified in the case of influent generators, it is essential to have such phenomena described in an integrated model.

Although the building blocks are available for describing some of the processes, it is obvious that considerable work is required in order to adapt them to an integrated modelling framework. For example, HH and IndS blocks should be used at different locations (for each sub-catchment) and also integrated with other source generation blocks (like pollution loads and wet weather phenomena). In conclusion, the model blocks (especially for flow rate/pollution load generation) in the DIPDSG represent a good starting point for the system-wide BSM. The fact that the model has been successfully calibrated with data from real WWTPs only adds to the confidence in the model and also makes a strong case for adapting it to the maximum extent possible for integrated modelling purposes.

3.7. Summary of Key Findings

The calibration and validation of the DIPDSG with full-scale data is successfully performed. The items below summarize the key findings.

- The flow rate model is able to describe the daily, weekly and yearly variations in domestic and industrial wastewater generation. The effect of rainfall and snowmelt are also well modelled.
- The temperature block can successfully model the daily and yearly trends in wastewater temperature variation. Also, the effect of precipitation events on temperature is well predicted.
- The pollutant loads in raw influent wastewater are reproduced by using the DIPDSG. The increased particulate pollutant load at the beginning of rain events is simulated using the model.
- The application of DIPDSG for future studies to produce raw influent wastewater date for different scenarios is demonstrated using various case studies.
- The suitability of the DIPDSG as a good starting point for catchment and sewer extensions to the BSM platform is established.

Chapter 4

BSM-UWS: Model Library

This chapter describes the modelling principles for the system-wide BSM (BSM-UWS) from Paper II, Paper III and Paper IV. The building blocks for all the four sections of an UWS (catchment, sewer network, WWTP and river water system) are summarized. Evaluation criteria for sewer overflows, WWTP effluent and river water quality are defined.

4.1. Introduction

It has been well established that WWTPs are strongly interconnected with other segments of an UWS (catchment, sewer network, river) (Rauch et al., 2002). Hence, the idea of operating and optimizing each of these sections individually does not necessarily lead to the best solution for the entire UWS. It is also important to evaluate the impact of the operation of individual sections on the river water quality. Based on the EU Water Framework Directive to improve the chemical and ecological quality of rivers (Council of the European Communities, 2000), it has become increasingly important to perform holistic assessment of the UWS. Integrated models are valuable tools for such an analysis (Mannina et al., 2006; Schütze et al., 2011; Solvi, 2007). These models can be used to study structural modifications (e.g. Astaraie-Imani et al., 2012; Weijers et al., 2012) and control strategies (Fu et al., 2009; Seggelke et al., 2013) spanning across different sections of the UWS.

In order to be able to model and evaluate the UWS in a holistic manner, a system-wide BSM model is envisioned. The first step towards this goal is to develop models for catchment, sewer network and river water system that can be easily integrated with the existing WWTP models. This chapter provides the details of the model library for all the four sections (catchment, sewer network, WWTP and river water system). In addition to the existing WWTP evaluation criteria (Gernaey et al., 2014), performance criteria for the sewer network (based on sewer overflows) and river quality (based on chemical quality of the river) are introduced.

4.2. Model Description

4.2.1. Catchment

The starting point for the catchment model is the BSM2 Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) (Gernaey et al., 2011), which is described in Chapter 3. The most important state variables included in the catchment model are flow rate and five pollutant variables. COD is divided into soluble (COD_{sol}) and particulate (COD_{part}) fractions. Ammonium (NH_4^+), nitrate (NO_3^-) and phosphate (PO_4^{3-}) are modelled as soluble components. The model simulates the generation of wastewater from four major sources, namely: i) domestic (DOM); ii) industrial (IND); iii) stormwater (SW); and iv) infiltration to sewers (INF).

Domestic (DOM) sub-model simulates the generation of domestic wastewater and pollutant loads. The variation in wastewater generation is modelled using three pre-defined profiles for daily, weekly and yearly variations in flow rate and pollutant loads (Figure 4.1). The daily variations are due to changes in wastewater generation depending on the time of day. Typically, more wastewater is generated during mornings and evenings with lowest generation during nights. Weekly changes are due to difference in lifestyle during weekdays and weekends. This leads to lower wastewater flow and pollutant loads during the weekend. Also, yearly trends with holiday periods (with lower wastewater generation) during summer are considered. All three source profiles are combined to generate a dynamic time series, which is further multiplied with the flow rate/pollutant load per population equivalent and the number of population equivalents at each sub-catchment to produce a dynamic wastewater profile for the sub-catchment. Zero mean white noise is added to the output in order to avoid having identical values during different days of the week and also to avoid too strong correlation between different variables.

Industrial (IND) sub-model considers weekly and yearly variation in flow rate and pollutant loads. The weekly variations are a reflection of the production cycles and maintenance periods (Figure 4.2a). Production cycles with peak production during day time and lower production during the night are assumed. A maintenance window on Fridays, which results in higher wastewater generation due to cleaning activities, is considered. Additionally, weekends are considered to show the lowest production. Holiday periods during summer and Christmas with lower production represent the yearly variations (Figure 4.2b).

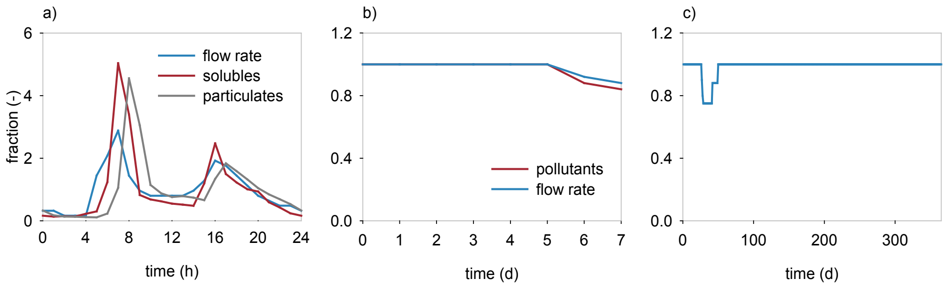

Figure 4.1: Diurnal variation in pollutant loads and flow rate (a). Weekly variation with two different profiles for flow rate and pollutants (day 0 is Monday) (b). Yearly profile (starting 1st July) with similar dynamics for pollutants and flow rate (c) (from Paper III).

Similar to the DOM sub-model, industrial wastewater generation is modelled by combining the source profiles for weekly and yearly variations and multiplying it with the mean daily pollutant load/flow rate. Also, the profiles for DOM and IND sub-models for daily, weekly and yearly variations are different. Additionally, the pollutant loads and their ratios (COD/N) are different for both sources. Similar to the DOM sub-model, zero mean white noise is added to the outputs.

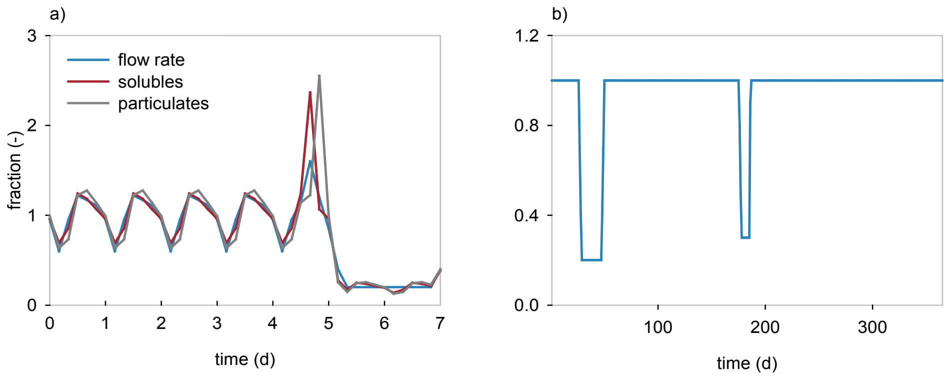

Figure 4.2: Weekly variation in industrial flow rate and pollutant loads (day 0 is Monday) (a). Yearly industrial wastewater production trends (b). The yearly profile begins on 1st July. For simplicity, it is assumed that the first day of July is a Monday (from Paper III).

Stormwater (SW) sub-model describes the generation of stormwater that is conveyed to the sewer network during rain events. A rainfall runoff model is used to simulate the conversion of rainfall to urban runoff. The model considers that impervious and pervious areas contribute differently to the urban runoff. The rainfall on impervious areas is directly converted into runoff using a runoff coefficient. For the pervious areas, rainfall leads to delayed runoff from the catchment. The contribution from pervious areas is not directly included in the model but a SOIL model describes the infiltration to sewers taking into account the pervious area contribution as an input.

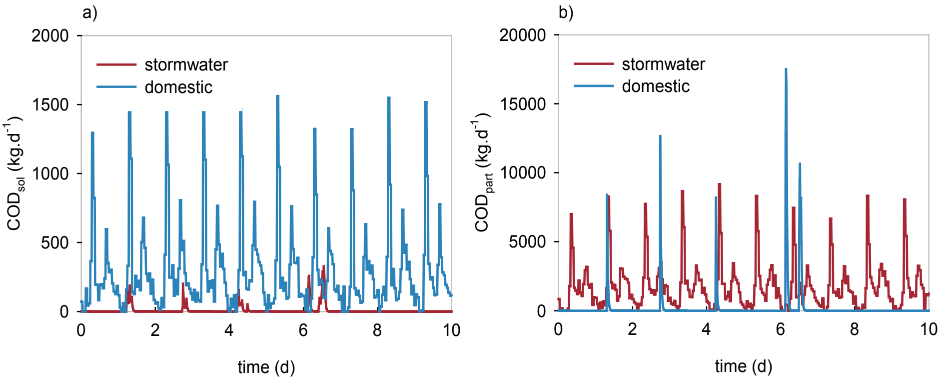

Figure 4.3: Effect of rain events on soluble pollutants (COD_{sol}) (a) and particulates (COD_{part}) (b) (from Paper III).

Pollution generation from storm events is modelled for soluble as well as particulate pollutants (Butler and Davies, 2011). Soluble pollutants reaching the sewer network due to rain events are considered to have a constant concentration for the entire rain event. Hence, event mean concentrations (*EMC*) are used (Figure 4.3a). For the particulate pollutant (COD_{part}), an accumulation and washoff model describes the generation of pollution during rain events. Figure 4.3b presents the difference between COD_{part} load (kg.d^{-1}) during dry weather and rain events illustrating the impact of the first-flush effect. A description of the model is provided in Section 2.2.1. During dry periods, the particulate pollutants accumulate on the catchment surface at a constant rate. A maximum limit is defined in order to avoid very high solids content on the surface, especially during long dry spells. During rain events, the accumulated mass of solids is washed off at a rate related to the rainfall intensity.

Infiltration to Sewers (INF) sub-model includes inputs from: i) upstream groundwater levels (modelled using a sinusoidal wave with a yearly frequency); and ii) percolation of stormwater from pervious areas (this input comes from the SW sub-model) into a hypothetical storage tank (SOIL). The outputs from the tank are: i) infiltration to the sewer network determined by a non-linear relationship between tank level and infiltration flow rate (Gernaey et al., 2011); and ii) water level in the groundwater system. As shown in Figure 4.4, the infiltration to sewers follows an annual trend as well as short term variations due to rain events. The purpose of this model is to describe the infiltration to the sewers and hence the groundwater system dynamics are not modelled in a detailed manner. Also, it is assumed that no pollutant loads reach the sewer from the infiltration model.

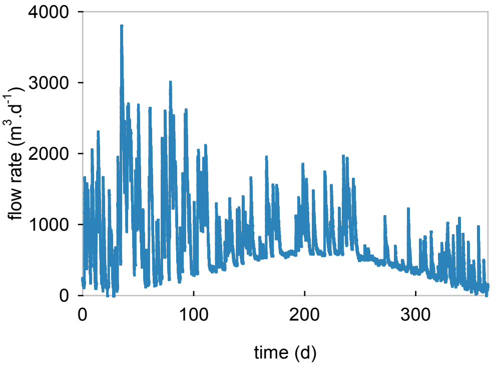

Figure 4.4: Infiltration to the sewer network depicting annual trends as well as rainfall dependent inflows. Day 0 is 1st July (from Paper III).

In addition to the above four models, a supplementary module that generates rainfall time series based on historic data is also developed. It uses a stochastic rainfall generator approach (Richardson, 1981; Talebizadeh et al., 2016). The first step is to determine the dry and wet periods during the entire evaluation period. A two state Markov chain model is used to represent the wet and dry periods. The transition between these states is estimated using historic rainfall data. Starting from a default state (assumed as dry), the model determines the wet and dry periods (with each period lasting 15 min). The next step is to determine the rainfall intensity during the wet periods. For this purpose, the historic rainfall data is fitted to a Gamma distribution (Buishand, 1978). Hence, for each wet period, the gamma distribution is sampled to determine the rainfall intensity for that period. Although monthly and annual total rainfall are well described, the model does not reproduce high intensity rainfall events well. This is due to the fact that such high intensity rainfall events are very rare and hence the probability of such an event being sampled from the fitted Gamma distribution is low. It is important to highlight that the approach presented herein is an empirical one and an accurate description of extreme rainfall events will require detailed modelling efforts.

4.2.2. Sewer Network

The generated wastewater and stormwater from the catchment model are conveyed to the WWTP through the sewer network. Also, excess flow beyond the sewer capacity is discharged into the river as sewer overflow. The model consists mainly of three elements, namely: i) TRANSPORT sub-model to describe the flow of wastewater/pollutants in the sewer; ii) STORAGE sub-model that represents various storage tank configurations and the control elements (CONTROL) present (e.g. pumps, throttle valves); and iii) FIRST-FLUSH sub-model that mimics the generation of high pollutant loads at the beginning of rain events.

Pollution/Flow Rate Transport (TRANSPORT) sub-model simulates the transport of wastewater through the sewer network. The conceptual modelling approach using a series of linear reservoirs is implemented (Viessman et al., 1989). Such reservoir models are used to connect different sub-catchments to the trunk sewer network, which is then connected to the WWTP. The number of such reservoirs and the residence time of each reservoir represent the capacity of the sewer network. Figure 4.5 shows that a longer sewer network will lead to longer time delays between the inflow and outflow from the sewer network and also lead to attenuation of the flow rate and pollutant concentrations.

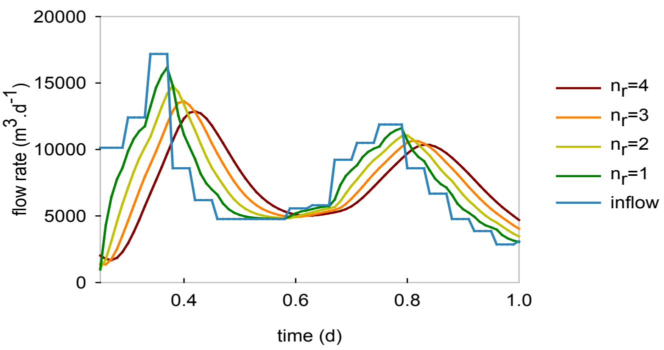

Figure 4.5: Effect of number of consecutive reservoirs on sewer outflow for a given inflow (blue) (from Paper III).

Storage Tank (STORAGE) sub-model represents the different possible configurations of the storage tanks. Four different configurations are modelled (Figure 4.6):

i. online pass-through tank;

ii. online bypass tank;

iii. offline pass-through tank;

iv. offline bypass tank.

Online tanks are in line with the sewer network, which means that all flow during dry weather as well as rain events reaches the storage tank. Offline tanks are utilized only when the flow rate crosses a threshold throttle limit for the downstream sewer pipe. In a pass-through configuration, the overflow weir is at the end of the pipe. Hence, any excess flow mixes with the wastewater that is already in the storage tank before overflowing. In a bypass configuration, the excess flow overflows without mixing with the constituents that are already in the storage tank (ATV, 1992; Schütze et al., 2011). A conceptual tank with inputs from upstream sewer network(s) and/or catchment(s) determines the storage tank volume or level. The two outputs from the model are: i) flow to the downstream sewer network; and ii) overflow to the receiving water (if any).

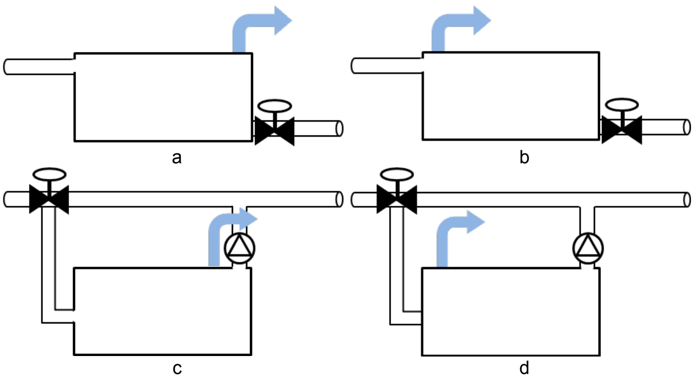

Figure 4.6: Various possible configurations for storage tanks. Online pass-through (a), online bypass (b), offline pass-through (c) and offline bypass (d) configurations. Actuator elements such as valves and pumps, are included in the model (from Paper III).

Other key aspects are the control elements (**CONTROL**) – pumps and valves that are available at the storage locations. The effect of valve opening on the throttle flow to the downstream sewer is modelled by assuming a linear relationship between valve opening and outflow rate. The valve opening can be defined either as a constant value for the entire period or as an input from a control system. For storage tanks where pumping is used to deliver the stored wastewater to the downstream sewer/WWTP (e.g. offline tanks), a pumping model (Kroll et al., 2016) with the possibility to implement frequency control and also define multiple pumps at a storage location (each with a different maximum pumping capacity) is used. The pumping rate can be altered using a control system.

Particulate First-Flush (**FIRST-FLUSH**) sub-model is used to represent the high particulate loads that are generated from the sewer network at the beginning of a rain event (Figure 4.7). It is modelled based on the approach described in Gernaey et al. (2011). A fraction of the particulate COD load is deposited in the sewer pipes during dry weather. This accumulated load is washed off during rain events as a function of the flow rate.

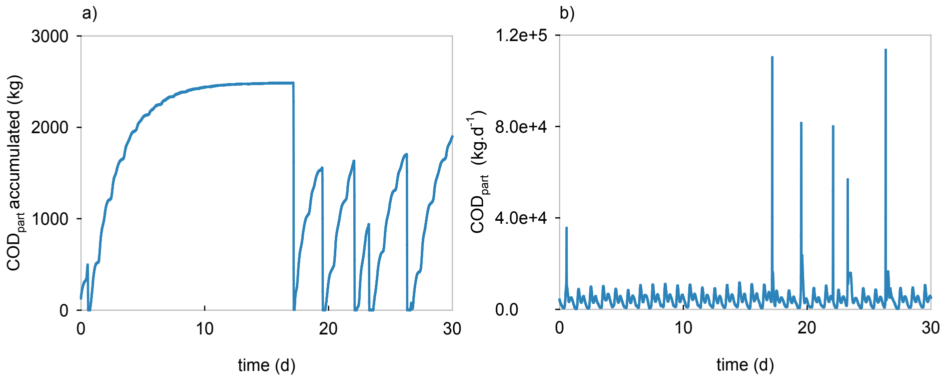

Figure 4.7: COD_{part} mass (kg) accumulated in the sewer (a) and the consequent washoff during rain events (b) described using the first-flush model (from Paper III).

4.2.3. Wastewater Treatment Plant

The model library for WWTP unit operations that is developed for BSM1 and BSM2 (Gernaey et al., 2014) is re-used when developing the WWTP section for BSM-UWS. The major unit operations are described below.

Primary Clarifier (PC) uses the Otterpohl & Freund model (Otterpohl & Freund, 1992) to describe the settling process. Settling efficiency factors for different state variables are defined based on the residence time of the settler. It is assumed that only particulate fractions undergo settling and all the soluble state variables reach the biological reactors. Additionally, adjustment factors to vary settling efficiency (e.g. with chemically enhanced settling) and also to define separate settling efficiencies for the different state variables are also introduced.

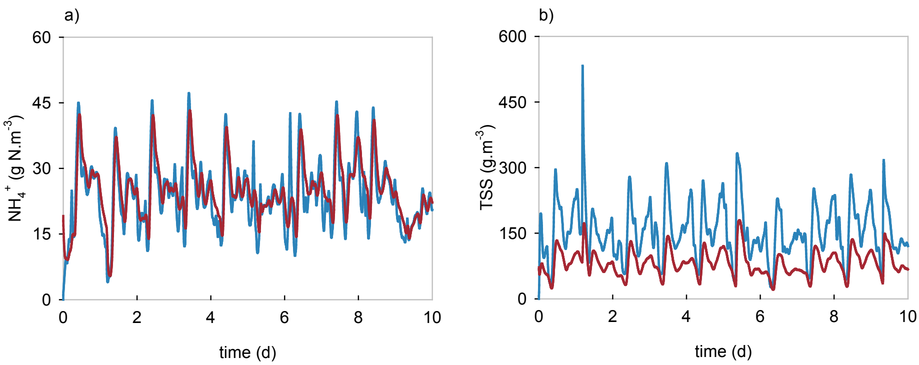

Figure 4.8: Input (blue) and output (red) concentrations for NH_4^+ (a) and TSS (b) from the primary clarifier model.

Figures 4.8a, b describe the solubles (NH_4^+) and particulate (total suspended solids (TSS)) concentrations at the inlet and outlet of the primary clarifier, respectively. It can be noticed that the NH_4^+ concentration is only attenuated due to the residence time in the clarifier while TSS undergoes settling.

Biological Reactors sub-model employs ASM2d (Henze et al., 1999) to describe the biological processes in the activated sludge reactors. ASM2d includes a description for chemical and biological removal of phosphorus. Inclusion of phosphorus accumulating organisms and cell internal storage compounds (organic and inorganic) are the major modifications in comparison to ASM1. Additionally, in ASM2d, the denitrifying ability of phosphorus accumulating organisms is also considered. ASM2d is generally the model of choice to describe nutrient removal processes when phosphorus removal is important (amongst the standard ASM models – ASM1, ASM2, ASM2d and ASM3).

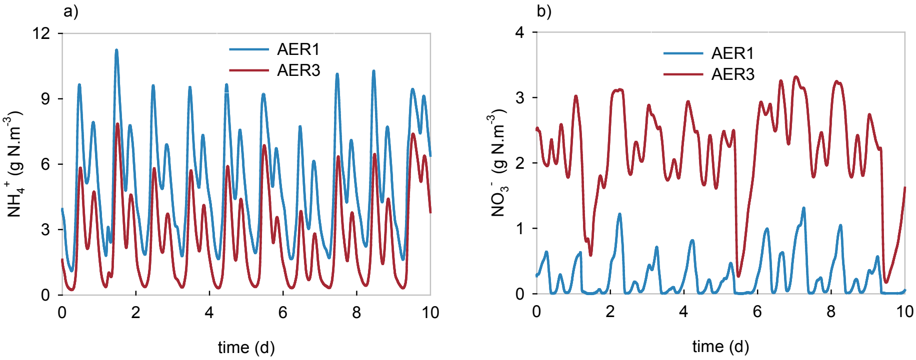

Figure 4.9: ASM2d model used in aerobic reactors depicting nitrification with removal of ammonium (a) and production of nitrate (b) in the aeration tanks (AER1 and AER3).

The process of nitrification in the aeration tanks (AER1 and AER3) is depicted in Figure 4.9 for the WWTP configuration used in BSM1_ASM2d (Flores-Alsina et al., 2012a). NH_4^+ is removed while NO_3^- is produced at the end of the aerated system. The produced NO_3^- is generally recirculated to the anoxic zone (ANOX1, ANOX2) for removal under denitrifying anoxic conditions.

Secondary Clarifier (**Sec. C**) is modelled in the default BSM1 and BSM2 versions using the Takács settler model (Takács et al., 1991). However, BSM-UWS uses the Bürger-Diehl settler model (Bürger et al., 2011; 2012; 2013), which is implemented in the BSM framework by Arnell (2015). The model is a 10-layer non-reactive settler model, which considers convection, compression and dispersion phenomena to describe the sludge concentration in the settling tank, effluent and underflow. The sludge concentration in the different layers of the settling tank is depicted in Figure 4.10. The dynamic variation of the sludge blanket level is visible.

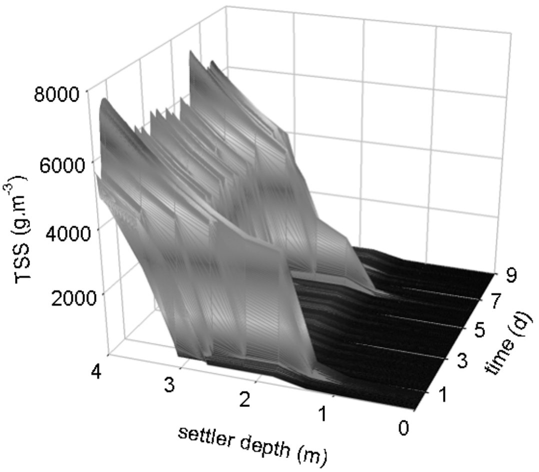

Figure 4.10: TSS concentration along the depth of the settler simulated using the Bürger-Diehl settler model.

In the current version of the BSM-UWS, the sludge line is not considered. Hence, the model blocks for anaerobic digester and other sludge handling units are not described.

4.2.4. River Water System

The complete river stretch is described by connecting a series of river system models, each describing the hydraulics and biochemical transformations taking place in that particular stretch. For the purpose of integrated modelling, the hydraulic processes are simplified and the biochemical transformations are described with a complexity that is similar to that of the ASM model used in the WWTPs.

River Hydraulics

Each river stretch is assumed to be of trapezoidal shape. Flow rate at the end of the stretch is described using the Manning's formula:

$$Q = \frac{k_r}{n} A R_h^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{(Eq. 4.1)}$$

where A is the cross sectional area of the river stretch (m^2), R_h is the hydraulic radius (m), S is the horizontal slope of the river stretch, n is the Manning's roughness coefficient ($s.m^{-1/3}$) and k_r is used for conversion of units (flow rate Q is expressed in $m^3.d^{-1}$).

River Biochemical Processes

A simplified version of RWQM1 (Reichert et al., 2001a) described in Section 2.2.4 is used to model the biochemical processes in the river. The major simplifications made to the RWQM1 model are highlighted below.

- pH calculations are omitted. Hence the state variables used exclusively for pH calculations are also removed.
- Higher organisms and consumers are not included in the model.
- Mass fractions for some of the state variables are modified to be similar to ASM2d mass fractions. This simplification is done in order to make the interfacing between ASM2d and RWQM1 easier and more logical.

All other kinetic and stoichiometric parameters are the same as the default values in the RWQM1 model. Solar irradiance is assumed to be dynamic with changes during day/night.

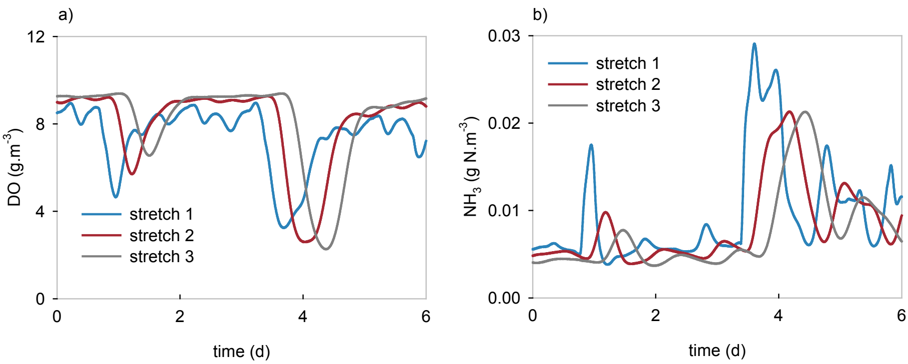

Figure 4.11: Effect of pollutant load on river water quality in terms of DO (a) and NH_3 (b) at three consecutive river stretches (stretches 1, 2 and 3) (modified from Saagi et al. (2015)).

Figure 4.11 describes the dynamics of dissolved oxygen (DO) and un-ionized ammonia (NH_3) in the river due to an intermittent organic load. It shows that the peak in NH_3 concentration is at the location where WWTP effluent discharges into the river and it gets attenuated in the downstream stretches for both the rain events, whereas for DO, the location of the lowest oxygen concentration varies depending on the amount of organic load discharged into the river and also river hydraulics and biochemical characteristics.

4.2.5. Interfaces

As the state variables for the sewer network, WWTP and river models are different, model interfaces are required to transform the state variables from one model to another. Some of the state variables can be directly mapped while others are transformed. Three different interfaces are developed for the BSM-UWS model.

- SEWER-WWTP – Translates the pollutant state variables in the sewer (described as daily loads) to ASM2d state variables (described as concentrations). While some of the pollutants (NH_4^+, PO_4^{3-}) are directly mapped, others (COD_{sol}, COD_{part}) are fractionated into multiple ASM2d state variables.
- SEWER-RIVER – Converts the sewer state variables (load based) into RWQM1 state variables (concentration based). As in the case of the sewer-WWTP interface, some of the pollutant state variables in the sewer are either directly mapped or fractionated into multiple RWQM1 state variables. Additionally, constant values are assumed for some state variables (e.g. inorganic carbon, algal biomass etc.) at the interface since these variables do not exist in the sewer model.
- WWTP-RIVER – Transforms the ASM2d state variables in the WWTP to RWQM1 state variables in the river. While some of the state variables can be directly mapped, state variables that are present only in the ASM2d model are first transformed to other variables that are present in both the models. It is assumed that the state variables undergo biological processes described in the ASM2d model instantaneously. Once transformed, they can be mapped directly to RWQM1 state variables.

For all interfaces, mass balances for COD, carbon, nitrogen and phosphorus are maintained using the principle described in Vanrolleghem et al. (2005b), Volcke et al. (2006), Nopens et al. (2009) and Flores-Alsina et al. (2016).

4.3. Evaluation Criteria

Sewer Network

Sewer network performance during rain events is generally assessed by the flow rate and pollutant loads that are discharged into the river system (the lower, the better). The major evaluation criteria are mentioned below.

1. Overflow duration (T_{ovf}, d.yr^{-1}): The total overflow duration for a given year /evaluation period.
2. Overflow frequency (N_{ovf}, events.yr^{-1}): Represents the number of overflow events annually. Two overflow events are separated if there is at least one hour difference in time between these events.
3. Overflow volume (V_{ovf}, m^3.yr^{-1}): The total volume of overflow from all overflow locations that reaches the receiving water system in a year.
4. Overflow quality index (*OQI*, kg pollutant units.d^{-1}): An aggregated pollution index similar to the indices used in BSM WWTP models. It considers the pollutant load from different pollutants (COD, BOD, TSS, TKN, NO_3^- and PO_4^{3-}) and assigns weights to each one of them. The *OQI* is the sum of the total load for each pollutant multiplied by its individual weight. The weights for individual pollutants are similar to those used in the BSM2 and BSM1 models.
5. Hourly maximum concentration (C_{max}, g.m^{-3}): The concentration that is continuously exceeded for a period of at least 1 hour. C_{max} is calculated for TSS, TKN and PO_4^{3-}.
6. Exceedance duration (T_{exc}, d.yr^{-1}): The total duration for which the pollutant concentration exceeds a pre-defined threshold limit. It represents the duration of acute pollutant discharge to the receiving water system. Pollutants considered are TSS, TKN and PO_4^{3-}.

All the above criteria are described for the entire sewer network but can also be computed for each overflow location individually.

Wastewater Treatment Plant

The evaluation criteria that have been developed for the BSM1 and BSM2 models can be calculated in the BSM-UWS as well. The major criteria are described here.

1. Influent Quality Index (*IQI*) (kg pollutant units.d^{-1}): An aggregated index that computes the cumulative pollutant load in the influent wastewater for six major pollutants (COD, BOD, TSS, TKN, NO_3^-, PO_4^{3-}). Each pollutant has a weight factor assigned to it.
2. Effluent Quality Index (*EQI*) (kg pollutant units.d^{-1}): An aggregated index computed for the wastewater effluent in a similar manner as the *IQI*. *EQI* includes both the bypass and the overflow from the secondary settler.

3. Operational Cost Index (*OCI*): It considers the operational costs from aeration, pumping, mixing, sludge handling and external carbon addition. Similar to the quality criteria, weights are assigned to each of the contributing operations and a net cost index is computed.

River Water System

Four evaluation criteria are described to assess the chemical quality of the river, mainly in terms of un-ionized ammonia (NH_3) and dissolved oxygen (DO). The criteria are calculated as a cumulative index for the entire river.

1. Exceedance duration (T_{exc}, d.yr^{-1}): $T_{exc,DO}$ and $T_{exc,NH3}$ represent the total duration in a year for which the respective concentrations exceed a threshold value. The threshold values used are: NH_3 – 0.018 g N.m^{-3} and DO – 6 g.m^{-3}. The values are based on the limits prescribed for salmonid species in the Urban Pollution Management (UPM) manual (FWR, 2012).
2. Hourly minimum oxygen concentration ($C_{min,DO}$, g.m^{-3}): Minimum dissolved oxygen concentration that is continuously reached for a duration of at least one hour.
3. Hourly maximum ammonia concentration ($C_{max,NH3}$, g N.m^{-3}): Un-ionized ammonia concentration that is continuously exceeded for a period of at least one hour.

4.4. Model Limitations

Hydrological phenomena in the catchment (like evapo-transpiration, depression losses, infiltration, groundwater levels etc.) are described in a simple manner. The sewer model does not include backwater effects and biological transformations in the sewer pipes. Owing to its conceptual nature, it is not suitable to assess the sewer pipe water levels, pressure etc. For the WWTP, only the water line is currently included but will be extended to the sludge line as well in the near future. The river system chosen is assumed to be a shallow river without significant sediment oxygen dynamics and it is assumed that the pH in the river stays constant across the stretch. However, recently developed physico-chemical models (Flores-Alsina et al., 2015; Solon, 2017) can be coupled with the river system model to predict pH dynamics in the future. Also, the limitation in the underlying models (reservoir models for sewer network, ASM2d for WWTP, RWQM1 for river) should be considered before deciding on the suitability of the model library for any case studies.

4.5. Summary of Key Findings

The chapter presents the underlying model library for BSM-UWS together with evaluation criteria for different sections of the UWS. The key findings are summarized below.

- 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 and SW) describing the different sources of wastewater generation from a catchment.
- The sewer model includes TRANSPORT and FIRST-FLUSH sub-models that can simulate: i) delay and attenuation noticed due to sewer network; and ii) increased pollutant loads at the beginning of rain events, respectively. Additionally, the STORAGE sub-model allows for modelling the storage tank dynamics and overflow discharges into the river system. It also includes all essential control elements (CONTROL) needed to implement sewer/integrated control strategies.
- The WWTP model describes the physical and biological transformations that lead to removal of pollutants from raw wastewater. The model is adapted from the existing BSM WWTP models.
- The river system model based on RWQM1 is implemented to predict the effect of sewer overflows and WWTP effluent on the chemical quality of the river.
- Evaluation criteria are defined to assess the performance of sewer overflows, WWTP effluent and more importantly the river water system quality. With the new evaluation criteria, BSM-UWS performance can be assessed both using traditional indirect metrics (sewer overflow and WWTP effluent quality based) as well as using direct metrics based on the river water quality.

The model library forms the basis for developing the system-wide BSM (BSM-UWS). The next step in the process is to define a general system layout and appropriate characteristics of the BSM-UWS.

Chapter 5

BSM-UWS: System Layout and Case Studies

The chapter details the final layout for BSM-UWS and the characteristics for various sections. The case studies for catchment and sewer extensions (Paper III) and the entire BSM-UWS (Paper IV) are summarized. Finally, global sensitivity analysis (GSA) results from Paper V are described. A brief discussion on the choice of the system layout, potential applications and model limitations is presented.

5.1. Introduction

The IWA Benchmark Simulation Models are developed primarily with the aim to objectively evaluate control strategies in WWTPs. These models (BSM1, BSM1_LT, BSM2) consist of a predefined WWTP layout, process models, sensor and actuator models, influent characteristics and evaluation criteria (Gernaey et al., 2014). They are extensively used for benchmarking purposes (e.g. Stare et al., 2007; Flores-Alsina et al., 2008; Sweetapple et al., 2014) and as model libraries for other full-scale studies (e.g. Lindblom et al., 2016; Kazadi Mbamba et al., 2016). Additionally, model extensions are also carried out using the BSM layouts (e.g. Snip et al., 2014; Solon et al., 2015). Currently, more than 500 publications using BSMs indicate the phenomenal success of the BSM family of models (Jeppsson et al., 2013). However, all these models are limited to the WWTP. For the first time, spatial extensions outside the fence of WWTP are attempted. An UWS-wide integrated model that can be used to directly evaluate the effect of various control strategies on river water quality is developed.

The Chapter describes the BSM-UWS layout that is developed using the model library presented in Chapter 4. The first set of case studies includes control strategies and structural modifications to the catchment and sewer BSM. Further, integrated and local control strategies using BSM-UWS are presented to demonstrate the applicability of the platform. A global sensitivity analysis (GSA) study to determine the most influential control handles and design parameters available in BSM-UWS is carried out.

5.2. Layout and Characteristics

The system layout consists of an urban catchment (with different sub-catchments) that generates sewage during dry weather and additionally stormwater during rain events (Figure 5.1). The sewer network connects all sub-catchments to the WWTP and transports all the collected wastewater to the treatment facility. During rain events, any excess flow beyond the capacity of the sewer network overflows into the river system.

Table 5.1: System characteristics (catchment and sewer network) for the BSM-UWS (from Paper IV).

Sub-catchment	Area (ha)	PE	DWF ($m^3.d^{-1}$)		Storage (m^3)
			DOM	IND	
1	99	15 920	2 390		5 000
2	21	3 920	590	2 500	1 000
3	29	2 960	440		
4	71	9 600	1 440		4 400
5	71	7 840	1 180		3 600
6	249	39 760	5 960		8 100
Total	**540**	**80 000**	**12 000**	**2 500**	**22 100**

DWF: Dry weather flow; DOM: Domestic; IND: Industrial

5.2.1. Catchment

The hypothetical urban catchment structure is adopted from the ATV A 128 case study (ATV, 1992). It consists of six sub-catchments ($SC_1 \ldots SC_6$) connected to the WWTP through a sewer network (Figure 5.1). The catchment has a total area of 540 hectares with 80 000 population equivalents. During dry weather, daily average wastewater generation is 19 000 $m^3.d^{-1}$. Contribution from domestic sources is 12 000 $m^3.d^{-1}$ and industrial sources is 2 500 $m^3.d^{-1}$. Daily average infiltration to sewers is assumed to be 4 500 $m^3.d^{-1}$. SC_2 has both an industrial and domestic section, while the remaining sub-catchments generate domestic wastewater only (Table 5.1).

5.2.2. Sewer Network

The sewer network consists predominantly of combined sewer networks. Five of the six sub-catchments (SC_1, SC_2, SC_3, SC_4 and SC_6) are connected to combined sewer networks whereas only SC_5 is connected to a separate sewer network (see Figure 5.1). The sewer network at each sub-catchment includes a storage tank (except at SC_3). Two different storage tank configurations are used. Online pass-through tanks are used at four locations (ST_1, ST_2, ST_5 and ST_6) whereas ST_4 is an offline bypass tank. The outflows from online tanks are regulated by throttle valves/pumps, whereas those from offline tank are regulated by pumps with fixed pumping capacity. The total available storage volume is 22 100 m^3 (approx. 40 $m^3.ha^{-1}$ of catchment area). Individual storage volume for each tank (connected to a sub-catchment) is detailed in Table 5.1. The sewer overflows are discharged at five locations to the river system.

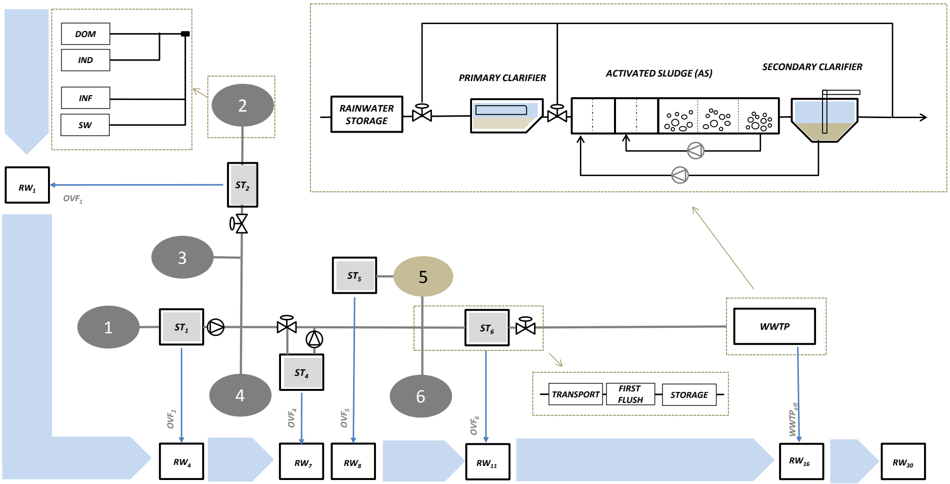

Figure 5.1: BSM-UWS layout – sub-catchments (SC_1, SC_2, SC_3, SC_4, SC_5 and SC_6), sewer network with storage tanks (ST_1,ST_2, ST_4, ST_5 and ST_6), WWTP and river water system (from Paper IV).

5.2.3. Wastewater Treatment Plant

An extended BSM1-ASM2d plant layout is used for the WWTP (Flores-Alsina et al., 2012a). The biological section includes two anaerobic tanks (ANAER1, ANAER2) (2 x 1 000 m^3), two anoxic tanks (ANOX1, ANOX2) (2 x 1 500 m^3) and three aerobic tanks (AER1, AER2, AER3) (3 x 3 000 m^3). A primary clarifier (PC) (900 m^3) and a secondary clarifier (Sec.C) (area – 2 500 m^2) are used for separation processes before and after the biological reactors, respectively. In addition, a rainwater storage tank (RST) (8 000 m^3) at the beginning of the WWTP and two bypass facilities (BP_1, BP_2) (before and after the primary clarifier) are included. BP_1 has a threshold of 90 000 $m^3.d^{-1}$ (any flow in excess of the threshold is bypassed and reaches the river system) while BP_2 has a threshold of 70 000 $m^3.d^{-1}$.

5.2.4. River Water System

A 30 km long urban river stretch is represented by a series of river model blocks (where each block contains the hydraulic and biochemical process model for a 1 km stretch of the river). It is assumed that the river has a uniform bottom width of 7 m and is trapezoidal in shape. From Figure 5.1, it can be seen that the river segment is modelled even after the WWTP discharge location. This is essential as the worst river quality does not necessarily occur at the point of effluent discharge. The river has a mean annual base flow rate of 72 500 $m^3.d^{-1}$. Additional runoff from an upstream catchment (area – 500 ha) reaches the river during rain events. The upstream pollutant concentrations are assumed to be constant and identical for both wet and dry weather conditions. WWTP effluent ($WWTP_{eff}$-RW_{16}) as well as sewer overflows from five overflow locations (OVF_1-RW_1, OVF_2-RW_4, OVF_4-RW_7, OVF_5-RW_8 and OVF_6-RW_{11}) reach the river system.

5.3. Catchment and Sewer BSM – Control Strategies and Structural Modifications

During the process of developing the BSM-UWS, the model blocks for the catchment and sewer network were developed initially. As this was already a significant extension to the BSM model family, case studies describing the applicability of the extensions (catchment and sewer BSM) were performed. The initial layout and system characteristics were gradually modified to reach the final version as described in the previous sections of this chapter. Major differences between the characteristics of: i) catchment and sewer BSM (Table 5.2); and ii) BSM-UWS (Table 5.1) are mentioned below.

- Reduction in the length of the sewer network – The initial sewer network length (modelled in terms of the number of reservoir models in series) is observed to have caused significant peak shaving of the influent flow rate. Hence, the length of the sewer network is reduced to keep the influent flow rate peaks at a realistic level.

- Addition of a detailed pumping model – The pumping model from Kroll et al. (2016) is included in the online and offline storage tanks providing the ability to model multiple pumps at a storage location as well as simulate frequency control of pumps.
- Changes to the storage tank locations – ST_1 is converted to a storage location with overflow. Initially, it was a storage location without any overflow. Also, ST_3 is removed. This is done as the location is very close to ST_4 and hence can be easily simplified as one storage location. Changes to the storage tank volumes, throttle flows and overflow rates are also made.

For the convenience of the reader, the system layout and characteristics of catchment and sewer BSM are presented in Figure 5.2 and Table 5.2 (details can be found in Paper III).

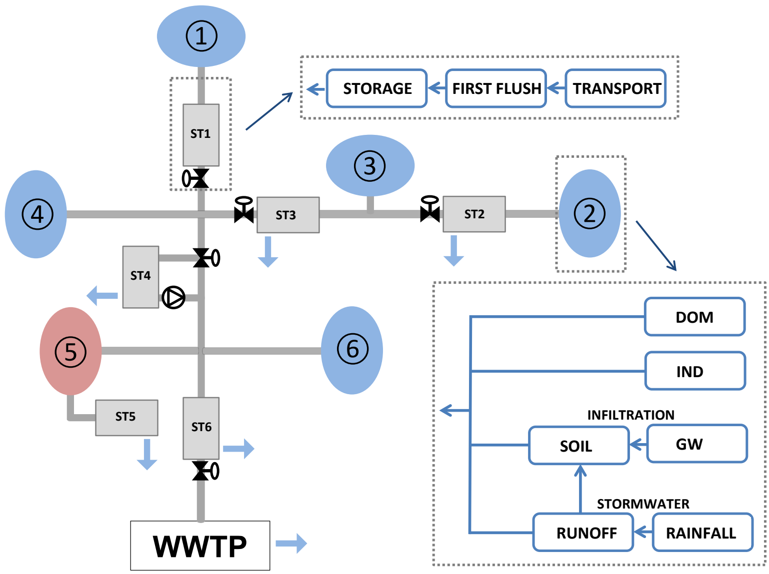

Figure 5.2: Catchment and sewer BSM layout indicating various sub-catchments, storage tanks and control elements (from Paper III).

In order to demonstrate the capability of the catchment and sewer BSM to simulate various control strategies and structural modifications, four case studies are implemented and analysed (see Table 5.3). No control (NC) represents the default configuration.

Table 5.2: Major characteristics for the catchment and sewer BSM. In comparison to BSM-UWS, the major change is the volume of storage tank (from Paper III).

Sub-catchment	Area (ha)	PE	DWF ($m^3.d^{-1}$)		Storage (m^3)
			DOM	IND	
1	99	15 920	2 390		5 500
2	21	3 920	590	2 500	1 000
3	29	2 960	440		2 000
4	71	9 600	1 440		4 000
5	71	7 840	1 180		4 000
6	249	39 760	5 960		15 000
Total	**540**	**80 000**	**12 000**	**2 500**	**31 500**

DWF: Dry weather flow; DOM: Domestic; IND: Industrial

These case studies include:

i. reducing the bypass at the WWTP (C1);

ii. reducing the total overflow from the system (C2);

iii. modification of SC_5 from a separate sewer network to a combined sewer network (S1);

iv. inclusion of an additional storage tank at the WWTP influent (S2).

Table 5.3: Evaluation of the catchment and sewer BSM for different scenarios. Evaluation criteria are described for cumulative as well as acute effects (from Paper III).

Criteria	NC	C1	C2	S1	S2
Cumulative effects					
N_{ovf} (events.yr^{-1})	137	142	141	82	137
T_{ovf} (d.yr^{-1})	71	71	71	21	71
V_{ovf} ($m^3.yr^{-1}$)	830 200	654 700	642 800	722 700	678 000
OQI (kg poll. units.d^{-1})	3 110	2 118	2 068	2 937	2 076
Acute effects					
$T_{exc,TKN}$ (d.yr^{-1})	49.0	50.7	50.6	20.3	47.6
$C_{max,TKN}$ (g N.m^{-3})	51.1	51.1	51.1	48.8	51.1

5.3.1. 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 $m^3.d^{-1}$) to the effluent section where it is mixed with the treated wastewater (Gernaey et al., 2014). A rule-based control strategy is developed to better utilize the available storage volume in ST_6 (storage tank located upstream of WWTP – Figure 5.1). The objective of the control strategy is to reduce the outflow from the storage tank as long as there is storage capacity available in the tank. The two conditions assessed by the control strategy are: i) inflow to ST_6 exceeds 60 000 $m^3.d^{-1}$; and ii) there is storage capacity available (ST_6 level < 4 m). When both these conditions are fulfilled, the outflow from the tank is restricted using a valve. The valve opening is reduced to 65 %. Otherwise, the valve is fully open.

The evaluation criteria at ST_6 and bypass (BP) are presented in Table 5.4. With a better utilization of the ST_6 capacity, there is an improvement noticed at the WWTP bypass. C1 leads to a decrease in yearly overflow volume ($V_{\mathrm{ovf,BP}}$) (39 %) and overflow quality index (OQI_{BP}) (50 %) at the bypass. However, the improvements at the bypass led to a drop in performance at ST_6. Yearly overflow volume ($V_{\mathrm{ovf,ST6}}$) increases by 54 % and the overflow quality index (OQI_{ST6}) increases significantly by 110 %. Additionally, the effect of the control strategy is also analysed using criteria that describe acute effects. Yearly exceedance duration for TKN ($T_{\mathrm{exc,TKN}}$) at both locations increases due to the control strategy. Hourly maximum concentration for TKN ($C_{\mathrm{max,TKN}}$) remains almost similar at the bypass while increasing at ST_6. From a global point of view, Table 5.3 reveals that C1 leads to a decrease in the yearly overflow volume (V_{ovf}) discharged into the receiving water by 21 %. Also, the overflow quality index (OQI) is reduced by 32 %. The control strategy did not have any major impact on the acute effects ($T_{\mathrm{exc,TKN}}$, $C_{\mathrm{max,TKN}}$). Summarizing, C1 successfully decreases the cumulative pollutant load to the receiving water but is not effective in handling critical situations.

Table 5.4: Effect of C1 on the performance of ST_6 and BP at WWTP (from Paper III).

Criteria	ST_6		Bypass (BP)	
	NC	C1	NC	C1
Cumulative effects				
N_{ovf} (events.yr^{-1})	5	8	79	75
T_{ovf} (d.yr^{-1})	0.6	0.9	18	21
V_{ovf} (m^3.yr^{-1})	21 400	32 900	473 300	286 400
OQI (kg poll. units.d^{-1})	32	67	2 072	1 045
Acute effects				
$T_{\mathrm{exc,TKN}}$ (d.yr^{-1})	0.3	0.7	17.2	18.8
$C_{\mathrm{max,TKN}}$ (g N.m^{-3})	8.2	12.2	47.8	47.5

5.3.2. Reducing the Total Overflow from the System (C2)

Having noticed an improvement in the sewer network performance by using a rule-based control strategy at only one location (C1), such local control strategies are now implemented at all storage locations with overflow structures (Figure 5.2). For ST_2, ST_3 and ST_6, if the water level is less than 4 m (max level = 5 m), the valve opening is reduced to 65 %. Otherwise, it is fully opened. The control strategy for ST_4 is different as it is an offline tank. The maximum throttle flow to the main sewer (any excess flow rate is directed to ST_4) is controlled based on water level measurement in ST_4. In the default case (NC), the maximum throttle flow is 45 000 $m^3.d^{-1}$. If the flow rate exceeds this value, the flow is directed to the ST_4. With control strategy C2 activated, if the level in ST_4 is less than 4 m, the maximum throttle flow is 40 000 $m^3.d^{-1}$, and when the tank is filled above 4 m, the throttle flow limit is increased to 55 000 $m^3.d^{-1}$. Hence, the algorithm tries to utilize the storage tank capacity as much as possible by limiting the maximum throttle flow. With local control strategies implemented at ST_2, ST_3, ST_4 and ST_6, C2 is an example of various non-interacting local control strategies developed with an overall aim to reduce the cumulative overflow volume/load.

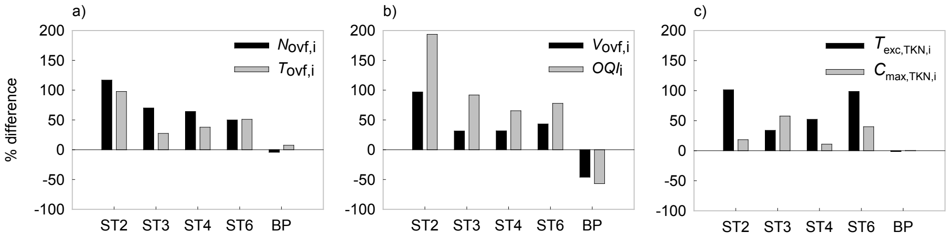

Figure 5.3: Performance comparison between NC and C2 evaluated for the criteria $N_{ovf,i}$ and $T_{ovf,i}$ (a), $V_{ovf,i}$ and OQI_i (b), and $T_{exc,TKN,i}$ and $C_{max,TKN,i}$ (c), at various storage tanks (ST_2, ST_3, ST_4, ST_6) and the bypass (BP) (from Paper III).

At the local level, a similar trend to that noticed in C1 is also observed in C2. The performance at the storage tanks where control is implemented has deteriorated while that at the downstream location (WWTP bypass in this case) has improved (Figure 5.3). At the bypass, criteria that show major improvements are yearly overflow volume ($V_{ovf,BP}$) (46 % lower) and OQI_{BP} (57 % lower). The acute effects at the bypass did not change much due to the control. Looking at the entire system (see Table 5.3), with an improved utilization of the available storage, a reduction in the yearly overflow volume (V_{ovf}) (23 %) and overflow quality index (OQI) (34 %) is observed. The major observations that can be made are:

i. the control strategy (C2) has a net positive impact on the sewer network performance although it leads to decreased performance at individual storage locations;

ii. the results obtained from C1 and C2 are very similar. 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.

5.3.3. Modification of Sewer Network for SC_5 from a Separate Sewer Network to a Combined Sewer Network (S1)

As SC_5 is connected to a separate sewer network, all rain events lead to stormwater overflows at ST_5 that eventually reaches the river. As an illustration of structural modification that can be simulated using the model, the sewer network for SC_5 is converted to a combined sewer network and the potential impact on overflow volume/load from SC_5 as well as the entire system is evaluated. It is assumed that the volume of the storage tank remains unchanged.

Table 5.5: Performance of ST_5 for scenario S1 compared to the default case NC (from Paper III).

Criteria	NC	S1
Cumulative effects		
N_{ovf} (events.yr^{-1})	134	2
T_{ovf} (d.yr^{-1})	71	0
V_{ovf} (m^3.yr^{-1})	268 800	2 100
OQI (kg poll. units.d^{-1})	864	2
Acute effects		
$T_{exc,TKN}$ (d.yr^{-1})	40.7	0.0
$C_{max,TKN}$ (g N.m^{-3})	51.1	2.9

The structural modification (S1), expectedly, has brought significant changes to the performance at ST_5. Only two overflow events ($N_{ovf,ST5}$) are observed after the system modification (Table 5.5). The overflow quality index (OQI_{ST5}) has also reduced significantly from 864 kg poll units.d^{-1} to only 2 kg poll units.d^{-1}. Also, the acute effects improved significantly. The yearly exceedance duration ($T_{exc,TKN}$) and hourly maximum concentration ($C_{max,TKN}$) reduced by a large extent (100 % and 94 %, respectively). From a global perspective (Table 5.3), a significant drop in the yearly overflow frequency (N_{ovf}) (41 %) and yearly overflow duration (T_{ovf}) (71 %) is observed due to the structural modifications. This is expected as the separate sewer network is now modified into a combined sewer. Although, there is a drop in yearly overflow volume (V_{ovf}) (13 %), the overflow quality index (OQI) has only improved by 6 %. This shows that a major portion of the polluted stormwater from SC_5 is leading to overflows at a different location. The change has also caused major improvements to the acute criteria. $T_{exc,TKN}$ and $C_{max,TKN}$ improved by 59 % and 5 %, respectively.

The system modification causes an impact at various levels with:

i. great improvements at the local level (ST_5) leading to almost negligible overflows;

ii. major change (positive) at the sewer network level when compared to the default case (NC). However, similar or better results are observed for some evaluation criteria with the implementation of control strategies.

5.3.4. Inclusion of an Additional Storage Tank at the WWTP Influent (S2)

As a modification to the sewer network, this scenario studies the impact of including an additional storage tank at the WWTP influent. An online pass-through tank with outflow regulated by a pump is added at the WWTP inlet. The volume of the storage tank is 8 000 m^3. With this additional storage capacity, it is expected that the bypass at the WWTP can be reduced.

Table 5.6: Evaluation criteria for S2 at the WWTP bypass (from Paper III).

Criteria	NC	S2
Cumulative effects		
N_{ovf} (events.yr^{-1})	79	35
T_{ovf} (d.yr^{-1})	18	10
V_{ovf} (m^3.yr^{-1})	473 300	321 200
OQI (kg poll. units.d^{-1})	2 072	1 037
Acute effects		
$T_{exc,TKN}$ (d.yr^{-1})	17.2	8.2
$C_{max,TKN}$ (g N.m^{-3})	47.8	31.7

At the bypass location, the effect of additional storage is clearly visible on the criteria for cumulative effects (Table 5.6). A drop in yearly overflow frequency ($N_{ovf,BP}$) (56 %) and yearly overflow duration ($T_{ovf,BP}$) (46 %) is observed. 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 ($T_{exc,TKN}$) and hourly maximum concentration ($C_{max,TKN}$) for TKN 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 (Table 5.3), no major changes are observed in yearly overflow frequency (N_{ovf}) and yearly overflow duration (T_{ovf}). A decrease is noticed in the yearly overflow volume (V_{ovf}) (18 %) and overflow quality index (*OQI*) (33 %). The modification has led to marginal decrease in $T_{exc,TKN}$ (3 %) while it has no effect on $C_{max,TKN}$. In spite of the high capital costs involved in construction of a storage tank at the WWTP influent, the overall performance improvement is similar to that from the control strategies.

To summarize, it is clear that overflow frequency and duration do not always reflect the pollution load discharged into the river. Based on the results, control strategy C2 leads to the biggest reduction in the cumulative effects determined by the volume and pollutant load discharged into the river system. The structural modification S1 reduces the acute effects significantly as this is the location where high concentration of pollutants (without treatment) are being discharged into the river.

5.4. BSM-UWS – Control Strategies

Three control strategies are devised and evaluated using the BSM-UWS. The case studies are developed to demonstrate the ability of the tool when modelling and evaluating local as well as integrated control alternatives. The focus has primarily been on developing simple yet realistic control strategies and not on identifying the best/optimum solution for the system. Open loop (OL) represents the default set up without any active control strategy. The three case studies evaluated are:

i. control of dissolved oxygen concentration in the WWTP aeration tanks (C3);

ii. modifying the biological capacity at the WWTP (by changing the bypass limits) based on river water quality (C4);

iii. optimize storage tank utilization based on influent flow rate to the WWTP (C5).

Table 5.7: Performance of various sections under OL, C3, C4 and C5 (from Paper IV).

	OL	C3	C4	C5
Sewer				
V_{ovf} ($m^3.yr^{-1}$)	203 400	203 400	203 400	207 700
OQI (kg poll. units.d^{-1})	940	940	940	957
WWTP				
IQI (kg poll. units.d^{-1})	92 714	92 714	92 714	94 377
EQI (kg poll. units.d^{-1})	6 778	6 466	6 409	6 505
River				
$T_{exc,NH3}$ (d.yr^{-1})	16.2	5.5	6.2	7.8
$T_{exc,DO}$ (d.yr^{-1})	12.8	13.7	12.1	11.4

5.4.1. Control of Dissolved Oxygen Concentration in the WWTP Aeration Tanks (C3)

The dissolved oxygen concentrations in the three aeration tanks (AER1, AER2 and AER3) are controlled using a feedback controller. The oxygen level in AER2 is compared to an oxygen set point of 2 g.m^{-3} and the error is used to regulate the oxygen supply in AER2 using a Proportional-Integral (PI) controller. For tanks AER1 and AER3, a less precise approach is chosen. The oxygen supply rate for AER2 is adjusted using correction factors in order to regulate the oxygen supply to these tanks. Although, this does not lead to precise control of oxygen concentrations in AER1 and AER3, it is considered as a practical simplification in order to avoid using a large number of control loops.

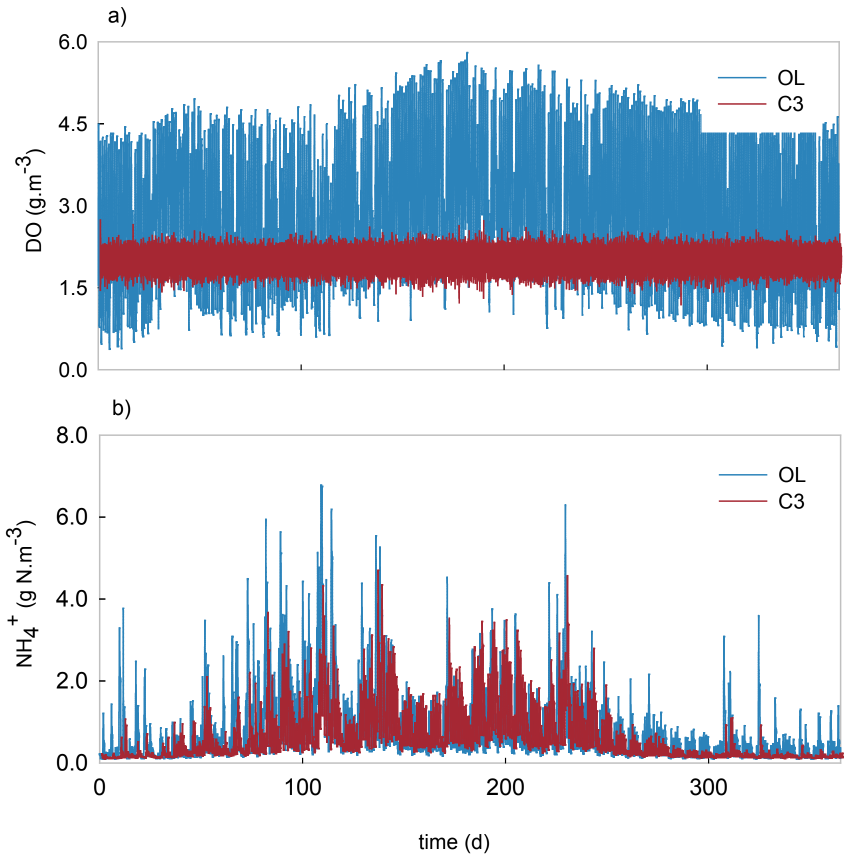

Figure 5.4: Variation in DO concentration in AER2 (a) and NH_4^+ concentration in the WWTP effluent due to the effect of DO controller (C3) (b). Day 0 is 1st July (from Paper IV).

From the WWTP perspective, C3 is successful in: i) maintaining the desired DO concentration set point in AER2 and also loosely regulating the oxygen supply in AER1 and AER3; and ii) improving the effluent quality (*EQI* decreases by 5 % in comparison to OL) due to improved nitrification. Figure 5.4a shows the oxygen concentration in AER2, which is well controlled around 2 g.m^{-3} in comparison to the OL oxygen concentration with periods of both higher and lower oxygen supply. It also illustrates the effect of improved oxygen supply on the ammonium concentration in the effluent. With sufficient oxygen supply for the entire duration, nitrification capacity in the WWTP has increased leading to better effluent quality (Figure 5.4b).

However, it does not lead to improvements in all the river criteria. While $T_{exc,NH3}$ reduces significantly (66 % lower than OL), $T_{exc,DO}$ increases by 7 % (Table 5.7). The drop in $T_{exc,NH3}$ is mainly due to the lower NH_4^+ concentration in the WWTP effluent. The reason for the increase in $T_{exc,DO}$ is not straightforward. It is observed that the DO control leads to marginally higher mixed liquor suspended solids concentration in the activated sludge reactors (better biomass growth with improved oxygen supply). During rain events, this causes higher TSS washoff concentration in the settler overflow leading to lower DO concentrations in the river.

5.4.2. Modifying the Biological Capacity of the WWTP Based on River Water Quality (C4)

The integrated control strategy (C4) regulates the bypass limits at the WWTP (thereby controlling the maximum treatment capacity of the WWTP) based on the river water quality (in terms of NH_4^+ concentration) at the point of WWTP effluent discharge. If the NH_4^+ concentration in the river exceeds 0.4 g N.m^{-3}, indicating that there is a high load of untreated wastewater reaching the river system, the maximum capacity of the WWTP is increased by 20 % (by rising the bypass limits). However, in order to ensure that this will not lead to loss of biomass (and reduced nitrification capacity), the control strategy is switched off when the effluent suspended solids concentration is higher than 60 g.m^{-3}. Also, the oxygen control at the WWTP (C3) is active.

The objective of utilizing the WWTP biological treatment capacity to the maximum extent possible is achieved by the control strategy. Figure 5.5a shows the reduction in bypass volumes. The strategy leads to a 45 % drop in bypass volume when compared to OL. The reduced overflow volume leads to a lower *EQI* (which means better effluent quality) in spite of sending more wastewater to treatment (3 % and 1 % lower than OL and C3, respectively).

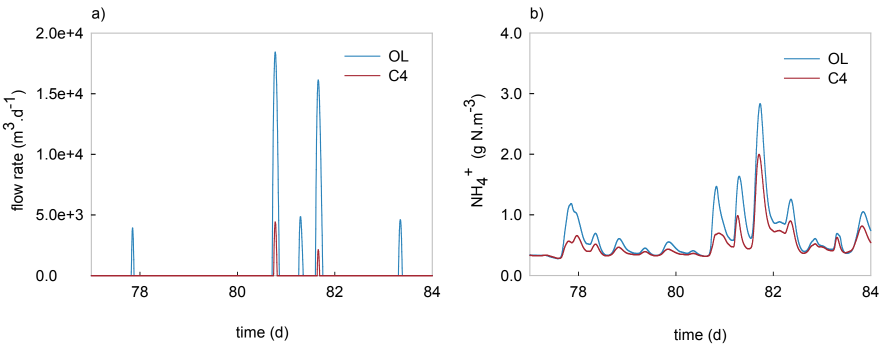

Figure 5.5: Effect of C4 on bypass flow rate (a) and NH_4^+ concentration in the river stretch where WWTP effluent is discharged (b). Day 77 is 16th September (from Paper IV).

The improvement in river quality in comparison to OL is clearly evident in both criteria. Figure 5.5b shows lower river ammonium concentration due to C4. $T_{exc,NH3}$ decreases by 62 % and $T_{exc,DO}$ decreases by 6 %. However, when compared to river water quality in C3, the results are mixed. While $T_{exc,NH3}$ increased by 10 %, $T_{exc,DO}$ decreased by 12 % (Table 5.7). The increase in $T_{exc,NH3}$ is due to increased NH_4^+ concentration in the effluent in spite of lower *EQI*. $T_{exc,DO}$ has reduced due to a lower bypass volume and hence less organic load to the river.

5.4.3. Optimize Storage Tank Utilization Based on Influent Flow Rate to WWTP (C5)

Taking inspiration from the control strategies implemented in Weyand (2002) and Kroll et al. (2016), a rule-based integrated control strategy that manipulates the behaviour of the storage tanks based on flow rate information at the inlet to the WWTP is implemented. Also, C5 can be considered as an extension of C1 (which manipulates only ST_6 operation) to other storage locations (ST_1, ST_2, ST_4). If the inflow ($Q_{in,WWTP}$) to the WWTP is higher than 80 000 $m^3.d^{-1}$ and there is capacity available in the storage tank (h_{ST} < 4 m): i) only one pump is used in the pumping station at ST_1 (i.e., the pumping capacity ($Q_{pump,ST1}$) is reduced to 63 % of the maximum capacity); ii) at ST_2 and ST_6, the valve openings ($Q_{max,ST2}$, $Q_{max,ST6}$) are reduced by 50 % and 30 %, respectively; and iii) at ST_4, the throttle flow ($Q_{throttle,ST4}$) is reduced by 50 %. C3 (WWTP DO control) is also active in C5.

The control strategy shows better utilization of the storage tanks. Figure 5.6a shows that ST_6 stores water for a longer duration in C5 than in the OL case. Also, the maximum throttle flow from ST_4 is reduced in C5 (Figure 5.6b). This means that more flow is directed to ST_4 instead of being sent downstream. This increases V_{ovf} and *OQI* marginally (1 % increase compared to OL). As the control strategy tries to store more water, there are situations where it leads to increased overflows from the storage tanks.

With better utilization of the storage tanks to reduce the peak flows, the inflow to the WWTP shows reduced peaks (Figure 5.6c). Surprisingly, this does not translate to improved influent quality. With the storage tanks storing more wastewater, an increased amount of pollution is sent to the WWTP leading to higher *IQI* (2 % higher than OL). However, the *EQI* decreases by 4 % compared to OL due to reduced peak flows.

The changes in the performance of the WWTP strongly affect the river water quality. Table 5.7 indicates that, while $T_{exc,NH3}$ is better (51 % lower) than the OL case, it is 41 % higher than that in C3. As the effluent NH_4^+ concentration from the WWTP increases (reflected in the higher *EQI* values compared to C3), $T_{exc,NH3}$ in the river also increases. However, due to reduced peak flows resulting in lower bypass volumes from the WWTP, $T_{exc,DO}$ improves by 11 % and 17 % in comparison to OL and C3. The reduced bypass flows lead to a drop in the organic load to the river thereby improving the oxygen levels in the river (Figure 5.6d).

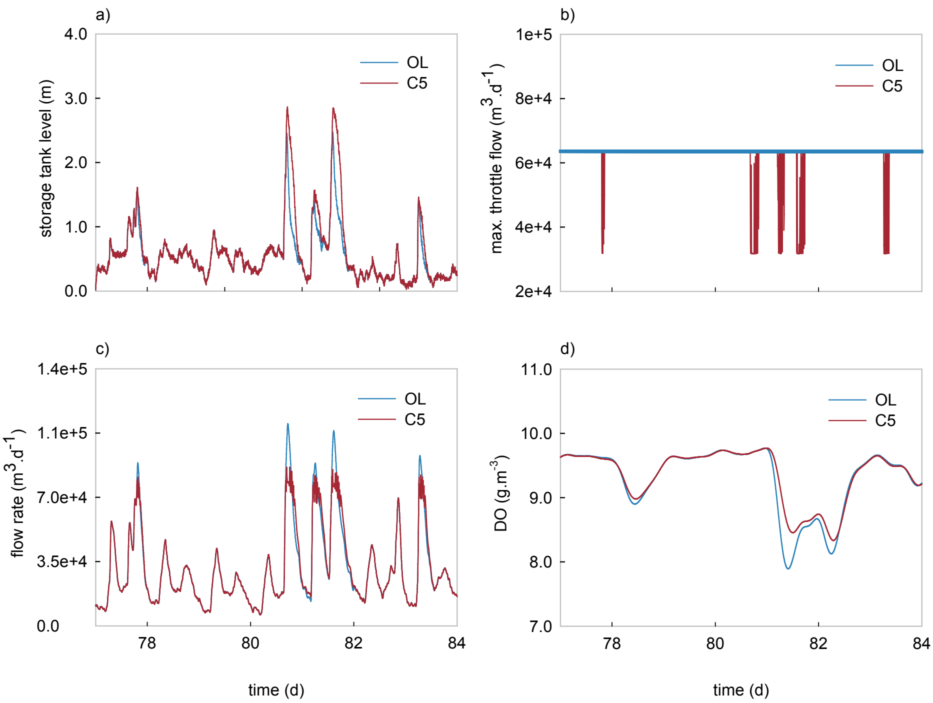

Figure 5.6: Effect of C5 on ST_6 level (a), ST_4 maximum throttle flow (b), WWTP influent flow rate (c) and DO concentration in the river (d) in comparison to OL. Day 77 is 16th September (from Paper IV).

In conclusion, a clear winner in terms of the evaluation criteria is not directly evident. The choice of control strategy depends on the needs of the actual UWS. In order to reduce the high un-ionized ammonia concentrations in the river, C3 is the choice (from the evaluated options) while C5 leads to improved oxygen concentrations and C4 can be considered as a good compromise considering both the criteria. Hence, a multi-criteria approach is needed in order to arrive at the final choice of control strategy from the evaluated case studies.

5.5. Global Sensitivity Analysis

Global sensitivity analysis (GSA) is performed for two different sets of input factors:

i. Control handles;

ii. Design parameters.

Table 5.8: List of control handles and design parameters used in the GSA study (from Paper V).

Control handles (25 % variation)			Design parameters (10 % variation)		
Section	Input factor	Description	Section	Input factor	Description
Sewer	$Q_{pump,ST1}$	Max. pump capacity for ST_1 ($m^3.d^{-1}$)	Sewer	V_{ST1}	ST_1 volume (m^3)
	$Q_{max,ST2}$	Max. throttle flow for ST_2 ($m^3.d^{-1}$)		V_{ST2}	ST_2 volume (m^3)
	$Q_{pump,ST4}$	Max. pump capacity for ST_4 ($m^3.d^{-1}$)		V_{ST4}	ST_4 volume (m^3)
	$Q_{throttle,ST4}$	Max. throttle flow for ST_4 ($m^3.d^{-1}$)		V_{ST5}	ST_5 volume (m^3)
	$Q_{max,ST5}$	Max. throttle flow for ST_5 ($m^3.d^{-1}$)		V_{ST6}	ST_6 volume (m^3)
	$Q_{max,ST6}$	Max. throttle flow for ST_6 ($m^3.d^{-1}$)	WWTP	V_{RST}	Rain storage tank volume (m^3)
WWTP	$Q_{max,RST}$	Max. throttle flow for RST ($m^3.d^{-1}$)		V_{PC}	Primary clarifier volume (m^3)
	Q_{BP1}	Max. flow rate after BP_1 ($m^3.d^{-1}$)		A_{SC}	Secondary settler area (m^2)
	Q_{BP2}	Max. flow rate after BP_2 ($m^3.d^{-1}$)		V_{ANAER1}	Anaerobic reactor 1 volume (m^3)
	Q_r	Sludge recycle rate ($m^3.d^{-1}$)		V_{ANAER2}	Anaerobic reactor 2 volume (m^3)
	Q_w	Sludge wastage rate ($m^3.d^{-1}$)		V_{ANOX1}	Anoxic reactor 1 volume (m^3)
	Q_{intr}	Internal recirculation rate ($m^3.d^{-1}$)		V_{ANOX2}	Anoxic reactor 2 volume (m^3)
	K_La_1	Oxygen transfer coefficient for AER_1 (d^{-1})		V_{AER1}	Aerobic reactor 1 volume (m^3)
	K_La_2	Oxygen transfer coefficient for AER_2 (d^{-1})		V_{AER2}	Aerobic reactor 2 volume (m^3)
	K_La_3	Oxygen transfer coefficient for AER_3 (d^{-1})		V_{AER3}	Aerobic reactor 3 volume (m^3)

The list of input factors for both sets is presented in Table 5.8. It is assumed that the control handles have a high uncertainty (25 %) as it should be possible to operate them within a broad range of variation. The uncertainty range in design values is based on Sin et al. (2009). A uniform distribution of input factors is assumed for both sets of input factors.

Morris screening is used as the GSA method. It is a simple, yet computationally effective, GSA technique that can be applied to identify influential parameters from a large set of input factors. It uses the concept of elementary effects to determine the relative sensitivity of various input factors to a model output. The mean (μ) and standard deviation (σ) of such an elementary effects distribution are used to assess the relative importance of each of these input factors for the output. Additionally, μ^*, the mean of the absolute values of the elementary effects is used to rank the parameters.

5.5.1. Control Handles

Only control handles that are influential to a limited extent are available for the sewer performance criteria. Both criteria (V_{ovf} and *OQI*) have identical sensitive control handles with similar μ^* values ($Q_{max,ST6}$ and $Q_{throttle,ST4}$) (Figures 5.7a, b).

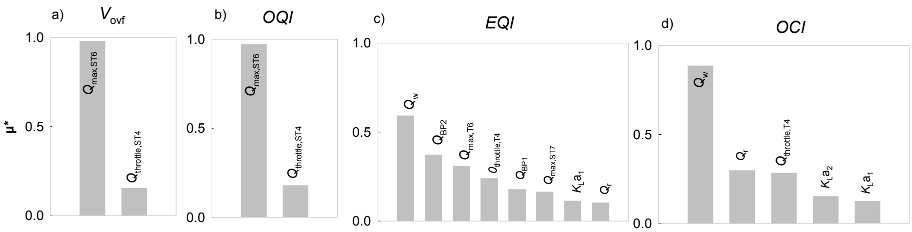

Figure 5.7: Ranking of sensitive control handles for sewer and WWTP performance for the criteria V_{ovf} (a), *OQI* (b), *EQI* (c) and *OCI* (d) (from Paper V).

The WWTP criteria *EQI* and *OCI* are affected by different input factors (Figures 5.7c, d). *EQI* is influenced by a wide number of WWTP control handles (Q_w, Q_{BP2}, $Q_{max,RST}$, Q_{BP1}, K_La_1 and Q_r) and sewer control handles ($Q_{max,ST6}$ and $Q_{throttle,ST4}$). *OCI* is understandably mainly influenced by pumping (Q_w and Q_r) and oxygen supply (K_La_1 and K_La_2) in the WWTP.

$C_{max,NH3}$ and $T_{exc,NH3}$ have an identical set of important control handles. They are mainly sensitive to WWTP control handles (Figures 5.8a, b). $Q_{throttle,ST4}$ is the only sewer control handle influencing both criteria. On the other hand, river DO criteria ($C_{min,DO}$ and $T_{exc,DO}$) are affected both by the sewer network ($Q_{max,ST6}$ and

$Q_{throttle,ST4}$) and WWTP control handles (Q_w, Q_{BP1}, Q_{BP2} and K_La_1) (Figures 5.8c, d). However, there are some control handles that influence only one of these criteria. Hence, while the river un-ionized ammonia quality is mainly affected by WWTP control handles, the DO quality criteria are influenced both by the sewer network and WWTP controls.

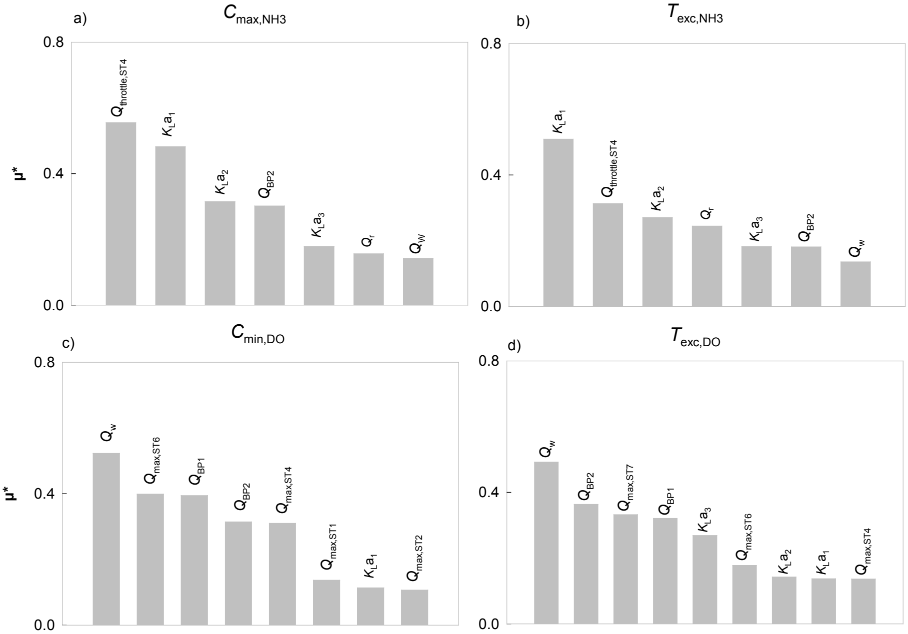

Figure 5.8: Important control handles for river water quality criteria in terms of NH_3 – $C_{max,NH3}$ (a) and $T_{exc,NH3}$ (b) and DO – $C_{min,DO}$ (c) and $T_{exc,DO}$ (d) (from Paper V).

5.5.2. Design Parameters

The overflow volume (V_{ovf}) and pollutant quality (*OQI*) are mainly influenced by the storage tank volumes (V_{ST6}, V_{ST4} and V_{ST1}) (Figures 5.9a, b). Although ST_5 is similar in volume to ST_4, it does not have any influence as it is a separate sewer network (and hence all the stormwater will eventually lead to overflows).

Both *EQI* and *OCI* have a similar set of influential design parameters with different rankings. The only exception is V_{RST} for *EQI* and V_{AER3} for *OCI*. While V_{SC} is the dominating design parameter for *EQI*, all the identified design parameters contribute similarly to the uncertainty in *OCI* (Figures 5.9c, d).

Most of the important design parameters for $T_{exc,NH3}$ and $C_{max,NH3}$ are identical and also have similar rankings (V_{AER1}, V_{AER2}, V_{AER3} and V_{PC}) (Figures 5.10a, b).

Additionally, $C_{max,NH3}$ is also affected by the volume of the secondary clarifier (V_{SC}). Hence, river un-ionized ammonia quality is mainly influenced by the biological treatment capacity in the WWTP and the effect of sewer overflows is limited.

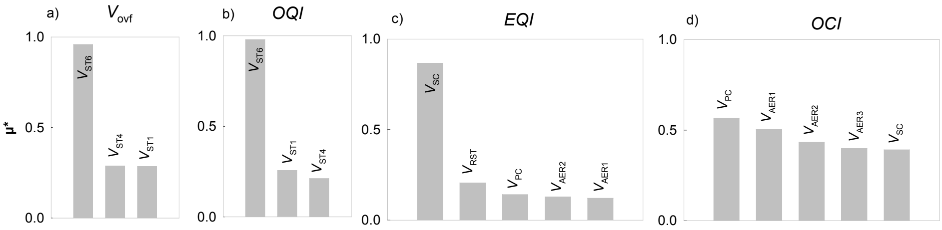

Figure 5.9: Influential design parameters affecting sewer (V_{ovf} (a) and *OQI* (b)) and WWTP (*EQI* (c) and *OCI* (d)) performance (from Paper V).

V_{SC} is the most influential parameter for both $C_{min,DO}$ and $T_{exc,DO}$. Other common design parameters are V_{PC}, V_{ST6} and V_{AER1}. V_{ST4} and V_{ST5} influence only $C_{min,DO}$, while V_{AER2}, V_{RST} and V_{AER3} have a strong effect on $T_{exc,DO}$ (Figures 5.10c, d). Hence, river DO quality criteria are affected by both WWTP and sewer design parameters.

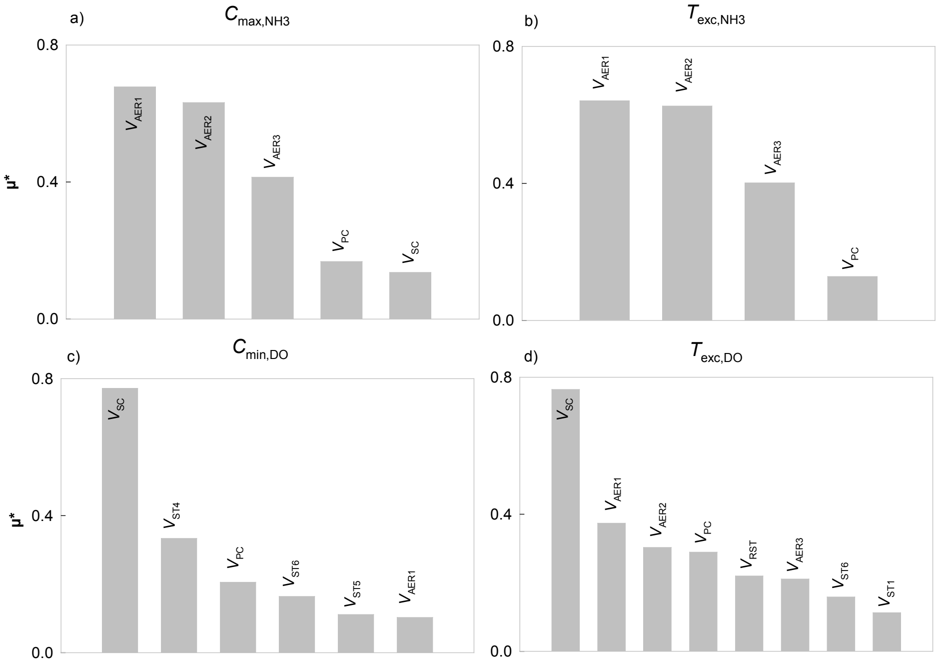

Figure 5.10: Ranking of the major design parameters influencing river water quality criteria – $C_{max,NH3}$ (a), $T_{exc,NH3}$ (b), $C_{min,DO}$ (c) and $T_{exc,DO}$ (d) (from Paper V).

5.6. BSM-UWS – Choice of Layout

During the course of model development, the layout for BSM-UWS has been extensively discussed. In the case of the WWTP, the choice is relatively straightforward, which is to use the existing BSM1 and BSM2 (in the future) layouts. The only modification to the WWTP is the addition of a storage tank at the WWTP inlet. For the river system, a simple layout with a single river running across the urban catchment with overflows at several locations is chosen. For the catchment and sewer network, the current layout is adapted from the ATV A 128 case study (ATV, 1992) and scaled up. However, it is clear that the sewer network layout varies significantly across catchments and also in different countries. Hence, it is impossible to propose a benchmark layout that represents the majority of sewer networks in the world (or at least in Europe). The provided layout is only one among the many possible layouts. It is also clear that the performance of a control strategy will vary depending on the layout. Therefore, a control strategy that performs well on the BSM-UWS does not necessarily perform similarly on another real catchment. In fact, the aim of the benchmark layout is to provide a standard layout for different research groups to simulate and evaluate their control ideas and not to find the best possible control strategy for a given UWS. However, the knowledge gained from the case studies using the BSM-UWS layout can be transferred to other layouts.

5.7. Potential Applications of BSM-UWS

The major areas where the BSM-UWS can potentially be used are mentioned below.

- Benchmarking control strategies – This is the primary objective behind the development of BSM models and it is expected that BSM-UWS will be used in a manner similar to BSM1 and BSM2 for this purpose. In particular, various rule-based control strategies (e.g. Seggelke et al., 2005; Vanrolleghem et al., 2005a), optimization routines (e.g. Fu et al., 2008; Muschalla, 2008) and permitting frameworks (e.g. Meng et al., 2016) can be evaluated using this layout.
- Adapting the model to other catchments – With the model library available (and distributed freely) for the BSM-UWS, system-wide models for real catchments can be developed and evaluated using the BSM model library as a software tool. Additionally, the toolbox can be used to model the individual sections (or select components) only.

- Including new model features – The standard layout and model library (with access to verified source code) makes the BSM family of models an ideal choice to implement new model features and evaluate them. Various model additions, such as biological reactions in the sewer network (Huisman & Gujer, 2002) and sediment dynamics in the river (Reichert et al., 2001a) can be implemented within the BSM-UWS layout.

5.8. Summary of Key Findings

- A standard layout for different sections of the BSM-UWS is presented and the major characteristics are described. An urban catchment with an area of 540 hectares and 80 000 population equivalents is connected to a sewer network with a total storage volume of 22 100 m^3. The WWTP is a BSM1_ASM2d model with unit operations for removal of organic matter and nutrients. A 30 km river stretch runs across the urban catchment with facilities to discharge sewer overflows and WWTP effluent at different locations in the river system.
- Control strategies and structural modifications to the catchment and sewer BSM are presented. The performance is evaluated using sewer overflow criteria. The control strategy C2 gives the best results in terms of cumulative effects whereas S1 reduces the acute effects significantly.
- Local and integrated control strategies are demonstrated using the BSM-UWS. The impacts of the control strategies on the performance of the sewer network, the WWTP and more importantly on the river water quality are assessed. The case studies highlight that there is no single winner in terms of all the evaluation criteria. While C3 is the choice when reducing $T_{exc,NH3}$ is important, C5 is the best strategy in terms of reaching the lowest $T_{exc,DO}$. The results highlight the difficulty in achieving improvement in both $T_{exc,NH3}$ and $T_{exc,DO}$ simultaneously.
- Influential control handles and design parameters for river water quality as well as sewer network and WWTP performance are determined.

Chapter 6

Conclusions and Future Perspectives

6.1. Conclusions

The thesis summarizes the work carried out towards extending the benchmark simulation models (BSM) “outside-the-fence” of WWTPs. The major conclusions from the work are mentioned below.

- A model library describing the: i) generation of wastewater from catchments during dry weather and rain events; ii) transport of the generated wastewater through the sewer network as well as various control possibilities in the form of storage tanks, pumps etc.; iii) physical separation processes (primary and secondary settling) and biochemical processes (activated sludge system) for the treatment of the wastewater; and iv) the effect of the discharged effluent and sewer overflows on the biochemical quality of the river is developed. Interfaces linking these different sections are developed ensuring conservation of COD, carbon, nitrogen and phosphorus. In addition to the model library, existing evaluation criteria for the WWTP, new criteria for the sewer network performance (based on sewer overflows) and river water quality in terms of river DO and NH_3 are presented.
- A system-wide benchmark model (BSM-UWS) for a pre-defined UWS layout is developed using the model library. The layout consists of: i) six sub-catchments generating wastewater; ii) a sewer network with various possibilities for control (storage tanks, pumps and throttle valves); iii) a

modified BSM1 WWTP model, which includes a rainwater storage tank, bypass facilities and primary clarifier in addition to the activated sludge system; and iv) a 30 km river stretch that receives the WWTP effluent as well as overflows from the sewer network during rain events.

- Several case studies are developed to describe the usefulness of the tools for evaluating control strategies and structural modifications on the pre-defined layout. The first set of case studies are developed using the catchment and sewer extensions. The evaluation criteria are based on sewer overflows. Case studies include both control strategies and structural modifications. The second set of case studies are focussed on control strategies using the system-wide model (BSM-UWS). Control alternatives at the local level (DO control) and integrated scale (sewer-WWTP, WWTP-river) are designed and evaluated using criteria for sewers, WWTPs and more importantly the river water quality.
- Finally, a global sensitivity analysis is performed to identify the major control handles and design parameters that affect the river water quality as well as the performance of the individual sections. It is noticed that the control handles in one section have a strong influence on the performance of the other sections. Also, the influential control handles for sewer, WWTP and river water quality are different. This indicates that control strategies that improve the performance of the sewer network or WWTP need not necessarily lead to positive results for the river water quality.

The work presents an open-source and freely distributed model library, pre-defined layout and evaluation criteria for integrated modelling and control that can be of great use to: i) develop and evaluate control strategies (local/integrated) using the system layout; ii) develop UWS models (or for the individual sections) for various real scenarios by using the model library; and iii) enhance the model development for integrated models by extending the model library.

6.2. Future Research

There are several areas for future research that can further enhance the capabilities and the application of the BSM-UWS.

The version of BSM-UWS presented in this thesis contains several simplifications. While some of these simplifications are justified in an integrated modelling set up, others can be improved to make the toolbox even more versatile. The main elements that need improvement are: i) sewer model to include backwater effects; ii) addition of sludge line to the WWTP (the sludge line already exists in other versions of the BSM but should be integrated with the system-wide model); and iii) river system model that includes sediment oxygen dynamics. Apart from these major additions, several other new features may be necessary depending on the individual case study. These include: i) processes to describe the accumulation and washoff of solubles and particulates in both the catchment and sewer network in an improved manner (the current version only includes particulate dynamics); ii) biological conversion models (to simulate the dynamics of hydrogen sulphide, methane, micropollutants etc.) in the sewer network; iii) the physico-chemical framework to model the pH variations in the WWTP and river (this is already developed but should be integrated with the existing models).

Only limited case studies on the potential application of the model library as a benchmarking tool are presented in this thesis (Chapter 5). Various new control strategies cases can be implemented using the layout. Multi-objective optimization studies as well as control strategy optimization can be performed on the layout. Also, several integrated control strategies and system modifications available in existing literature as well as new ideas can be implemented on the layout. This will be a major utilization of the developed modelling software.

As is the case with the WWTP benchmark systems, it is expected that the BSM-UWS will be used by various researcher and practitioners to describe their UWSs and evaluate scenarios that are important for the local conditions. Hence, utilizing the BSM-UWS and adapting it to other catchments is an important area for future research. Here, it is anticipated that users will also use the individual model blocks to describe only an individual section or a set of unit operations (e.g. pumping models, storage tank models etc.).

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Paper I

WATER RESEARCH 51 (2014) 172–185

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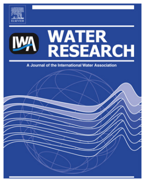

Calibration and validation of a phenomenological influent pollutant disturbance scenario generator using full-scale data

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ARTICLE INFO

Article history:
Received 29 July 2013
Received in revised form
30 September 2013
Accepted 4 October 2013
Available online 24 October 2013

Keywords:
Catchment modelling
Flow prediction
Influent modelling
Load prediction
Scenario analysis

ABSTRACT

The objective of this paper is to demonstrate the full-scale feasibility of the phenomenological dynamic influent pollutant disturbance scenario generator (DIPDSG) that was originally used to create the influent data of the International Water Association (IWA) Benchmark Simulation Model No. 2 (BSM2). In this study, the influent characteristics of two large Scandinavian treatment facilities are studied for a period of two years. A step-wise procedure based on adjusting the most sensitive parameters at different time scales is followed to calibrate/validate the DIPDSG model blocks for: 1) flow rate; 2) pollutants (carbon, nitrogen); 3) temperature; and, 4) transport. Simulation results show that the model successfully describes daily/weekly and seasonal variations and the effect of rainfall and snow melting on the influent flow rate, pollutant concentrations and temperature profiles. Furthermore, additional phenomena such as size and accumulation/flush of particulates of/in the upstream catchment and sewer system are incorporated in the simulated time series. Finally, this study is complemented with: 1) the generation of additional future scenarios showing the effects of different rainfall patterns (climate change) or influent biodegradability (process uncertainty) on the generated time series; 2) a demonstration of how to reduce the cost/workload of measuring campaigns by filling the gaps due to missing data in the influent profiles; and, 3) a critical discussion of the presented results balancing model structure/calibration procedure complexity and prediction capabilities.

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http://dx.doi.org/10.1016/j.watres.2013.10.022

Nomenclature

α_i Fraction of a given ASM1 state variable, where i can be S_I (α_{SI}), S_S (α_{SS}), X_I (α_{XI}), X_S (α_{XS}) and X_{BH} (α_{XBH})
A_1 Surface area of the variable volume tank ('Soil' model block) [m^2]
aH A parameter determining the direct contribution of rainfall falling on impermeable surfaces in the catchment area to the flow rate in the sewer ('Rain generator' model block) [%]
ASMs Activated Sludge Models
BSM1 Benchmark Simulation Model No.1
BSM2 Benchmark Simulation Model No.2
COD Chemical Oxygen Demand
COD_{part} Particulate COD
$COD_{part\_gperPEperd}$ COD_{part} load per person equivalent per day ('Households pollutants' model block) [(g COD pe^{-1}) d^{-1}]
$COD_{part\_Ind\_kgperd}$ COD_{part} load from industry per day ('Industry pollutants' model block) [kg COD d^{-1}]
COD_{sol} Soluble COD
$COD_{sol\_gperPEperd}$ COD_{sol} load per person equivalent per day ('Households pollutants' model block) [(g COD pe^{-1}) d^{-1}]
$COD_{sol\_Ind\_kgperd}$ COD_{sol} load from industry per day ('Industry pollutants' model block) [kg COD d^{-1}]
COD_{tot} Total COD
FFfraction Fraction of TSS that can settle in the sewer ('First flush effect' model block) [–]
G_{rain_Temp} Proportional gain to adjust the temperature after a rain event ('Temperature' model block) [–]
G_{snow_Temp} Proportional gain to adjust the temperature after a snow event ('Temperature' model block) [–]
InfBias Mean value of the sine wave signal for generating seasonal effects due to infiltration ('Seasonal correction factor' model block) [m^3 d^{-1}]
IWA International Water Association
K_{down} Gain for adjusting the flow rate to downstream aquifers ('Soil' model block) [m d^{-1}]
K_{inf} Infiltration gain ('Soil' model block) [$m^{2.5}$ d^{-1}]
M_{max} Maximum mass of stored sediment in the sewer system ('First flush effect' model block) [kg]
MC Monte Carlo simulation technique
N_{tot} Total N concentration [g N m^{-3}]
N_{org} Total organic N concentration [g N m^{-3}]
PE Person equivalent ('Households' model block) [–]
$Q_{Ind\_weekday}$ Average wastewater flow rate from industry on normal weekdays (Monday to Thursday) ('Industry' model block) [m^3 d^{-1}]
Q_{lim} Flow rate limit triggering a first flush effect ('First flush effect' model block) [m^3 d^{-1}]
Qpermm Flow rate per mm rain ('Rain generator' model block) [m^3 mm^{-1}]
Qpercm Flow rate per cm of snow ('Rain generator' model block) [m^3 cm^{-1}]
QperPE Wastewater flow rate per person equivalent ('Households' model block) [m^3 d^{-1} PE^{-1}]
S_U Inert soluble COD [g COD m^{-3}]
NH_{4Ind_kgperd} S_{NHX} load from industry per day ('Industry pollutants' model block) [kg N d^{-1}]
$NH_{4gperPEperd}$ S_{NHX} load per person equivalent per day ('Households pollutants' model block) [(g N pe^{-1}) d^{-1}]
S_B Readily biodegradable COD [g COD m^{-3}]
Subareas A parameter that forms a measure of the size of the catchment area. It will determine the number of variable volume tanks in series that will be used for describing the sewer system ('Sewer' model block) [–]
TKN Total Kjeldahl nitrogen [g N m^{-3}]
T_{Amp} Seasonal temperature variation, amplitude ('Temperature' model block) [°C]
T_{Bias} Seasonal temperature variation, average ('Temperature' model block) [°C]
Td_{Amp} Daily temperature variation, amplitude ('Temperature' model block) [°C]
Td_{Freq} Daily temperature variation, frequency ('Temperature' model block) [rad d^{-1}]
Td_{Phase} Daily temperature variation, phase shift ('Temperature' model block) [rad]
T_{Freq} Seasonal temperature variation, frequency ('Temperature' model block) [rad d^{-1}]
$TKN_{gperPEperd}$ *TKN* load per person equivalent per day ('Households pollutants' model block) [(g N pe^{-1}) d^{-1}]
TKN_{Ind_kgperd} TKN load from industry per day ('Industry pollutants' model block) [kg N d^{-1}]
T_{Phase} Seasonal temperature variation, phase shift ('Temperature' model block) [rad]
TSS Total suspended solids concentration [$g.m^{-3}$]
WWTP Wastewater treatment plant
X_{ANO} Autotrophic biomass [g COD m^{-3}]
X_{OHO} Heterotrophic biomass [g COD m^{-3}]
X_U Inert particulate COD [g COD m^{-3}]
$XC_{B,N}$ Particulate organic nitrogen [g N m^{-3}]
XC_B Slowly biodegradable particulate COD [g COD m^{-3}]
WWTP1 Data set for WWTP1 (Bromma, Stockholm, Sweden)
WWTP2 Data set for WWTP2 (Lynetten, Copenhagen, Denmark)

1. Introduction

The use of activated sludge models (ASM) is constantly growing and both industry and academia are more and more introducing these tools when performing WWTP benchmarking (Copp, 2002; Jeppsson et al., 2007; Gernaey et al., 2014), diagnosis (Rodriguez-Roda et al., 2002; Olsson, 2012), design (Flores et al., 2007; Rieger et al., 2012), teaching (Hug et al., 2009) and optimization (Rivas et al., 2008). The level of detail and the specific data required for a modelling exercise strongly depend on the project objectives (Rieger et al., 2012). In general, the more detailed and accurate the results of the simulation study, the more detailed is the set of data needed. For instance monthly or yearly average values may be sufficient for a large variety of steady-state modelling projects, such as developing a process configuration for biological nutrient removal (Flores-Alsina et al., 2012a) or calculating sludge production (Spérandio et al., 2013). However, if dynamic processes need to be investigated, daily values and typical diurnal patterns are required. Indeed, there is a need for high frequency influent data to accurately optimize aeration systems (Rieger et al., 2004), develop control strategies to handle storm flows (Maruejouls et al., 2012) or design treatment systems to meet effluent nitrogen/phosphorus limits (Phillips et al., 2009).

Full characterization of influent profiles requires a large amount of work and there is a high cost involved when analysing the samples taken from the influent wastewater for the most relevant influent pollutants (soluble and total chemical oxygen demand (COD), total suspended solids (TSS), ammonium nitrogen (NH_4–N), total Kjeldahl nitrogen (TKN), orthophosphate phosphorus (PO_4–P), total phosphorus (TP), etc.). Recent developments in measurement technology make nitrogen, phosphorus and total organic carbon sensors more reliable and cheap. Nevertheless, there is often a severe mismatch between the assumed and the real values entering the treatment plant, for example as a consequence of drift of the sensor signal (Rieger et al., 2004). Indeed, several standard lab analyses, such as COD, can still not be performed reliably in on-line mode on the influent of a WWTP (Olsson, 2012).

In essence, the success of many modelling studies strongly depends on having sufficiently long influent time series – the main disturbance of a typical WWTP – representing the inherent natural variability of the flow rate, pollutant concentrations and temperature at the plant inlet as accurately as possible (Ráduly et al., 2007). This is an important point to consider since most modelling projects suffer from lack of realistic data representing the influent wastewater dynamics. When including a catchment-specific detailed list of perturbations, model predictions are both more reliable and useful in practice and the “real” capacity of the plant under study to handle different types of events is better understood.

Literature offers a wide range of tools reproducing influent characteristics by means of mathematical models. The simplest approaches are based on black-box models and generate the required time series by means of different modelling principles. For example, dynamic influent flow rate data can be reproduced by means of a simple equation, such as a Fourier series (sum of sinusoids with varying frequencies and phase shifts) whose parameters are fitted to dynamic influent data (e.g. Carstensen et al., 1998; Bechmann et al., 1999; Langergraber et al., 2008; Alex et al., 2009). Stochastic models are a bit more complex and they are especially suited to describe compounds without a clear pattern of occurrence. For example Ort et al. (2005) developed a stochastic model describing short-term variations of benzotriazole concentrations (a chemical contained in dishwasher detergents). Additionally, during the development of the BSM1_LT platform (BSM1 Long Term, Rosen et al., 2004), the influent model for ASM variables was combined with a Markov chain modelling approach to describe the occasional occurrence of either toxic or inhibitory influent shock loads (Rosen et al., 2008). The same principle was later used by Snip et al. (2013) to describe the occurrence of diclofenac and other xenobiotic compounds without a clear administration pattern. Finally, the most complex approaches are constructed using first principles' models and contain a full deterministic description of the different draining units (Achleitner et al., 2007; Elliott and Trowsdale, 2007; Devesa et al., 2010). As model complexity increases, the predictive capabilities of the model might be improved, i.e. it can capture the dynamics related to a larger variety of phenomena. However, the calibration effort increases exponentially. This can be overcome by using sophisticated/computer time-demanding calibration procedures based on Bayesian methods (Lindblom et al., 2011; Rieckermann et al., 2011) or increasing the required quantity of data/measurements to adjust the different model parameters (Gamerith et al., 2009, 2011).

Phenomenological models have also shown to be useful tools to handle these types of problems (Gernaey et al., 2006; De Keyser et al., 2010; Gernaey et al., 2011). This is mainly due to three basic principles which are applied during the model development: 1) model parsimony, i.e. limiting the number of parameters as much as possible; 2) model transparency, by using model parameters that have a physical meaning when possible; and, 3) model flexibility, such that the proposed influent model can be easily modified/extended for other applications where long influent time series are needed. Phenomenological models represent a good compromise between model complexity and prediction capability. For this reason the authors believe that such approaches can be valuable tools to improve influent data quality during modelling studies.

The objective of this paper is to demonstrate the usefulness of the phenomenological dynamic influent pollutant disturbance scenario generator (DIPDSG) that was previously used to create the influent data of the International Water Association (IWA) Benchmark Simulation Model No. 2 (BSM2) with two full-scale applications. The Benchmark Simulation models (BSM) were originally developed to objectively compare control/monitoring strategies in wastewater treatment systems (Gernaey et al., 2014). More than 300 scientific papers and theses on work related to the benchmark systems have been published to date (Jeppsson et al., 2013) demonstrating the interest and need of the scientific community for the tools developed in this framework. In fact, some of the modelling tools originally developed to compare control strategies are being used as stand-alone tools in full-scale applications. Examples are: computationally efficient

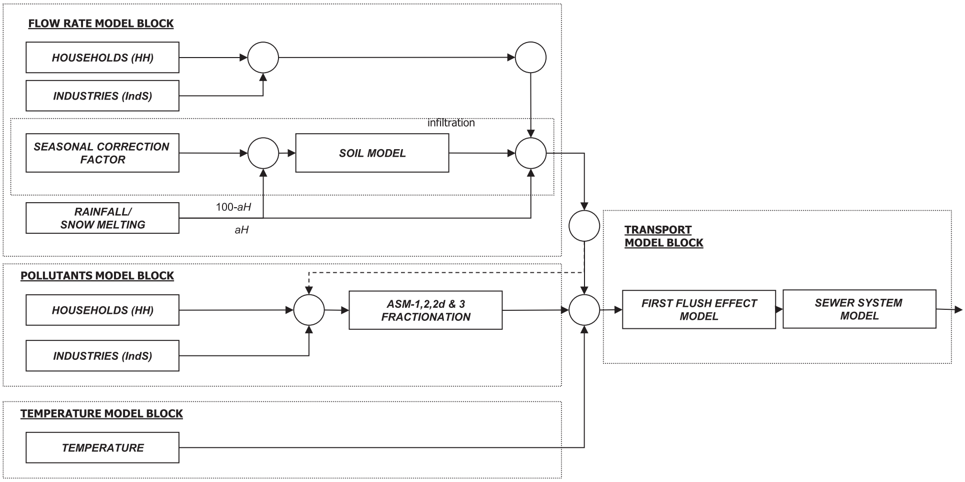

Fig. 1 – Schematic representation of the modified influent generator to describe flow rate/pollutants/temperature dynamics.

implementations (Rosen et al., 2006) of the Anaerobic Digestion Model No. 1 (Batstone et al., 2002), the ASM/ADM interfaces (Nopens et al., 2009) and the BSM2 final plant layout (Nopens et al., 2010).

In this study, flow rate, pollution loads and temperature characteristics of two large Scandinavian treatment facilities (*WWTP1*, *WWTP2*) are studied for a period of two years. *WWTP1* is located in Stockholm (Sweden), while *WWTP2* is near Copenhagen (Denmark). The paper is organized as follows: Firstly, the two studied facilities and the phenomenological modelling approach used in the case studies are described. Next, the calibration/validation results of the model predictions with respect to flow rate, temperature and organic material/nitrogen loads are presented. Finally, the benefits of using such an approach are demonstrated with a scenario analysis demonstrating situations where the DIPDSG can be especially useful. The paper ends with a critical discussion of the results.

2. Methods

2.1. *WWTPs under study*

Wastewater treatment plant Bromma (*WWTP1*) receives wastewater from the western part of Stockholm (Sweden), based on a catchment area of 25 km^2. Bromma has a capacity of 300,000 PE and removes organic matter (COD), phosphorus (P) and nitrogen (N) by mechanical, biological and chemical processes. A data set comprising the years 2009 and 2010 is used for the case study. During this period the average influent flow rate was 120,000 $m^3\ d^{-1}$ and the pollution loads were approximately 35,000 kg COD d^{-1}, 3100 kg N d^{-1} and 380 kg P d^{-1}. The maximum (design) plant capacity (into the activated sludge system) is 10,800 $m^3\ h^{-1}$.

Wastewater treatment plant Lynetten (*WWTP2*) receives wastewater from the central and north-eastern part of Copenhagen, Denmark (8 municipalities), based on a catchment area of 76 km^2. Lynetten has a capacity of 750,000 PE and removes COD, N and P from wastewater by mechanical, biological and chemical processes. A data set comprising the years 2010 and 2011 is used for the case study. During this period the average influent flow rate was 170,000 $m^3\ d^{-1}$ and average pollution loads were 95,000 kg COD d^{-1}, 7300 kg N d^{-1} and 1200 kg P d^{-1}. The maximum plant capacity (into the activated sludge system) is 23,000 $m^3\ h^{-1}$.

In both cases (*WWTP1* & *2*) high frequency data were available for flow rate and temperature (1 sample every 6 min). Rainfall and snow melting data were reduced to three samples per day (1 sample every 8 h). Rainfall, snow melting and outdoor temperature data for Stockholm were obtained from the Swedish Meteorological and Hydrological Institute (SMHI) (www.smhi.se). Two to three (daily-averaged) flow proportional samples were taken per week for wastewater characterisation. COD was measured by the standard dichromate method, while N was determined by a colorimetric method in a flow-injection system. In all cases, the measurements reproduce wastewater quantity and quality at the inlet of the WWTP (before by-pass and primary clarifier).

2.2. *Description of DIPDSG*

The phenomenological DIPSDG model used in the case studies (Gernaey et al., 2011) is sub-divided in four main blocks: 1) flow rate; 2) pollutants (carbon, nitrogen and phosphorus); 3) temperature; and, 4) a transport model block. The software is implemented in Matlab/Simulink (www.mathworks.com) and can be obtained for free by contacting Prof. Krist Gernaey at the Department of Chemical and Biochemical Engineering, Technical University of Denmark (kvg@kt.dtu.dk).

The generation of the influent flow rate is achieved by combining the contributions from households (*HH*), industry (*IndS*), rainfall (*RAIN*), snow melting (*SNOW*) and infiltration (*Inf*) (see Fig. 1, top). Rainfall and snow melting contribute to the total flow rate in two ways: the largest part (*aH*) of the rainfall contribution to the flow rate originates from runoff from impermeable surfaces, and is thus transported directly to the sewer. Rainfall and snow melting on permeable surfaces (fraction 100 − *aH* of the total flow rate) will influence the groundwater level, and thus the contribution of infiltration to the influent flow rate. Assuming a dry, a snow-melting and a rainy season, the 'Seasonal correction factor' model block creates the desired seasonal effects. This seasonal correction is combined with the rainfall falling on permeable surfaces, and the sum of both flows is passed through the soil (SOIL) model block. Afterwards, the net contribution of infiltration – an output of the 'Soil' model block – is combined with the overall flow rate resulting from industry and households and the flow rate contribution from rainfall on impermeable surfaces.

Similar to the flow rate generation model, it is assumed (see Fig. 1, middle) that there are two pollutant sources, households and industry, which is an acceptable simplification. Thus, the complexity of the model is reduced by neglecting other sources (infiltration and run-off). Including contributions from run-off can for example be important if micro-pollutant influent profiles are needed, but that is not the purpose of the work presented here. Pollutants from both sources are combined and converted to ASM-X state variables, where *X* can be 1, 2, 2d or 3 (Henze et al., 2000).

The temperature of the wastewater is included as an additional state variable in the model influent (see Fig. 1, bottom). The temperature profile includes a seasonal effect, i.e. it is assumed that there is a warm and a cold season. In addition, the model includes a daily temperature effect since the temperature during daytime is slightly different from the wastewater temperature at night.

Finally the last model block is related to transport (Fig. 1, right). The particulate pollutants of each submodel are subsequently passed through the 'First Flush Effect' model block, where first flush effects are mimicked as a function of flow rate: first flush effects will for example occur during severe rain events following dry weather periods. Finally, the size of the sewer system is incorporated in the influent dynamics as well: the larger the sewer system, the smoother the simulated diurnal flow rate and concentration variations.

This phenomenological modelling approach is used to describe the 2009/2010 and 2010/2011 time series of *WWTP1* and 2, respectively. Measured data from the first year are used for model calibration purposes and model output/measurements for the second year are used for validation.

3. Results

3.1. *Dynamic modelling of influent flow rate in WWTP1*

A stepwise procedure based on the results of the global sensitivity analysis (GSA) performed by Flores-Alsina et al. (2012b) is used to calibrate/validate the parameter set of the used DIPDSG. This GSA study demonstrated that a stronger/weaker impact of the model parameters on the flow rate dynamics depends on the analysed time scale. Indeed, the dynamics of the generated influent time series are affected differently by a certain parameter if the considered time scale is months, days or hours. In this case study we opted to first calibrate flow rate per person equivalent (*QperPE*), the industrial contribution ($Q_{Ind\_weekday}$) and the parameters related to the soil model: 1) the gain to adjust the flow rate to the downstream aquifers (K_{down}); and, 2) the infiltration gain, a measure of the quality of the sewer system pipes (K_{inf}), since it was found out in the GSA that they have the strongest impact on the yearly flow rate profile. Next, wet weather conditions were fine-tuned when the time scale was reduced to days by modifying both *Qpermm* and *Qpercm*, two parameters which convert either rain intensities or snow level measurements into flow. Finally, at the shortest time scale (hours), the suitable parameter values are identified to correctly describe the hourly dynamics (flow rate peaks), which is achieved in practice by extending/reducing the sewer length (parameter *subareas*).

The (measured) yearly averaged influent flow rate is around 120,000 $m^3\ d^{-1}$. Default parameter values are used to estimate the dry/wet weather fractions and the contributions from households (*HH*), industry (*IndS*) and infiltration (*Inf*) (Gernaey et al., 2011). The total (120,000 $m^3\ d^{-1}$) to dry weather (98,000 $m^3\ d^{-1}$) influent flow-rate ratio is around 1.22.

3.1.1. *Households, industry and infiltration (DRY weather flow)*

In dry weather conditions 56% of the influent flow rate is assumed to originate from *HH* and industrial wastewater *IndS*. Specifically, 82% of this dry weather flow fraction is produced by *HH*, whereas the other 18% represents industrial wastewater. The remaining 44% of the influent flow rate (dry weather conditions) originates from groundwater infiltration.

For example, the 'Households' (*HH*) model block contributes to the final influent flow rate dynamics by diurnal variations, a weekend effect and a holiday effect. This is achieved by combining three normalized user-defined data files containing: 1) a diurnal profile; 2) a weekly household flow pattern including the weekend effect; and, 3) a yearly pattern including the holiday effect. The relative contributions from the data files are combined by multiplication. The generated signal is then multiplied by two gains corresponding to the flow rate per person equivalent ($QperPE = 0.150\ m^3\ d^{-1}$) and the number of person equivalents in the catchment area (*PE* = 300,000 population equivalent), respectively. The dynamic flow rate pattern is obtained by repeating the data files in a cyclic manner.

The industrial contribution to the influent flow rate is generated similarly to the *HH* model block. The industry (*IndS*) model block is also based on user-defined files describing diurnal variations, and weekend and holiday effects. Again, the dynamic flow rate pattern sampled in a cyclic manner from source files is multiplied by a gain representing the use of water by industry ($Q_{Ind\_weekday} = 10{,}000\ m^3\ d^{-1}$) similar to the generation of the diurnal flow rate patterns in the *HH* model block.

A continuous groundwater contribution due to infiltration processes is assumed. Thus, the quantity of water originating

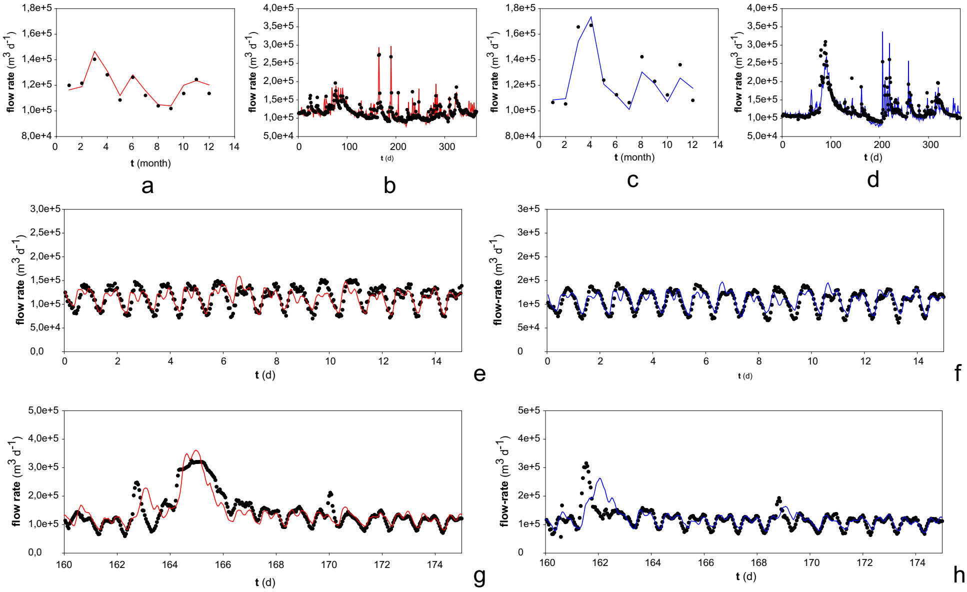

Fig. 2 – Calibration (red)/validation (blue) results (dots: measurements, solid line: simulations) of the influent flow rate for 2009 (a,b,e,g) and 2010 (c,d,f,h) for WWTP1 at different time scales: months (a,c), days (b,d) and four (15 day) snapshots (winter/summer) in hours (e,f,g,h). Hourly variation shows one morning peak, one evening peak and late night minima. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

from upstream aquifers has to be modified (*InfBias* = 80,000 m^3 d^{-1}) in order to reach the pre-established percentage of flow rate due to infiltration, i.e. 44% of the dry weather flow rate. Also, it is necessary to adjust the quantity of water going to: 1) downstream aquifers (K_{down} = 30,000 m^2 d^{-1}); and, 2) infiltrated through the sewer pipes (K_{inf} = 15*K_{down}). Again, default parameters were used to consider seasonal changes of the amount of groundwater throughout the year (*Inf*). These effects are assumed to be the result of seasonal, temperature-related changes in the amount of evaporation.

3.1.2. *Rainfall and snow melting (WET weather flow)*

In the *RAIN* model block rainfall intensities are converted to flow rate values using a conversion factor. The parameter *Qpermm*, was estimated to be equal to 25,000 m^3(mm rain)$^{-1}$. For this particular case, the original model structure (Gernaey et al., 2011) had to be modified in order to take into account snow melting (*SNOW*), which has a strong influence on influent dynamics in large parts of Sweden. In essence the snow-melting module is based on the same principle as the rain generator. Snow level differences within Stockholm are used to calculate the contribution to the influent flow rate. In order to differentiate snow melting (surface runoff produced from melting snow) from snow compaction (decrease of the snow level due to gravity forces but without actual melting), the snow-melting module is only activated when the ambient temperature is positive. Differences in snow levels are converted into flow rate using the conversion factor *Qpercm* = 35,000 m^3(cm snow)$^{-1}$. There is also a parameter *aH*, which is set to 75% and corresponds to the contribution of rainfall/snow melting falling on impermeable surfaces in the catchment area (A_1 = 25 km^2, porosity = 2.0 g cm^{-3}).

Finally, the length of the sewer is adjusted by the parameter *subareas* in order to refine the hourly peaks and the overall dynamics of the generated time series (*subareas* = 8). Each subarea is comprised of 3 continuous stirred tank reactors (CSTRs) with varying volumes.

3.1.3. *Flow-rate calibration and validation results*

Fig. 2 illustrates that the simple phenomenological model allows reproducing the yearly variation in the influent flow rate data of WWTP1 for the period comprised between 2009 (calibration, Fig. 2a,b,e,g) and 2010 (validation, Fig. 2c,d,f,h). Fig. 2 a,b,c,d show that the proposed approach correctly describes the main rain events. The calibrated influent model can also capture the snow-melting period, which mainly takes place during March and April (months 3–4, days 80–100) during both calibration and validation periods. In addition, the model also captures the catchment's seasonal variation, which is basically a flow decrease during summer and winter caused by the holiday periods. Fig. 2b,d depict an interesting feature of the soil model that allows including a 'memory effect',

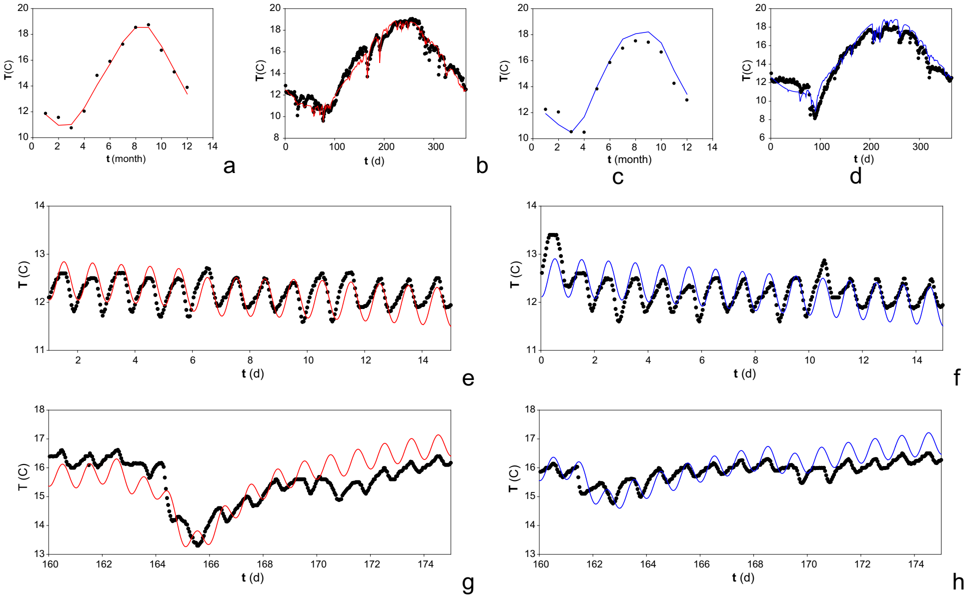

Fig. 3 – Calibration (red)/validation (blue) results (dots: measurements, solid line: simulations) of the influent temperature for 2009 (a,b,e,g) and 2010 (c,d,f,h) for *WWTP1* at different time scales: months (a,c), days (b,d) and four (15 day) snapshots (winter/summer) in hours (e,f,g,h). Hourly variation shows the difference between day/night wastewater temperature. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

following a rainfall/snow melting, i.e. each major rain event is followed by a 'tail' in the flow rate. This tail illustrates the effect of passing a percentage ($100 - aH = 25\%$) of the rainfall through the soil model block (see Fig. 1). Delay of rainfall water due to the accumulation in the soil model is a feature that can be commonly observed in full-scale facilities (Tchobanoglous et al., 2003). This phenomenon can be calibrated by modifying the parameter K_{inf} and can describe different infiltration dynamics, thereby providing an indication of the quality of the sewer pipes. Finally, Fig. 2e,g,f,h describe the daily flow rate profile which represents a general behaviour, namely one morning peak, one evening peak, and late night and mid-day minima. The morning and evening peaks represent the increased activity of the residents just before going to work (morning) or after returning from work. The late night minimum flow rate corresponds to the night hours with strongly reduced water consumption. The daytime flow rate shows a small decrease corresponding to the residents' working hours. Finally, it should be mentioned that some discrepancies can be observed (delays/bias) when reproducing rain events at high frequency. This is mainly due to the lack of high resolution rain data when performing this study (the authors had only access to three measurements per day). More complex rainfall models could provide a better correlation between the measurements and the simulation data. A more in-depth analysis of the required model complexity and the prediction capabilities of the model are provided in the Discussion section.

3.2. *Dynamic modelling of influent temperature in WWTP1*

In this section, the same type of approach is used to calibrate/validate the influent temperature. First of all, the parameters, which have a strong effect on the largest time scale, are estimated. These parameters represent the yearly average temperature (T_{Bias}), amplitude (T_{Amp}), frequency (T_{Freq}) and phase shift (T_{Phase}). Secondly, parameters describing the effects of rainfall (G_{rain_Temp}) and snow melting (G_{snow_Temp}) on the overall temperature are adjusted. Those two parameters are additional modifications of the original model proposal (Gernaey et al., 2011) in order to account for the effect of rainfall/snow melting on the temperature dynamics. Finally, the high frequency variation in the data is mimicked by calibrating daily amplitude Td_{Amp} (maximum and minimum temperature during the day), frequency (Td_{Freq}) and phase (Td_{Phase}).

3.2.1. *Seasonal and daily temperature variation*

The temperature profile is modelled by a sine function with a determined average temperature ($T_{Bias} = 15$ °C), amplitude ($T_{Amp} = 5$ °C) and frequency ($T_{Freq} = 2\pi$ rad year^{-1}). A phase shift ($T_{Phase} = 28.8\,\pi$ rad) was applied, such that the maximum flow rate due to infiltration approximately corresponds to the lowest temperature, and vice versa. Two proportional gains G_{rain_Temp} and G_{snow_Temp}, both with a value of 0.4, are multiplied to the rain and snow-melting data and subsequently subtracted from the generated time series in order to reproduce the observed temperature drop in the influent when these phenomena take place. Finally, a daily temperature effect is included in the temperature model, assuming that the temperature in the WWTP influent varies according to a sinusoidal wave with amplitude Td_{Amp} of 0.5 °C. The parameters for the daily temperature dynamics were tuned such that the minimum temperature for each day occurs around 3 a.m. ($Td_{Freq} = 2\pi$ rad $^{-1}$ and $Td_{Phase} = 1.5\ \pi$ rad^{-1}).

3.2.2. *Temperature calibration and validation results*

Fig. 3 shows the calibration results for 2009 (Fig. 3a,b,e,g) and the validation results for 2010 (Fig. 3c,d,f,h) based on *WWTP1* data. In both cases, the modelled temperature can reproduce the seasonal effect, i.e. there is a warm and a cold season (Fig. 3a,b,c,d). In addition, when the level of detail is expanded by switching to a time scale of days, the effect of snow melting and heavy rain events can be observed (Fig. 3b,d). For example, in both 2009 and 2010, it can be seen that snow melting takes place around March and April (months 3–4, days 80–100), which decreases the water temperature even more. In 2009, two heavy rain events (Fig. 2b) can be observed in the modelled temperature profile as two noticeable temperature drops (Fig. 3b). In the validation data set, a similar phenomenon can be observed at the end of the summer and beginning of the fall. Finally, at the highest frequency the hourly temperature variations can be observed, i.e. temperature increases during daytime and decreases at night. Again, the model is able to capture the general trend in these variations (Fig. 3e,f,g,h). However, some differences can be observed. A further model extension could be added in order to better represent: 1) the daily variations between winter and summer time (i.e., the temperature does not change in the same way); and, 2) the recovery after a snow melting/heavy rain period. This issue is addressed in more detail in the Discussion section.

3.3. *Dynamic modelling of influent loads in WWTP2*

In the third case study the DIPDSG is adapted to reproduce the influent characteristics of *WWTP2*. Again, the same approach of calibrating the model parameters at different time scales has been used. First of all, at the largest time scale (months), the *HH* and *IndS* contributions have been adjusted based on the different COD and N loads. For *HH* these loads represent the quantity of pollution per person equivalent (COD_{sol_gperPE}, COD_{part_gperPE}, $NH_{4gperPE}$ and TKN_{gperPE}) in terms of soluble and particulate COD, ammonium and total Kjeldahl nitrogen. A similar parameter set is used to quantify the industrial contribution, which is represented by the parameters $COD_{sol\_Ind\_kgperd}$, $COD_{part\_Ind\_kgperd}$, NH_{4Ind_kgperd} and TKN_{Ind_kgperd}. Using this approach, the complexity of the model is reduced by neglecting other potential sources of organic material and nitrogen (infiltration and runoff). Thus, the calibration procedure is enormously simplified, but on the other hand it decreases the prediction capabilities of the model. Clearly, this is a trade off which has to be decided for each case study. Secondly, at a medium time scale (days), the parameters of the first flush model are fine-tuned to correctly describe the accumulation-release of particulate material in the sewer system. The 'First flush effect' model block introduces additional influent dynamics corresponding to a flushing of the sewer system following a severe rain event. The model is based on the assumption that a fraction of the particulate material can settle in the sewer system. Part of the accumulated material is released when there is a sudden increase of flow modifying the dynamics of the particulate compound concentrations at the inlet of the WWTP. The parameters to adjust are: 1) the quantity of particulate material that can settle in the sewer ($FF_{fraction}$); 2) the total mass of solid particulates that can be accumulated (M_{Max}); and, 3) the flow rate limit triggering the first flush event (Q_{lim}).

3.3.1. *Households and industry (DRY weather loads)*

Similarly as when re-creating the influent flow rate, the *HH* model block contributes to the final pollution profile with daily/weekly/seasonal variation by means of user defined profiles for COD (COD_{sol} and COD_{part}), Kjeldahl nitrogen (*TKN*) and ammonium nitrogen (NH_4–N). The relative contributions of the data files are combined by multiplication and then transformed into g COD PE^{-1} and g N PE^{-1} units, by using the following gains: COD_{sol_gperPE} = 16.42 g COD PE^{-1} d^{-1}, COD_{part_gperPE} = 97.82 g COD PE^{-1} d^{-1}, $NH_{4gperPE}$ = 4.82 g N PE^{-1} d^{-1} and TKN_{gperPE} = 7.23 g N PE^{-1} d^{-1}. The assumed number of inhabitants in the catchment is 750,000 (PE). Again, the industrial contribution is generated similarly to the *HH* model block. Pollutant-specific user-defined profiles are sampled cyclically describing diurnal variations, weekend and holiday effects. The industrial contribution represents 10% and 5% of the total organic and nitrogen loads, respectively. These industrial loads represent a daily quantity of 18,000 kg COD d^{-1} ($COD_{sol\_Ind\_kgperd}$ + $COD_{part\_Ind\_kgperd}$) and 2700 kg N d^{-1} (TKN_{Ind_kgperd}). The ratio between soluble and particulate COD has been adjusted on the basis of available experimental data (results not shown) and is 0.16. The same procedure is followed to determine the ratio between NH_4–N and *TKN*. No nitrate nitrogen is considered to be present in the influent.

3.3.2. *First flush effect (WET weather loads)*

During dry weather conditions it is assumed that 75% ($FF_{fraction}$) of the total particulate material can settle in the sewer system. This builds up a mass of total suspended solids until a maximum storage capacity of 120·10^3 kg (M_{Max}) is reached. This quantity of solids is supposed to remain at the bottom of the sewer until it is flushed out after a heavy rain event. In the model, such a first flush is triggered if the current flow rate is higher than a threshold Q_{lim} (Q_{lim} = 560,000 m^3 d^{-1}). The M_{max} and Q_{lim} values are estimated by trial and error to match the simulation results with the experimental data.

3.3.3. *COD and N loads calibration and validation results*

Fig. 4 shows the full-scale data and the model simulations for COD (Fig. 4a,b,c,d) and TKN (Fig. 4e,f,g,h). The simulation results show that the set of phenomenological models presented herein can describe the general (yearly) variation of

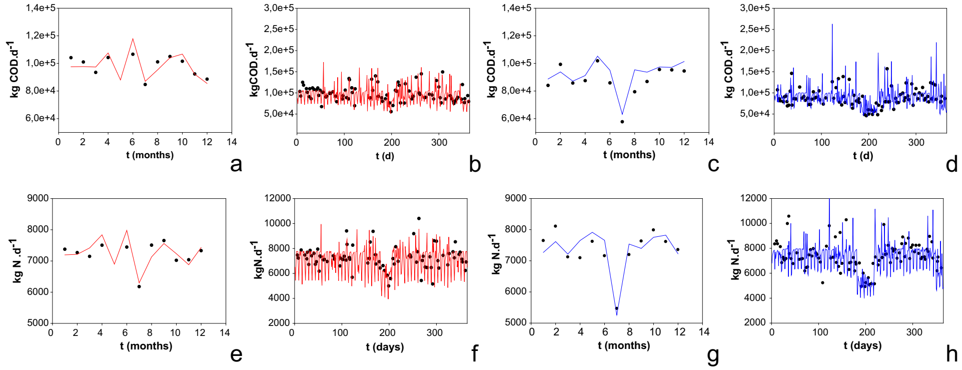

Fig. 4 – Calibration (red)/validation (blue) results (dots: measurements, solid line: simulations) of the influent COD (a,b,c,d) and TKN (e,f,g,h) for 2010 (a,b,e,f) and 2011 (c,d,g,h) for *WWTP2* at different time scales: months (a,c,e,g) and days (b,d,f,h). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

both compounds. In both 2010 and 2011, a substantial reduction in the pollutant load can be observed during the summer months (7–8), which corresponds to the holiday period (see Fig. 4, month 7, day 200). Finally, the sudden increases in the pollution load are mainly caused by the flush out of the particulate fraction that is stored in the sewer system during dry weather conditions (first flush). Other sources of pollution can be included in the model as well, e.g. run-off, but again this would add a higher degree of complexity to the model structure and the associated calibration procedure (see Discussion section).

4. Scenario analysis

The scenario analysis presented in this section demonstrates some benefits of using a DIPDSG when performing WWTP model studies. Therefore, for exemplary purposes, we make use of the calibrated and validated results of the two analyzed plants (*WWTP1* and *2*) and study how the results are affected by changing some settings in three different scenarios. *Scenario 1* evaluates the variation of the influent flow rate for different rainfall patterns. In *Scenario 2* the effect of parameter uncertainty on the predicted influent biodegradability is studied. Finally, *Scenario 3* shows how the workload of measuring campaigns can be reduced by synthetically generating high frequency data on the basis of available low frequency data.

4.1. *Scenario 1: effect on different rainfall patterns*

The contribution of rainfall has a huge impact on the yearly influent flow rate profile. Rainfall patterns affect the quantity of water arriving via: 1) run-off from impervious surfaces; and, 2) infiltration dynamics to the sewer pipes through the soil model. The rain generator proposed by Gernaey et al. (2011) is

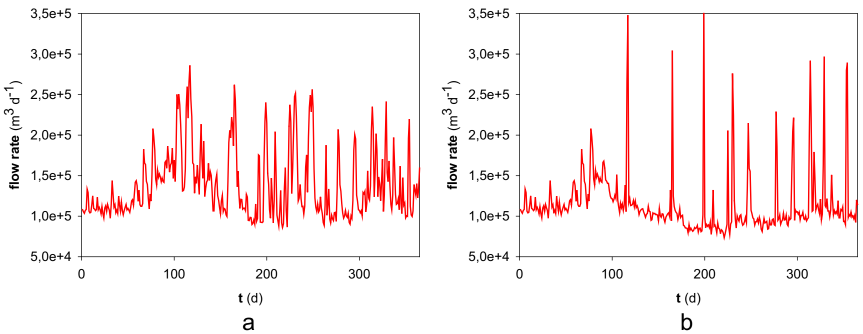

Fig. 5 – Scenario analysis of the 2009 influent flow rate for *WWTP1* assuming two different future rainfall patterns: a) low intensity and high frequency; and, b) high intensity and low frequency.

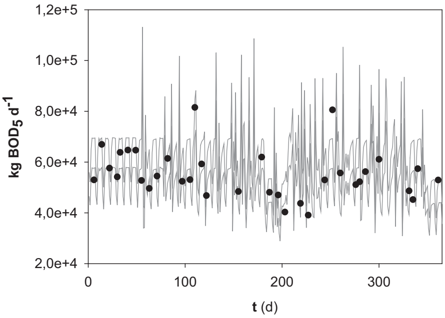

Fig. 6 – Scenario analysis of the 2011 influent organic matter biodegradability for *WWTP2*. The solid lines represent the 5th and 95th percentiles resulting from 250 MC simulations. Dots correspond to the experimental measurements.

used to reproduce different rainfall patterns. Snow melting is defined by the same characteristics as for *WWTP1* in 2009. The influence of the different rainfall patterns is analyzed assuming: 1) a year with a high frequency of events of low intensities; and, 2) a year with a low frequency of events of high intensities. Fig. 5 shows the effect of different rainfall patterns (and related rain water volumes) on the overall flow rate dynamics. For example, severe storm events are depicted in Fig. 5b particularly during summer/fall. On the other hand, Fig. 5a shows more frequent events but always with lower peaks.

Such new influent profiles will have a strong impact on the plant design and operation in wet weather conditions. For example, additional modelling studies can then be performed testing different control strategies to handle violent storm events. The focus of such model studies could then be on when/how to activate by-pass, decrease the waste flow rate to avoid the wash-out of biomass, activate aeration tank settling procedures etc. Another important point that could be studied on the basis of such influent time series is how to operate primary and secondary clarifiers under wet weather conditions. In addition, the effect that these storm events can have on the occurrence of particulate material can be further analyzed by using the first flush model included in the proposed approach. Thus, the quantity of particulate material and the way in which it arrives to the WWTP can be predicted. Again, strategies to smoothen the effects of such load peaks on the overall process performance could be developed, tested and finally implemented at full-scale level.

4.2. *Scenario 2: uncertainty analysis of influent organic biodegradability*

The objective of the second scenario analysis is to show the impact of ASM1 fractionation parameters on the influent organic matter biodegradability (Henze et al., 2000). In order to handle this problem, uncertainty analysis by means of Monte Carlo (MC) simulations was used. 50% of variation is assumed in the default parameters used to calculate S_U (α_{SU}), S_B (α_{SB}), X_U (α_{XU}), XC_B (α_{XCB}) and X_{OHO} (α_{XOHO}) from the calibrated *HH* and *IndS* organic loads (COD_{sol_gperPE}, COD_{part_gperPE}, $COD_{sol\_Ind\_kgperd}$, $COD_{part\_Ind\_kgperd}$). S_U and S_B are calculated from COD_{sol_gperPE} and $COD_{sol\_Ind\_kgperd}$ while X_U, XC_B and X_{OHO} originate from the COD_{part_gperPE} and $COD_{part\_Ind\_kgperd}$. In order to keep the mass balance consistent and avoid negative numbers we assume that $\alpha_{SB} = 1 - \alpha_{SU}$ and $\alpha_{XCB} = (1 - \alpha_{XU} - \alpha_{XOHO})$ (see Corominas et al. (2010) for further specifics about ASM nomenclature). Fig. 6 shows the BOD_5 simulation results after sampling the input uncertainty space 250 times using the Latin Hypercube (LH) method (Helton and Davis, 2003). The solid dynamics time series

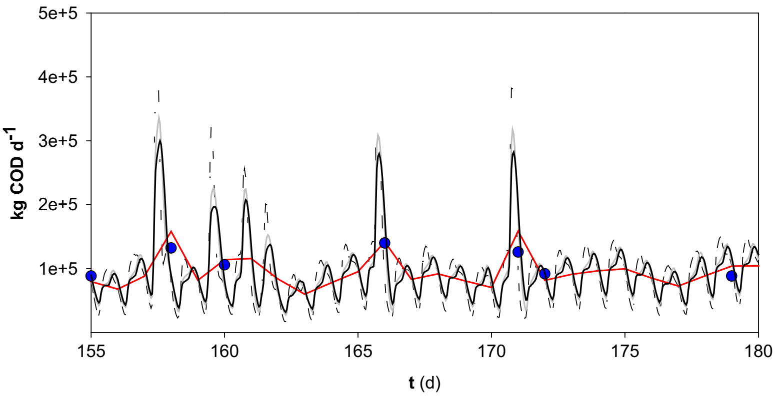

Fig. 7 – Increase of data frequency of the daily calibrated COD data (2010 snapshot for *WWTP2*). Dots (measurements), red solid line (daily simulations) and grey/black/dashed line (hourly simulation assuming different sewer lengths). The longer the sewer length the higher the effect on the hourly profile. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

comprise the 5th and 95th percentiles of the 250 MC simulations while the dots correspond to the experimental data series. The resulting simulations demonstrate that measurements are comprised within the limits of the calculated output uncertainty.

Using this type of analysis has an important impact from a design point of view. For example, when the uncertainty of the influent organic matter biodegradability is accounted for, it is possible to reduce safety factors and consequently the calculated anoxic volumes can be made smaller (Tchobanoglous et al., 2003). As a result, the capital cost of upgrading activated sludge systems to achieve simultaneous removal of organic carbon and nitrogen can be significantly decreased (Belia et al., 2009; Flores-Alsina et al., 2012a).

4.3. *Scenario 3: increased data frequency*

Data frequency is critical in any dynamic modelling exercise. This last scenario will demonstrate how the presented phenomenological influent generator can reduce the cost and workload of measuring campaigns by synthetically increasing the frequency of the available data. Fig. 7 shows the average daily COD values, both the measurements and the simulation results. In addition, this plot is complemented by a "realistic" high frequency dynamic profile generated by the model assuming different sewer lengths (see the smoothing effect on the daily dynamics). For this study, the daily profiles were constructed based on the investigations carried out by Butler et al. (1995). These studies showed that a pollutant peak in C and N occurs in urban catchments in the morning, followed by a smaller one in the evening. This is mainly due to the fact that organic matter primarily originates from kitchen sinks and WCs. Default profiles are being used for the *IndS* block (Gernaey et al., 2011), but since the presence of industrial activity in this catchment is rather low, the effect of the *IndS* block is unnoticeable.

Optionally, zero-mean white noise can be added to the variables by the user. Noise is added by multiplying a random signal to a time series. Specifically in this case study, adding noise is done with the purpose of avoiding that subsequent days have exactly the same flow rate profile.

It is important to highlight that for the development of process configurations for nitrogen and phosphorus removal average values might be sufficient as data source (Flores-Alsina et al., 2012a). However, typical diurnal patterns are required if dynamic processes are to be investigated, such as the design of a treatment system to meet peak effluent nitrogen/phosphorus limits or to optimize aeration control (Rieger et al., 2012).

5. Discussion

This study has demonstrated that dynamic influent pollutant disturbance scenario generators (DIPDSG) are promising tools to improve model-based simulation studies in WWTPs since they can: 1) significantly reduce the cost and workload of measuring campaigns; 2) fill gaps due to missing data in influent flow-rate/pollution/temperature profiles; and, 3) create additional disturbance scenarios following the same catchment principles as a previously calibrated phenomenological influent model. Even though the set of advantages derived from using these tools is extensive, they also open the door to several discussion points, which are analysed below.

5.1. *Model complexity versus prediction capabilities*

The phenomenological model originally developed to generate the influent data for BSM2 (Nopens et al., 2010) and comprehensively presented in Gernaey et al. (2011) represents a good compromise between model complexity and predictive capability. The tool generates realistic influent data to perform simulation studies. Naturally, additional expansions could be included as additional modules by increasing the number of accounted drainage phenomena. For example there are existing models that are more accurate in accounting for pollution run-off (Bertrand-Krajewski et al., 1993; Ashley et al., 2002), combined sewer overflows (Ashley et al., 2004), storm tanks (Schutze et al., 2002) and back-flow effects (Borsányi et al., 2008) in the sewer system. More realistic mathematical representations of flow and pollution transportation through the sewer pipes can be found for example in Hager (1999). Also, additional phenomena such as seasonal differentiation between daily temperature amplitude could be easily included, i.e. the daily amplitude is different during winter/summer. Likely the consideration of such additional aspects could increase the predictive capabilities of both flow rate and pollution loads provided by the presented approach (particularly for high frequency dynamics). Nevertheless, the increased calibration effort to adjust the additional parameter(s) would come with the drawback of making this tool less attractive for process engineers and water/wastewater designers. The guidelines provided in this paper will help future users of the influent generator to easily adapt the generated time series to the desired influent conditions.

5.2. *Description of non-traditional pollutants and consumption rates*

Current research is undertaken towards expanding the influent generator to describe non-traditional pollutants such as pharmaceuticals (Snip et al., 2013). Daily, weekly and seasonal influent dynamics for these compounds are generated following a phenomenological approach based on user-defined pollutant profiles and for more random patterns an additional stochastic module based on Markov chains. Hence, administration patterns, bioavailability, half-life and total annual consumption rates are the basis to generate the user-defined profiles, while the Markov chains are constructed assuming a set of transition probabilities. Additional physico-(bio)chemical reactions in the sewer system, allow considering the effect of transport conditions on these compounds. As a result it is possible to back calculate "consumption rates" or pinpoint areas with the highest contribution (considering the sewer hydraulic retention time) using reverse engineering.

5.3. *Manual versus automatic calibration*

During the development of the DIPDSG user protocol we balanced the pros and cons of manual versus automatic calibration. With respect to automatic calibration the use of traditional frequency-based techniques to estimate the parameters of the model presented herein was very difficult, i.e. the model presents apparent identifiability problems (Weijers and Vanrolleghem, 1996). This poor parameter identifiability was primarily attributed to a couple of factors. First, the model outputs were not sufficiently sensitive to individual changes of each parameter. For example, the effect of sewer length cannot be observed when the analysed time scale is months or days. Another example is related to the parameters determining the memory effect of the soil model, which cannot be observed when the time scale is in the order of months. For this reason, different model parameters must be calibrated at different time scales. Secondly, changes in the model outputs due to changes in certain parameters may be approximately cancelled by appropriate changes in other parameters. In other words some of the parameters can compensate each other's impacts depending on the analysed time scale, e.g. *PE* vs *QperPE*, *PE* vs $Q_{Ind\_weekday}$, K_{down} vs *InfBias*. This could be addressed by estimating only the parameter which can be identified and assuming good knowledge of the non-identifiable parameters (Brun et al., 2002). However, this was not the case here. Another solution would be the use of Bayesian techniques. Bayesian approaches are better choices for model predictions when there are poorly identifiable model parameters (Omlin and Reichert, 1999). Nevertheless, the necessary computational burden to run this study made these options less attractive for the authors. Also, the calibration of the influent model for both *WWTP1* and *WWTP2* and the justification of the available knowledge used to obtain the a posteriori distribution of the model parameters can be considered a whole study on its own.

5.4. *Tool to complement ASM based simulation studies and forecast future scenarios*

Traditional design and upgrade concepts for WWTPs are based on forecasting flow and loads over a period of 25–40 years. Nevertheless, dynamics and complexity of wastewater systems make reliable predictions very difficult, i.e. the characteristics of the catchment area can change substantially over the years. For this reason, it is necessary to improve the planning and design of wastewater treatment infrastructures through methodologies that systematically account for future uncertainty (Dominguez and Gujer, 2006). The use of the presented tool can be very beneficial to answer "what-if" questions. For example, what would happen if the industrial contribution is increased? What is the effect of including a storm-tank on the particulates in the wastewater influent? Another good example would be for planners to add (town expansion) or remove catchment areas (divert a part of town to another plant) to check the future dynamic capacity of a WWTP. The presented DIPDSG is expected to be useful to design engineers, regulators, operators and students/junior engineers.

6. Conclusions

This study presented results of applying the phenomenological DIPDSG that was originally developed to create the BSM2 influent data set to two full-scale applications. The key points of this paper can be summarized as follows:

1) The DIPDSG can reproduce daily, weekly and seasonal variations of the influent flow rate/temperature under dry/wet weather conditions;
2) The DIPDSG can reproduce influent daily and monthly variations of soluble/particulate compounds as well as the effect of particulates' accumulation/wash-out within the sewer system;
3) The DIPDSG has proven to improve future modelling studies, for example by generating additional scenarios, and reducing cost and workload of measuring campaigns by increasing data frequency and finally by running uncertainty analyses.

Acknowledgements

Dr. Flores-Alsina and Mr. Saagi gratefully acknowledge the financial support provided by the People Program (Marie Curie Actions) of the European Union's Seventh Framework Programme FP7/2007-2013 under REA agreement 329349 (PROTEUS) and 289193 (SANITAS) respectively.

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Paper II

Benchmarking integrated control strategies using an extended BSM2 platform

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ABSTRACT

The need for integrated management of urban wastewater systems (UWS) is highly acknowledged in both research and industry. Modelling studies and practical experiences with integrated control of UWS clearly suggest the benefits of a holistic control approach. Given the success of benchmark simulation models (BSMs) in the study of control strategy options for wastewater treatment plants (WWTPs), a spatial extension of the BSM2 model to include sewer and catchment descriptions is presented in this paper. The extension includes detailed model descriptions for wastewater generation (flow rate/pollution) in a hypothetical catchment and its transport through a sewer network. The extension can be linked directly to the existing BSM2 model. Possibilities for the control of loads and flow rate exist in the form of storage tanks with pumps/valves as actuators. As an example, an integrated control scheme of the extended system is presented. The study demonstrates the potential (beneficial) effects that integrated control might have on the UWS by balancing the sewer overflows and/or WWTP overloading.

KEYWORDS

Benchmark simulation models, Integrated control, Modelling, Urban wastewater system.

INTRODUCTION

The International Water Association (IWA) Benchmark Simulation Models (BSM1, BSM1_LT, BSM2) consist of a predefined plant layout, process models, sensor and actuator models, influent wastewater characteristics and evaluation criteria (e.g. **Jeppsson et al., 2013**). They were originally developed with the objective to evaluate control strategies in wastewater treatment plants (WWTPs) and have been widely used in both industry and academia. Up to date, more than 400 publications are related to the BSM family products (**Gernaey et al., 2014**). Over the years, it has become very clear that sewer networks, WWTPs and receiving waters are strongly interconnected, forming the urban wastewater system (UWS). Their (model-based) design and operation should be developed and evaluated in a more holistic manner (**Benedetti et al., 2013**). Integrated control has been studied for quite some years and the main benefits of using such an approach are demonstrated in several scientific contributions (e.g. **Schütze et al., 2002; Vanrolleghem et al., 2005; Langeveld et al., 2013**). For this reason, there is a need to extend the benchmarking platform to include catchment, sewer network and receiving water models. As a result, it will be possible to create a decision support tool that provides water/wastewater engineers with an improved overall picture of the implications of each control strategy to different elements of the UWS.

The objective of this paper is to present a first attempt at the spatial extension of the plant-wide benchmark simulation model BSM2 (**Jeppsson et al., 2007**) to include a catchment and sewer network model. Different components of the model: 1) dry weather pollutant generation; 2) catchment runoff generation; 3) simplified sewer system description; and, 4) storage tanks and combined sewer overflow, are explained in detail. Finally, the model is combined with the plant-wide BSM2 model. In addition, an integrated control strategy based on WWTP influent flow rate control is implemented, simulated and evaluated. Results are used to demonstrate the usefulness/benefits of the proposed platform.

MODEL DESCRIPTION

Catchment characteristics

The ATV A 128 catchment case study (**ATV, 1992**) was scaled up to meet the BSM2 population equivalent (80,000) and total dry weather flow (18,500 m^3/d). Population density and percentage of total area for each sub catchment is considered to be the same as in the ATV A 128 case study catchment. **Table 1** describes the characteristics of the upscaled hypothetical catchment. It includes six sub catchments (SC), each with different areas and population densities. SC1, SC2, SC3, SC4 and SC6 are connected to a combined sewer system whereas SC5 is connected to a separate sewer network. All the defined SCs are considered to be urban/domestic (HH) except SC2, which is modelled as an industrial (IndS) area. The studied system has four storage structures (three online pass-through tanks and an offline bypass tank). The volume of each of these tanks is 25 m^3/ha of the catchment area connecting to the storage tank. One of the online pass-through tanks is connected to the separate sewer system in SC5. The catchment also has two combined sewer overflows (CSOs) without storage (**Figure 1**). The 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 wastewater treatment effluents are discharged at various locations into the receiving waters. A receiving media model will be developed and included in the future based on the model by **Reichert et al. (2001)**. The integration of the catchment, the WWTP and the river models in the same software will make it possible to study control strategies at a system-wide scale.

Modelling the catchment pollutant loads and flow rate

Dry weather pollutant load and flow rate generation

The Dynamic Influent Pollutant Disturbance Scenario Generator (DIPDSG) proposed by **Gernaey et al. (2011)** is used as the starting point to model the pollutant loads and flow rates from the previously defined domestic/industrial sub catchments (see **Figure 1**). The proposed approach can generate the pollutant loads for soluble and particulate COD (kg COD/d), ammonia (kg N/d), nitrate (kg N/d), total Kjeldahl nitrogen (TKN) (kg N/d), total phosphorus (kg P/d) and flow rate (m^3/d). The domestic model blocks (HH) contributes to the final influent flow rate dynamics with diurnal variations, a weekend and a holiday effect. This is achieved by combining three normalized user-defined data files containing: 1) a diurnal profile; 2) a weekly household flow pattern including the weekend effect; and, 3) a yearly pattern including the holiday effect. The relative contributions from the data files are combined by multiplication. The generated outputs are then multiplied by factors corresponding to the flow rate per population equivalent (*QperPE*) (m^3/PE.day), pollution per population equivalent (kg/PE.day) and the number of population equivalents in the catchment area (*PE*). The dynamic flow rate pattern is obtained by repeating the data files in a cyclic manner. The industrial contribution is generated in a similar manner. Again, the dynamic flow rate and pollutant patterns are sampled cyclically from source files and multiplied by a factor representing the use of water (Q_{ind}) (m^3/d) and pollution release (kg/d) from the industry. The main differences between HH and IndS are: i) the dynamics of the generated time series; ii) the ratios between soluble and particulate components and between COD, N, and P. Zero-mean white noise is added to create a different stochastic component in each variable and make the generated time series more realistic. Noise variance is a tuneable parameter that depends on the nature and temporal scale of the generated time series.

Table 1: Catchment characteristics of the extended BSM2 model.

Sub catchment	Area (ha)	PE	Wastewater flow (m^3/d)	
			Domestic (HH)	Industrial (IndS)
1	99	15920	2390	0
2	21	3920	590	2500
3	29	2960	440	0
4	71	9600	1440	0
5	71	7840	1180	0
6	249	39760	5960	0
Total	**540**	**80000**	**12000**	**2500**

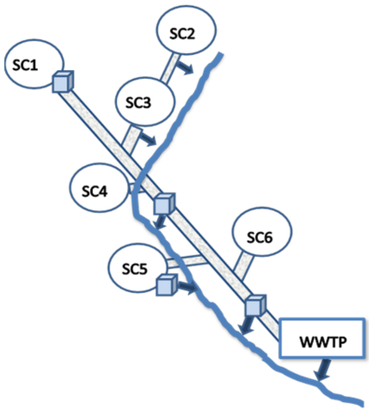

Figure 1: Extended BSM2 catchment and sewer system layout.

Wet weather pollutant load and flow rate generation

Rainfall data (mm/d) is given as an external input time series, e.g. Gernaey et al. (2011). A linear reservoir model is used to mimic the transport of rainwater to the sewer network. This approach is based on the concept of Nash cascades used for hydrological routing models in catchments (Viessman et al., 1989).

Additionally, build up and washoff of particulate material on the catchment surface is also included (Butler and Davies, 2000). **Equation 1** describes the mass balance for pollutants accumulated on the surface (M_s) (kg). The first two parameters of the equation refer to pollution build up. Pollutant accumulation is attributed to various factors like land use, population, seasonal variations, street cleaning, surface conditions etc. All these factors have been lumped into a single representative parameter named the *surface accumulation rate* *constant* (a) (kg/m^2.s). This parameter defines the weight of pollutants accumulated for a given area and time period. A represents the catchment area (m^2). The accumulation model can be further extended to avoid the pollutant concentration from increasing linearly. The equation has a removal rate characterized by the parameter b (decay rate constant (1/s)).

The washoff term is formulated as a first-order equation. The negative sign indicates removal of pollutants from the surface as a result of washoff. The rate of pollutant washoff depends on the *Washoff constant* (k_4) (mm) and on the rain intensity (i) (mm/s). The wastewater flow generated using the catchment runoff model is added to the dry weather flow and connected to the sewer network model.

$$\frac{dM_s}{dt} = aA - bM_s - k_4 iM_s \quad \textbf{Eq. 1}$$

Transport model

Infiltration

In addition to the dry weather flow and rain, infiltration into the sewers also represents a significant contribution to the sewer flow. The seasonal change in the amount of infiltration is attributed to changes in the groundwater level over the year. Seasonal variation depends on average values (*InfBias*), amplitude (*InfAmp*) and frequency (*InfFreq*) (**Gernaey et al., 2011**). The infiltration flow rate obtained is added to the inflow from the sub catchment and serves as the input to sewer system.

Sewer network

A conceptual linear reservoir modelling approach with varying tank volume is used to model the attenuation and delay generally observed in urban sewer systems (**Schütze et al., 2002**). A linear relationship between the sewer flow rate and volume is used (see **Equations 2** and **3**).

$$\frac{\mathrm{d}V}{\mathrm{dt}} = Q_{\mathrm{in}} - Q_{\mathrm{out}} \quad \textbf{Eq. 2}$$

$$V = \frac{1}{K} Q_{out} \quad \textbf{Eq. 3}$$

where V is the volume of the reservoir (m^3), Q_{in} and Q_{out} are the inflow and outflow from the reservoir (m^3/d), respectively, and *1/K* is residence time constant (d). The pollutants are transported along with the flow and have the same residence time in the system. **Equations 4** and **5** show the variation of a pollutant X (kg), assuming that X_{in} and X_{out} are input and output loads (kg/d) of the reservoir model.

$$\frac{\mathrm{d}X}{\mathrm{dt}} = X_{\mathrm{in}} - X_{\mathrm{out}} \quad \textbf{Eq. 4}$$

$$X = \frac{1}{K} X_{\mathrm{out}} \quad \textbf{Eq. 5}$$

Storage structures

For the purpose of control strategy evaluation, storage tanks in the sewer system act as one of the important control elements. These storage elements can be used to limit or increase the flow and pollutant loads to the WWTP and combined sewer overflow (CSO). Different configurations of the storage tanks exist. The extended benchmark system includes models for the four major configurations of the storage elements in an urban drainage system. Online and offline modes of storage are modelled. Within each of these modes, pass-through and bypass configurations are described by the models. Valves are used as the control handles for online tanks and pump models are used in offline storage tanks.

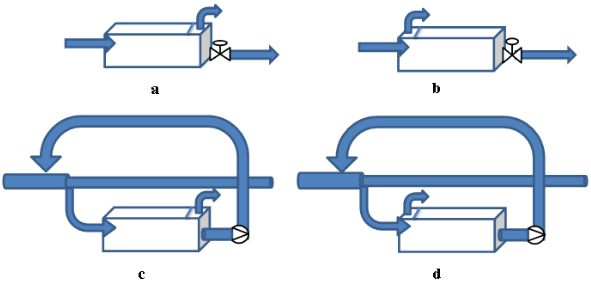

Figure 2: Different configurations of storage tanks: a) online pass-through tank; b) online bypass tank; c) offline pass-through tank; d) offline bypass tank. Pumps and valves are used as flow control elements in offline and online tanks, respectively.

Online tanks are in line with the sewer system and the sewage flows through the storage tank to the downstream sewer and WWTP. In offline mode, any flow beyond the predefined throttle level of the sewer is diverted towards a storage tank. The stored flow is pumped back into the sewer system at a later point in time. If the overflow structure is present after the storage tank, it is termed as pass-through tank and those with an overflow structure before the storage tank are named bypass tanks as the excess flow bypasses the storage tank and reaches the receiving waters (see **Figure 2**).

$$Q_{\text{throttle}} = \frac{Q_{\max}(h - h_{\min})^{n}}{h_{o}^{n} + (h - h_{\min})^{n}} \qquad \textbf{Eq. 6}$$

$$Q_{\text{overflow}} = CL_{\text{weir}}(h - h_{\text{overflow}})^{1.5} \qquad \textbf{Eq. 7}$$

The pollutant mass and flow balance around the tank vary depending on the configuration of the system. The throttle flow (Q_{throttle}) (m^3/d) and overflow (Q_{overflow}) (m^3/d) are described by **Equations 6 (**Vallet, 2011**) and 7 (**Hager, 2010**),** respectively. $Q_{\max}$ is the maximum throttle flow (m^3/d), h_o is the height in the storage tank (m) when $Q=Q_{\max}/2$, $h_{\min}$ is the minimum water level in the tank (m), h is the water level in the tank (m), C is a constant for weir overflow, L_{weir} is the length of the weir (m) and h_{overflow} is the height of the overflow weir measured from the bottom of the tank (m).

For the specific system, SC2 and SC3 only have a CSO without storage. These are modelled as online pass-through tanks. As there is no storage in the actual system, the storage volumes in these tanks are considered to be a part of the sewer volume. An online pass-through tank is present downstream of SC1, SC5 and SC6. An offline bypass tank that can store the excess pollutant loads generated during the first flush is situated downstream of SC4 (see **Figure 1**).

First flush model

A model describing the first flush effects during rain is added before the storage elements. This will allow studies of the influence of particulates on the CSOs and WWTP influent and effluent. Also, it will help studying control strategies that are aimed at handling the high first flush solid loads. The first flush model developed by **Gernaey et al. (2011)** is used in the model. It is assumed that only a fraction of the particulates is settleable (*FFfraction*). During dry weather periods, this fraction gradually settles in the sewer until it reaches a maximum mass of solids (M_{max}). A switching function is used to describe the onset and intensity of the first flush during rain events. The intensity of the first flush can be tuned using the parameter *FF*. The mass balance in the first flush model is defined as follows:

$$\frac{\mathrm{d}M}{\mathrm{dt}} = TSS_{\mathrm{in}}\left(1 - \frac{M}{M_{\mathrm{max}}}\right) - \frac{Q_{\mathrm{in}}^{\mathrm{n}}}{Q_{\mathrm{lim}}^{\mathrm{n}} + Q_{\mathrm{in}}^{\mathrm{n}}}.M.FF \qquad \textbf{Eq. 8}$$

This equation describes the accumulation of the total mass of solids in the sewer as a function of the flux of solids that is entering (M_{in}) and leaving (M_{out}) the system. Q_{in} represents the influent flow rate (m^3/d), TSS_{in} represents the suspended solids concentration that forms the input to the model, M_{max} (kg) is the maximum amount of TSS that can be stored in the sewer system, Q_{lim} (m^3/d) is the flow rate limit triggering the first flush effect, and *FF* (d^{-1}) and *n* (-) are adjustable parameters to tune the desired strength of the first flush effect.

Wastewater treatment plant under study

The WWTP under study (BSM2) has the same layout as the BSM2 described by **Jeppsson et al. (2007).** The activated sludge (AS) unit is a modified Ludzack-Ettinger configuration consisting of 5 tanks in series. Tanks 1 and 2 are anoxic, while tanks 3, 4 and 5 are aerobic. Tanks 5 and 1 are linked by means of an internal recycle. The BSM2 plant further contains a primary and a secondary clarifier, a sludge thickener, an anaerobic digester, a storage tank and a dewatering unit. Further information about benchmark simulation models can be found in **Gernaey et al. (2014)**.

Description of the integrated control strategy

The extended BSM2 model is evaluated by an integrated control strategy for equalization of the flow rate to the WWTP inlet. A feedback controller based on the information from a sensor located at the outlet of the storage tank situated downstream of SC6 determines the valve opening for the pass-through storage tank. The set point for the controller is determined offline and varies depending on the time of day, week and year. A PI (proportional-integral) controller is used to regulate the valve opening in order to achieve the desired throttle level.

RESULTS

Generation of the influent flow rate

Figure 3 shows the simulated flow rate variation of the WWTP influent using the extended BSM2 proposed in this paper. The population equivalent (*PE*) and area (*A*) for each catchment are defined in **Table 1**. The average wastewater flow generation per capita (*QperPE*) is assumed to be 0.15 m^3/PE.day. The industrial flow (Q_{ind}) from SC2 is 2500 m^3/d. In dry weather conditions 75% of the influent flow rate is assumed to originate from

HH and *IndS*. Strong differences in the daily and weekly influent dynamics can be appreciated between *HH* and *IndS* (see **Figures 3a** and 3**b**). The remaining 25% of the influent flow rate (dry weather conditions) originates from groundwater infiltration to the sewers. Thus, infiltration model parameters (*InfBias*, *InfFreq and InfAmp*) are modified accordingly. The stochastic model suggested by **Gernaey et al. (2011)** provides rainfall data, which is added to previously generated time series in order to create wet weather conditions (**Figure 3c**). **Table 2** summarizes the simulation results of the BSM2 influent generator and the model presented herein.

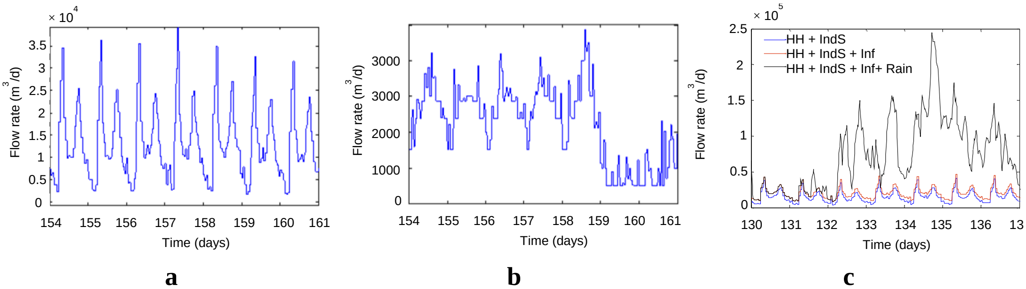

Figure 3: Dynamic profiles for household (a) and industry (b) flow rate generation. Different components of the influent flow rate to the WWTP (c).

Generation of the influent loads

In dry weather conditions, average domestic (*HH*) and industrial daily (*IndS*) pollution loads (soluble COD, particulate COD, ammonia, nitrate, total Kjeldahl nitrogen and total phosphorus), for the six (SC1-SC6) different sub catchments (in kg/PE.day) are given as inputs to the model. **Table 2** shows the yearly average concentrations at the WWTP inlet of the model presented in this paper and those of the well-established BSM2 influent generator for comparison. It is important to highlight the effect that wet weather conditions have on the dynamics of the particulates. **Figure 4** shows the first flush effect on the TSS profile. During dry weather conditions, TSS can accumulate in the sewer system as long as the total amount of sediments stored in the sewer system is below $M_{\max}$ (**Eq. 8**). During rain events, the function in the second part of the equation will induce the switching. Hence, the sudden increase of flow rate should result in a washout of sediments from the sewer system (first flush).

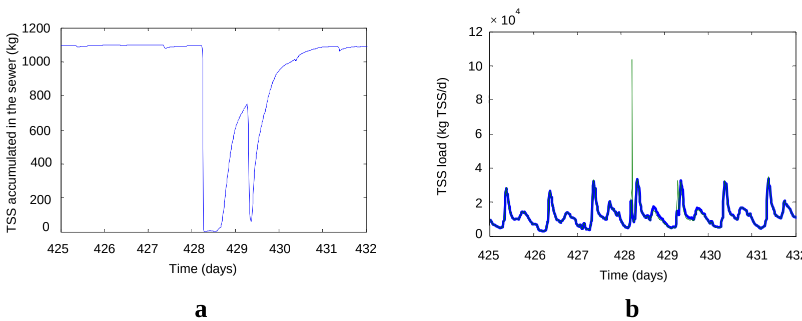

Figure 4: Accumulation and washoff of TSS in the sewer system (a), effect of first flush on sewer inflow (blue) and outflow (green) TSS load (b).

Table 2: Comparison of yearly average influent flow rate and pollutant loads between the extended BSM2 presented in this study and the original BSM2 WWTP influent generator

Parameter	BSM2	This study
Influent flow rate (m^3/d)	20 669	24 147
COD (kg/d)	592	573
BOD (kg/d)	305	293
TSS (kg/d)	380	368
TN (kg/d)	55	52

Effect of the proposed control strategy

This section shows the simulation results of a control strategy that was aimed at equalizing the flow rate to the WWTP inlet using storage tank 6. The possibility to have a constant flow rate throughout the year does not exist due to limited storage volume. Different normalized levels for the throttle flow based on the time of day and week are defined. As infiltration is one of the major contributions to variation in daily dry weather flow, a normalized sine curve with the same frequency as the infiltration model (*InfFreq*) is used. This defines the seasonal variation in throttle flow. The final set point for the throttle is achieved by multiplication of the mean throttle flow (Q_{throttle}) with the normalized values for daily, weekly and seasonal variations. This value is dynamically fed to the PI controller and is compared with the measured flow rate at the storage tank outlet to define the valve opening. **Figure 5** depicts the results when Q_{throttle} is set to 22 800 m^3/d. This strategy was developed to avoid WWTP overloading at the expense of creating CSO events. The throttle flow is always maintained within the operational limits of the WWTP thereby generating sewer overflows during the intermittent rain events. Identification of the optimum throttle flow is critical to achieve flow equalization without causing significant sewer overflows and/or WWTP overloading. A balance between sending the excess flow to CSOs and overloading the treatment plant has to be evaluated. An integrated platform like this provides the ideal tool to perform such studies.

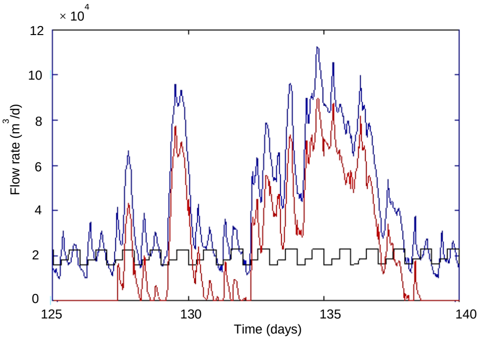

Figure 5: Storage tank 6 inflow (blue), throttle flow (black) and combined sewer overflow (red) rates during a rainy period. The control strategy is aimed at reducing the overload to WWTP during rain events.

CONCLUSIONS

An extension to the existing plant-wide BSM2 model is developed to account for a catchment and sewer system description. The study explains in detail the salient features and the modelling methodology that was necessary to include this extension. In addition, the proposed tool will provide the research/industry community with a platform for development and evaluation of integrated control strategies (not possible within the original BSM2). The authors illustrated the possibility to evaluate integrated/system-wide control strategies using a simplified case study. As the next step forward, the effect of the above mentioned control option on the WWTP and receiving water will be evaluated to show the potential/benefits/usefulness of urban wastewater system-wide control.

ACKNOWLEDGEMENTS

Dr. Flores-Alsina and Mr. Saagi gratefully acknowledge the financial support provided by the People Program (Marie Curie Actions) of the European Union's Seventh Framework Programme FP7/2007-2013 under REA agreement 329349 (PROTEUS) and 289193 (SANITAS), respectively.

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Paper III

Environmental Modelling & Software 78 (2016) 16–30

Contents lists available at ScienceDirect

Environmental Modelling & Software

journal homepage: www.elsevier.com/locate/envsoft

Catchment & sewer network simulation model to benchmark control strategies within urban wastewater systems

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ARTICLE INFO

Article history:
Received 1 September 2015
Received in revised form
7 December 2015
Accepted 11 December 2015
Available online xxx

Keywords:
Benchmark simulation models
Combined sewer overflows
Integrated control
System-wide evaluation
Urban drainage models

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.

Software availability

Name of the software:

BSMsewer.

Developers:

R. Saagi, X. Flores-Alsina, G. Fu, L., D. Butler, K.V. Gernaey, U. Jeppsson.

Programming language:

Matlab 13.0.

Software availability: The source code for the catchment & sewer model can be obtained for free. Contact Dr Ulf Jeppsson. Division of Industrial Electrical Engineering and Automation (IEA), Lund University, Box 118, SE-221 00 Lund, Sweden.

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

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http://dx.doi.org/10.1016/j.envsoft.2015.12.013
1364-8152/

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 (m^3/d) (soil block)
$Q_{lim,ff}$	flow rate limit triggering the first flush effect (m^3/d) (first flush block)
$Q_{max,st}$	maximum throttle flow (m^3/d) (storage block)
$Q_{out,r}$	outflow from the reservoir (m^3/d) (transport block)
$Q_{out,st}$	outflow from the storage tank (m^3/d) (storage block)
$Q_{ovf,st}$	overflow from the storage tank (m^3/d) (storage block)
$Q_{pump,st}$	pumping rate at the storage tank (m^3/d) (storage block)
$Q_{throttle,st}$	throttle flow from the storage tank (m^3/d) (storage block)
RDI_{in}	rainfall dependent inflow from pervious areas (m^3/d) (soil block)
rrc	rainfall runoff coefficient
$T_{exc(c)}$	yearly exceedance duration for pollutant c (d)
T_{ovf}	yearly overflow duration (d)
V_{ovf}	yearly overflow volume (m^3)
V_r	volume of reservoir (m^3) (transport block)
V_{soil}	storage volume of the tank (m^3) (soil block)
V_{st}	volume of the storage tank (m^3) (storage block)
w	washoff constant (mm^{-1}) (runoff block)
$X_{i,st}$	inflow load for pollutant “i” in the storage tank (kg/d) (storage block)
$X_{in,ff}$	particulate flux entering the first flush block (kg/d) (first flush block)
$X_{in,r}$	input load to the reservoir (kg/d) (transport block)
$X_{out,ff}$	particulate flux leaving the first flush block (kg/d) (first flush block)
$X_{out,r}$	output load from the reservoir (kg/d) (transport block)
$X_{ovf(c)}$	yearly overflow pollutant load for pollutant c (kg)
α	shape parameter for Gamma distribution (rainfall block)
β	scale parameter for Gamma distribution (rainfall block)

or during model use (e.g. Astaraie-Imani et al., 2012; Benedetti et al., 2008, 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. Harremoës 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 analysing 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.

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. Fig. 1 illustrates the catchment configuration and its main characteristics. The total catchment area (A_c) is 540 ha and comprises 80,000 population equivalents (PE_c). Dry weather flow is scaled up to be similar to the BSM2 influent characteristics (18,500 m^3/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 m^3/d; 2) industrial contribution with a DAF of 2,500 m^3/d; and, 3) infiltration to sewers which corresponds to 25% of the dry weather flow.

The system under study is comprised of six sub-catchments (SC_1,…, SC_6) with different areas (A_1,…, A_6) and population densities (PE_1,…, PE_6) (see Table 1). All the defined SCs are considered to be domestic except SC_2, which has both domestic and industrial contributions. SC_1, SC_2, SC_3, SC_4 and SC_6 are connected to a combined sewer system whereas SC_5 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 Fig. 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 Fig. 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 (Fig. 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 (Fig. 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 (Fig. 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).

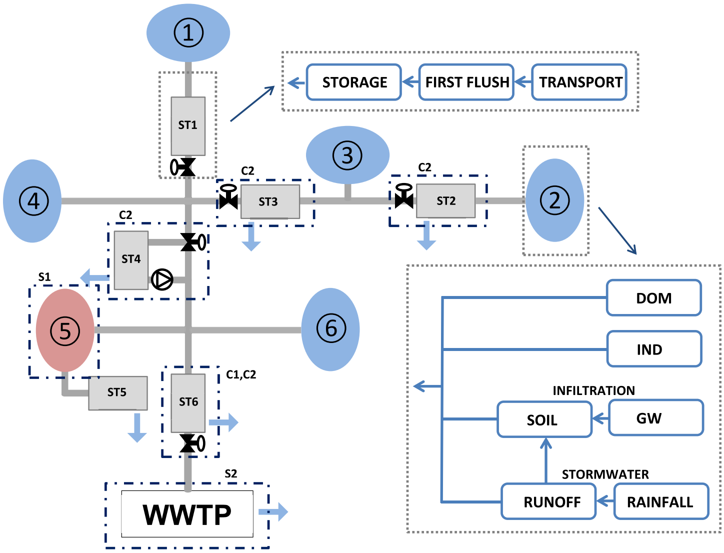

Fig. 1. Catchment and sewer BSM layout indicating various sub-catchments (①, ②, ③, ④, ⑤, ⑥ represent SC_1, SC_2, SC_3, SC_4, SC_5 and SC_6 respectively), storage tanks (ST_1, ST_2, ST_3, ST_4, ST_5, ST_6) and control elements. Overflows are assumed to enter a receiving water (not modelled here). A snapshot 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.

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

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

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.

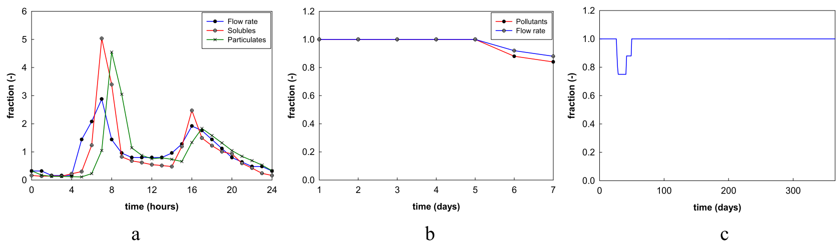

Fig. 2. Diurnal variation in pollutant loads and flow rate (a). Weekly variation with two different profiles (red = pollutants, blue = flow rate) (b) and yearly profile (starting first week of July) with similar dynamics for pollutants and flow rate (c). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

- *Weekly profile*: As can be seen in Fig. 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 Fig. 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 (Fig. 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.

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 $P_{\mathrm{d|w}}$ represents the probability for the next period to be wet given that the current period is dry and vice-versa for $P_{\mathrm{w|d}}$. The other probability values can also be interpreted in a similar fashion. Each period lasts for 15 min. 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).

$$P = \begin{bmatrix} P_{\mathrm{d|d}} & P_{\mathrm{d|w}} \\ P_{\mathrm{w|d}} & P_{\mathrm{w|w}} \end{bmatrix} \tag{1}$$

Finally, a gamma distribution (Equations (2) and (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.

$$f(x) = \frac{\left(\frac{x}{\beta}\right)^{\alpha-1} e^{-\frac{x}{\beta}}}{\beta\Gamma(\alpha)} \tag{2}$$

$$\Gamma(\alpha) = \int_0^\infty e^{-t} t^{\alpha-1} \mathrm{d}t \tag{3}$$

Fig. 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.

3.3.2. Runoff contribution block (runoff)

The *runoff* block is used to convert the rainfall intensities (mm/

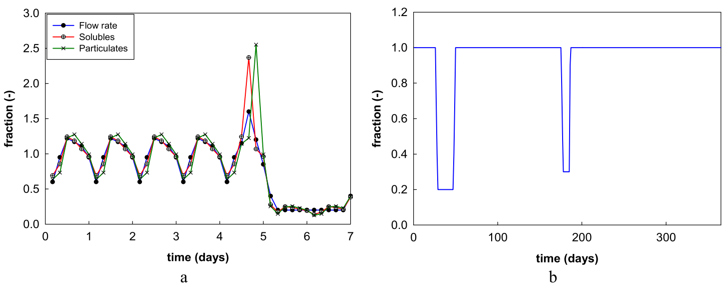

Fig. 3. Dynamics of industrial dry weather pollutant and flow rate generation with weekly (a) and yearly (b) variations. The yearly profile begins in the first week of July. (For simplicity, we assume that the first day of july is a Monday).

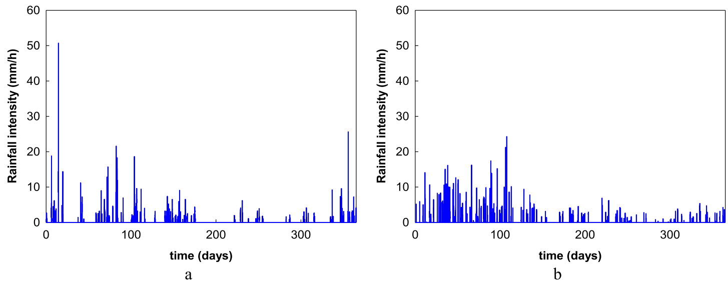

Fig. 4. Rainfall intensity time series from data (a) and the one generated using the model (b) for a period of 1 year. The time series begins in the first week of July.

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*) (Fig. 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 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) (Fig. 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 Fig. 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.

$$\frac{dM_{s,i}}{dt} = a_i A_i - b_i M_{s,i} - 24 w_i i_{rain} M_{s,i} \tag{4}$$

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

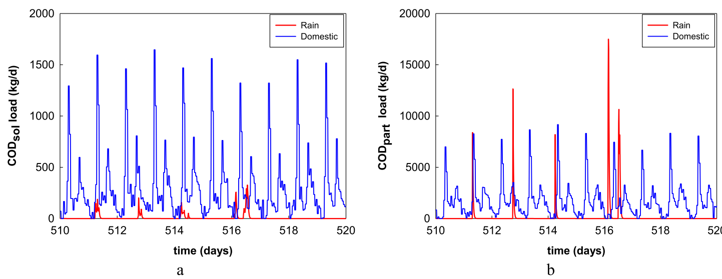

Fig. 5. Effect of EMC during rainfall on soluble pollutant (COD_{sol}) (a), and effect of the accumulation and washoff model on COD_{part} (b).

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 (Fig. 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.

$$\frac{dV_{soil,i}}{dt} = GW_{in,i} + RDI_{in,i} - Q_{inf,i} - Q_{GW,i} \quad (5)$$

$$\frac{A_{soil,i} dh_{soil,i}}{dt} = GW_{in,i} + RDI_{in,i} - K_{inf,i}\sqrt{h_{soil,i} - h_{inv,i}} - K_{down,i} h_{soil,i} \quad (6)$$

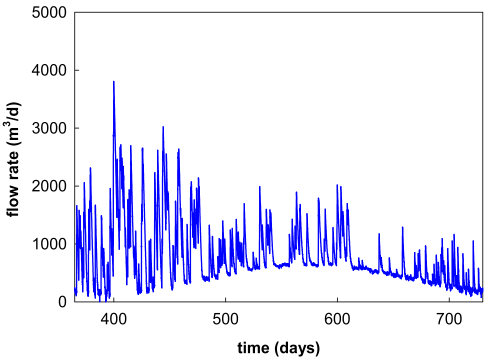

Fig. 6. Infiltration to sewers from SC_1 depicting the annual variations and also rainfall dependent variations.

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).

$$\frac{dV_r}{dt} = Q_{in,r} - Q_{out,r}; \quad Q_{out,r} = \frac{1}{K_r} V_r \quad (7)$$

$$\frac{dM_r}{dt} = X_{in,r} - X_{out,r}; \quad X_{out,r} = \frac{1}{K_r} M_r \quad (8)$$

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). Fig. 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 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).

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.

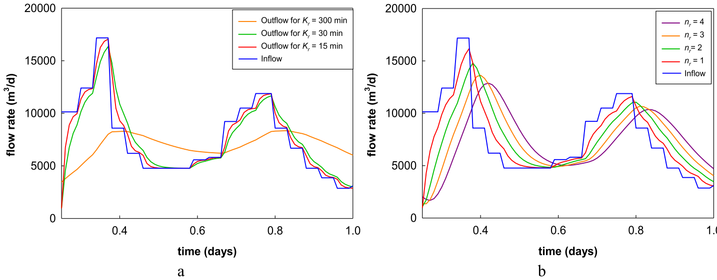

Fig. 7. Reservoir model used for the sewer network. Effect of different residence time constants (K_r) for a given inflow (a). Variations in the outflow based on the number of such reservoirs (n_r) in series (b).

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

Fig. 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 Fig. 8. Due to 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.

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 m^3/ha of catchment area. In Europe, storage volumes range from 30 m^3/ha to 200 m^3/ha (Schütze et al., 2002). There are four different configurations of the tanks which are mainly classified into on-line and off-line modes (Fig. 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.

Only two of the four available configurations are used in the current layout (Fig. 1). ST_1, ST_2, ST_3 and ST_6 are on-line pass-through tanks while ST_4 is an off-line bypass tank.

Table 2 summarizes the mass balance and equations used for the previously described storage tanks. V_{st} is the volume of the tank

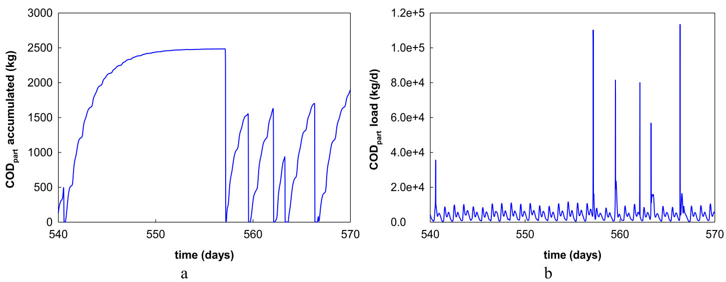

Fig. 8. Accumulation of COD_{part} in the sewer system (a), and the sewer particulate load (blue) (b). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

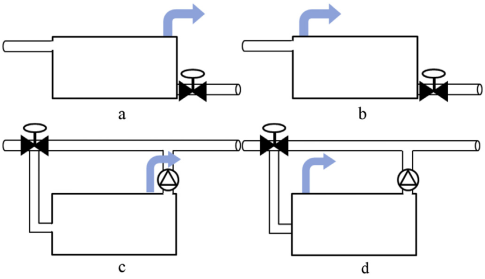

Fig. 9. Different configurations of storage tanks: a) on-line pass-through tank; b) on-line bypass tank; c) off-line pass-through tank; d) off-line bypass tank. Pumps and valves are used as flow control elements in off-line and on-line tanks, respectively.

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 (m^3/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).

Fig. 10 presents the behaviour of an *on-line* (ST_6) (a) and an *off-line* (ST_4) (b) storage tank model. In the case of *on-line tanks* (Fig. 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. Fig. 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.

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 (10)). 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:

$$T_{ovf,i} = \frac{1}{y}\sum_{j=1}^{n} t(j) \tag{10}$$

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 (11)). 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:

$$V_{ovf,i} = \frac{1}{y}\sum_{j=1}^{n}\int_{t_0(j)}^{t_e(j)} Q(t)\mathrm{d}t \tag{11}$$

4. *Yearly overflow pollutant load* ($X_{ovf(c),i}$) (kg/year): This represents the total load in the overflow for a given pollutant X_c(t) at any given overflow location (see Equation (12)). 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 $t_e(n)$).

$$X_{ovf(c),i} = \frac{1}{y}\sum_{j=1}^{n}\int_{t_0(j)}^{t_e(j)} X_c(t)\mathrm{d}t \tag{12}$$

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 (13)). 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

Table 2
Summary of modelling details for various storage tank models used in the system-wide BSM (Note that X stands for pollutant load).

	On-line		Off-line	
	Pass-through	Bypass	Pass-through	Bypass
V_{st}	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$	$\frac{dV_{st}}{dt} = \frac{1}{A_{st}}(Q_{in,st} - Q_{out,st} - Q_{ovf,st})$
$M_{c,st}$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}(Q_{out,st} + Q_{ovf,st})$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}Q_{out,st} - X_{c,in}\frac{Q_{ovf,st}}{Q_{in,st}}$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}(Q_{out,st} + Q_{ovf,st})$	$\frac{dM_{c,st}}{dt} = X_{c,in,st} - \frac{M_{c,st}}{V_{st}}Q_{out,st} - X_{c,in}\frac{Q_{ovf,st}}{Q_{in,st}}$
$Q_{out,st}$	$\frac{Q_{max,st}(h_{st}-h_{min,st})^{n_{st}}}{h_{o,st}^{n_{st}} + (h_{st}-h_{min,st})^{n_{st}}}$	$\frac{Q_{max,st}(h_{st}-h_{min,st})^{n_{st}}}{h_{o,st}^{n_{st}} + (h_{st}-h_{min,st})^{n_{st}}}$	$Q_{pump,st}$	$Q_{pump,st}$
$X_{c,out,st}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$	$M_{c,st}\frac{Q_{out,st}}{V_{st}}$
$Q_{ovf,st}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$	$C_{st}L_{weir,st}(h_{st} - h_{ovf,st})^{1.5}$
$X_{c,ovf,st}$	$M_{c,st}\frac{Q_{ovf,st}}{V_{st}}$	$X_{c,in,st}\frac{Q_{ovf,st}}{Q_{in,st}}$	$M_{c,st}\frac{Q_{ovf,st}}{V_{st}}$	$X_{c,in,st}\frac{Q_{ovf,st}}{Q_{in,st}}$

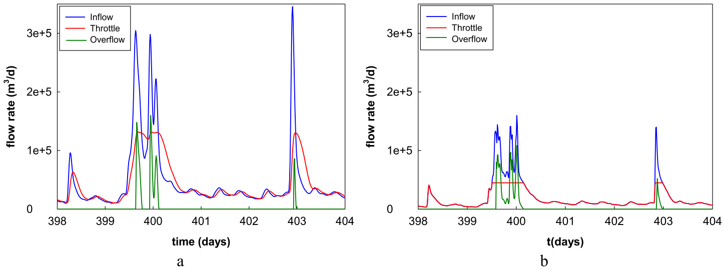

Fig. 10. Effect of different configurations of the storage tanks on throttle (to the sewer) and overflow: 1) On-line (a); and, 2) Off-line (b).

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.

$$OQI_i = \frac{1}{t}\int_0^t \Big[w_{BOD}X_{ovf(BOD),i}(t) + w_{COD}X_{ovf(COD),i}(t) + w_{TSS}X_{ovf(TSS),i}(t) + w_{TKN}X_{ovf(TKN),i}(t) + w_{NO_3}X_{ovf(NO_3),i}(t) + w_{PO_4}X_{ovf,(PO_4),i}(t)\Big]dt \tag{13}$$

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 (14) and (15). 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.

$$t_{exc,i}(c,n) = \sum\{(t+1)-t\} \quad when \;\; C(t) > C_{th} \tag{14}$$

$$T_{exc(c),i} = \frac{1}{y}\sum_{j=1}^{n} t_{exc,i}(c,j) \tag{15}$$

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-h maximum exceedance is used. It is the highest concentration that is continuously discharged for a period of at least 1 h. Similarly, maximum concentrations for 2-h, 6-h 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 (Schütze 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 analysed:

- 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).

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

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.

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 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 analysed 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 ($T_{exc(TKN)}$, $C_{max(TKN)}$). 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 Fig. 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.

The implementation of C2 has led to mixed results (Fig. 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%) ($V_{ovf,bp}$) 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 ($C_{max,TKN}$, $T_{exc,TKN}$). 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.

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

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 behaviour. It is assumed that the volume of the storage tank remains unchanged.

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

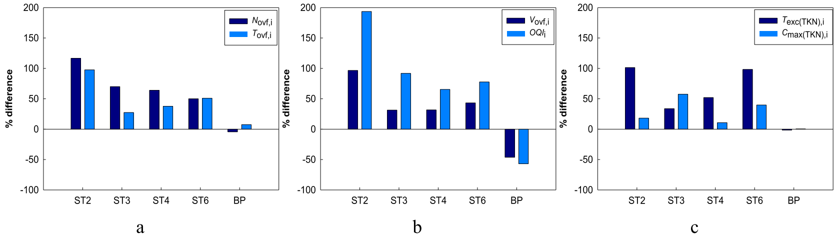

Fig. 11. Evaluation of various performance criteria comparing the default case (NC) with the global control strategy (C2). The percentage difference in performance between NC and C2 is shown, evaluated for the criteria: a) $N_{ovf,i}$ and $T_{ovf,i}$; b) $V_{ovf,i}$ and OQI_i; and, c) $T_{exc(TKN),i}$ and $C_{max(TKN),i}$ in various storage tanks (ST_2, ST_3, ST_4, ST_6) and the bypass (BP).

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

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 ST_5 is exceeded (2 events/year in this case). Although, there is a drop in *yearly overflow volume* (V_{ovf}) (13%), the discharges at ST_5 and downstream are now more polluted due to mixing with the domestic wastewater from SC_5. A significant drop in *OQI* is observed at ST_5, but this does not lead to overall improvement in *OQI*. This is due to the fact that the discharges are now happening elsewhere (at ST_6 and the 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 analysed 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 8000 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 SC_5 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

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

total overflow volumes.

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 Schütze 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 analysed.

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.

Acknowledgements

Dr. Flores-Alsina and Mr. Saagi gratefully acknowledge the financial support provided by the People Program (Marie Curie Actions) of the European Union's Seventh Framework Programme FP7/2007–2013 under REA agreement 329349 (PROTEUS) and 289193 (SANITAS) respectively. The authors gratefully acknowledge the feedback provided by Professor Peter A. Vanrolleghem, Canada Research Chair in Water Quality Modelling, Université Laval, Quebec, Canada during the development of the model.

Appendix A. Supplementary data

Supplementary data related to this article can be found at http://dx.doi.org/10.1016/j.envsoft.2015.12.013.

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Supplementary Information

Paper title: Catchment & sewer network simulation model to benchmark control strategies within urban wastewater systems

Authors: Saagi, R., Flores-Alsina, X., Fu, G., Butler, D., Gernaey, K.V. and Jeppsson, U.

Table 1.1: Main parameters for the catchment & sewer system extensions.

Model section	Parameter	Value	Units	Remarks
Domestic	*QperPE*	0.15	m^3/PE.d	Domestic wastewater flow rate per population equivalent
	PE_c	80,000	PE	Population equivalents
	domestic_avg	19.31, 115.08, 5.85, 0, 1.5	g poll/PE.d	Average daily pollutant loads per PE for COD_{sol}, COD_{part}, NH_4, NO_3, PO_4
Industrial	Q_{ind}	2,500	m^3/d	Daily average wastewater flow rate from industry on normal week days (Monday to Thursday)
	industry_avg	386.24, 2301.8, 52.06, 0, 0	kg poll/d	Average daily pollutant loads for COD_{sol}, COD_{part}, NH_4, NO_3, PO_4
Rainfall runoff	*rrc*	0.7		Rainfall runoff coefficient to account for the continuing losses
	imp_frac	0.75		Impervious area as a fraction of total area
Rainfall generator (values for January provided here)	*P*	[0.995,0.005; 0.134,0.866]		Markov transition matrix for dry/wet periods
	α	0.88		Gamma distribution parameter
	β	3.08		Gamma distribution parameter
Pollutant accumulation and washoff	*a*	5	kg/m^2.s	Surface accumulation rate
	b	0.2	1/s	Decay rate constant for the pollutant accumulation model
	w	0.3	1/mm	Washoff constant
	EMC_{codsol}	9	g/m^3	EMC for COD_{sol}
	EMC_{NH4}	0.56	g/m^3	EMC for NH_4
Groundwater (SC_1 values provided here)	*gwbias*	7100	m^3/d	Mean yearly infiltration. Values for each SC are a fraction of *gwbias*.
	amp	25	%	Amplitude of the sine wave
	freq	$2\pi/365$	rad/d	Frequency of the sine wave (1 year)
	phase	$15\pi/24$	rad	Phase shift
Soil (SC_1 values provided here)	h_{max}	2.8	m	Maximum level of the tank
	h_{inv}	0.8	m	Invert level of the tank
	A_{soil}	24.75×10^4	m^2	Area of the tank
	K_{soil}	0.4	m/d	Soil permeability
	K_{inf}	2.98×10^4		Gain for infiltration to sewer
	K_{down}	298.2		Gain for infiltration to groundwater
Sewer	n_r			Number of sewer reservoirs in series. This is done as a model modification and not as a parameter change.
	K_r	0.0104	d	Time constant for each sewer tank
First-flush (SC_1 values provided here)	*FFfraction*	0.25		Fraction of TSS that can settle in the sewer system
	$Q_{lim,ff}$	19104	m^3/d	Limit flow rate triggering a first flush effect
	n_{ff}	15		Exponent for Hill function
	$M_{max,ff}$	990	kg	Maximum sediment mass stored in sewer system
	FF	5000	1/d	Gain for first flush effect
Storage tanks (ST_2 values provided here)	A_{st}	294	m^2	Area of storage tank.
	C_{st}	1.53×10^5		Constant for weir overflow
	$Q_{max,st}$	7.25×10^3	m^3/d	Maximum throttle flow
	$h_{o,st}$	2.5	m	Height at which $Q=Q_{max}/2$
	$h_{ovf,st}$	5	m	Height above which overflow occurs
	$L_{weir,st}$	3	m	Length of overflow weir
	$Q_{throttle,st}$	4.56×10^4	m^3/d	Throttle flow (offline tanks only) (ST_4)
	$Q_{pump,st}$	1000	m^3/d	Pumping flow rate (offline tanks only) (ST_4)

Paper IV

Environmental Modelling & Software 93 (2017) 282–295

Contents lists available at ScienceDirect

Environmental Modelling & Software

journal homepage: www.elsevier.com/locate/envsoft

A model library for simulation and benchmarking of integrated urban wastewater systems

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A R T I C L E   I N F O

Article history:
Received 21 September 2016
Accepted 28 March 2017

Keywords:
Benchmark simulation models
Urban wastewater system modelling
Integrated control strategies

A B S T R A C T

This paper presents a freely distributed, open-source toolbox to predict the behaviour of urban wastewater systems (UWS). The proposed library is used to develop a system-wide Benchmark Simulation Model (BSM-UWS) for evaluating (local/global) control strategies in urban wastewater systems (UWS). The set of models describe the dynamics of flow rates and major pollutants (COD, TSS, N and P) within the catchment (CT), sewer network (SN), wastewater treatment plant (WWTP) and river water system (RW) for a hypothetical, though realistic, UWS. Evaluation criteria are developed to allow for direct assessment of the river water quality instead of the traditional emission based metrics (for sewer overflows and WWTP discharge). Three case studies are included to illustrate the applicability of the proposed toolbox and also demonstrate the potential benefits of implementing integrated control in the BSM-UWS platform. Simulation results show that the integrated control strategy developed to maximize the utilization of the WWTP's capacity represents a balanced choice in comparison to other options. It also improves the river water quality criteria for unionized ammonia and dissolved oxygen by 62% and 6%, respectively.

Software availability

Name of the software:
BSM-UWS.
Developers:
R. Saagi, X. Flores-Alsina, S. Kroll, K.V. Gernaey, U. Jeppsson.
Programming language:
Matlab 13.0.
Software availability: The source code for the system-wide BSM can be obtained for free. Contact Dr Ulf Jeppsson, division of Industrial Electrical Engineering and Automation (IEA), Lund University, Box 118, SE-221 00 Lund, Sweden (ulf.jeppsson@iea.lth.se). The software is documented and interested readers will be able to reproduce the results summarized in this article, and then modify the software for their own purposes as well.

1. Introduction

The main objective of integrated modelling is to link various sections of the urban wastewater system (UWS) (catchment (CT), sewer network (SN), wastewater treatment plant (WWTP) and receiving water system (RW)) together to provide a unified platform for design and analysis of wastewater infrastructures in urban areas (Benedetti et al., 2013). Such a tool enables direct evaluation of UWS dynamic performance (or of individual sections) based on river water quality instead of relying on traditional emission based evaluation. Significant progress has been made in the field of integrated modelling ever since it was first proposed by Beck (1976) (e.g. Fronteau et al., 1997; Rauch et al., 2002; Muschalla et al., 2009; Benedetti et al., 2013; Bach et al., 2014). It is now well established that optimization of sub-system performance (SN or WWTP) does not necessarily lead to improvements in river quality (Rauch and Harremoës, 1999) and a more holistic approach is required (Lijklema et al., 1993). Although research has highlighted the need of integrated modelling for a long time, a strong incentive for receiving water quality based evaluation of UWS performance has been provided by the EU Water Framework Directive, which calls

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http://dx.doi.org/10.1016/j.envsoft.2017.03.026
1364-8152/

Nomenclature

$(P)_{eff,WWTP}$	Pollutant (P) total load at WWTP effluent (kg) (P = BOD_5, COD, TSS, TKN, P_{org} and P_{inorg})
$(P)_{EMC}$	Pollutant (P) EMC (g/m^3)
$(P)_{in,WWTP}$	Pollutant (P) total load at WWTP inlet (kg)
$(P)_{ovf}$	Pollutant (P) total load in overflow (kg)
AER1	Aerobic reactor 1
AER2	Aerobic reactor 2
AER3	Aerobic reactor 3
ANAER1	Anaerobic reactor 1
ANAER2	Anaerobic reactor 2
ANOX1	Anoxic reactor 1
ANOX2	Anoxic reactor 2
BOD_5	5-day biological oxygen demand
BP_1	Bypass 1 (before primary clarifier)
BP_2	Bypass 2 (after primary clarifier)
$C_{max,NH3}$	Hourly maximum concentration for unionized ammonia (g N/m^3)
$C_{min,DO}$	Hourly minimum concentration for dissolved oxygen (g/m^3)
COD	Chemical oxygen demand
COD_{part}	Particulate COD
COD_{sol}	Soluble COD
DO	Dissolved oxygen
EMC	Event mean concentration (g/m^3)
EQI	Effluent quality index (kg pollution units/d)
h_{STi}	Height of storage tank i
IQI	Influent quality index (kg pollution units/d)
$K_La_{AER(i)}$	Oxygen transfer coefficient for aerobic reactor i (d^{-1})
MLSS	Mixed liquor suspended solids
NH_3	Unionized ammonia
NH_4^+	Ammonia
NO_3^-	Nitrate
N_{ovf}	Yearly overflow frequency (events/year)
$OVF_{(i)}$	Overflow at location no. i
OQI	Overflow quality index (kg pollution units/d)
PC	Primary clarifier
P_{inorg}	Inorganic phosphorus
PO_4^3	Phosphate
P_{org}	Organic phosphorus
Q_{intr}	Internal recirculation rate (m^3/d)
$Q_{in,WWTP}$	Inflow to WWTP (m^3/d)
$Q_{max,BP1}$	Maximum flow at bypass 1 (m^3/d)
$Q_{max,BP2}$	Maximum flow at bypass 2 (m^3/d)
$Q_{max,ST2}$	Maximum throttle flow for ST_2 (m^3/d)
$Q_{max,ST5}$	Maximum throttle flow for ST_5 (m^3/d)
$Q_{max,ST6}$	Maximum throttle flow for ST_6 (m^3/d)
$Q_{pump,ST1}$	Pumping rate at ST_1 (m^3/d)
$Q_{pump,ST4}$	Pumping rate at ST_4 (m^3/d)
Q_r	Sludge recycle rate (m^3/d)
$Q_{throttle,ST4}$	Maximum throttle flow for off-line tank ST_4 (m^3/d)
Q_w	Sludge wastage rate (m^3/d)
RST	Rainwater storage tank
$RW_{(i)}$	River water stretch i
$SC_{(i)}$	Sub-catchment i
Sec.C	Secondary clarifier
$S_{NH4,RW16}$	Sensor measurement for ammonia (NH_4^+) at river stretch 16 (g N/m^3)
$S_{O2,AER2}$	Sensor measurement for oxygen concentration at aerobic reactor 2 (AER2) (g/m^3)
$ST_{(i)}$	Storage tank i
$S_{TSS,eff}$	Sensor measurement for total suspended solids (TSS) at WWTP effluent (g N/m^3)
$T_{exc,DO}$	Yearly exceedance duration for dissolved oxygen in river (h)
$T_{exc,NH3}$	Yearly exceedance duration for unionized ammonia in river (h)
TKN	Total Kjeldahl nitrogen
T_{ovf}	Yearly overflow duration (days/year)
TSS	Total suspended solids
V_{ovf}	Yearly overflow volume (m^3)
$WWTP_{eff}$	Wastewater treatment plant effluent discharge into the river system

for achieving “good ecological and chemical status in all rivers” (EU, 2000). Today, tools for integrating sub-system models running on different platforms exist (Gregersen et al., 2007) and case studies illustrating their usage are available in the literature (Reußner et al., 2008; Van Assel et al., 2010). Commonly used commercial simulation software packages (e.g. SIMBA (ifak, Germany), WEST (DHI, Denmark)) provide libraries that allow users to develop system-wide models on a single platform. Additionally, several modelling libraries are developed by various researchers (e.g. Schütze, 1998; Achleitner et al., 2007; Mannina, 2005; Freni et al., 2010b; Willems and Berlamont, 2002).

Design and evaluation of (local/global) control strategies are two of the major areas where integrated models showed their full potential (e.g. Schütze et al., 2002; Meirlaen et al., 2002; Langeveld et al., 2013; Seggelke et al., 2005). Some of the studies were extremely successful, provided a lot of scientific inspiration for further control development and clearly demonstrated the benefits of using integrated approaches. Nevertheless, the evaluation/comparison of these control strategies, either real or model-based, is difficult. This is due to a number of reasons, including: i) variation in the characteristics of the UWS (catchment layout, sewer and WWTP design, river water quality etc.); ii) differences in the underlying models for describing the hydraulic, biological and physico-chemical processes in the UWS; and iii) the lack of a common evaluation method to compare the results. Hence, the objective comparison of the reported strategies has been a challenge.

A similar problem in the WWTP modelling community is addressed by using Benchmark Simulations Models (BSMs). Several researchers working under the umbrella of the International Water Association (IWA) benchmarking task group developed different benchmarks (BSM1, BSM1_LT, BSM2) to facilitate an unbiased comparison of control strategies in WWTPs (Copp, 2002; Rosen et al., 2004; Nopens et al., 2010). These BSMs consist of pre-defined layouts, process models, sensor/actuator models, influent characteristics and evaluation criteria (Gernaey et al., 2014). They have seen huge success (500 + publications) and are widely accepted in the research/practice community (Jeppsson et al., 2013). Similar efforts in the urban drainage community have been made where pre-defined sewer system layouts are used for comparing various real time control strategies and static design options (Borsányi et al., 2008; Schütze et al., 2015). Besides the original objective of comparing control strategies (Stare et al., 2007; Flores-Alsina et al., 2008; Sweetapple et al., 2014), the different tools developed by the BSM group are also used to develop better solvers (Rosen et al., 2008; Flores-Alsina et al., 2015), model development/comparison (Daelman et al., 2014; Solon et al., 2015) or full-scale optimization (Flores-Alsina et al., 2014; Lindblom et al., 2016; Kazadi Mbamba et al., 2016) using the model library as a software tool.

Hence, the idea of benchmarking is now extended to the UWS. The objective of this manuscript is to present the Benchmark Simulation Model (BSM-UWS) toolbox describing flow rate and pollution dynamics within the UWS. The toolbox considers the interactions between several sub-sections (CT, SN, WWTP and RW) and is developed on a single platform (Matlab). The BSM-UWS is a freely distributed and open-source toolbox and is intended to be used: i) as a software for developing integrated models for different UWSs; and ii) for unbiased evaluation of local/global control strategies using the presented system layout. The paper details the development of the new BSM platform, the simulation procedure and the evaluation criteria. In addition, a set of case studies are presented to illustrate the use of BSM-UWS for evaluation of control strategies. These case studies investigate the effect of: i) aeration control; ii) variation in bypass limits; and iii) optimal utilization of storage tank volumes on sewer overflows, performance of WWTP and river water quality. A discussion on various applications of the toolbox (both as a model library and as a BSM platform) is included.

In order to develop the toolbox: i) a hypothetical UWS layout comprising all the sub-systems is defined; ii) well-established process models such as the IWA BSM2 influent generator (Gernaey et al., 2011), the Activated Sludge Model no. 2 (ASM2d) (Henze et al., 2000) and the River Water Quality Model no. 1 (RWQM1) (Reichert et al., 2001) are used as the basis for the description of the catchment, WWTP and river water system, respectively. A conceptual modelling approach is used to describe the transport of wastewater in the sewer system (Viessman et al., 1989); and finally iii) a set of criteria to perform both emission/river quality based evaluations are described.

The paper contributes to the field of integrated modelling by enhancing various aspects of model development within the benchmark framework such as: i) improved catchment model with the ability to describe various sources of wastewater generation at different time scales; ii) storage tank models in different possible configurations; iii) pumping station model that can be used to describe rising mains as well as storage tanks with pumps; iv) interfaces between all the sub-sections ensuring conservation of elemental mass balances (for COD, carbon, nitrogen and phosphorus); and v) a concise list of evaluation criteria describing the performance of all the sub-sections. Additionally, it also contributes to the field of wastewater engineering by providing a freely distributed, ready-to-use UWS model that can be valuable for academics/practitioners to evaluate novel control/operational strategies and system modifications either on the pre-defined layout or any specific UWS model developed using the provided library.

2. Methods

2.1. Model description

2.1.1. Catchment (CT)

The catchment model has largely been derived from the BSM2 dynamic influent pollutant disturbance scenario generator (DIPDSG) (Gernaey et al., 2011). A detailed description of all model blocks used in the catchment model is available in Saagi et al. (2016). Flow rate (Q in m^3/d) and five pollutant variables (COD_{sol}, COD_{part}, NH_4^+, NO_3^- and PO_4^{3-}) (kg/d) are described in all the sub-models. The model facilitates simulation of wastewater generation from various sources using four sub-models, each for a specific source of wastewater generation.

- **Domestic (DOM)** sub-model simulates the generation of domestic wastewater and pollutant loads. Three user defined profiles for daily, weekly and yearly variations in flow rate and pollutant loads are included in the model as source files. All three source profiles are combined to generate a dynamic time series and further multiplied with the flow rate/pollutant load per population equivalent and the number of population equivalents at each sub-catchment to produce a dynamic wastewater profile from each sub-catchment. Additionally, zero-mean white noise can be added to the source profiles in order to avoid exactly similar values during different days.
- **Industrial (IND)** sub-model is used to represent the generation of industrial wastewater with daily, weekly and yearly variations in flow rate and pollutant loads. These variations can be due to: fluctuations in production times, holiday periods and maintenance periods. Similar to the DOM model, industrial wastewater generation is modelled by combining the source profiles for daily, weekly and yearly variations and multiplying it with the mean daily pollutant load/flow rate. DOM and IND models have different daily, weekly and yearly profiles. The mean pollutant loads as well as COD/N ratios are also different for the DOM and IND models. Similar to DOM source profiles, zero-mean white noise can also be added to the IND profiles.
- **Stormwater (SW)** is modelled using a simplified rainfall runoff model. It takes into account impervious areas and rainfall runoff coefficients for each sub-catchment to determine the amount of runoff from the surface of the sub-catchment to the sewer system (RUNOFF). The rainfall (RAIN) on the pervious area reaches the groundwater system and serves as an input for the SOIL module. Additionally, pollution generation from storm events is accounted for using two different approaches for soluble and particulate pollutants (Butler and Davies, 2011). Event mean concentrations (*EMC*) are used for soluble pollutants ($COD_{sol,EMC}$, $NH_{4\,EMC}^+$). This assumes constant pollutant concentrations for the entire rain duration. An accumulation and washoff model is used to describe the generation of particulate pollutants (COD_{part}) during rain events. It considers the accumulation of particulate pollutants on the sub-catchment surface during dry periods and subsequent washoff of the accumulated particulates during rain events as a function of the rainfall intensity. Additionally, a Markov-chain-based stochastic rainfall generator that can generate rainfall time series with similar properties to a user specified historic rainfall data series is also included (Richardson, 1981; Talebizadeh et al., 2016).
- **Infiltration to sewers (INF)** sub-model considers a hypothetical storage tank (SOIL) with inputs from the annual variation in groundwater (GW) levels and the percolation of surface runoff during rain events to the soil. The output from the tank is the infiltration to the sewer system determined by a non-linear relationship between tank level and infiltration flow rate (Gernaey et al., 2011). In the existing implementation, no pollutant loads are considered to reach the sewer from the infiltration model. However, this is something that can be modified by the model user, if required.

2.1.2. Sewer network (SN)

The sewer model simulates the transport of the wastewater generated in the catchment to the WWTP and also to the river system as overflows during rain events. A brief overview of the model blocks is provided here. (see Saagi et al. (2016) for more details). The model consists mainly of:

- **Pollution/flow rate transport (TRANSPORT)** sub-model that simulates the sewer network used for the transport of wastewater. It is formulated as a series of linear reservoirs with varying volumes (Viessman et al., 1989) connecting different sub-catchments and the WWTP. The number of such reservoirs for each sub-catchment is determined by the catchment area and the sewer flow that has to be conveyed through the network.

- **Storage tanks (STORAGE)** sub-models used to represent the storage tanks at various locations in the sewer system. The storage tanks are modelled in various possible configurations (on-line and off-line, pass-through and bypass) (Schütze et al., 2002; ATV, 1992). Outflow from the upstream sewer model or catchment determines the input to the storage tank while the throttle flow and overflow are calculated based on Vallet (2011) and Hager (2010), respectively. In case of storage tanks where the throttle flow is regulated by pumps, a pumping model (**PUMPING**) (Kroll et al., 2015) is used. The pumping model includes features for frequency control and the possibility to account for multiple pumps with different capacities at each storage tank. Hence, the throttle flows and pumping rates from the storage tanks limit the maximum possible flow through the pipes while there is no direct implementation of the maximum pipe capacity in the model. The storage tank models are also key control handles (throttle valves and pumps act as flow regulators) for implementing sewer-based control strategies in the system-wide model.
- **Particulate first flush (FIRST FLUSH)** sub-model used to describe the washoff of particulates that are accumulated in the sewer system during dry weather periods. It mimics the sudden increase in particulate pollutants at the beginning of rain events. This model assumes that a fraction of the particulate load during dry events is accumulated in the sewer system. During rain events, the accumulated particulates are washed off as a function of the flow rate (Gernaey et al., 2011).

2.1.3. Wastewater treatment plant (WWTP)

Major sub-model blocks of the WWTP model are the primary clarifier, biological reactors and secondary clarifier. The primary clarifier model is an empirical model with settling fractions for various soluble and particulate state variables (Otterpohl and Freund, 1992). The biological reactors are described using ASM2d (Henze et al., 2000). The ASM2d considers organic matter degradation, nitrification/denitrification and also phosphorus accumulation dynamics. The secondary clarifier is modelled using the Bürger-Diehl settling model (Bürger et al., 2011, 2012; 2013). This model considers convection (includes bulk flux and hindered settling), compression (above a critical concentration) and dispersion (due to mixing, particularly at the feed inlet) to describe the concentration of sludge at various layers in the settling tank. A 10-layer non-reactive settler model approach is used in this study (Arnell, 2015). Also, the WWTP layout has a rainwater storage tank that is modelled as an on-line storage tank.

2.1.4. River water system (RW)

The river water system model is based on a simplified version of the IWA River Water Quality Model (Reichert et al., 2001). Main processes included are growth of bacteria and algae using organic substrate, nutrients, oxygen and sunlight. The formula described in Owens et al. (1964) is used to calculate the reaeration coefficient based on the velocity and water depth at each river stretch. The default values for kinetic and stoichiometric coefficients from the RWQM1 Technical Report (Reichert et al., 2001) are used. The conceptual river system model consists of a series of varying volume tanks ($RW_1 \ldots RW_n$), each representing a stretch of the river. Other parameters like solar irradiance, temperature, river characteristics (river length/tank, bottom width, bed slope, pH) can be defined. The model does not include higher organism dynamics and the effect of a sediment layer.

2.1.5. Interfaces

In order to link different sub-system models, three different interfaces have been developed: i) SEWER-WWTP interface; ii) WWTP–RIVER interface; and iii) SEWER-RIVER interface. In all cases, mass balances are verified and COD/elemental continuity is ensured according to the principles stated by Vanrolleghem et al. (2005b), Volcke et al. (2006), Nopens et al. (2009) and Flores-Alsina et al. (2016).

2.2. Evaluation criteria

Evaluation criteria are defined to assess the performance of different control/operational strategies implemented in the BSM-UWS. While the sewer and WWTP performance criteria represent the performance of the sub-systems, criteria for the river system can be used for a holistic evaluation of the integrated UWS performance considering that one of the main objectives of an UWS is to ensure good river water quality (EU, 2000). It should be noted that the threshold concentrations and weightings for individual pollutant loads can easily be changed by the users, for example to suit local regulations.

2.2.1. Sewer network (SN)

Major evaluation criteria for the sewer system are: overflow duration (T_{ovf}, h); ii) overflow frequency (N_{ovf}, events/year); iii) overflow volume (V_{ovf}, m^3); and iv) overflow quality index (*OQI*, kg pollutant units/day). All the above criteria are calculated for the entire sewer system as well as for individual overflow locations. Overflow quality index (*OQI*) is an aggregated criterion to represent the pollutant load reaching the river system from the sewer network. Overflow pollutant loads $BOD_{5,ovf}$, COD_{ovf}, TSS_{ovf}, TKN_{ovf}, $NO_3^-{}_{ovf}$, $P_{org,ovf}$ and $P_{inorg,ovf}$ are calculated and individual weights are used for each of the pollutants to calculate an overall pollution index. Additional criteria to represent acute conditions are the hourly maximum concentration and exceedance duration for BOD_5, TKN and PO_4^{3-} from overflows. Also, all the above criteria can be computed for each overflow location. Further details of these criteria can be found in Saagi et al. (2016).

2.2.2. Wastewater treatment plant (WWTP)

Traditional WWTP evaluation criteria for BSMs (Jeppsson et al., 2007) that are mainly used in this study are the influent quality index (*IQI*, kg pollutant units/day) and the effluent quality index (*EQI*, kg pollutant units/day). Both indices are calculated in a similar fashion and represent the pollution load for the influent and effluent of the WWTP. Total pollution load for key pollutants (for *IQI*: $BOD_{5,in,WWTP}$, $COD_{in,WWTP}$, $TSS_{in,WWTP}$, $TKN_{in,WWTP}$, $NO_3^-{}_{in,WWTP}$, $P_{org,in,WWTP}$, $P_{inorg,in,WWTP}$; for *EQI*: $BOD_{5,eff,WWTP}$, $COD_{eff,WWTP}$, $TSS_{eff,WWTP}$, $TKN_{eff,WWTP}$, $NO_3^-{}_{eff,WWTP}$, $P_{org,eff,WWTP}$, $P_{inorg,eff,WWTP}$) is multiplied with the respective weights to calculate the total pollution index. The weights used in *IQI*, *EQI* and also for the *OQI* calculation for sewer overflows are identical.

2.2.3. River water system (RW)

Evaluation criteria for the river are based mainly on two major pollutants (dissolved oxygen (DO) and unionized ammonia (NH_3)) that are commonly used to assess river water quality. The four major river quality criteria used in this study are: i) exceedance duration for NH_3 ($T_{exc,NH3}$, h); ii) exceedance duration for DO ($T_{exc,DO}$, h); iii) maximum hourly unionized ammonia concentration ($C_{max,NH3}$, g/m^3); and iv) minimum hourly DO concentration

($C_{min,DO}$, g/m^3). $T_{exc,NH3}$ (h) represents the total time for which the ammonia concentration in any river stretch exceeds a threshold concentration (0.018 g/m^3). $T_{exc,DO}$ (h) denotes the total time for which the DO concentration is below a threshold value (6 g/m^3) at any river stretch. While $T_{exc,NH3}$ and $T_{exc,DO}$ are the building blocks for Fundamental Intermittent Standards specified in the UPM manual (FWR, 2012), $C_{min,DO}$ and $C_{max,NH3}$ are used as additional complimentary criteria inspired by Schütze et al. (2002). The threshold values for $T_{exc,NH3}$ and $T_{exc,DO}$ are chosen from the limit values prescribed in the UPM manual for salmonid species (FWR, 2012).

2.3. BSM-UWS layout and characteristics

The urban wastewater system layout used for the BSM-UWS is presented in Fig. 1. It consists of an urban catchment that generates sewage as well as stormwater. A sewer network collects and transports the wastewater to a WWTP. Treated effluent from the WWTP is discharged into the river. Additionally, during rain events, any excess flow beyond the capacity of the sewer system and storage tanks reaches the river system. The layouts for urban catchment and sewer system are hypothetical and inspired by ATV A 128 case study (ATV, 1992) and Schütze (1998). The WWTP layout is similar to that used in BSM1-ASM2d (Flores-Alsina et al., 2012). The river section is also hypothetical and assumed to be a shallow river stretching across the urban catchment.

2.3.1. Catchment (CT)

The catchment layout consists of six sub-catchments ($SC_1 \ldots SC_6$) connected to the WWTP through a sewer network. The catchment has a total area of 540 ha with 80,000 population equivalents. Mean dry weather wastewater generation is 19,000 m^3/d (domestic flow = 12,000 m^3/d, industrial flow = 2,500 m^3/d and infiltration = 4,500 m^3/d). While SC_2 has both an industrial and domestic section, the remaining sub-catchments are assumed to only generate domestic wastewater. An impervious area ratio of 75% is assumed for all sub-catchments. Table 1 contains the area, population equivalents and dry weather flow information for all sub-catchments.

2.3.2. Sewer network (SN)

Five of the six sub-catchments ($SC_1 \ldots SC_4$, SC_6) are connected to a combined sewer system whereas SC_5 is connected to a separate sewer network (see Fig. 1). Five storage tanks (ST_1, ST_2, ST_4, ST_5 and ST_6) with overflows (OVF_1, OVF_2, OVF_4, OVF_5 and OVF_6) are available in SC_1, SC_2, SC_4, SC_5 and SC_6, respectively. The total available storage volume is 22,100 m^3 (approx. 40 m^3/ha of catchment area). Individual storage volume for each tank (connected to a sub-catchment) is detailed in Table 1. The storage volumes for individual tanks vary from 15 m^3/ha to 50 m^3/ha of the connected upstream catchment area. This uneven distribution of storage volumes can be exploited for implementing various control strategies. Four of the storage tanks (ST_1, ST_2, ST_5 and ST_6) are on-line pass-through tanks while ST_4 is an off-line bypass tank. Any excess flow above $Q_{throttle,ST4}$ is diverted to ST_4 while the rest is sent to the downstream sewers. The stored wastewater in ST_4 is pumped ($Q_{pump,ST4}$) back to the main sewer line after the rain event. The throttle flow from ST_1 is regulated by pumps ($Q_{pump,ST1}$) while the rest are regulated by throttle valves (ST_2, ST_5 and ST_6).

2.3.3. Wastewater treatment plant (WWTP)

The WWTP is designed for an average influent flow rate of 20,500 m^3/d. A rainwater storage tank (RST) (8,000 m^3) at the beginning of the WWTP, two bypass facilities (BP_1, BP_2) (before and after the primary clarifier) and a primary clarifier (PC) (900 m^3) are added to the original configuration. BP_1 has a threshold ($Q_{max,BP1}$) of 90,000 m^3/d (any flow excess of the threshold is bypassed and reaches the river system) while BP_2 has a threshold ($Q_{max,BP2}$) of

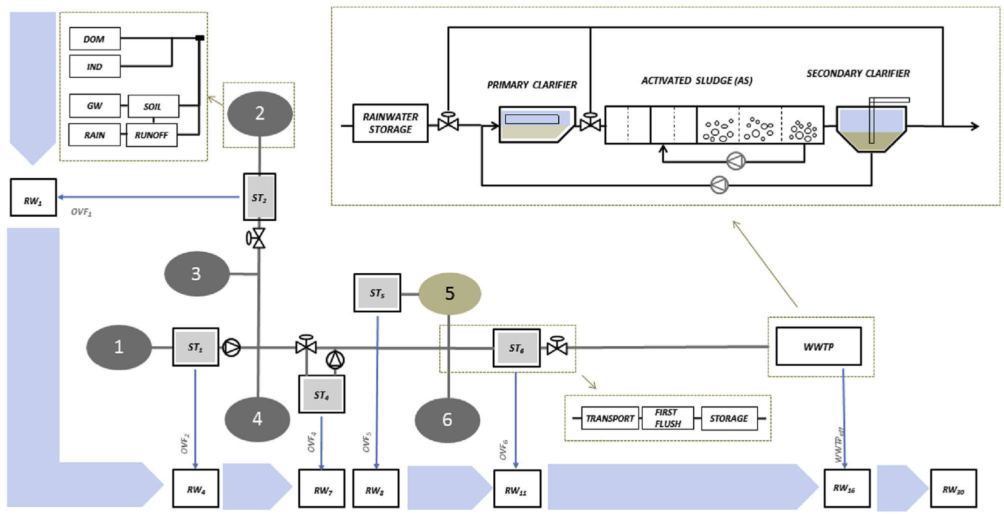

Fig. 1. BSM-UWS layout indicating various sub-catchments (①, ②, ③, ④, ⑤, ⑥ represent SC_1, SC_2, SC_3, SC_4, SC_5 and SC_6, respectively), storage tanks (ST_1, ST_2, ST_4, ST_5 and ST_6) and control elements (throttle valves and pumps). The WWTP includes primary clarifier (PC), anaerobic (ANAER), anoxic (ANOX) and aerobic (AER) tanks followed by a secondary clarifier (Sec.C). Sewer overflows (OVF_1, OVF_2, OVF_4, OVF_5 and OVF_6) and WWTP effluent ($WWTP_{eff}$) are discharged into the river system ($RW_1 \ldots RW_{30}$).

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

Sub-catchment (SC)	Total area (ha)	PE	DWF (m^3/d)		Storage volume (m^3)
			DOM	*IND*	
1	99	15,920	2,390		5,000
2	21	3,920	590	2,500	1,000
3	29	2,960	440		
4	71	9,600	1,440		4,400
5	71	7,840	1,180		3,600
6	249	39,760	5,960		8,100
Total	540	80,000	12,000	2,500	22,100

PE: Population equivalents; DOM: Domestic; IND: Industrial.

70,000 m^3/d. The two bypass facilities are used so that a higher flow rate can be sent to the PC resulting in partial treatment (removal of particulates to some extent) and a lower bypass level can be used before the biological reactors to avoid significant loss of biomass and treatment capacity. It will also provide more control handles for the layout. Other components of the WWTP include biological reactors (two anaerobic tanks (ANAER1, ANAER2) (2 $\times$ 1,000 m^3), two anoxic tanks (ANOX1, ANOX2) (2 $\times$ 1,500 m^3) and three aerobic tanks (AER1, AER2, AER3) (3 $\times$ 3,000 m^3) followed by a secondary clarifier (Sec.C) (area = 2,500 m^2). The sludge line (Solon et al., 2017) is not included in the current layout.

2.3.4. River water system (RW)

The river system stretches across the length of the catchment and extends even after the WWTP. The river system in the urban catchment is 30 km long with a bottom width of 7 m. The mean annual base flow rate of the river during dry weather conditions is 72,500 m^3/d (diltution ratio of 4 when compared to daily average dry weather flow). The chosen dilution ratio is similar to that in Schütze (1998) and higher than those used in Langeveld et al. (2013) and Vanrolleghem et al. (2005a). Increasing the dilution ratio further will lead to a less polluted river thereby reducing the potential for any further improvements using control strategies. Additional runoff from an upstream catchment (area = 500 ha) reaches the river during rain events. Reaeration coefficients obtained are in the range of 7–12 d^{-1}. These values are similar to those reported in Borchardt and Reichert (2001) while some other literature sources show higher values (Mannina and Viviani, 2010). The difference in values can be due to the different kinds of rivers and also due to the choice of modelling approach. The upstream pollutant concentrations are assumed to be constant and identical for both wet and dry weather conditions. 30 varying volume tanks (RW_1…RW_{30}) are used to represent the river system, each tank representing a 1 km stretch of the river. WWTP and various overflow locations in the urban catchment discharge into the river system (OVF_1-RW_1, OVF_2-RW_4, OVF_4-RW_7 OVF_5-RW_8, OVF_6-RW_{11} and $WWTP_{eff}$-RW_{16}). It is necessary to include the long river stretch (14 km) after the WWTP so as to completely capture the effect of organic matter discharge on the dissolved oxygen (DO) levels in the river. Depending on the flow conditions and pollutant loads, the lowest DO concentration can occur very far from the point of discharge.

2.4. Simulation methodology

Model development and simulation is carried out in Matlab (version 2014b). Firstly, a steady state simulation for 100 days of dry weather conditions is performed to generate the initial states for model blocks in WWTP and river system (RW). The steady state values obtained are used as initial states for all further simulations. For the catchment (CT) and sewer network (SN) sub-sections, initial states are assumed. One year simulation is carried out for the open loop as well as control case studies. A three-step simulation approach similar to other BSM models is followed: i) all the model parameters and input files are loaded using an initialization script; ii) the model is then simulated using "ode45" (an explicit Runge-Kutta solver); and iii) on completion of the simulation, an evaluation script computes the performance criteria based on the dynamic outputs from various model blocks.

2.5. Control strategies

The control strategies used in the study are summarized in Table 2. It is important to highlight that the aim here is not to find the best control strategy (optimal) but to demonstrate the versatility of the tool to evaluate various control strategies. Sensor and actuator models from BSM2 are used (Jeppsson et al., 2007). These models consider noise and time delays for sensor measurements and actuator responses. No fault models for sensors and actuators are included (Rosén et al., 2008).

There is no active control strategy in the OL case. Pumping rates ($Q_{pump,ST1}$, $Q_{pump,ST4}$) are only influenced by the storage tank level. Maximum throttle flows ($Q_{throttle,ST4}$) and throttle valve positions (indirectly influencing $Q_{max,ST2}$, $Q_{max,ST5}$, $Q_{max,ST6}$) are constant. In the WWTP, the sludge recirculation rate (Q_r), sludge wastage rate (Q_w) and internal recirculation rate (Q_{intr}) are fixed. The aeration is supplied at a constant rate (K_La_{AER1} = 120 d^{-1}, K_La_{AER2} = 120 d^{-1}, K_La_{AER3} = 60 d^{-1} to AER1, AER2 and AER3, respectively).

2.5.1. Control strategy 1 (CL1)

CL1 is a local control strategy at the WWTP aimed at maintaining the required dissolved oxygen concentration in the aeration tanks (AER). The dissolved oxygen level in AER2 ($S_{O2,AER2}$) (measured variable) is sent to a feedback controller (PI) (control action) that determines the required oxygen transfer coefficient (K_La_{AER2}) (control variable) (K_La is used as a surrogate for air flow rate) to maintain a predetermined dissolved oxygen level (set point = 2 g/m^3) in AER2. For aeration tanks AER1 and AER3, the K_La values are adjusted with a correction factor (K_La_{AER3} = 1.5 $\times$ K_La_{AER2}, K_La_{AER3} = 0.5 $\times$ K_La_{AER2} for AER1 and AER3, respectively). In this way, a single control loop is used to provide precise control of oxygen level in AER2 and also a less accurate control for the oxygen levels in AER1 and AER3. Further information can be found in Gernaey et al. (2014).

2.5.2. Control strategy 2 (CL2)

CL2 is a rule-based integrated control strategy that uses information from the river system to modify the operation of the WWTP (Vanrolleghem et al., 2005a). The principle behind this control strategy is to maximize the utilization of the biological treatment capacity available in the WWTP. Control strategy CL2 includes CL1 as well. Ammonia measurements in river stretch 16 (at the point of

Table 2
Details of various control strategies evaluated using the system-wide BSM model.

No.	Sub systems	Measured variable	Control variable	Manipulated variable	Control strategy
CL1	WWTP	$S_{O2,AER2}$ (Oxygen conc. at aerobic reactor 2)	$S_{O2,AER1}$, $S_{O2,AER2}$ and $S_{O2,AER3}$ (Oxygen conc. at aerobic reactors 1, 2 and 3)	K_La_{AER1}, K_La_{AER2} and K_La_{AER3} (Oxygen transfer coefficient at aerobic reactors 1, 2 and 3)	Feedback control (PI)
CL2	WWTP-River	$S_{NH4,RW16}$ (Ammonia conc. at river stretch 16)	$Q_{max,BP1}$ and $Q_{max,BP2}$ (Threshold for bypass flow before and after the primary clarifier)	$Q_{max,BP1}$ and $Q_{max,BP2}$ (Threshold for bypass flow before and after the primary clarifier)	Rule-based control
CL3	Sewer-WWTP	$Q_{in,WWTP}$ (Flow rate at WWTP inlet)	Throttle flow from ST_1, ST_2, ST_4 and ST_6 to downstream sewer	$Q_{max,ST2}$, $Q_{throttle,ST4}$, $Q_{max,ST6}$, $Q_{pump,ST1}$ (Throttle valve opening for ST_2, ST_4, ST_6 Pumping rate for ST_1)	Rule-based control

Open loop (OL).

WWTP effluent discharge) ($S_{NH4,RW16}$) are used to manipulate the bypass limits ($Q_{max,BP1}$ and $Q_{max,BP2}$) at the WWTP inlet. If the ammonia concentration ($S_{NH4,RW16}$) in the river stretch exceeds 0.4 g/m^3, $Q_{max,BP1}$ and $Q_{max,BP2}$ are increased by 20% thereby sending more wastewater to the biological treatment. Also, an additional condition (effluent TSS ($S_{TSS,eff}$) < 60 g/m^3) is used to ensure that there is no significant loss of solids/nitrification capacity from the secondary settler/biological reactors due to increased flow to the activated sludge section.

2.5.3. Control strategy 3 (CL3)

The last alternative also consists of a rule-based integrated control strategy that modifies the behaviour of the storage tanks (throttle valve opening ($Q_{max,ST2}$, $Q_{throttle,ST4}$, $Q_{max,ST6}$) and pumping rate ($Q_{pump,ST1}$)) in the sewer system based on the information from the WWTP. The control strategy is inspired from Weyand (2002) and Kroll et al. (2015). In scenario CL3, CL1 is also active. The control strategy aims to utilize the storage capacity available in the sewer network to reduce hydraulic shocks to the WWTP. The flow rate at the inlet to the WWTP ($Q_{in,WWTP}$) (measured variable) is used by the rule-based controllers at storage tanks ST_1, ST_2, ST_4 and ST_6 to reduce the throttle flow to the downstream sewer network when storage capacity is available. If the inflow ($Q_{in,WWTP}$) to the WWTP is higher than 80,000 m^3/d and there is still capacity available in the storage tank ($h_{ST} < 4$ m): i) only one pump is used in the pumping station at ST_1 (i.e., the pumping capacity ($Q_{pump,ST1}$) is reduced to 63% of the maximum capacity); ii) at ST_2 and ST_6, the valve openings ($Q_{max,ST2}$, $Q_{max,ST6}$) are reduced by 50% and 30%, respectively; and iii) at ST4, the throttle flow ($Q_{throttle,ST4}$) is reduced by 50%.

3. Results

3.1. Open loop dynamics

3.1.1. Catchment (CT)

The catchment model is capable of simulating the dynamic profiles for wastewater generation during dry weather as well as rain events (Fig. 2a, 2b & 2c). Flow rate and pollutant loads follow a daily, weekly and yearly variation based on the profiles used in the catchment model (Gernaey et al., 2011; Saagi et al., 2016). Fig. 2a depicts the total wastewater generation from all the sub-catchments (the data collected is the total sum of flow rates to the sewer system from each sub-catchment) with peak flows during rain events. Fig. 2b and c represent the variation in COD_{sol} and COD_{part} generation from the entire catchment. Fig. 3a highlights the contribution of different sub-models to the wastewater flow rate generation from the catchment. Rain events (e.g. days 78, 81, 84) lead to a prolonged increase in the infiltration to sewers (INF) due to an increase in groundwater levels during rains. Dry weather flow (DRY) represents the contribution from domestic (DOM) and industrial (IND) sub-models with diurnal as well as weekly and yearly changes. A substantially high wastewater contribution comes from the stormwater (SW) model during rain events. The contributions from DRY (DOM + IND sub-models) and SW sub-models to the COD_{part} generation are shown in Fig. 3b. While the dry weather generation is based on the dynamic profiles and the mean pollutant loads, the SW sub-model (accumulation and washoff model for COD_{part}) leads to an increased particulate load due to washoff of particulate organic matter from the catchment surface during rain events.

3.1.2. Sewer network (SN)

Fig. 2d shows the total overflow from all the locations in the sewer network. Higher overflow volumes are observed during the first part of the evaluation period (evaluation starts in the first week of July) as there is heavy rainfall during summer and autumn. Fig. 3c shows the attenuation and delay in flow rate due to the sewer network (TRANSPORT & STORAGE sub-models). The STORAGE sub-model describes various storage tank configurations and also simulates the overflow generation from the sewer network. The storage volumes considered here are only for the storage tanks and do not consider the storage capacity in the sewers. The extent of overflows depends on the available volume in the storage tanks and the operation of throttle valves and pumps. Fig. 3d shows the functioning of pumps at different storage tanks. ST_1 is an on-line tank (where all wastewater passes through the tank) and hence the pump is in operation during the entire period. ST_4 is an off-line tank where flow reaches the tank only when it crosses the maximum permitted throttle flow to downstream ($Q_{throttle,ST4} = 83{,}000$ m^3/d in open loop). The stored water in ST_4 is emptied after the end of a rain event and hence the ST_4 pump is operational only during certain periods. Additionally, the FIRST FLUSH model in the sewer system simulates the storage of particulate pollutants in the sewer during dry periods and subsequent washoff of the stored particulates during rain events. A more detailed analysis of the catchment and sewer network models is presented in Saagi et al. (2016).

3.1.3. Wastewater treatment plant (WWTP)

The WWTP model has been well established and used in numerous other studies (Jeppsson et al., 2007). An average sludge retention time of 17 days is maintained in the biological system. The functioning of the WWTP is depicted through Fig. 2e and f. The dissolved oxygen (DO) profile in AER3 indicates that the aeration system is highly inefficient (insufficient during daytime and

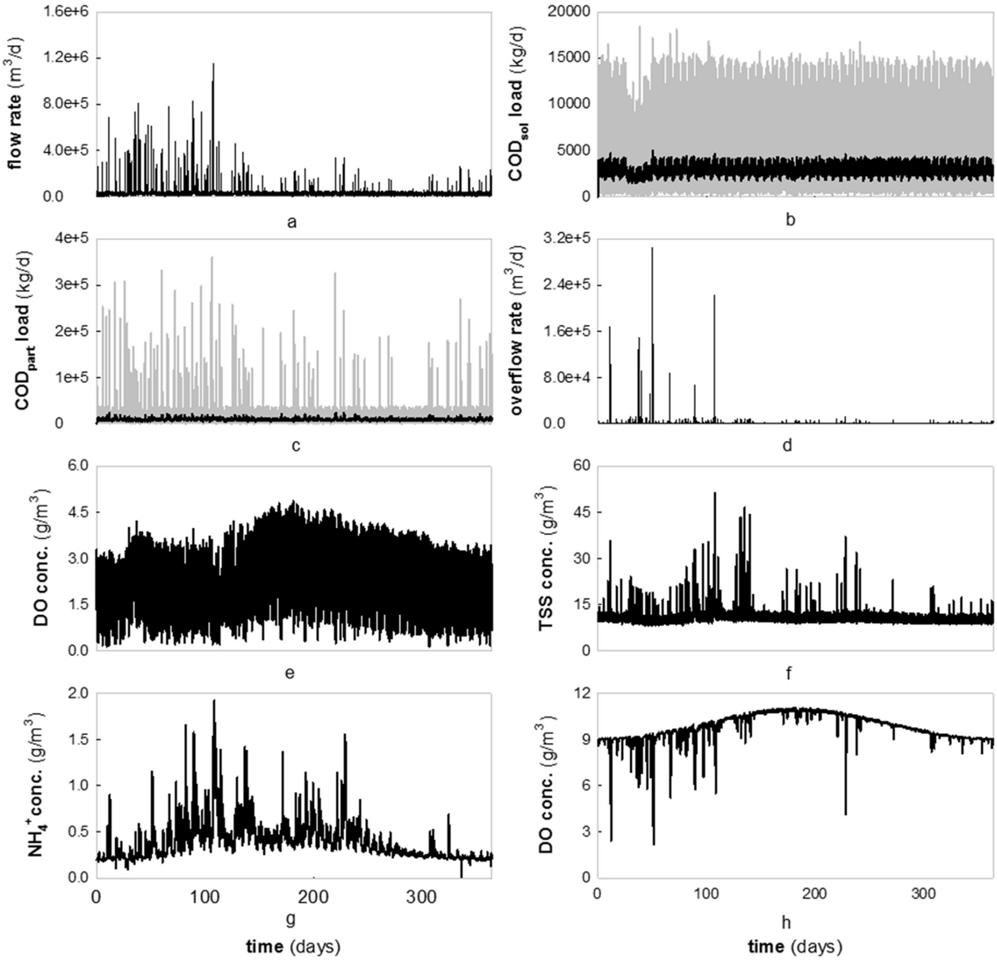

Fig. 2. Yearly dynamic profiles for the OL case, describing: a) total sum of all flow rates originating from each sub-catchment; b) COD_{sol} pollutant load generated from the entire catchment (grey-unfiltered; black-filtered); c) COD_{part} pollutant load generated from the entire catchment (grey-unfiltered; black-filtered); d) total overflow from the sewer system to the river; e) DO concentration in the AER3; f) effluent TSS concentration from the secondary settler; g) NH_4^+ concentration at river stretch 16 (after WWTP effluent discharge); and h) DO concentration at river stretch 16. The profiles start in the first week of July (= day 0).

excessive at nights). The settler model has been able to describe TSS washoff during rain events.

3.1.4. River water system (RW)

During dry weather, the river water quality is mainly affected by the effluent discharge from the WWTP. There are no significant variations in the river pollution from the upstream stretch until the WWTP (Fig. 4a & 4b). For the river stretch upstream of the WWTP effluent discharge point ($RW_1...RW_{16}$), a slight increase in DO concentration along the stretch is observed. This is due to river reaeration and lack of organic pollutants that demand oxygen for degradation. Downstream of the WWTP discharge, a major drop in river quality in terms of NH_4^+ and DO is observed (see Fig. 2g and h). The additional NH_4^+ load from the WWTP effluent leads to an increased NH_4^+ concentration in the river. A drop in the DO concentration in the river is observed mainly due to the degradation of organic matter (from the WWTP effluent) by (suspended) aerobic heterotrophic bacteria in the river. The DO concentration has dropped at some of the discharge locations (e.g. RW_7) and mainly at the WWTP discharge location (RW_{16}) (Fig. 4a) due to an increase in the organic load to the river. The DO concentration values eventually recover as the organic matter is degraded and river reaeration leads to increase in the DO levels. Fig. 4b represents the NH_4^+ dynamics at different river stretches for a specific time period.

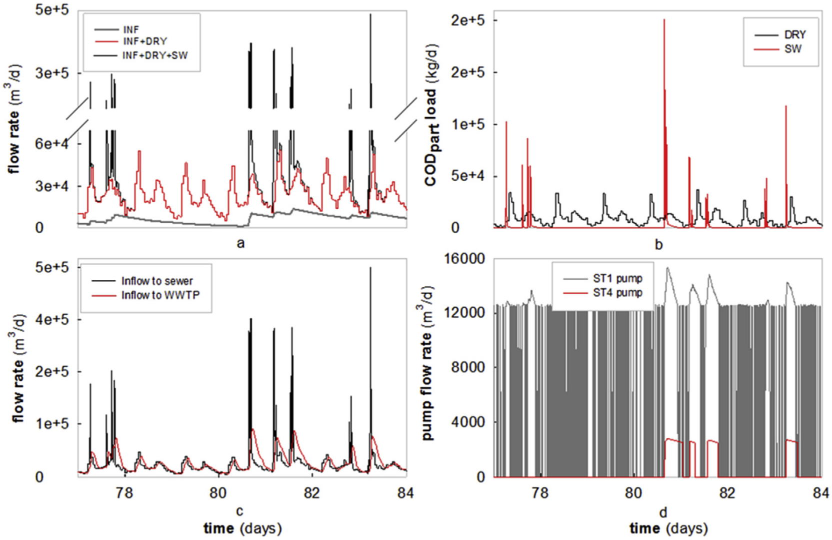

Fig. 3. a) Contribution from the infiltration (INF), dry weather (domestic + industrial) (DRY) and rainfall runoff from storm events (SW) to the catchment flow rate generation. b) Generation of COD_{part} from different sub-models: DRY (DOM & IND sub-models) and SW (stormwater sub-model using the accumulation and washoff model). c) Comparison between the inflow and outflow from the sewer system describing the effect of sewer network on flow rate. d) Pump outflow from different storage tanks – ST_1 (on-line tank), ST_4 (off-line tank). All results are for the OL case.

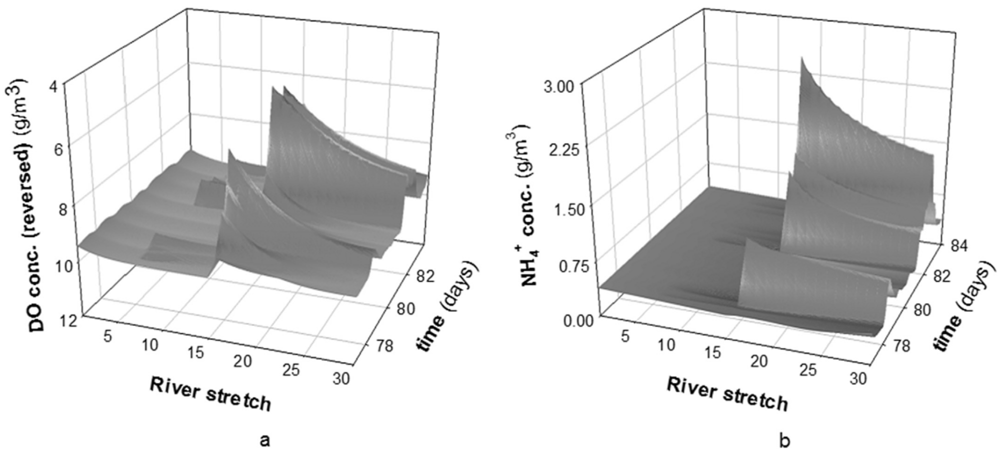

Fig. 4. a) DO and b) NH_4^+ mesh plots depicting the concentration changes as functions of time and river stretches. WWTP discharges into the river at stretch 16 (RW_{16}).

During rain events, at some overflow locations (OVF_5), there is a dilution in NH_4^+ concentration in the river (e.g. RW_{10}) as the overflow NH_4^+ concentration is less than that in the river. A big increase in NH_4^+ concentration is observed at the WWTP effluent (RW_{16}) and attenuates as we move further downstream. It can be concluded that the WWTP is the major source of river pollution for this catchment. Additionally, the sewer overflows aggravate the problem that is caused by the WWTP.

3.2. Control strategies

3.2.1. CL1: DO control at WWTP

Simulation results show that the first control action is successful in maintaining the desired DO set point in AER2 (Fig. 5a). *The*

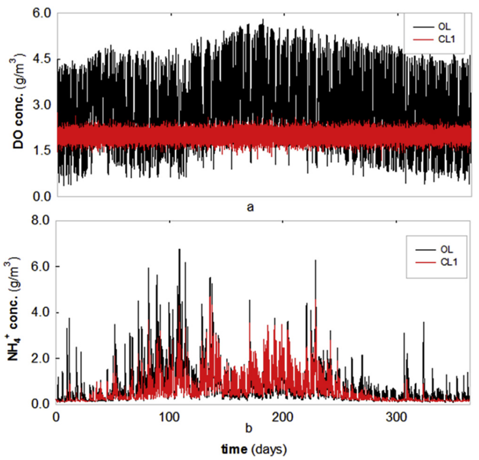

Fig. 5. Effect of CL1 (red) on: (a) DO concentration in AER2; and (b) effluent NH_4^+ from the secondary settler. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

control strategy (CL1) increases the efficiency of the aeration system, leading to improved nitrification and thereby enhances effluent quality. The increase in nitrification capacity is visible in the peak shavings in NH_4^+ concentration due to the effect of CL1. This is indicated by the NH_4^+ profile at the WWTP effluent (Fig. 5b) and also by a decrease in effluent quality index (*EQI*) (5%) in comparison to OL (Table 3). For the river system, CL1 reduces the exceedance duration for NH_3 ($T_{exc,NH3}$) by 256 h (66%) due to lower NH_4^+ concentrations in the WWTP effluent. In contrast, it increases the exceedance duration for DO ($T_{exc,DO}$) by 21 h (7%). This can be attributed to the fact that aeration control leads to a marginally higher mixed liquor suspended solids (MLSS) concentration (mean MLSS values in reactor 7 are 3675 g/m^3 and 3710 g/m^3 for OL and CL1, respectively) in the biological treatment system (higher biomass growth due to improved oxygen supply). Although, the increase is not significant, this causes a higher TSS washoff from the settler during rain events, which is increasing the severity of the DO concentration drop in the river (see results in Table 3).

Table 3
Performance of various sub-systems (sewer, WWTP, river) applying different control strategies (CL1, CL2, CL3) compared to the OL case.

	OL	CL1	CL2	CL3
Sewer				
V_{ovf}(m^3)	203,393	203,393	203,393	207,733
OQI (kg poll units/day)	342,262	342,262	342,262	348,373
WWTP				
IQI (kg poll units/day)	92,714	92,714	92,714	94,377
EQI (kg poll units/day)	6,778	6,466	6,409	6,505
River				
$T_{exc,NH3}$ (h)	389	133	148	187
$T_{exc,DO}$ (h)	308	329	291	274

3.2.2. CL2: integrated control of bypass to WWTP based on river ammonia concentration + CL1

Fig. 6a shows that CL2 reduces the bypass volume and sends more wastewater to the biological treatment line in comparison to OL and CL1 during a rain event. Compared to OL and CL1, a drop of 45% is observed in the volume of wastewater bypassed from the WWTP. The control strategy has also led to a decrease in *EQI* by 3% and 1% in comparison to OL and CL1, respectively (Table 3). The changes in river quality are considerable when compared to OL (62% (256 h) decrease in $T_{exc,NH3}$ and 6% (21 h) decrease in $T_{exc,DO}$). The same can be noticed in Fig. 6b indicating a drop in NH_4^+ concentration at river stretch 16 after the WWTP discharge point when compared to OL. However, when compared to CL1, $T_{exc,NH3}$ increased by 10% (13 h) and $T_{exc,DO}$ decreased by 12% (38 h). The increase in $T_{exc,NH3}$ is due to higher NH_4^+ loads from WWTP effluent although *EQI* is lower. The drop in $T_{exc,DO}$ is due to the reduction in bypass volume which has higher organic load than the treated effluent. To summarize, it can be said that CL2 does not improve both $T_{exc,DO}$ and $T_{exc,NH3}$ simultaneously when compared to CL1. *CL2 can be chosen over CL1, when improving oxygen concentration in the river is prioritized over lowering ammonia concentrations.*

3.2.3. CL3: integrated control of sewer system based on inlet flow to wastewater treatment plant + CL1

Fig. 7a shows that ST_6 is utilized to store more water in CL3 than in OL during a rain event. Fig. 7b shows a similar phenomenon for ST_4 throttle flow. With CL3, the maximum permissible flow ($Q_{throttle,ST4}$ − constant value in OL) is reduced under certain conditions as defined in the rule-based control strategy thereby sending less flow to the WWTP and instead directing the wastewater to ST_4. The overflow volume (V_{ovf}) and *OQI* increased only marginally (1%) in CL3 in comparison to OL (see Table 3). The additional overflows occur as the control strategy tries to maximize the utilization of storage volume in order to reduce peak flows to the WWTP. The effect of the control strategy on the inflow to the WWTP can be observed in Fig. 7c. CL3 reduces the peak inflow to the WWTP as the storage tanks are able to absorb some of the peak flows. For the WWTP, the drop in peak flow reduces the bypass volume by 30% in comparison to CL1 and OL. While the control strategy led to a drop in the inflow volume to the WWTP, it has increased the *IQI* (2% higher than OL). The increase is mainly due to better utilization of the storage tanks where highly polluted wastewater is being sent to the WWTP instead of reaching the overflows. This has also led to a marginal increase in *EQI* in comparison to CL1 although it is 4% lower than OL. Fig. 7d shows that the DO concentration in the river is higher in CL3 than in OL. This is due to reduced organic load to the river system. This has led to a significant reduction in $T_{exc,DO}$ (11% lower than OL and 17% lower than CL1). Although $T_{exc,NH3}$ is 51% lower than OL, it is 41% higher than CL1. The reason is the higher NH_4^+ loads (compared to CL1) discharged into the rivers during rain events. In conclusion, for a system with emphasis on reducing $T_{exc,DO}$, CL3 can be chosen over the other controls but it should be remembered that this will lead to higher $T_{exc,NH3}$ in comparison to other control choices. *Additionally, it should be noted that CL3 does not have any significant effect on the performance of the sewer system (where the control is operational) but provides major changes to the river water quality*. This indicates that the interactions between the sub-systems are important as well as complex in many cases and also puts a strong emphasis on the need for integrated modelling and control.

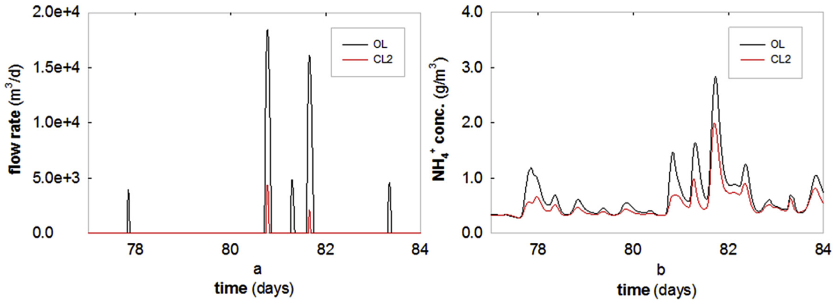

Fig. 6. Comparison between OL (black) and CL2 (red) for: (a) bypass from the WWTP; and (b) river stretch 16 NH_4^+ concentrations. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

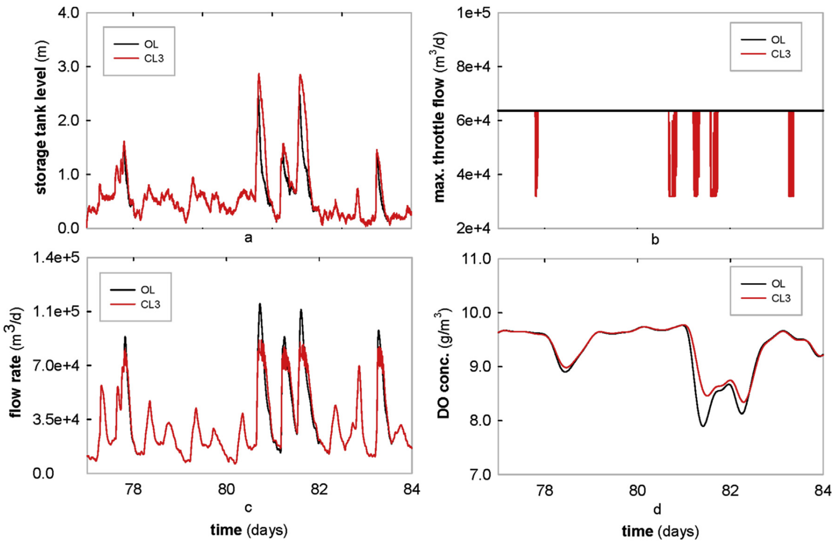

Fig. 7. Variation in: (a) ST_6 level; (b) maximum permissible throttle flow ($Q_{max,ST4}$) from ST_4 to downstream sewer; (c) inflow to WWTP; and (d) DO concentration at river stretch 30 for OL (black) and CL3 (red). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

4. Discussions

4.1. BSM-UWS for benchmarking control strategies

Similar to other BSM models (BSM1, BSM2), BSM-UWS has been primarily designed as a platform to perform objective evaluation of control strategies on an UWS scale. Both local and global control strategies at different sections of the UWS can be easily implemented, simulated and evaluated with the proposed set of criteria as demonstrated in this paper. One of the major challenges during the model development is the defining of the catchment layout. It has been understood that the complexity and variety in the design of sewer systems, WWTP and variations in river structures cannot be captured in one benchmark model. In fact, it is impossible to have a single benchmark layout that is representative of the vast number of possible UWS. Hence, it has been agreed to use a realistic layout of the CT, SN, WWTP and RW. This is only one of the many catchment layouts that exist in Europe and elsewhere. Hence, the control strategies that are evaluated cannot be directly applied to other catchments. However, given that a pre-defined layout already exists, future users can directly develop various control strategies without having to start from the model development from scratch. All the control strategies evaluated using the layout can serve as a repository that can be explored for ideas when implementing site-specific alternatives. That is what the authors see as one of the main valuable contributions of having such a simulation tool available.

4.2. BSM-UWS as a platform for other integrated modelling studies

Like other models from the BSM family, the BSM-UWS can be used for different purposes apart from the purpose of control strategy development and evaluation. As a platform with pre-defined UWS layout, i) it can be used to evaluate future scenarios arising due to changes in the rainfall pattern, catchment layout and characteristics etc. The impact of such changes on the sub-system performance (sewer overflows and WWTP effluent quality) and also on the river water quality can be studied (Astaraie-Imani et al., 2012); ii) it can be linked with an optimization/control design routine to determine best possible control/design parameters for the given layout (Fu et al., 2008; Muschalla, 2008; Mauricio-Iglesias et al., 2015; Mollerup et al., 2015, 2016; Vezzaro and Grum, 2014); iii) various existing effluent permit standards (FWR, 2012) as well as novel permitting approaches (e.g. Meng et al., 2016) can be easily integrated and evaluated for compliance; and finally, iv) it can provide a layout to include new model features (e.g. sewer biological modelling (Huisman, 2001), micropollutants transport and treatment (Vezzaro et al., 2014; Snip et al., 2014), river sediment model (Reichert et al., 2001)) and evaluate the effects of such improvements.

4.3. Developing site-specific case studies using the BSM-UWS model library

As a model library, the individual model blocks can be used to develop site-specific UWS models (and also for the individual sub-systems). The Matlab/Simulink toolbox is used as the graphical interface for the model development. The first step in developing an UWS model is to assemble various model blocks (sub-catchments, sewer network, storage tanks, aeration tanks, clarifiers, river stretches etc.) to build the specific UWS model. Various physical characteristics for each sub-catchment (area, runoff coefficient, EMC, dry weather pollution loads etc.), sewer system (residence time, storage tank volumes, throttle flows from storage tank, pump capacities etc.), WWTP design values (reactor volumes, wastage rate, recycle rate, oxygen supply etc.) and river characteristics (base flow, length, slope, reaeration rate etc.) can be easily defined/modified using initialization scripts before the beginning of the simulation. Initialization of the WWTP and river biochemical processes can be made by running a dry weather simulation for a long time period (e.g. 100 days). Rainfall time series and solar irradiance data for the river system are given as inputs to the model. Currently, the model considers homogeneous rainfall across all the sub-catchments although it is possible to include rainfall heterogeneity by supplying different input rainfall time series to the different sub-catchments. The model is then ready to be run for the intended simulation period. A performance script can be used to calculate the evaluation criteria for all the sub-systems. Additionally, dynamic time series data for all the state variables (e.g. flow rates, pollution loads at each sub-catchment, storage tank overflows, biological reactor effluent composition, river outputs) is also available in the model workspace. Once the model is developed following the above described steps, any of the above-mentioned applications for the BSM layout can also be performed for the specific real UWS.

4.3.1. Choice of control strategies

The control strategies described in this paper are based on commonly applied control actions on the UWS. DO control is the most commonly applied control strategy in WWTPs and hence included in the paper (Åmand et al., 2013; Olsson, 2012). Although various integrated control strategies exist, the ones selected here are inspired from the existing literature (e.g. Vanrolleghem et al., 2005a; Weyand, 2002; Kroll et al., 2015). Only a limited set of control handles are utilized in the case studies and other potentially effective control options (e.g. control of RWT before WWTP, river reaeration) can be explored by future users. When analysing the control strategies, the set points and control thresholds are obtained using a trial and error method, and hence there exists further scope for optimization of the set points and threshold limits. The choice of these strategies is mainly to demonstrate that the tool can be used to evaluate control strategies of varying complexities across different sub-systems. Hence, the authors choose to present three control strategies that are implemented in: i) WWTP (CL1); ii) Sewer network-River (CL2); and iii) Sewer network-WWTP (CL3). The results of these selected strategies illustrate that the final decision of choosing the best control is not straightforward for the demonstrated control options on the BSM-UWS layout due to the systems complexity. It has not been possible to improve both the river water quality criteria ($T_{exc,DO}$ and $T_{exc,NH3}$) simultaneously and hence the choice of control strategy depends on prioritizing one criterion over the other. In case $T_{exc,NH3}$ is critical, CL1 is the best control strategy. CL3 can be chosen if reducing $T_{exc,DO}$ is the most important objective. CL2 can be considered as a good balance between CL1 and CL3 as it improves $T_{exc,DO}$ without causing major decrease in $T_{exc,NH3}$. The study also highlights the multi-criteria approach that is typically needed when selecting optimal control strategies for a given system (Flores-Alsina et al., 2008). Finally, the comparison between CL2 and CL3 clearly demonstrates that a complex control strategy (in this case CL3) does not necessarily lead to superior system performance.

4.3.2. Assumptions & limitations

There are several model limitations that should be noted before applying the model blocks to other catchments. Hydrological processes in urban catchments like rainfall-runoff are currently described in a simplistic manner. Various phenomena like evapotranspiration, depression losses etc. are not considered (Butler and Davies, 2011). Also, only infiltration to sewers is included and no exfiltration is considered (Rutsch et al., 2008). The catchment model does not have the ability to simulate the effect of various low-impact developments/best management practices (Freni et al., 2010a). The sewer model does not include backwater effects (Wolfs et al., 2013) and other detailed hydraulics (e.g. in-line storage in the sewers, water levels etc.). Hence, it cannot be used to describe complex flow phenomena and urban flooding. Due to the conceptual approach used for modelling the sewer system, a direct calibration of the sewer parameters based on physical characteristics of the sewer system (e.g. Solvi, 2007; van Daal-Rombouts et al., 2016) is not possible. The description of particulate accumulation and washoff in catchments and sewers is not yet fully understood and hence, the model used may not be perfectly suitable for all catchments. Also, biological conversions in the sewer are not included (Huisman, 2001). In the WWTP, no sludge line is included yet (but will soon be) and hence the effect of ammonia and phosphorus recycle from the reject water line is not considered. Including the sludge line substantially increases model complexity, in particular when describing phosphorus and its close interlink with the sulphur and iron cycles (Flores-Alsina et al., 2016; Kazadi Mbamba et al., 2016; Solon et al., 2017). The sediment oxygen consumption is not included in the river model (e.g. Reichert et al., 2001). Due to lack of a sediment layer, the current model is only applicable to shallow waters showing no significant effects of a sediment layer on the chemical quality of the river. Also, any limitations present in the underlying biochemical models (ASM2d, RWQM1) need to be considered. In terms of the evaluation criteria, the operational costs in the sewer system are currently not accounted for (only WWTP operational costs are evaluated) but can easily be updated if information from the pump manufacturer data sheet is available. The evaluation criteria also do not include other novel criteria like greenhouse gas emissions from WWTPs and sewers. Changes to the ASM models in the WWTP and inclusion of biological transformations in the sewer are essential to evaluate GHG emissions. Although the list seems exhaustive as all the model limitations are clearly elaborated, many of these limitations are either currently being addressed or will be addressed in the future.

5. Conclusions

The major contributions of the study are:

- A model library for integrated modelling of UWS (BSM-UWS) which includes a pre-defined urban catchment layout (and the underlying models) and objective performance evaluation criteria is developed.
- The BSM-UWS is capable of simulating: i) dynamics of wastewater generation from catchments during dry as well as rain events (CT); ii) transport of wastewater to the WWTP and discharge of excess stormwater to the river during rain events (SN); iii) physical, chemical and biological aspects of treating the transported wastewater (WWTP); and finally iv) the effect of overflow discharges and WWTP effluent on biological and chemical constituents of the river (RW).
- Using the BSM-UWS platform, several control strategies (local/global) are implemented and their performance analysed using the evaluation criteria defined for sewer, WWTP and river systems.
- The control strategies highlight the complex relationships between the different sub-systems and hence strongly support the need for integrated modelling and control of UWS.
- The described toolbox can be used to develop integrated models for various urban catchments. The integrated model with the pre-defined layout can be used for evaluating local/global control strategies in a holistic manner.

With the freely distributed ready-to-use integrated UWS model, it is expected that the BSM-UWS will play a key role in disseminating the benefits of integrated control and analysis of the UWS to a larger audience.

Acknowledgements

Dr. Flores-Alsina and Mr. Saagi gratefully acknowledge the financial support provided by the People Program (Marie Curie Actions) of the European Union's Seventh Framework Programme FP7/2007–2013 under REA agreement 329349 (PROTEUS) and 289193 (SANITAS), respectively. Dr Flores-Alsina is grateful for the financial support from the REWARD project, funded by the Innovation Fund Denmark (project 1308-0027B) and the collaborative international consortium WATERJPI2015 WATINTECH of the Water Challenges for a Changing World Joint Programming Initiative (Water JPI) 2015 call (project ID 196). The authors gratefully acknowledge the feedback provided by Professor Peter A. Vanrolleghem, Canada Research Chair in Water Quality Modelling, Université Laval, Québec, Canada during the development of the model.

Appendix A. Supplementary data

Supplementary data related to this article can be found at http://dx.doi.org/10.1016/j.envsoft.2017.03.026.

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Supplementary Information

Paper title: A model library for simulation and benchmarking of integrated urban wastewater systems

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Table 1.1: Sewer evaluation criteria at the individual overflow locations in the open loop (OL) case

OL	ST_1	ST_2	ST_4	ST_5	ST_6	Total
N_{ovf} (events/year)	6	2	5	109	13	109
T_{ovf} (h)	5	1	10	2,928	15	2,928
V_{ovf} (m^3)	2,581	111	7,381	163,044	30,277	203,393
OQI (kg poll units/day)	2,604	165	5,045	304,008	30,440	342,262

Table 1.2: Sewer evaluation criteria at the individual overflow locations for CL1

CL1	ST_1	ST_2	ST_4	ST_5	ST_6	Total
N_{ovf} (events/year)	6	2	5	109	13	109
T_{ovf} (h)	5	1	10	2,928	15	2,928
V_{ovf} (m^3)	2,581	111	7,381	163,044	30,277	203,393
OQI (kg poll. units/day)	2,604	165	5,045	304,008	30,440	342,262

Table 1.3: Sewer evaluation criteria at the individual overflow locations for CL2

CL2	ST_1	ST_2	ST_4	ST_5	ST_6	Total
N_{ovf} (events/year)	6	2	5	109	13	109
T_{ovf} (h)	5	1	10	2,928	15	2,928
V_{ovf} (m^3)	2,581	111	7,381	163,044	30,277	203,393
OQI (kg poll. units/day)	2,604	165	5,045	304,008	30,440	342,262

Table 1.4: Sewer evaluation criteria at the individual overflow locations for CL3

CL3	ST_1	ST_2	ST_4	ST_5	ST_6	Total
N_{ovf} (events/year)	6	4	9	109	13	109
T_{ovf} (h)	5	3	14	2,928	16	2,928
V_{ovf} (m^3)	2,955	678	12,422	163,044	28,634	207,733
OQI (kg poll. units/day)	3,097	1,133	9,809	304,008	30,327	348,373

Table 1.5: WWTP evaluation criteria for the different case studies

	OL	CL1	CL2	CL3
IQI (kg poll. units/day)	92,714	92,714	92,714	94,377
EQI (kg poll. units/day)	6,777	6,466	6,409	6,505
EQI_bio (biological treatment)	6,181	5,870	6,129	6,145
EQI_bp (bypass)	596	596	280	360

Table 1.6: River evaluation criteria for the different case studies

	OL	CL1	CL2	CL3
$C_{max,NH3}$ (g/m^3)	0.04	0.03	0.03	0.03
$C_{min,DO}$ (g/m^3)	2.2	2.1	1.9	1.8
$T_{exc,NH3}$ (h)	389	133	148	187
$T_{exc,DO}$ (h)	308	329	291	274

Paper V

KEY CONTROL HANDLES AND DESIGN PARAMETERS FOR IMPROVING RECEIVING WATER QUALITY

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1. INTRODUCTION

Integrated modelling of urban wastewater systems (UWS) is increasingly being recognized as a valuable tool to understand and thereby improve the performance of urban wastewater infrastructure (Bach et al., 2014; Benedetti et al., 2013). Many authors (e.g. Harremoës et al., 1993; House et al., 1993; Beck, 1976) identified the interactions between different sub-systems (catchment, sewer network, wastewater treatment plant (WWTP) and receiving waters) in an UWS and stressed for the development of holistic approaches to improve the quality of receiving waters, instead of focusing on sewer system or WWTP performance individually. Various case studies demonstrating the potential of integrated modelling to optimize the performance of UWS are available in research literature (Weijers et al., 2012; Nielsen & Nielsen, 2005; Sharma et al., 2013; Seggelke et al., 2013; Kroll et al., 2016). Also, several theoretical studies highlight the great potential of integrated modelling tools to effectively improve river water quality by analysing the entire UWS as a single unit (Fu & Butler, 2012; Muschalla, 2008; Rauch & Harremoës, 1999; Vanrolleghem et al., 2005).

In order to provide a common platform for several researchers interested in developing control strategies and evaluating system modifications on an UWS scale, a benchmark simulation model (BSM-UWS) consisting of a model library for all the sub-systems (catchment, sewer system, WWTP and river water system) and a pre-defined UWS layout is developed (Saagi et al., 2016) . As such models are highly complex with a lot of interactions, it is difficult to determine the most influential control handles and design parameters that will lead to improvements in the river water quality. Hence, a global sensitivity analysis (GSA) is provided to identify the effects of various input factors (e.g. storage tank volumes, pumping rates) on the outputs (e.g. WWTP effluent quality, river water quality). The most common approaches for enhancing the performance of an UWS are: i) implementing control/operational strategies using the available capacity; and/or ii) upgrading the design capacity of the existing infrastructure. Therefore, the study focuses on identifying influential: i) operational/control handles; and ii) design parameters influencing river water quality as well as the sewer system and WWTP performance.

Additionally, a survey of the existing literature on GSA for integrated models (focusing on control handles and design parameters) highlighted that such an analysis is rarely performed. Fu et al. (2009) and Astaraie-Imani et al. (2012) employed GSA on a semi-hypothetical case study in the UK to determine the effects of urbanization, climate change and operational parameters on river water quality. Only a limited set of control handles are evaluated in these studies and a short term rainfall time series (six days) is used for the simulations. Langeveld et al. (2013) employed a more thorough GSA to determine different control handles that have influence on river ammonia and DO concentrations. Three different rain events (with different return periods) are used to study the response of the modelled river system with an 11 day evaluation period for each scenario. It can be noticed from the above examples that short term rainfall data is used as an alternative to long term simulations, mainly to reduce computational costs. However, such short term evaluation periods may lead to underestimating the importance of processes that have long time constants (e.g. biological processes in WWTPs). Hence, an additional objective of this study is to assess the influence of evaluation period (short term vs long term) and rainfall intensity on the results of the GSA.

The paper presents an overview of the BSM-UWS layout and the evaluation criteria used for the analysis. Morris screening is used as the GSA method of choice owing to its low computational foot print. Various control handles/design parameters in the sewer system and WWTP and their uncertainty ranges are described. GSA is performed separately to determine the influential control handles and design parameters using a long term rainfall evaluation period (100 days). Additionally, a comparison is made between the GSA results for control handles using i) three different short term rainfall time series (5 days) with varying intensities; and ii) the long term evaluation (100 days). The results from this study can provide valuable information to future BSM-UWS users interested in developing integrated control strategies and system modifications. The knowledge obtained can further strengthen our understanding of the interactions between the different sub-systems and the need for holistic evaluation using river water quality criteria instead of focusing on optimizing the sub-systems. Additionally, implications of the choice of evaluation period on the sensitivity analysis results can be understood.

2. METHODS

2.1. Urban wastewater system layout

The BSM-UWS layout consists of: i) an urban catchment (with different sub-catchments) that generate sewage as well as stormwater; ii) a sewer system transporting the generated wastewater from all the sub-catchments to the treatment facility; iii) a WWTP where different physical and biological unit operations are used to remove the pollutants from wastewater; and finally iv) a river system into which sewer overflows and treated effluent from the WWTP are discharged.

Table 1: Catchment characteristics and sewer storage volumes for the BSM-UWS layout.

Sub-catchment	Area (ha)	PE	DWF ($m^3.d^{-1}$)		Storage (m^3)
			DOM	IND	
1	99	15 920	2 390		5 000
2	21	3 920	590	2 500	1 000
3	29	2 960	440		
4	71	9 600	1 440		4 400
5	71	7 840	1 180		3 600
6	249	39 760	5 960		8 100
Total	**540**	**80 000**	**12 000**	**2 500**	**22 100**

DWF: Dry weather flow; DOM: Domestic; IND: Industrial

The hypothetical urban catchment (adapted from ATV, 1992; Schütze et al., 2011) consists of six sub-catchments ($SC_1 \ldots SC_6$) with a total area of 540 hectares and 80 000 population equivalents. The characteristics of the individual sub-catchments are described in Table 1. During dry weather, the daily average wastewater generation is 19 000 $m^3.d^{-1}$ with the contribution from domestic and industrial sources being 12 000 $m^3.d^{-1}$ and 2 500 $m^3.d^{-1}$, respectively. Daily average infiltration to sewers is assumed to be 4 500 $m^3.d^{-1}$. While all sub-catchments generate domestic wastewater, SC_2 has an industrial source as well.

The sewer network consists of a combination of combined as well as separate sewer systems. Five sub-catchments (SC_1, SC_2, SC_3, SC_4 and SC_6) are connected to a combined sewer system, whereas SC_5 is connected to a separate sewer network. Five storage tanks are located in different sub-catchments (SC_1, SC_2, SC_4, SC_5 and SC_6). Total storage volume available is 22 100 m^3 (approx. 40 $m^3.ha^{-1}$ of catchment area). Online pass-through tanks are used at four locations (ST_1, ST_2, ST_5 and ST_6) whereas ST_4 is an offline bypass tank. The outflow from online tanks is regulated by throttle valves/pumps and that from AN offline tank is regulated by a pump with fixed pumping capacity. Sewer overflows from all the storage tanks are discharged into the river. Individual storage volume for each tank is mentioned in Table 1.

The WWTP consists of an extended BSM1-ASM2d plant layout (Flores-Alsina et al., 2012). The biological section includes: i) two anaerobic tanks ($ANAER_1$, $ANAER_2$) (2 x 1 000 m^3) for biological phosphorus removal; ii) two anoxic tanks ($ANOX_1$, $ANOX_2$) (2 x 1 500 m^3) that form the denitrification zone; and iii) three aerobic tanks (AER_1, AER_2, AER_3) (3 x 3 000 m^3) for nitrification and organic matter removal. A primary clarifier (PC) (900 m^3) and a secondary clarifier (Sec.C) (area = 2 500 m^2) are used for separation of sludge and particulates before and after the biological reactors, respectively. A rainwater storage tank (RST) (8 000 m^3) before the primary clarifier helps balance peak loads and store excess rain water. In order to protect the WWTP from peak wet weather flows, two bypass facilities (BP_1, BP_2) (before and after the primary clarifier) are included. BP_1 has a threshold of 90 000 $m^3.d^{-1}$ (any flow in excess of the threshold is bypassed) and BP_2 has a threshold of 70 000 $m^3.d^{-1}$.

A 30 km long shallow river runs across the urban catchment and receives sewer overflows (OVF_1-RW_1, OVF_2-RW_4, OVF_4-RW_7, OVF_5-RW_8, OVF_6-RW_{11}) and WWTP effluent ($WWTP_{eff}$-RW_{16}). The river has a mean annual base flow rate of 72 500 $m^3.d^{-1}$. Additional runoff from an upstream catchment (area = 500 ha) reaches the river during rain events. The river has a uniform bottom width of 7 m and is assumed to be trapezoidal in shape.

2.2. Evaluation criteria

Several evaluation criteria exist for different sub-systems of the UWS. In this case study, a selective list of two criteria each for sewer and WWTP performance and four criteria for river water quality are used.

- Sewer system
  i. Total overflow volume (V_{ovf}, m^3): This is the total overflow volume from all the overflows in the catchment.
  ii. Overflow quality index (OQI, kg poll. units.d^{-1}): This is an aggregated pollution index that considers the total pollution load for five different pollutants (COD, BOD, NH_4, NO_3, PO_4).

OQI is the weighted sum of the individual pollution loads. The weight for each pollutant is the same as that used in the BSM WWTP criteria described below.

- WWTP

  iii. Influent quality index (*IQI*, kg poll. units.d^{-1}): This is the weighted sum of all the major pollutants at the inlet of the WWTP. This represents the daily influent pollutant load to the WWTP.

  iv. Effluent quality index (*EQI*, kg poll units.d^{-1}): It is calculated in the same manner as *OQI* and *IQI*. The *EQI* considers the pollutant load both from bypass and the effluent from the secondary clarifier.

- River

  v. Exceedance duration for NH_3 ($T_{exc,NH3}$, hrs): This is the duration for which the un-ionized ammonia (NH_3) concentration is above a particular threshold value (= 0.018 g N.m^{-3}).

  vi. Exceedance duration for DO ($T_{exc,DO}$, hrs): This is the duration for which the dissolved oxygen (DO) concentration is below a particular threshold value. (= 6 g.m^{-3}).

  vii. Maximum concentration for NH_3 ($C_{max,NH3}$, g N.m^{-3}): This is the maximum concentration of NH_3 at any river stretch continuously for at least 1 hour.

  viii. Minimum concentration for DO ($C_{min,DO}$, g.m^{-3}): This is the minimum concentration of DO at any river stretch continuously for at least 1 hour.

2.3. Global sensitivity analysis

Morris screening

Morris screening is a computationally efficient tool to perform GSA studies, especially for large input factor sets. It offers the advantage of a global analysis at a cost similar to local one-at-at-time sensitivity analysis. The method uses the concept of "elementary effects" to analyse the impact of various input factors on the outputs. Consider *k* input factors ($X_1 \ldots X_k$) leading to an output *Y*. The elementary effect for input factor X_i at a point X_{i0} in the input space is given by:

$$EE_{\mathrm{i}} = \frac{[Y(X_1, X_2, X_{\mathrm{i}-1}, X_{\mathrm{i}} + \Delta) - Y(X_1 \ldots X_{\mathrm{k}})]}{\Delta}$$

where Δ is the change in X_i between two different model runs. The input factor space for each input X_i is varied across p levels. Assuming that p is even, a value of $\Delta = p/2(p-1)$ guarantees uniform sampling across the input space. After *r* such repetitions, a distribution of elementary effects (EE_i) for the input factor X_i is obtained by randomly sampling the input space for X_i. The mean (μ) and standard deviation (σ) of this distribution are the sensitivity measures for the given input X_i towards the output *Y*. A higher mean indicates that the parameter is important and a higher standard deviation shows that there are interactions and non-linearities affecting the input factor. In general μ and σ are plotted against each other. A wedge is plotted on the graph for $\mu = \pm 2\, \sigma/\sqrt{r}$. Any factor outside the wedge is considered to be influential. The factors inside are considered non-influential. A modification of the measure is to use $\mu*$, which is the mean of the distribution of absolute values for elementary effects. The advantages of using $\mu*$ are: i) parameters can now be ranked; ii) it will avoid type II errors which can be caused by two elementary effects of opposite signs cancelling each other and resulting in a small mean value. In general, a combination of μ vs σ and $\mu*$ values can provide insights into the ranking of the factors and also any non-linearites and interactions in the factors.

Input factors and uncertainty framing

Morris screening is performed for two different sets of input factors, namely:

i. Control handles;

ii. Design parameters.

The list of input factors for both sets is presented in Table 2. It is assumed that the control handles have a high uncertainty (25 %) as it should be possible to operate them with a wide range of variation. Although even higher uncertainty range was used for control handles in the earlier studies (Astaraie-Imani et al., 2012; Langeveld et al., 2013), the variation is limited to 25 % to ensure that extreme and improbable values for the control handles are not considered. The uncertainty in design values is limited to 10 %. Similar range for design parameters is used in Sin et al. (2009). With 15 input factors (k) and 50 repetitions (r), a total of 800 simulations ($r\times$ (k+1)) are performed for each set of input factors. Finally, only input factors with the $\mu*$ values higher than 0.1 are considered for evaluation of results.

Table 2: List of input factors for the two (control handles and design parameters) GSA studies.

CONTROL HANDLES (25 % variation)			**DESIGN PARAMETERS (10 % variation)**		
Section	**Input factor**	**Description**	**Section**	**Input factor**	**Description**
Sewer	$Q_{pump,ST1}$	Max. pump capacity for ST_1 ($m^3.d^{-1}$)	**Sewer**	V_{ST1}	ST_1 volume (m^3)
	$Q_{max,ST2}$	Max. throttle flow rate for ST_2 ($m^3.d^{-1}$)		V_{ST2}	ST_2 volume (m^3)
	$Q_{pump,ST4}$	Max. pump capacity for ST_4 ($m^3.d^{-1}$)		V_{ST4}	ST_4 volume (m^3)
	$Q_{throttle,ST4}$	Max. throttle flow rate for ST_4 ($m^3.d^{-1}$)		V_{ST5}	ST_5 volume (m^3)
	$Q_{max,ST5}$	Max. throttle flow rate for ST_5 ($m^3.d^{-1}$)		V_{ST6}	ST_6 volume (m^3)
	$Q_{max,ST6}$	Max. throttle flow rate for ST_6 ($m^3.d^{-1}$)	**WWTP**	V_{RST}	Rain storage tank volume (m^3)
WWTP	$Q_{max,RST}$	Max. throttle flow rate for RST ($m^3.d^{-1}$)		V_{PC}	Primary clarifier volume (m^3)
	Q_{BP1}	Max. flow rate after BP_1 ($m^3.d^{-1}$)		A_{SC}	Secondary settler area (m^2)
	Q_{BP2}	Max. flow rate after BP_2 ($m^3.d^{-1}$)		V_{ANAER1}	Anaerobic reactor 1 volume (m^3)
	Q_r	Sludge recycle rate ($m^3.d^{-1}$)		V_{ANAER2}	Anaerobic reactor 2 volume (m^3)
	Q_w	Sludge wastage rate ($m^3.d^{-1}$)		V_{ANOX1}	Anoxic reactor 1 volume (m^3)
	Q_{intr}	Internal recirculation rate ($m^3.d^{-1}$)		V_{ANOX2}	Anoxic reactor 2 volume (m^3)
	K_La_1	Oxygen transfer coefficient for AER_1 (d^{-1})		V_{AER1}	Aerobic reactor 1 volume (m^3)
	K_La_2	Oxygen transfer coefficient for AER_2 (d^{-1})		V_{AER2}	Aerobic reactor 2 volume (m^3)
	K_La_3	Oxygen transfer coefficient for AER_3 (d^{-1})		V_{AER3}	Aerobic reactor 3 volume (m^3)

Additionally, in order to compare the impact of evaluation period on the sensitivity analysis results, three different rainfall time series (5 days evaluation period) are chosen that consist of a single rain event each (4 hour duration) with varying return periods (0.5 year, 2 years and 5 years) based on historic data from Copenhagen, Denmark (Figure 1a, 1b & 1c). GSA is performed for identifying the most influential control handles under each rainfall time series and the results are compared with the long term evaluation.

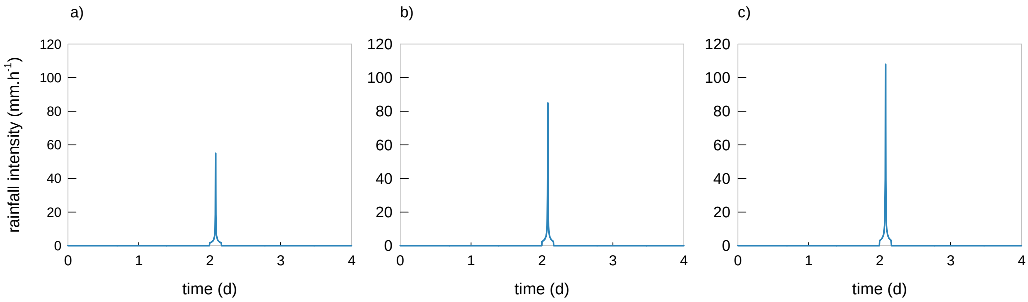

Figure 1: Rainfall data for the three different return periods (0.5 year (a), 2 years (b) and 5 years (c)).

3. RESULTS

3.1. Control handles

The influential control handles for both overflow volume (V_{ovf}) and overflow quality index (*OQI*) are $Q_{max,ST6}$ and $Q_{throttle,ST4}$. Also, the $\mu*$ values are similar for both cases (Figure 2) indicating a correlation between V_{ovf} and *OQI*. Hence, it can be said that, for the chosen uncertainty range, the control handles available for manipulating sewer performance are limited (Figure 2a & 2b).

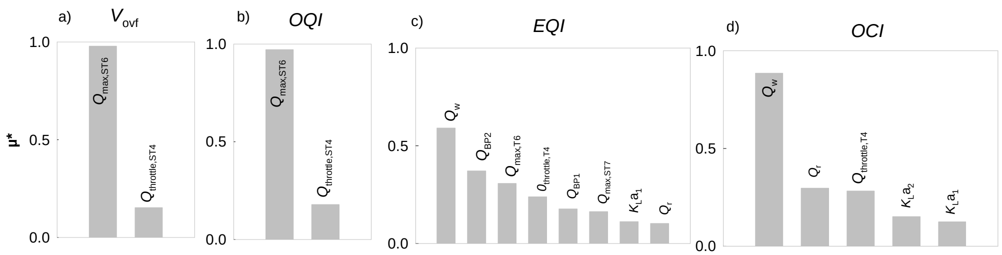

Figure 2: Influential control handles for sewer and WWTP performance criteria – V_{ovf} (a), *OQI* (b), *EQI* (c) and *OCI* (d).

The WWTP criteria *EQI* and *OCI* are affected by different input factors (Figure 2c & 2d). While *EQI* is influenced by a wide number of WWTP control handles (Q_w, Q_{BP2}, $Q_{max,RST}$, Q_{BP1}, K_La_1 and Q_r) and sewer control handles ($Q_{max,ST6}$ and $Q_{throttle,ST4}$), *OCI* is understandably mainly influenced by pumping (Q_w and Q_r) and oxygen supply (K_La_1 and K_La_2) in the WWTP. Also, the sewer control handle $Q_{throttle,ST4}$ influences *OCI* although no direct cost is calculated related to it. It is interesting to note that the influence of $Q_{throttle,ST4}$ is higher than that of aeration cost for the individual reactors (K_La_1 and K_La_2).

In terms of river water quality, $C_{max,NH3}$ and $T_{exc,NH3}$ have similar influential control handles but differ in their rankings (Figure 3a & 3b). These two criteria are strongly influenced by WWTP control handles with $Q_{throttle,ST4}$ being the only important sewer control handle. Control handles in the sewer system and WWTP impact river DO criteria ($C_{min,DO}$ and $T_{exc,DO}$). Sewer control handles – $Q_{max,ST6}$ and $Q_{throttle,ST4}$ and WWTP control handles – Q_w, Q_{BP1}, Q_{BP2} and K_La_1 impact both the criteria. $Q_{pump,ST1}$ and $Q_{max,ST2}$ influence only $C_{min,DO}$ while $Q_{max,RST}$ and K_La_1 impact $T_{exc,DO}$. (Figure 3c & 3d) Hence, while the river ammonia quality is mainly affected by WWTP control handles, the DO quality criteria are influenced both by the sewer system and WWTP controls. As the wet weather ammonia load from the catchment is limited (only constant *EMC* values are used), the major ammonia discharge to the river happens at the WWTP while the overflows as well as WWTP effluent (and bypass) contribute to the organic loads reaching the river and hence the difference in influencing control handles for the two criteria.

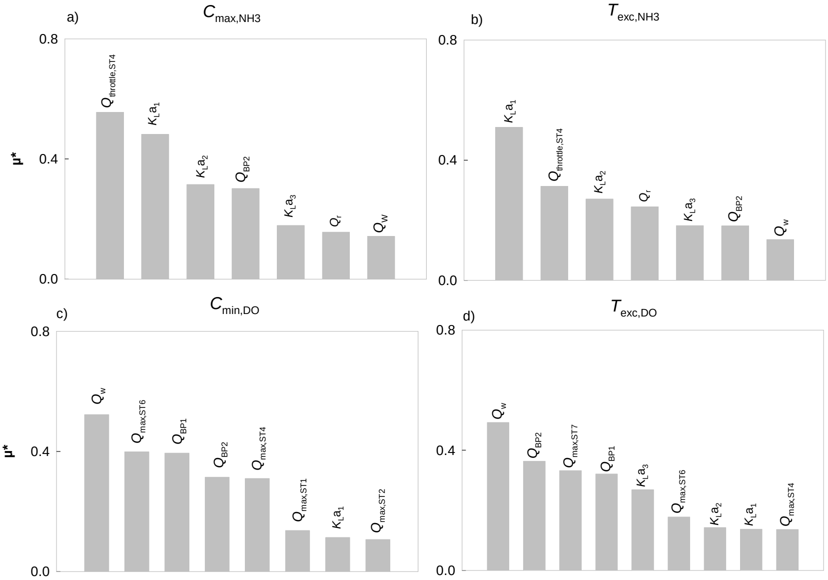

Figure 3: Important control handles for river water quality in terms of NH_3 ($C_{max,NH3}$ (a) and $T_{exc,NH3}$ (b)) and DO ($C_{min,DO}$ (c) and $T_{exc,DO}$ (d)).

3.2. Design parameters

The overflow volume (V_{ovf}) and pollutant quality (*OQI*) are influenced by the three large storage tank volumes (V_{ST6}, V_{ST4} and V_{ST1}) (Figure 4a & 4b). Although ST_5 is similar in volume to ST_4, it does not influence V_{ovf} and *OQI* as it is a separate sewer system and all the flow from rain events will eventually lead to overflows.

Most of the design parameters that are influential on *EQI* and *OCI* are the same although their ranking is different (Figure 4c & 4d). The only minor difference is that while V_{RST} is influential for *EQI*, V_{AER3} is influential for *OCI*. Only WWTP design parameters have a strong influence on both these WWTP performance criteria. Also, it should be noted that V_{SC} contributes to a large part of the variation in *EQI* while there is no dominating input factor that contributes to the variation in *OCI*. All the sensitive input factors contribute similarly to the uncertainty in *OCI*.

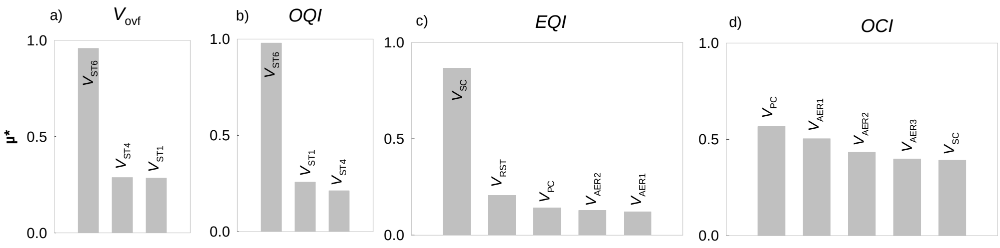

Figure 4: Influential design parameters affecting sewer (V_{ovf} (a) and *OQI* (b)) and WWTP performance (*EQI* (c) and *OCI* (d)).

In terms of river quality, $T_{exc,NH3}$ and $C_{max,NH3}$ are influenced by the volume of the aeration tanks (V_{AER1}, V_{AER2} and V_{AER3}) and the primary clarifier (V_{PC}). Additionally, $C_{max,NH3}$ is also affected by the volume of the secondary clarifier (V_{SC}) (Figure 5a & 5b). This indicates that the river ammonia quality is strongly impacted by the nitrification capacity in the WWTP and the influence of sewer overflows on river ammonia is limited for this particular UWS.

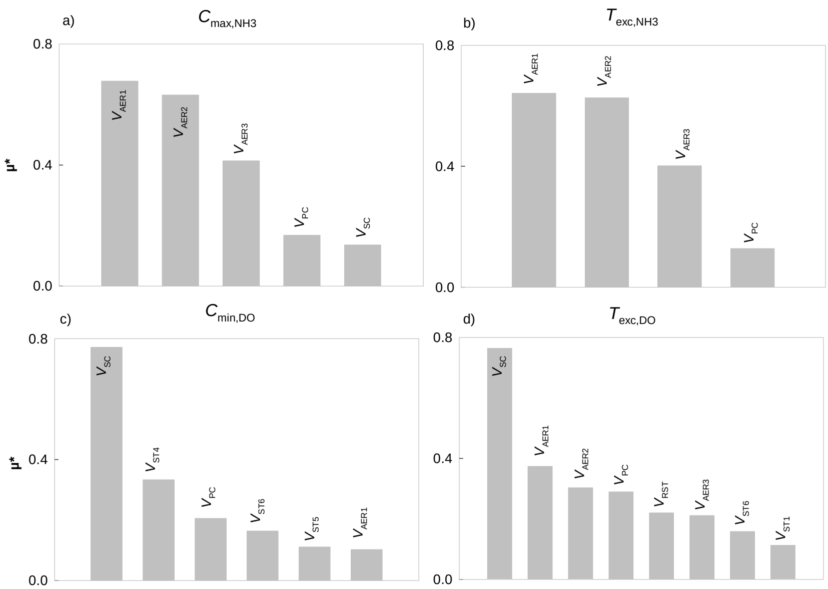

Figure 5: Influential design parameters for river water quality criteria ($C_{max,NH3}$ (a), $T_{exc,NH3}$ (b), $C_{min,DO}$ (c) and $T_{exc,DO}$ (d)).

The important parameters affecting river DO are not always the same as those for NH_3. V_{SC} is the most influential parameter for both $C_{min,DO}$ and $T_{exc,DO}$. Other important factors for these criteria are different (Figure 5c & 5d). For $C_{min,DO}$, storage tank volumes (V_{ST4}, V_{ST5} and V_{ST6}) are important design parameters from the sewer system. The volume of the primary clarifier (V_{PC}) and that of the first aeration tank (V_{AER1}) are important factors from the WWTP. In case of $T_{exc,DO}$, while V_{PC}, V_{AER1} and V_{ST6} are also important, their ranking is different than that for $C_{min,DO}$. Other important factors are V_{ST1} and V_{RST}. It can be seen that river NH_3 quality is mainly influenced by WWTP design whereas DO quality is affected by both WWTP and sewer design. Also, it can be clearly seen that the most important parameters for sewer performance, WWTP effluent quality and river water quality are different.

3.3. Comparison between short term and long term evaluation

For the sewer evaluation criteria, the cumulative list of influential control handles identified from using all the three rain events is identical to the active control handles determined using a long term simulation ($Q_{max,ST6}$ and $Q_{throttle,ST4}$) (Figure 6a & 6b). The ranking as well as the sensitivity (μ* value) are also similar. For the *OQI*, all three short rain events as well as the long term evaluation gave identical results in terms of the list of control handles and their ranking. Hence, the sewer evaluation criteria have identical influential control handles and they can be determined using a combination of different rain events instead of using a long term evaluation.

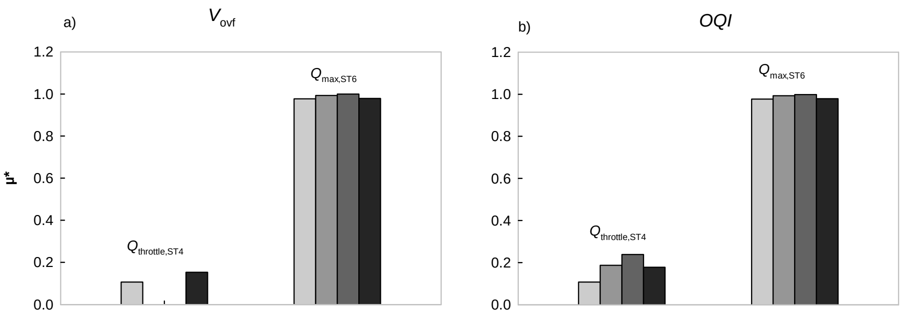

Figure 6: Comparison of GSA results for sewer performance criteria (V_{ovf} (a) and *OQI* (b)). The μ* values from three different short rainfall series are compared with one long term evaluation.

For the WWTP performance criteria in terms of *EQI*, firstly, the results from the different rainfall events are not similar (Figure 7a). For the half year rain event only three influential control handles are determined ($Q_{max,ST6}$, Q_{BP1} and Q_{BP2}) while the two years and five year rain events also list these three control handles as the most influential, but additional WWTP control handles are also determined to be influential (Q_w, Q_r, Q_{intr} and K_La_1). Although, the list of control handles cumulatively from all rain events is similar to that from the long term evaluation, the ranking is different. Two additional control handles ($Q_{max,RST}$ and $Q_{throttle,ST4}$) appear only in the long term evaluation. For *OCI*, while all single rain events identify the same set of control handles as well as identical ranking for the important control handles, they could not identify $Q_{throttle,ST6}$ as an influential control handle when compared to the long term evaluation (Figure 7b). With the exception of $Q_{throttle,ST4}$, the list of control handles from the three single rain events and the long term evaluation is identical. Hence, for the WWTP performance, the short term rain events could only identify a limited set of influential control handles.

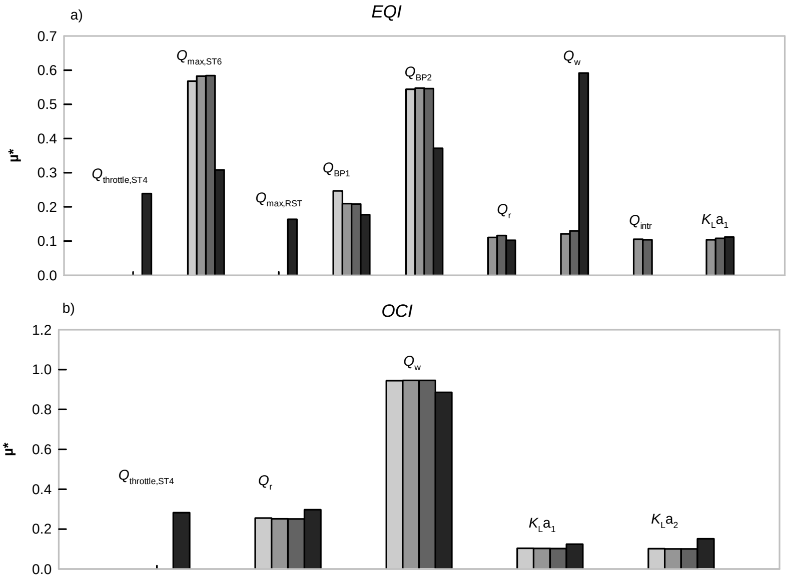

Figure 7: The GSA results for WWTP performance (*EQI* (a) and *OCI* (b)) from long term evaluation compared with those from three short term rainfall series.

For the river quality criteria $C_{max,NH3}$, all three single rain events have identical influential control handles (with the exception of $Q_{pump,ST1}$ for the half year rain event) although with differences in ranking (Figure 8a). However, there is a major difference in ranking as well as the list of handles when compared with the long term evaluation (only Q_{BP1} is the common control handles between the short term evaluations and the 100 day scenario). For $T_{exc,NH3}$, there is a common set of control handles identified both by the short term as well as long term rain scenarios (Q_{BP2}, K_La_1 and K_La_2) (Figure 8b). Other influential control handles in the short term ($Q_{max,RST}$ and $Q_{max,ST6}$) do not appear in the list from the 100 days case. As noticed in the earlier cases, the control handle $Q_{throttle,ST4}$ gains more importance during long term simulation. Other control handles identified only by the long term rainfall are Q_w, Q_r and K_La_3. For $C_{min,DO}$, all control handles identified by short term rain events also appear in the long term scenario. However, Q_w is the most influential parameter in the long term and does not appear in the short term evaluation (Figure 8c). Other influential control handles missing from the short term evaluations are $Q_{max,ST2}$ and K_La_1. For $T_{exc,DO}$, the cumulative list of parameters from all short term events represents most of the control handles from the 100 day evaluation except for Q_w (the most influential for long term) and K_La_2 (Figure 8d). For the river system, the importance of WWTP control handles has increased when long term evaluation is used while the short term scenarios over-estimate the sewer control handles influence.

In conclusion, it should be stressed that a replacement of the long term scenario by a limited number of short term rainfall events does not always reflect the actual list of influential control handles that are identified using the long term evaluation. In the case of sewer system evaluation, this assumption seems to hold but does not in the case of the WWTP and especially the river system.

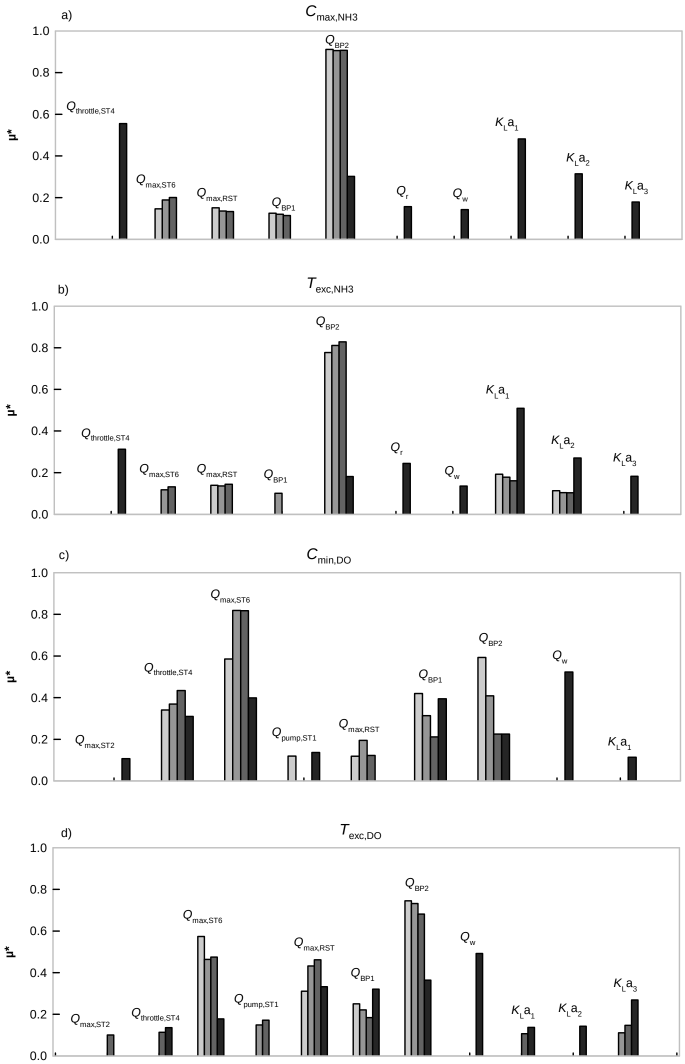

Figure 8: GSA results comparison for river water quality criteria ($C_{max,NH3}$ (a), $T_{exc,NH3}$ (b), $C_{min,DO}$ (c) and $T_{exc,DO}$ (d)).

3.4. Convergence analysis

In order to make sure that convergence (similar results with increasing number of repetitions) is achieved during the GSA runs, an evaluation of the model results, as described by Vanrolleghem et al. (2015) is performed for all the scenarios for design and control handles. All results converge (Figure 9a & 9b). The convergence is achieved after about 15-20 repetitions although higher repetition number is used in this study. The results differ from the conclusions in Vanrolleghem et al. (2015) regarding the convergence of Morris screening and are similar to those observed by Kroll et al. (2016).

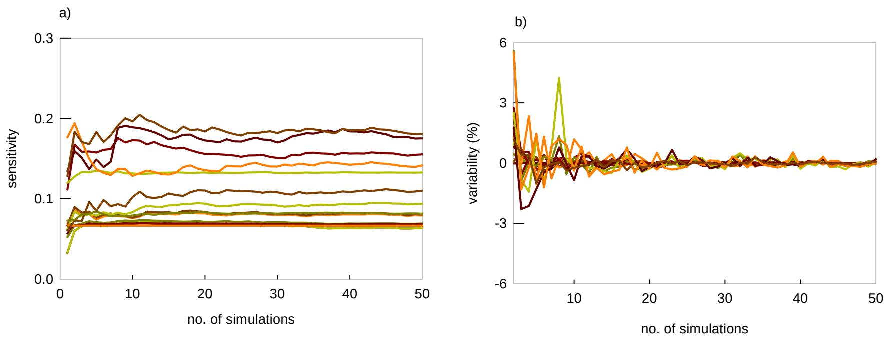

Figure 9: Total sensitivity (a) and the corresponding variability (b) for different evaluation criteria.

4. CONCLUSIONS

The major contributions of the study are:

- The most influential control handles and design parameters influencing river water quality for the BSM-UWS layout are identified.
- Input factors (control handles and design parameters) affecting the uncertainty in the performance of the sub-systems (sewer and WWTP) does not necessarily have the same influence on river water quality criteria. Hence, the most influential control handles and design parameters for sewer system/WWTP performance do not always have a similar impact on river water quality criteria.
- A combination of different short term simulation events is sufficient to replace a long term simulation to perform GSA for the sewer system performance but does not produce similar results for the WWTP and river water quality criteria.

ACKNOWLEDGEMENTS

Dr. Flores-Alsina and Mr. Saagi gratefully acknowledge the financial support provided by the People Program (Marie Curie Actions) of the European Union's Seventh Framework Programme FP7/2007-2013 under REA agreement 329349 (PROTEUS) and 289193 (SANITAS), respectively. Dr Flores-Alsina is grateful for the financial support from the REWARD project, funded by the Innovation Fund Denmark (project 1308-0027B) and the collaborative international consortium WATERJPI2015 WATINTECH of the Water Challenges for a Changing World Joint Programming Initiative (Water JPI) 2015 call (project ID 196).

The authors gratefully acknowledge the feedback provided by Professor Peter A. Vanrolleghem, Canada Research Chair in Water Quality Modelling, Université Laval, Québec, Canada during the development of the model.

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Printed by Media-Tryck, Lund 2017 NORDIC ECOLABEL, 3041 0903

LUND UNIVERSITY

Faculty of Engineering
Department of Biomedical Engineering
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ISBN 978-91-88934-81-9
CODEN: LUTEDX/(TEIE-1083)/1-208/(2017)