Probabilistic modelling and evaluation of wastewater treatment plant upgrades in a water quality based evaluation context

Lorenzo Benedetti, Webbey De Keyser, Ingmar Nopens and Peter A. Vanrolleghem

ABSTRACT

Process choice and dimensioning of wastewater treatment plants (WWTPs) is difficult while ensuring regulatory standards are met and cost-efficiency is maintained. This step only accounts for a small fraction of the upfront costs, but can lead to substantial savings. This paper illustrates the results of a systematic methodology to evaluate system upgrade options by means of dynamic modelling. In contrast to conventional practice, the presented approach allows the most appropriate trade-off between cost of measures and effluent quality to be chosen and the reliability of a process layout to be assessed by means of uncertainty analysis. In a hypothetical case study, thirteen WWTP upgrade options are compared in terms of their effluent quality and economic performance. A further comparison of two options with regard to the resulting receiving water quality reveals the paramount importance of this aspect, and highlights the inadequacy of evaluation frameworks limited to the performance relative to a sub-system (WWTP effluent) when a wider perspective (as induced by the EU Water Framework Directive) has to be adopted.

Key words | cost efficiency, integrated modelling, probabilistic design, uncertainty assessment, Water Framework Directive, water quality

Lorenzo Benedetti (corresponding author)
Webbey De Keyser
Ingmar Nopens
BIOMATH, Ghent University,
Coupure Links 653,
9000 Ghent,
Belgium
E-mail: *lorenzo.benedetti@ugent.be*

Peter A. Vanrolleghem
modelEAU, Département de Génie Civil,
Pavillon Pouliot, Université Laval,
Québec, QC, G1K 7P4,
Canada

INTRODUCTION

The introduction of the EU Water Framework Directive (CEC 2000)–which enforces good practices long advocated (Lijklema 1993)–requires compliance both with effluent quality standards and with receiving water quality standards. The US Federal Clean Water Act of 1972 also advocates the use of emissions load allocation on a river basin scale. Therefore, the boundaries of the system to be managed expand from single structures (e.g. wastewater treatment plant) or sectors (e.g. agriculture) to all activities affecting the water environment in the river basin.

This increased complexity implies that the evaluation of the impact of pollution mitigation measures on the water quality should be evaluated with instruments able to cope with such complexity both from the methodological point of view–by developing and applying systems analysis and modelling uncertainty assessment tools–and by making the developed methodology applicable in practice by means of adequate software tools.

Urban wastewater systems (UWWSs) are crucial components of river basins, since they usually contribute significantly to the pollution loads affecting the receiving water body. They also have more flexibility in their operation and management than other subsystems such as agriculture.

On the one hand, the question of where to improve the UWWS can be answered by means of systems analysis.

doi: 10.2166/hydro.2010.011

This allows to identify where pressures exist and potential measures can be successful within a river basin (Benedetti *et al.* 2006, 2008*a*) to be identified.

On the other hand (the focus of this paper), we also need to address the question of how to improve the UWWS. For this, we propose a systematic methodology to design upgrade measures. In this paper, the methodology is illustrated by means of a hypothetical but realistic example of a wastewater treatment plant (WWTP) upgrade. The evaluation of the options is divided into emission-based criteria (considering the quality of the plant effluent), immission-based criteria (judging on the basis of the receiving water quality, in this case a river stretch) and economic criteria (capital and operational costs).

The evaluation of all criteria is performed probabilistically by means of the propagation of parameter uncertainty into output uncertainty, in order to assess the risk of non-compliance with regulatory limits. Such probabilistic approaches are becoming quite popular, in particular regarding decision support in river management (de Kort & Booji 2007; McCormick *et al.* 2007; Reichert *et al.* 2007). This concept already has a history of three decades in electronics and structural design. The first applications in water engineering had to wait for another decade had to pass (e.g. Melching 1995; Tchobanoglous *et al.* 1996; Rousseau *et al.* 2001).

Previous work which contributed to the development of integrated modelling, especially dealing with transient events, include Bauwens *et al.* (1996), Vanrolleghem *et al.* (1996*a*, 2005*a*) and Meirlaen *et al.* (2001). Other approaches to tackle the problem were presented by Achleitner *et al.* (2007)–which can be considered a continuation of the earlier work of Rauch & Harremoës (1999), with the CITY DRAIN open source software allowing to run long-term simulations of simple models of the urban drainage system to optimise its global performance towards receiving water quality–and by the group working on the SYNOPSIS software–specifically developed to evaluate the benefits of integrated real-time control, with the application of genetic algorithms for multi-objective optimisation of integrated real-time control–e.g. Schütze *et al.* (1999), Butler & Schütze (2005) and Fu *et al.* (2008). Mannina *et al.* (2006) included sensitivity and uncertainty analysis concepts in calibration of integrated models.

However, such publications did not aim to establish a methodology to exploit the capabilities of the developed models and software tools and do not include probabilistic design aspects, as this paper does.

MATERIALS AND METHODS

The evaluation methodology proposed by Benedetti (2006) requires that: (1) a sufficiently long and representative influent time series be provided to the WWTP model, in order to consider the process influent disturbances at different timescales (from minutes in 'first flush' effect to months in infiltration); (2) the WWTP upgrades are modelled; (3) the river is modelled; (4) the WWTP and river models are integrated; (5) the model uncertainties are characterized and propagated to the model outputs by means of Monte Carlo simulations (since uncertainty in WWTP model predictions is considered to be large, it must always be quantified), and probabilistic simulation results are evaluated from economic and environmental points of view. Finally, options for implementation are decided upon (see Figure 1). Each step is described in this section. As for the software tools used and developed for the methodology, the reader is referred to Benedetti *et al.* (2008*a*).

This type of probabilistic analysis on modelled WWTP effluent is definitively complementary to the reliability analysis of real WWTPs (Oliveira & Von Sperling 2008; Bott *et al.* 2009; Parker *et al.* 2009). Reliability can be defined as the percentage of time at which the expected effluent concentrations comply with specified discharge standards or treatment targets. Applications of reliability analysis include the introduction of stochastic concepts into the design process and selection of appropriate parameters for use in the operation of processes, based on reasonable expectation of performance. This may be done from past experience or using the results from other similar treatment plants. Probabilistic considerations may be introduced both in design and in setting discharge requirements. The probability of failure is extremely sensitive to the distribution function of the effluent concentration. Actual WWTP effluent distributions may help validating the modelling results and improve their predictions.

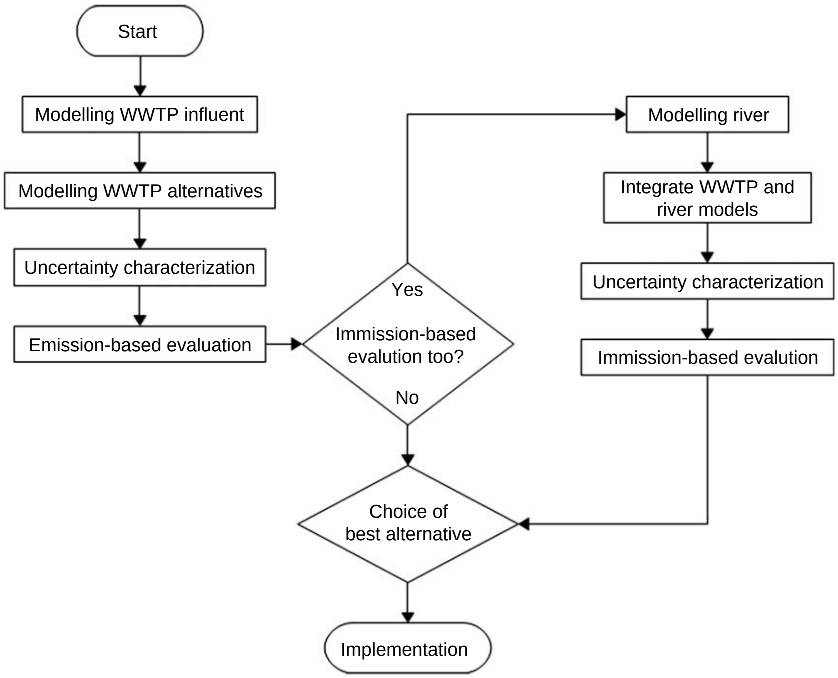


Figure 1 | Methodology flow chart.

Modelling the WWTP influent

The first step consists of the generation of influent time series to be fed to the WWTP models. This is done by submitting an actual rain series to a phenomenological dynamic model of the draining catchment and sewer system, taking into account the number of inhabitants, the presence of industry, the loads per capita of households and industry (see Table 1), the size of the catchment, the length of the sewer system, rainfall data and the interactions with groundwater (infiltration). One year time series with data every 15 minutes are produced which realistically represent the influent dynamics with time scales varying from minutes (e.g. first flush effect) to months (e.g. seasonality in infiltration rate). For a description of this dynamic influent generation model, see Gernaey *et al.* (2006).

Modelling WWTP upgrades

Thirteen options to upgrade a low loaded activated sludge (LLAS) system were selected for evaluation, partly requiring real-time control (RTC) and partly the construction of additional treatment volumes. All configurations were modelled by using a slightly modified (Gernaey & Jørgensen 2004) Activated Sludge Model no.2 (ASM2d) (Henze *et al.* 2000) to describe the dynamics of the activated sludge processes and were implemented in the WEST modelling and simulation software (MOSTforWATER NV, Kortrijk, Belgium) (Vanhooren *et al.* 2003), which allows for high flexibility of use and short calculation time (Claeys *et al.* 2006*b*). Figure 2 shows the general WWTP layout implemented in WEST, which includes a combined sewer overflow (CSO) splitter and a by-pass with a storm tank.

Table 1 | Domestic loads for Continental and Mediterranean climates

Climate	COD soluble (g/d/PE)	COD particulate (g/d/PE)	TKN (g/d/PE)	TP (g/d/PE)	TSS (g/d/PE)	Water (l/d/PE)
Continental	35	65	11	1.8	60	130
Mediterranean	45	55	11	1.8	60	160

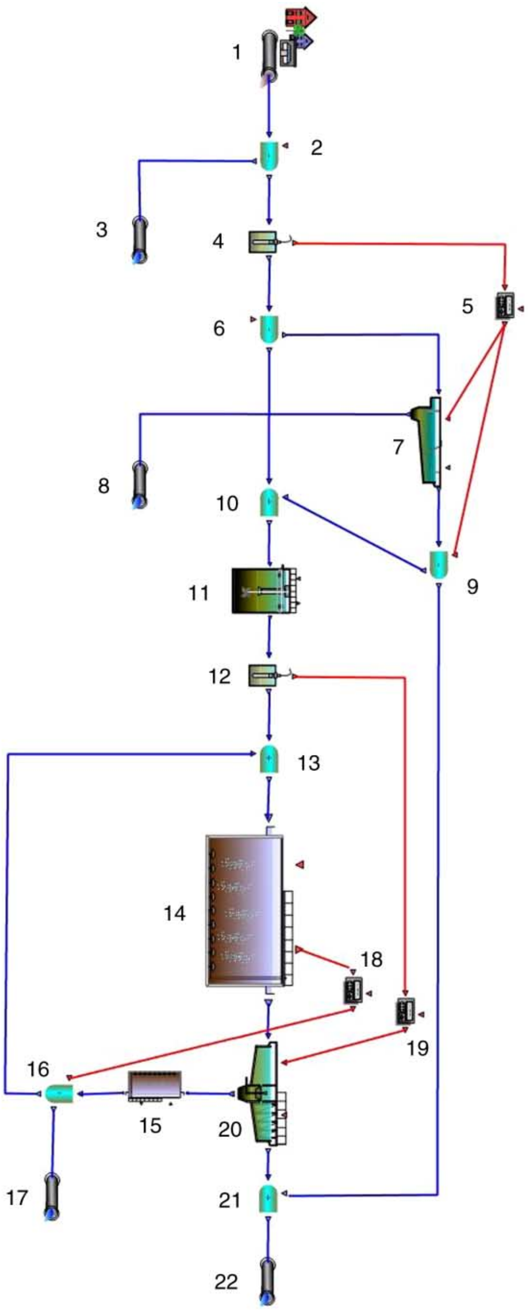


Figure 2 | General plant layout in WEST; for node numbers explanation see Table 1.

The specific configuration for "PROCESS" (see Figure 2 and Table 2) in the LLAS layout consists of one anoxic tank for pre-denitrification, followed by the dosage of a P-precipitant and by six aerated tanks in series. More details on the implementation of the WWTP models can be found in Benedetti (2006).

Table 2 | Legend for nodes of Figure 2

Number	Description
1	Influent data
2	Splitter for CSO structure
3	'Dump' output for CSO spilling
4	Flow sensor
5	Controller for buffer tank pump
6	Splitter for by-pass of water line to storm tank
7	Storm tank
8	'Dump' output for storm tank sediment
9	Splitter to treatment line and WWTP effluent
10	Combiner of flow returning from storm tank to treatment line
11	Fixed volume buffer tank to account for the HRT of pre-treatments
12	Flow sensor
13	Combiner of secondary sludge recirculation to treatment line
14	Represents a generic process, combination of several tanks, controllers, recirculations, etc.
15	AS tank accounting for the anoxic part of the sludge blanket in the clarifier
16	Splitter for secondary sludge to wastage
17	'Dump' output for wasted secondary sludge
18	Controller of waste sludge as a function of TSS measured in the process tanks
19	Controller for clarifier underflow as a function of measured treatment line inflow
20	Secondary clarifier
21	Combiner of treatment line effluent and storm tank effluent
22	Effluent data

The upgrades were implemented for a 300,000 population equivalent (PE) plant treating typical municipal sewage from a combined system. In order to see the impact of different climatic conditions on the plant performance evaluation, the upgrade scenarios were simulated for Continental and Mediterranean climate types. These were characterized by specific influent characteristics driven by temperature and rainfall (see Figure 3), fed to the influent generation model introduced above. An increase in loads of 33% (from 300,000 PE to 400,000 PE) was applied to the influent of the plant to justify the need for upgrading.

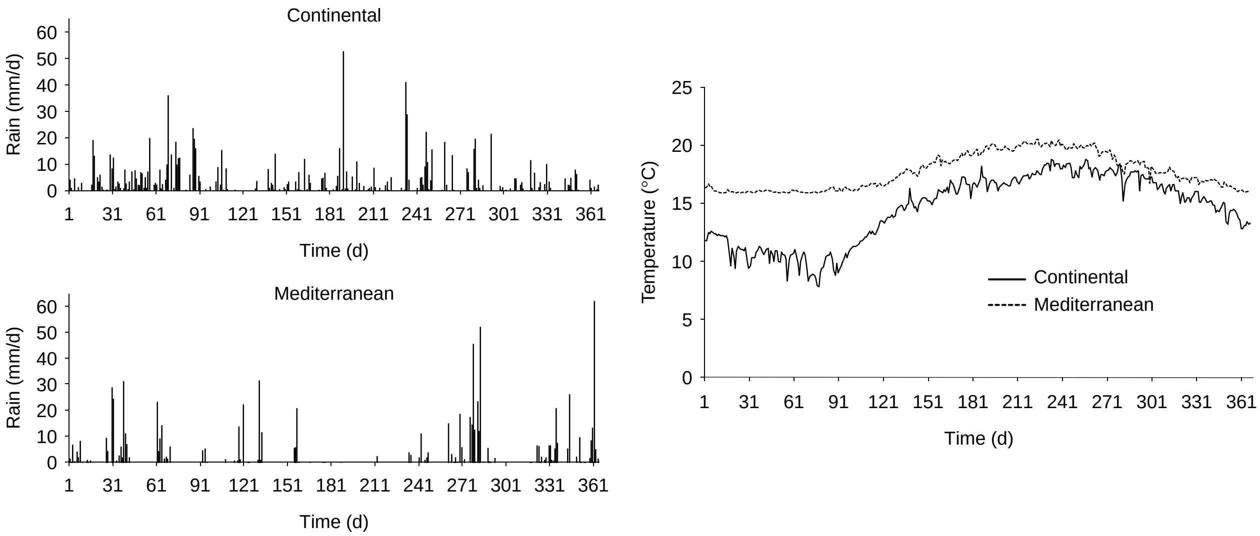


Figure 3 | 1-year time series (January to December) of rainfall (left) and influent temperature (right) in Continental and Mediterranean climates.

For all upgrades, the total suspended solids (TSS) concentration in the activated sludge tanks was set to 3.5 gTSS/l in summer and 4.5 gTSS/l otherwise, with summer defined as the period with mixed liquor temperature above 16°C.

Compared to the original 300,000 PE LLAS plant (Benedetti 2006) dimensioned according to the German ATV-131 guidelines (ATV 2000)–resulting in anoxic, aerated and settling volumes of 32,200 m^3, 42,300 m^3 and 16,400 m^3 respectively–some changes were made to obtain the basic configuration referred to as U0 ("upgrade zero", not upgraded), in order to mimic a situation where upgrades are needed due to load increase. The safety margins included in the ATV dimensioning guidelines were removed by reducing the plant size to 60% of its original volume. With this reduced tank volume, the plant effluent was still complying with the standards set in the EU Urban Waste Water Directive (UWWD) (CEC 1991) with the influent for 300,000 PE, but was not complying with the influent for 400,000 PE (+33%). This means that to have the plant designed with ATV guidelines not complying with the UWWD, it was necessary to more than double the load (1.33/0.6 > 2).

It should be noted that the above-mentioned compliance was checked only for the yearly average limits set in the UWWD, which are the regulatory limits in several Member States. However, some Member States (e.g. Germany) have applied stricter limits and/or limits based on effluent concentrations measured over short periods (e.g. threshold exceedance not more than 20% of 2-h composite samples) or where maximum allowed concentrations are set in the receiving water. Such regulations require an analysis on the exceedance frequencies and lengths of the given concentration thresholds. Of course, such restrictions challenge the treatment performance of WWTPs, and justify the dimensioning suggested by the ATV guidelines.

The list of possibilities for upgrading a WWTP is extensive and case dependent. The upgrades that were chosen for evaluation seemed to be the most applicable scenarios for LLAS. They were also selected because of the established modelling practice, while upgrade options with limited modelling history (e.g. membrane bioreactors) were not considered. Four of the upgrades are pure RTC upgrades, and therefore only require the installation of sensors, wiring and controllers. The other seven upgrades also require constructions and equipment such as pumping, piping and building of new reactor volumes.

Below, the different upgrade scenarios will be referred to as U1, U2, … ,U13. Table 3 provides an overview of the studied upgrade scenarios. The reference case without upgrade is referred to as U0.

In RTC options, controller tuning is extremely important because an ill-tuned controller can be the cause of suboptimal results. The same controller with well-tuned parameter values could allow savings in operational costs

Table 3 | Overview of the upgrade options

Short name	Description	Requires construction	Requires RTC
U0	Reference case with no upgrade		
U1	Increase of aerated tank volume by 33%	X	
U2	U1 + increase of final clarifier area by 33%	X	
U3	U1 + pre-anaerobic tank + C dosage to denitro + lower DO set-point	X	X
U4	Dosage of external carbon source	X	X
U5	DO control based on ammonia		X
U6	Internal recycle control based on nitrate		X
U7	U4 + U6	X	X
U8	Spare sludge storage	X	X
U9	Sludge wastage control		X
U10	Dynamic step feed	X	X
U11	Increase in anoxic volume, decrease in aerated volume		X
U12	Buffering ammonia peak loads with the storm tank	X	X
U13	More wastewater to treatment line, less to storm tank and CSO		

and/or improvements in effluent quality. Tuning of controllers was conceived as a two-step iterative process, since a controller has two types of parameters: (1) target specification (e.g. set-point) and (2) control algorithm parameters (e.g. proportional gain).

1. Once a particular control strategy has been chosen with a particular target, tuning of the algorithm constants is carried out by trial and error until the performance of the controller satisfies the *a priori* defined targets.
2. The definition of the target can be modified according to the result of the evaluation of the operational costs or the overall effluent quality.

An example illustrates this: if the chosen strategy is to keep a certain nitrate concentration at a pre-set value of 2 mg NO_3-N/l, control parameters have to be adjusted until the controller succeeds in maintaining that nitrate concentration in the range between e.g. 1.5 and 2.5 mgNO_3-N/l. The second step consists of an evaluation of the controller's performance in terms of operational costs and effluent quality. This second evaluation level may reveal that the set-point of 2 mg NO_3-N/l would better be lowered to 1 mg NO_3-N/l.

In many cases, WWTP upgrades turn out to be a trade-off between investment costs and effluent quality, which makes it hard to decide the endpoint of the iteration. In this work, the end target has been defined as making the plant comply with the effluent standards if those were not met without any upgrades. In the case of the plant already having complied with the standards, the aim was to reduce operating costs without exceeding the yearly average effluent quality limits with the 95th percentile.

Modelling the river

To provide an example of immission-based comparison of upgrade options, the model of a river stretch has been connected to the WWTP model.

A sub-model of the River Water Quality Model no. 1 (RWQM1) (Reichert *et al.* 2001) has been implemented, based on the work of Solvi *et al.* (2006) to model the river Sure in Luxembourg. This sub-model does not include processes and state variables for which there were no data available or which were of no relevance to the river Sure. This is the case for all chemical pH-dependent reactions (the river's buffer capacity is high) and for the state variable 'consumers' (and connected processes). An RWQM1 sub-model similar to that adopted in this study was successfully tested on a South African basin (Deksissa *et al.* 2004) and on an Italian basin (Benedetti *et al.* 2007).

Hydrolysis, bacterial and algal growth and especially dissolved oxygen concentration are functions of water temperature, which is therefore of great importance and should be adequately estimated. A simple heat balance model, based on that of Talati & Stenstrom (1990), was implemented in the river model to consider the effect of atmospheric changes on water temperature.

The river stretch hydraulic model consists of 5 tanks in series, each representing a river stretch 1,000 m long and 30 m wide, for a total length of 5,000 m. The first tank receives input from the upstream river which is adapted from real river measurement data (Solvi *et al.* 2006) by rescaling the flow to have a dilution factor of 5 between yearly river flow and yearly WWTP flow. Another input to this first tank is the effluent of the LLAS treatment plant model which includes the biological treatment effluent, the storm tank effluent and the CSO effluent. The same river stretch model has been used for both climate types, since it is plausible that such a river type is present in both climate areas. In any case, this is only as an illustrative hypothetical case study.

Integrating WWTP and river models

For the immission-based evaluation, the required integration of the WWTP model with the river stretch model was made by means of the continuity-based interfacing method (CBIM), which allows any model expressed in the Petersen matrix format (Vanrolleghem *et al.* 2005*b*) to be consistently connected. The whole integrated model was implemented in WEST.

The interface consists of a list of algebraic equations expressing concentration inputs in the river in terms of concentration outputs from the sewer or WWTP models, and closes all elemental mass balances in the passage from one system to the other. More details on connecting WWTP and river models can be found in Benedetti *et al.* (2004).

Evaluation

Probabilistic aspects

In this study, the modified ASM2d parameters considered as uncertain were chosen according to Rousseau *et al.* (2001) and to expert knowledge. Also, two parameters of the influent fractionation model–the fraction of fermented products in soluble chemical oxygen demand (COD) and the fraction of slowly biodegradable matter in particulate COD–are assumed uncertain since the influent composition is considered as uncertain. No uncertainty was considered for the river model parameters in order to simplify the assessment, as it is only demonstrative of the methodology. The propagation of parameter uncertainty to the outputs was performed by Monte Carlo simulations (McKay 1988) with Latin Hypercube Sampling from the probability density functions.

The large amount of data (e.g. 100 yearly time series for each process configuration to evaluate) generated by the Monte Carlo simulations with the help of distributed computing (Claeys *et al.* 2006*b*) needs to be presented to the user effectively. For this purpose, some methods of summarizing the uncertainty information have been devised. In particular, percentile polygons have been introduced. These allow the performance of the simulated options, with regard to the yearly averages for two variables of interest, to be evaluated by drawing for each option a polygon which joins the 5th and 95th percentiles of the 100 simulated averages. For further details on the uncertainty characterization, see Benedetti *et al.* (2008*b*).

Emission-based

The emission-based performance evaluation of the different upgrade scenarios introduced in Benedetti (2006) was performed using the yearly average NH_4, total nitrogen (TN) and total phosphorus (TP) effluent concentrations (which show the accumulation of nutrients) the effluent quality index (EQI) and the effluent violations for NH_4 (>2 mg/L), TN (>10 mg/L) and TP (>1 mg/L), indicators of acute pollution events. The limits for effluent violations were selected according to the UWWD except for COD, for which the limit was chosen so that the different performance of the upgrades could be appreciated. The percentage of time that the constraints are not met was calculated from the simulation output data generated at 15-minute intervals. The EQI is the weighted sum over one complete year of the pollution loads due to TSS, COD, BOD_5, TN and TP. The used weights–2 for TSS, 1 for COD, 2 for BOD_5, 20 for TN and 100 for TP–are based on Vanrolleghem *et al.* (1996*b*) which cited a Flanders' effluent quality formula for calculating fines. The EQI is of course sensitive to the values of the weights, which should therefore be carefully chosen according to the specific situation of the study. This approach was adopted since effluent fees are applied in several countries, a strong incentive for upgrading plants.

Immission-based

The assessment of the effect of different WWTP upgrades on the receiving water quality (immission-based evaluation) was carried out by analyzing quality variables at one or more points of the river. In this study, the yearly averages and exceedance periods of concentration thresholds were measured in the last tank of the river model (5,000 m downstream of the WWTP effluent) for dissolved oxygen (DO) and in the first tank (1,000 m downstream of the WWTP effluent) for NH_4, NO_3, PO_4 and COD. The choice of location was determined by evaluation of the critical sections for those water quality parameters. The values of the thresholds for the exceedance analysis are 0.5 mg NH_4/l and 5 mg DO/l.

Three WWTP options were compared for the immission-based evaluation: two were already present in the emission-based evaluation (U0 and U2) and an additional upgrade (U13) was included since it would have a positive effect if the comparison is made on the receiving water quality effects (Bixio *et al.* 2004). U13 consists of an increase of the maximum treated flow from 2.5 times the dry weather flow (DWF) to 5 DWF, an increase of the flow going to treatment and to the storm tank from 5 DWF to 10 DWF and a doubling of the maximum recirculation and return sludge pumping capacity. Only three (instead of thirteen) options were selected in order to simplify the comparison with very distinct behaviours.

Costs

A detailed description of cost calculations makes the assessment more transparent and comparable with other studies or available data. The main focus of this study is the water treatment line, while sludge treatment was considered in less detail.

The cost categories used in this study are:

- aeration energy cost (AEC);
- energy cost (EC) including aeration, pumping and mixing costs;
- sludge cost (SC) which comprises sludge treatment and disposal;
- variable cost (VC) incorporating energy, sludge and chemicals cost; and
- total cost (TC) which includes variable, personnel, maintenance (proportional to capital cost) and annualized capital costs.

All cost figures provided below and not clearly referenced were received from Aquafin (Belgium). Since capital costs information was available for Germany, the operational costs were given for the same country. Personnel costs amount to zero in all comparisons, since it was assumed that no extra or further specialized personnel were required in the upgraded plant, given the large size of the plant.

RESULTS

Before the results are presented, the following must be noted. In terms of variable costs, U4 is quite expensive due to the consumption of C-source. Therefore, it should only be applied if effluent nitrogen levels are higher than the applicable standards. For the Mediterranean climate, the yearly average nitrogen and ammonia concentrations in the effluent never exceed the standard and, therefore, U4 was only incorporated into the comparison of different upgrade scenarios for the Continental and not for the Mediterranean climate. U11 was only included in the comparison for the Mediterranean climate, since in the Continental climate the system with the upgrade was not able to nitrify sufficiently. U13 is only included in the immission-based evaluation section, since it can be argued *a priori* that its effluent quality would not be better than that of U0.

Concerning the performance of the methodology itself–i.e. of the software tools used and developed for the methodology–the reader is referred to Benedetti *et al.* (2008*b*).

Emission-based evaluation

The emission-based evaluation is performed by an economic assessment and by an environmental assessment of

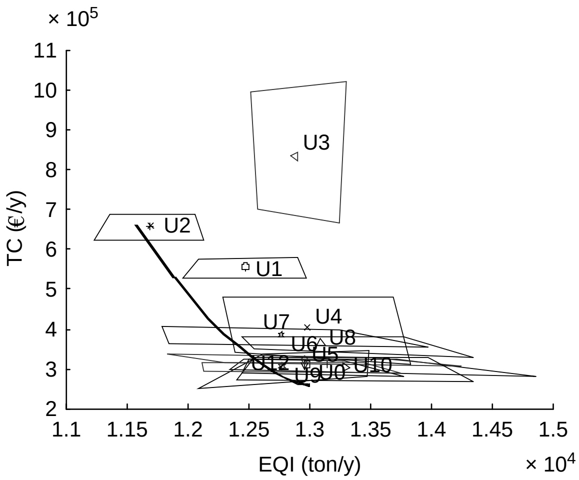


Figure 4 | EQI and TC for LLAS upgrades in Continental climate; bold line: Pareto front.

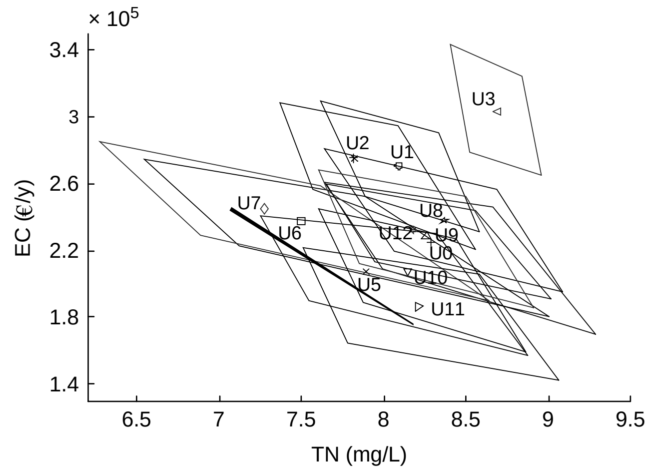


Figure 6 | TN and EC for LLAS upgrades in Continental climate; bold line: Pareto front.

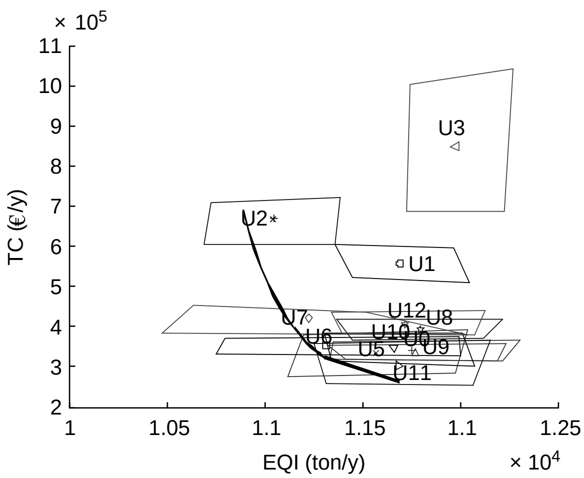


Figure 5 | EQI and TC for LLAS upgrades in Mediterranean climate; bold line: Pareto front.

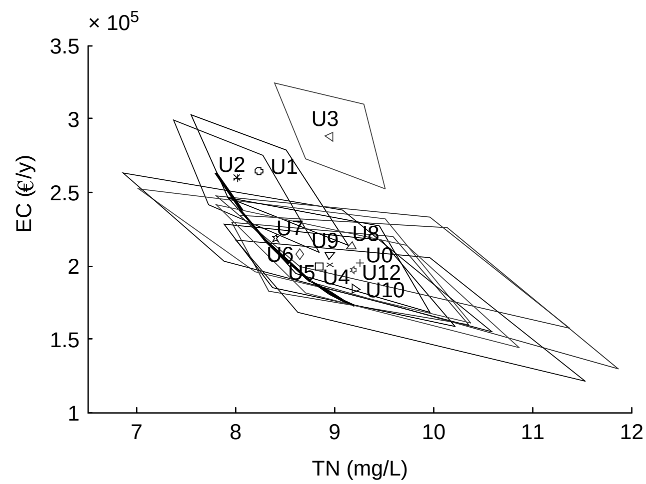


Figure 7 | TN and EC for LLAS upgrades in Mediterranean climate; bold line: Pareto front.

the options to be evaluated. Figures 4–9 show the percentile polygons of the upgrade options for some of the variables of interest. In these figures, the bold line approximates the Pareto-optimality front (i.e. the set of non-dominated options), which helps to determine the option with the preferred trade-off between the two plotted variables.

Costs

The economic performance was evaluated on the basis of the difference in costs of the upgrade (including U0) fed by the 400,000 PE influent minus the costs of U0 fed by the 300,000 PE influent.

In terms of total costs (Figures 4 and 5), the difficult upgrades U1, U2 and U3, which involve mainly constructional intervention, are clearly more expensive than the RTC upgrades. The larger volumes of difficult upgrades also entail higher energy costs mostly due to higher aeration costs, where it can be noted that lower NH_4 effluent concentrations are synonymous with higher aeration costs.

Figure 10 illustrates that the majority of the additional total costs for upgrade is due to variable costs, and that additional capital costs are definitively minor. It also shows that variable costs are mostly constituted by aeration, that P-precipitant and sludge costs are of similar magnitude and that the main differences are due to the presence of

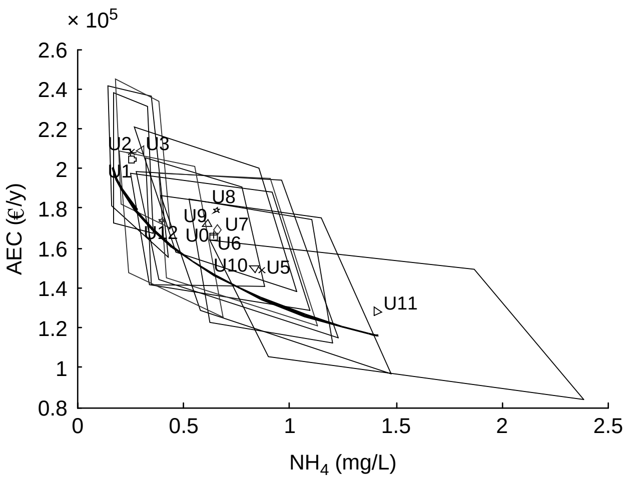


Figure 8 | NH_4 and AEC for LLAS upgrades in Continental climate; bold line: Pareto front.

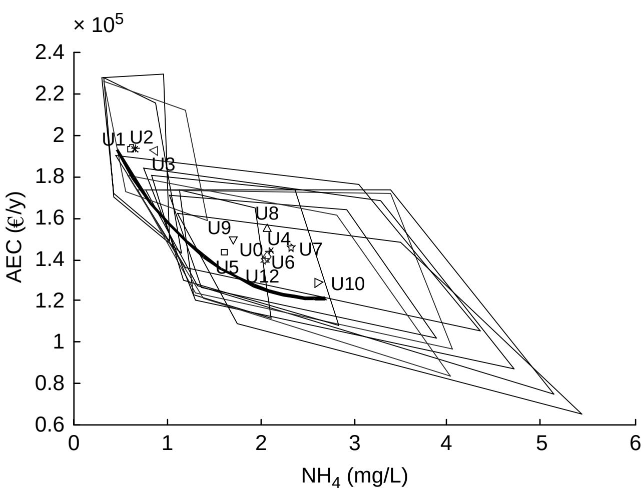


Figure 9 | NH_4 and AEC for LLAS upgrades in Mediterranean climate; bold line: Pareto front.

C-source dosage. Although it might seem from these figures that all upgrade options have total annual costs that are nearly the same as U0, it should be stressed that the difference between the most and the least expensive scenarios is about €500,000 per year. This means that, in absolute terms, there is certainly a difference worth consideration.

It can be noted that U2 shows the best environmental performance in Continental climate conditions (Figure 4), together with U7 in Mediterranean climate (Figure 5) especially for TN (Figures 6 and 7). All upgrades have a 50th percentile EQI that is lower than that of U0 in Continental conditions. This is not the case in Mediterranean conditions but it should be considered that, for the Mediterranean condition, the EQI of U0 was already more than 10% lower than in Continental conditions due to the better temperature conditions.

Such results are valid under the given assumptions; in order to provide sound investment policy advice for a specific case, as much information as possible therefore has to be gathered to reduce the uncertainties in the uncertainty estimations.

Effluent quality

Concerning the effluent concentrations, it can be seen that almost all upgrades have better nitrogen removal than U0. Because of the less favourable conditions for nitrification in the Continental climate, the box plots in Figure 11 show a larger spread in exceedance values compared to the Mediterranean plots on the right side, which is a sign of process instability. This is also reflected in Figures 4–9, where the 50th percentile values are higher and the 5th/95th percentile interval is larger for the Continental than for the Mediterranean plots. These figures show that U2 performs better than U1 with respect to TN removal, but not with regard to effluent ammonia concentrations which are about the same in both scenarios. This means that U2 demonstrates a better denitrification performance. This can partly be attributed to the larger final clarifier, the model of which includes anoxic processes that take place in the lower part of the sludge blanket.

When comparing the results of the first three upgrade options, which all require the construction of additional volumes, it can be seen that U2 always performs better than U1 and U3. The difference compared to U1 proves that an extension of the final clarifier area (U2) is a clear added value to the increase in aerated volume (U1). U3 aimed at biological phosphorus removal by adding extra anaerobic tank volume and a dosage of external carbon source. Despite those extra investments, the figures show that the environmental performance of U3 is poorer than that of U1 and U2. The higher effluent ammonia and TN concentrations in U3 can be attributed to the lower DO set-point used–an attempt to lower the aeration costs–and to the introduction of biological phosphorus removal

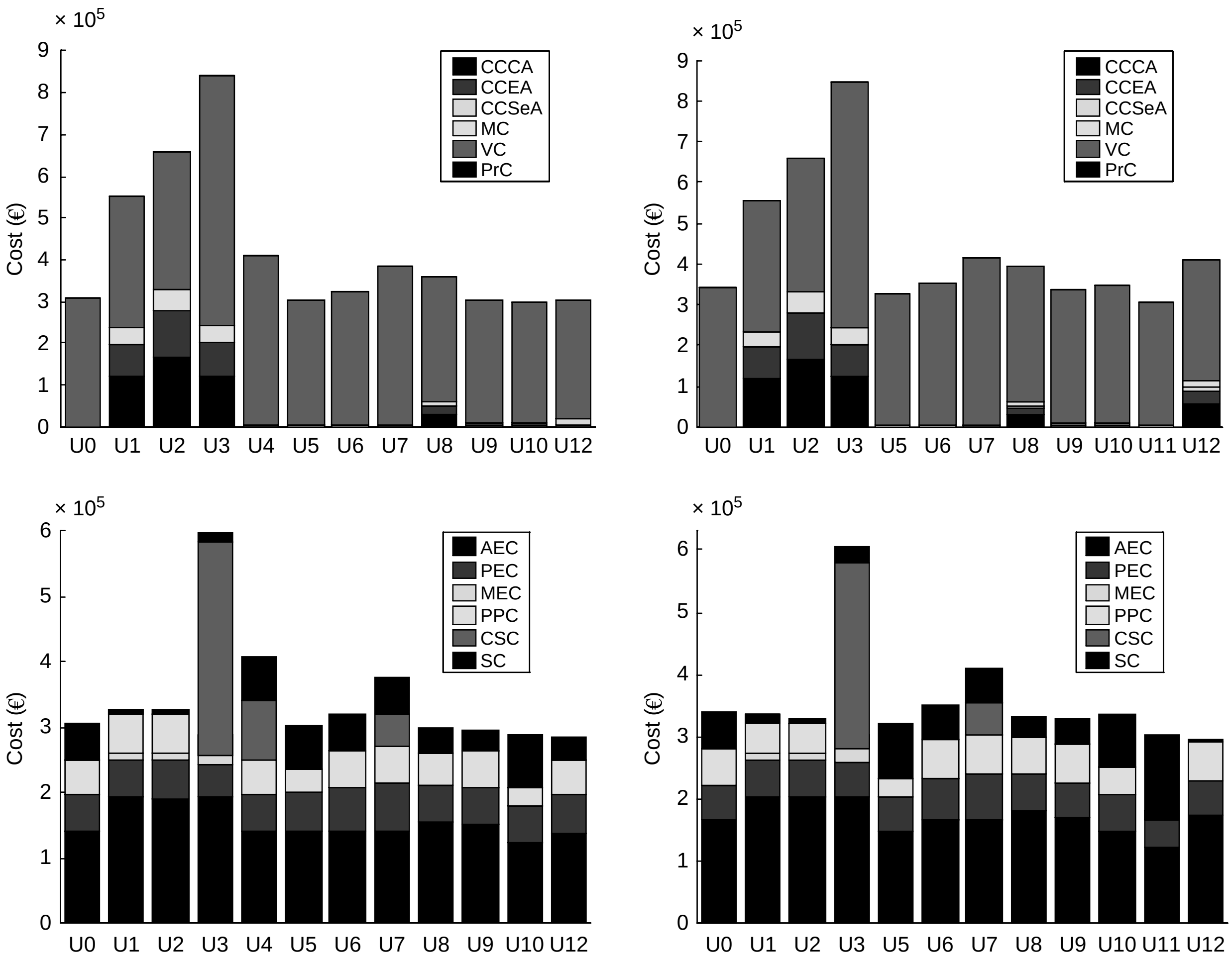


Figure 10 | TC (top) and VC (bottom) for LLAS 300,000 PE upgrades in Continental (left) and Mediterranean (right) climates; CCCA: capital cost for construction annualized; CCEA: capital cost for equipment annualized; CCSeA: capital cost for sensors annualized; MC: maintenance cost; VC: variable cost; PrC: personnel cost; AEC: aeration energy cost; PEC: pumping energy cost; MEC: mixing energy cost; PPC: P-precipitant cost; CSC: C-source cost; and SC: sludge cost.

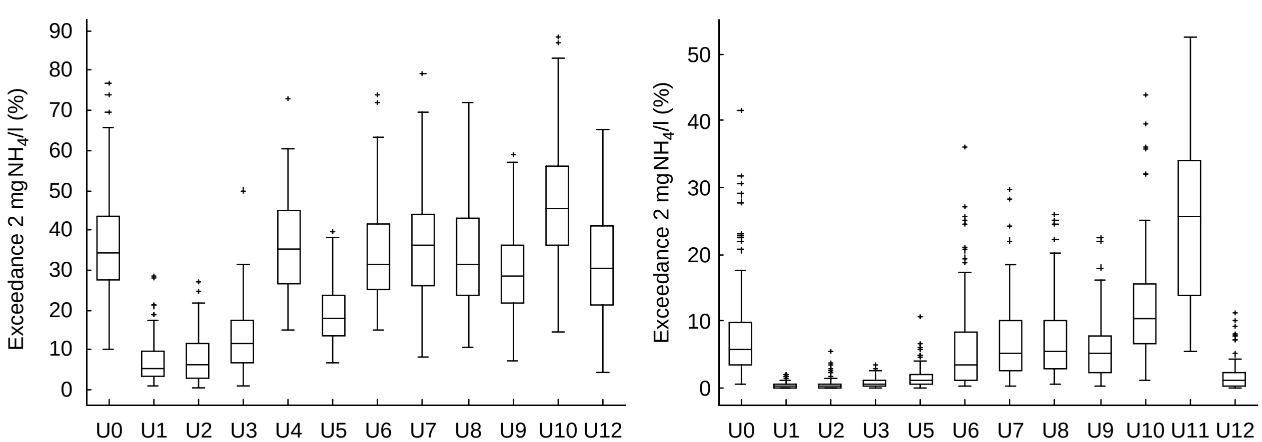
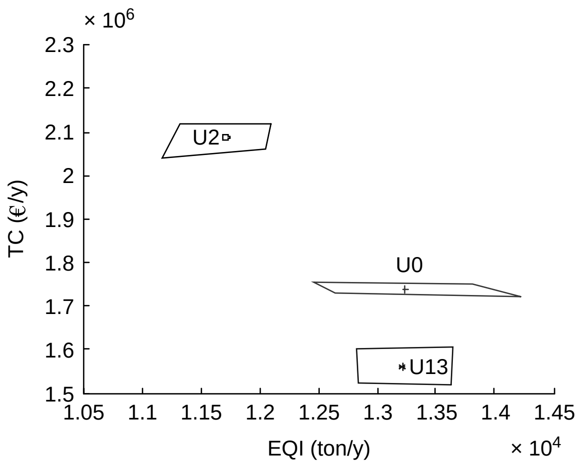


Figure 11 | Exceedance time of 2 mgNH$_4$/l for LLAS 300,000 PE upgrades in Continental (left) and Mediterranean (right) climates.



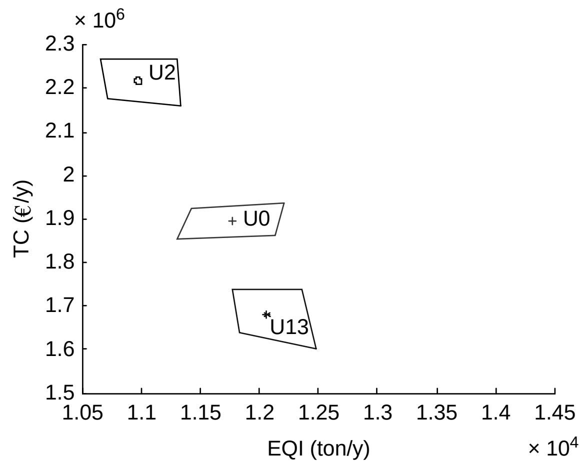


Figure 12 | Yearly average EQI and TC for LLAS 300,000 PE upgrades in Continental (left) and Mediterranean (right) climates.

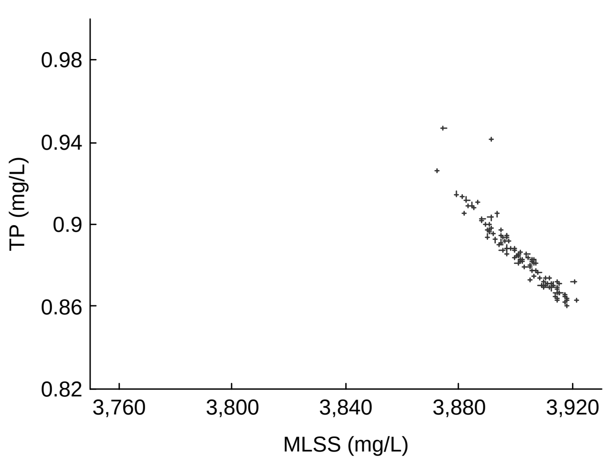
before the denitrification tank. This leads to the use of most of the carbon source by the phosphorous accumulating organisms, hence decreasing the denitrification performance.

The poor performance and process instability (large polygon) of U10 concerning nitrogen removal in Continental conditions (Figure 6) indicates that the loss of nitrification capacity due to the decrease in aerated volume cannot be compensated by the benefits of the increased anoxic tank volume for denitrification.

Immission-based evaluation

A basic emission-based evaluation of the three alternatives considered in this section (U0, U2 and U13) is initially performed. From Figure 12, it can be deduced that U2 implies higher costs (in particular capital cost) and that U13 has lower costs than U0, for both climates.

Further analysis reveals that the higher hydraulic load through the treatment plant under U13 leads to a lower mixed liquor suspended solids (MLSS) concentration in the aerated tanks due to larger effluent TSS in wet weather. This entails lower aeration requirements and also lower sludge production (Figure 13). The larger dilution in U13 also plays a role in this result, since the extra flows allowed to the treatment line and to the storm tank occur only in wet weather flow. Another aspect which helps to explain the good performance of U13 is the increased maximum pumping capacity. This allows more nitrates to be recirculated to the anoxic tank, allowing for a better



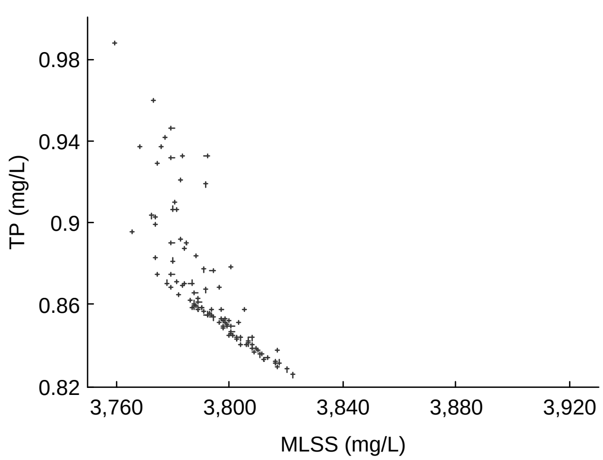


Figure 13 | MLSS in the tanks and TP in the effluent for LLAS 300,000 PE in Continental climate for U0 (left) and U13 (right).

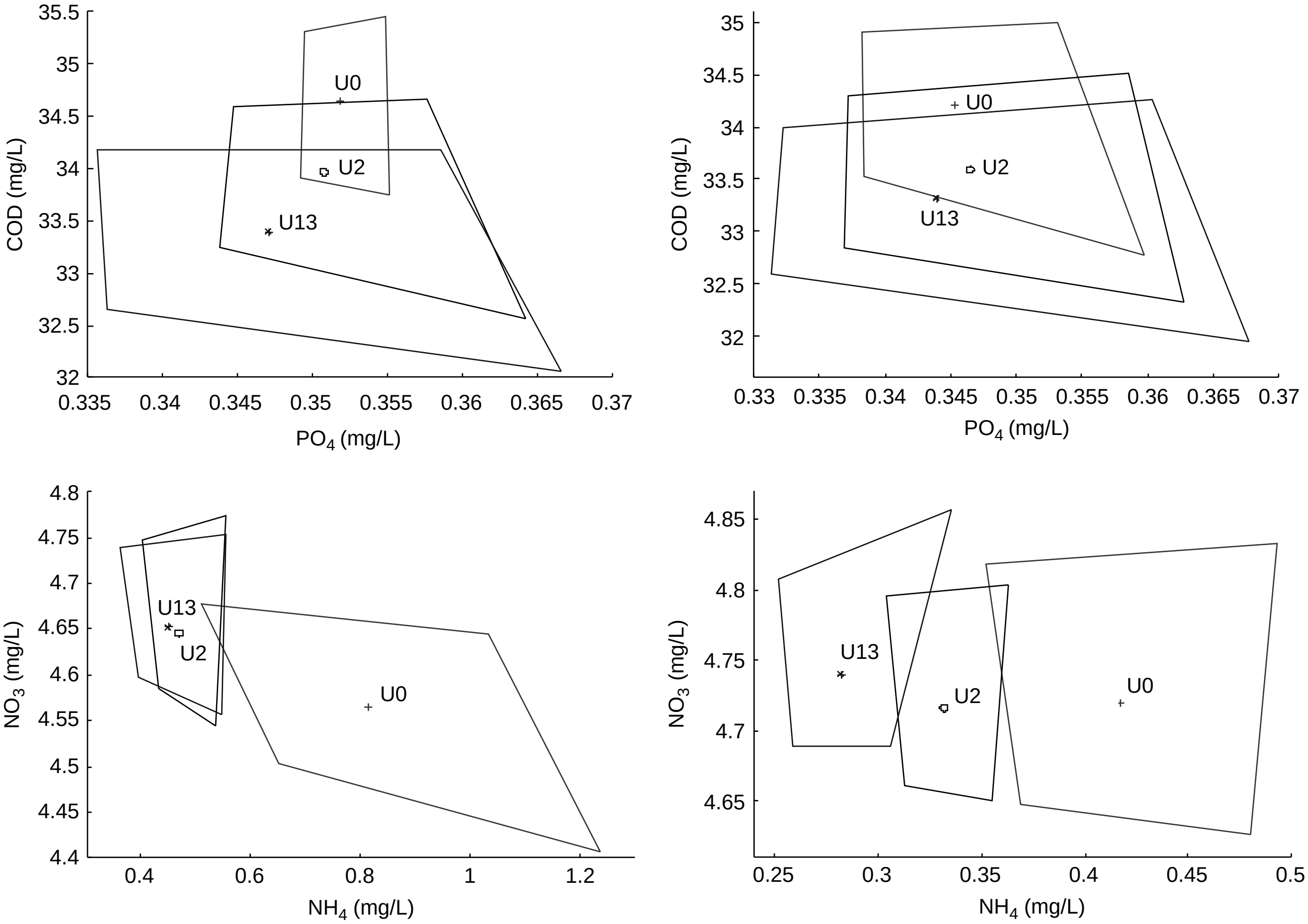


Figure 14 | Yearly average NH_4 and NO_3 in the river 1,000 m downstream of the WWTP effluent (top) and yearly average PO_4 and COD in the river 1,000 m downstream of the WWTP effluent (bottom) for LLAS 300,000 PE upgrades in Continental (left) and Mediterranean (right) climates.

utilization of oxygen in the form of nitrates during the denitrification step.

No sludge losses occur in U13 because of the dimensioning of the secondary settler. Note that settling problems (e.g. bulking or insufficient hydraulic capacity) are not the topic of this study, therefore a good sludge volume index (100 mL/g) was assumed in all simulations.

On the other hand, the EQI (pollutant loads) of U13 is similar to that of U0 (in the Mediterranean climate it is even slightly better), and both are around 20% worse than U2 in Continental climate and 10% worse in Mediterranean climate.

Considering the immission-based evaluation, a better situation with U13 can be noted when analyzing the average concentrations in the river (Figure 14). For NH_4, the cold winter in the Continental climate penalizes U0 for its difficulties with respect to nitrification, both in terms of 50th percentile and of process stability. In the Mediterranean climate, such a difference is not very significant. U13 achieves lower NH_4 in the river than U2, while NO_3 is lower with U2 but only very slightly. For DO and COD the pattern is also similar, with U13 performing slightly better than U2 and U0 clearly showing its deficiencies.

Concerning the exceedance periods for NH_4 and DO (Figure 15), they all show the same behaviour. U0 clearly has larger exceedance periods than U2 and U13, which perform similarly in both climates. In general, a slightly larger variance (instability) can be observed for U13 due to the smaller process volumes which give less stability than U2.

It can be noted that in both the emission- and the immission-based evaluations, the differences in process

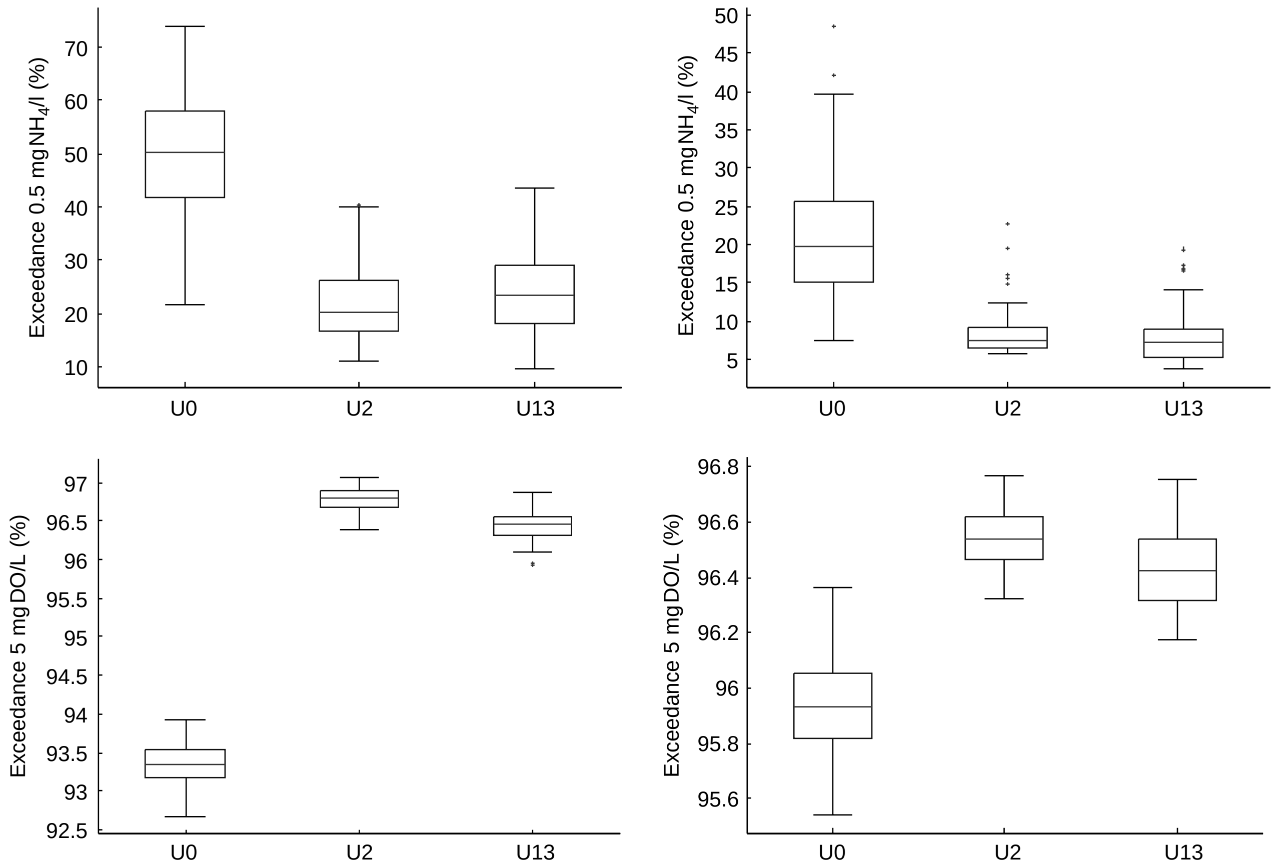


Figure 15 | Exceedance of 0.5 mgNH$_4$/l in the river 1,000 m downstream of the WWTP effluent (top) and exceedance of 5 mg DO/L in the river 5,000 m downstream of the WWTP effluent (bottom) for LLAS 300,000 PE upgrades in Continental (left) and Mediterranean (right) climates.

stability between the three configurations in Mediterranean climate are much less pronounced than in Continental climate. This is a feature of the huge impact of periods of low temperature on the activated sludge process.

CONCLUSIONS

The comparison of eleven WWTP upgrade options highlights the advantages and disadvantages of upgrades that require either construction of volumes or real-time control. The former generally provides more process stability (less spread of the Monte Carlo simulations, i.e. less output uncertainty) at high cost, and the latter delivers good performance improvement at low cost but with more risk of compliance failure.

An important conclusion is that WWTPs designed with ATV guidelines can accept almost double the design load and still comply with the yearly average limits of the EU Urban Waste Water Directive.

The immission-based evaluation of some plant upgrade options revealed that considering the system from a holistic point of view (although requiring more modelling efforts and calculation time) can lead to substantial savings. The option which consisted of simply allowing more water to be treated in the plant (implying lower effluent quality but less untreated water to be directly discharged in the river) resulted in better environmental and economic performance than that involving the extension of the treatment volume. This option, more beneficial for the receiving water, would have been discarded by only considering the WWTP emission quality.

Such results are valid under the given assumptions. In order to provide sound investment policy advice for a specific case, as much information as possible therefore has to be gathered.

Concerning the practical applicability of the results, strong incentives can be found in the national/regional legislation. The limits may be expressed in statistical terms or maximum allowed concentrations in the receiving water.

It is therefore evident that the actual availability of well-accepted models, uncertainty characterization and propagation techniques, sufficient computational power and specific software tools can move the design practice from conventional procedures suited to a relatively stiff context as imposed by emission limits to more advanced, transparent and cost-effective procedures. The latter are more appropriate to cope with the flexibility and complexity introduced by integrated water management approaches.

ACKNOWLEDGEMENTS

The results presented in this publication have been elaborated in the frame of the EU project CD4WC, contract no EVK1-CT-2002-00118. The programme is organized within the Energy, Environment and Sustainable Development Programme in the 5th Framework Programme for Science Research and Technological Development of the European Commission. Lorenzo Benedetti is a post-doctoral researcher of the Special Research Fund (BOF) of Ghent University. Peter A. Vanrolleghem holds the Canada Research Chair in Water Quality Modelling. Aquafin NV (Belgium) and Ruhverband (Germany) are kindly acknowledged for the provision of data.

REFERENCES

Achleitner, S., Möderl, M. & Rauch, W. 2007 CITY DRAIN©–an open source approach for simulation of integrated urban drainage systems. *Environ. Model. Softw.* **22** (8), 1184–1195.

ATV 2000 Standard ATV-DVWK-A 131E. Dimensioning of Single-Stage Activated Sludge Plants.

Bauwens, W., Vanrolleghem, P. A. & Smeets, M. 1996 An evaluation of the efficiency of the combined sewer–wastewater treatment system under transient conditions. *Water Sci. Technol.* **33** (2), 199–208.

Benedetti, L. 2006 Probabilistic design and upgrade of wastewater treatment plants in the EU Water Framework Directive context. PhD Thesis, Ghent University, Belgium, p. 304. http://biomath.ugent.be/publications/download/

Benedetti, L., Meirlaen, J. & Vanrolleghem, P. A. 2004 Model connectors for integrated simulations of urban wastewater systems. In *Sewer Networks and Processes within Urban Water Systems* (ed. J.-L. Bertrand-Krajewski, M. Matos & S. Abdul-Talib), pp. 13–21. IWA Publishing, London, UK.

Benedetti, L., Dirckx, G., Bixio, D., Thoeye, C. & Vanrolleghem, P. A. 2006 Substance flow analysis of the wastewater collection and treatment systems. *Urban Water J.* **3** (1), 33–42.

Benedetti, L., Meirlaen, J., Sforzi, F., Facchi, A., Gandolfi, C. & Vanrolleghem, P. A. 2007 Dynamic integrated modelling: a case study on the river Lambro. *Water SA* **33** (5), 627–632.

Benedetti, L., Dirckx, G., Bixio, D., Thoeye, C. & Vanrolleghem, P. A. 2008*a* Environmental and economic performance assessment of the integrated urban wastewater system. *J. Environ. Manage.* **88**, 1262–1272.

Benedetti, L., Bixio, D., Claeys, F. & Vanrolleghem, P. A. 2008*b* Tools to support a model-based methodology for emission/immission and benefit/cost/risk analysis of wastewater treatment systems which considers uncertainties. *Environ. Model. Softw.* **23** (8), 1082–1091.

Bixio, D., Thoeye, C. & De Gueldre, G. 2004 Biological solution to stormwater? *Water Sci. Technol.* **50** (7), 171–177.

Bott, C. B., Neethling, J. B., Parker, D. S., Pramanik, A. & Murthy, S. 2009 WEF/WERF Cooperative Study of BNR Plants Approaching the Limit of Technology: II. Effluent Reliability. In: *Proceedings of WEF Nutrient Removal Conference 2009*, Washington, DC, USA, 28 June–1 July 2009.

Butler, D. & Schütze, M. 2005 Integrating simulation models with a view to optimal control of urban wastewater systems. *Environ. Model. Softw.* **20**, 415–426.

CEC 1991 Council Directive 91/271/EEC of 21 May 1991 concerning urban wastewater treatment.

CEC 2000 Directive 2000/60/EC of the European Parliament and of the Council establishing a framework for the Community action in the field of water policy.

Claeys, F., De Pauw, D. J. W., Benedetti, L. & Vanrolleghem, P. A. 2006*a* Tornado: a versatile and efficient modeling & virtual experimentation kernel for water quality systems, In: *Proceedings of iEMSs 2006*, 10–13 July 2006, Burlington, VT, USA.

Claeys, F., Chtepen, M., Benedetti, L., Dhoedt, B. & Vanrolleghem, P. A. 2006 Distributed virtual experiments in water quality management. *Water Sci. Technol.* **53** (1), 297–305.

de Kort, I. A. T. & Booji, M. J. 2007 Decision making under uncertainty in a decision support system for the Red River. *Environ. Model. Softw.* **22** (2), 128–136.

Deksissa, T., Meirlaen, J., Ashton, P. J. & Vanrolleghem, P. A. 2004 Simplifying dynamic river water quality modeling: a case study of inorganic dynamics in the Crocodile River (South Africa). *Water Air Soil Pollut.* **155**, 303–320.

Fu, G., Butler, D. & Khu, S.-T. 2008 Multiple objective optimal control of integrated urban wastewater systems. *Environ. Model. Softw.* **23** (2), 225–234.

Gernaey, K. & Jørgensen, S. B. 2004 Benchmarking combined biological phosphorous and nitrogen removal wastewater treatment processes. *Control Eng. Pract.* **12** (3), 357–373.
Gernaey, K., Rosen, C. & Jeppsson, U. 2006 WWTP dynamic disturbance modelling-an essential module for long-term benchmarking development. *Water Sci. Technol.* **53** (4–5), 225–234.
Henze, M., Gujer, W., Mino, T. & van Loosdrecht, M. C. M. 2000 *Activated Sludge Models ASM1, ASM2, ASM2d and ASM3*. IWA, London.
Lijklema, L. 1993 Integrated management of urban waters. *Water Sci. Technol.* **27** (12), vii–x.
Mannina, G., Freni, G., Viviani, G., Sægrov, S. & Hafskjold, L. S. 2006 Integrated urban water modelling with uncertainty analysis. *Water Sci. Technol.* **54** (6–7), 379–386.
McCormick, J. F., Johnson, B. & Turner, A. 2007 Analyzing risk in wastewater process design: using Monte Carlo simulation to move beyond conventional design methods, In: *Proceedings of WEFTEC 2007*, San Diego, CA, USA, 14–17 October 2007.
McKay, M. D. 1988 Sensitivity and uncertainty analysis using a statistical sample of input values. In *Uncertainty Analysis* (ed. Y. Ronen), pp. 145–186. CRC Press, Inc., Boca Raton, Florida.
Meirlaen, J., Huyghebaert, B., Sforzi, F., Benedetti, L. & Vanrolleghem, P. A. 2001 Fast, simultaneous simulation of the integrated urban wastewater system using mechanistic surrogate models. *Water Sci. Technol.* **43** (7), 301–309.
Melching, C. S. 1995 Reliability estimation. In *Computer Models of Watershed Hydrology* (ed. V. P. Singh), pp. 69–118. Water Resources Publications, Littleton, Colorado.
Oliveira, S. C. & Von Sperling, M. 2008 Reliability analysis of wastewater treatment plants. *Water Res.* **42** (4–5), 1182–1194.
Parker, D., Bott, C., Neethling, J. B., Pramanik, A. & Murthy, S. 2009 WEF/WERF Cooperative Study of BNR Plants Approaching the Limit of Technology: I. What Can We Learn About the Technologies? In: *Proceedings of WEF Nutrient Removal Conference 2009*, Washington DC, USA, 28 June–1 July 2009.
Rauch, W. & Harremoës, P. 1999 Genetic algorithms in real time control applied to minimize transient pollution from urban wastewater systems. *Water Res.* **33** (5), 1265–1277.
Reichert, P., Borchardt, D., Henze, M., Rauch, W., Shanahan, P., Somlyody, L. & Vanrolleghem, P. A. 2001 *River water quality model No. 1* (RWQM1), IWA Publishing.
Reichert, P., Borsuk, M., Hostmann, M., Schweizer, S., Spörri, C., Tockner, K. & Truffer, B. 2007 Concepts of decision support for river rehabilitation. *Environ. Model. Softw.* **22** (2), 188–201.
Rousseau, D., Verdonck, F., Moerman, O., Carrette, R., Thoeye, C., Meirlaen, J. & Vanrolleghem, P. A. 2001 Development of a risk assessment based technique for design/retrofitting of WWTPs. *Water Sci. Technol.* **43** (7), 287–294.
Schütze, M., Butler, D. & Beck, M. B. 1999 Optimisation of control strategies for the urban wastewater system–an integrated approach. *Water Sci. Technol.* **39** (9), 209–216.
Solvi, A.-M., Benedetti, L., Vandenberghe, V., Gillé, S., Schlosseler, P., Weidenhaupt, A. & Vanrolleghem, P. A. 2006 Implementation of an integrated model for optimised urban wastewater management in view of better river water quality: a case study. In: *Proceedings of IWA World Water Congress 2006*, 10–14 September, Beijing, China.
Talati, S. N. & Stenstrom, M. 1990 Aeration basin heat loss. *J. Environ. Eng.* **116** (1), 70–86.
Tchobanoglous, G., Loge, F., Darby, J. & Devries, M. 1996 UV design: comparison of probabilistic and deterministic design approaches. *Water Sci. Technol.* **33** (10–11), 251–260.
Vanrolleghem P. A., Fronteau C. & Bauwens W. 1996*a* Evaluation of design and operation of the sewage transport and treatment system by an EQO/EQS based analysis of the receiving water immission characteristics. In: *Proceedings WEF Specialty Conference on Urban Wet Weather Pollution: Controlling sewer overflows and stormwater runoff*. 16–19 June 1996, Quebec City, Canada.
Vanrolleghem, P. A., Jeppsson, U., Carstensen, J., Carlsson, B. & Olsson, G. 1996*b* Integration of wastewater treatment plant design and operation–a systematic approach using cost functions. *Water Sci. Technol.* **34** (3–4), 159–171.
Vanhooren, H., Meirlaen, J., Amerlink, Y., Claeys, F., Vangheluwe, H. & Vanrolleghem, P. A. 2003 WEST: modelling biological wastewater treatment. *J. Hydroinform.* **5** (1), 27–50.
Vanrolleghem, P. A., Benedetti, L. & Meirlaen, J. 2005*a* Modelling and real-time control of the integrated urban wastewater system. *Environ. Model. Softw.* **20** (4), 427–442.
Vanrolleghem, P. A., Rosen, C., Zaher, U., Copp, J., Benedetti, L., Ayesa, E. & Jeppsson, U. 2005*b* Continuity-based interfacing of models for wastewater systems described by Petersen matrices. *Water Sci. Technol.* **52** (1–2), 493–500.

First received 6 February 2008; accepted in revised form 30 June 2009. Available online 25 March 2010