Probabilistic WWTP Design and Upgrade in a Water Quality Based Regulation Context

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ABSTRACT

Process choice and dimensioning of WWTPs is a particularly sensitive step to cost-efficiently comply with regulatory standards. This step accounts only for a small fraction of the upfront costs, but it can lead to substantial savings.

This paper illustrates the results of a systematic methodology to evaluate system design/upgrade options with regard to both effluent quality and receiving water quality. In contrast to conventional practice, the presented approach allows choosing the most appropriate trade-off between cost of measures and effluent quality, and to assess the reliability of a process layout by means of uncertainty analysis. It is therefore a flexible instrument to cope with the flexibility and complexity of integrated water management regulations.

KEYWORDS

Activated sludge, Integrated modelling, Probabilistic design, Uncertainty assessment

INTRODUCTION

The European Water Framework Directive (WFD) requests to achieve good quality of ground and surface waters by organising water management on a river-basin scale and – with regard to impacts on natural water bodies originating from wastewater release – applying a combined emission and water quality (immission) based approach (CEC, 2000). This requires both assessing the current wastewater system performance more broadly (Benedetti *et al.*, 2008b) and evaluating and quantifying what potential costs and benefits to improve the system could result from setting water quality goals in the natural water bodies instead of prescribing the design of urban wastewater systems. A consequence is that the design of the systems is by far less predetermined and the options to meet the goals become much more numerous. 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. This article presents a new methodology (Benedetti, 2006) to identify and quantify the costs and benefits for the development of the urban wastewater system, with regard to its environmental and economic consequences.

To design or upgrade a treatment plant, dynamic mechanistic models were used in this work, since their parameters have a straightforward physical meaning and can be directly measured in the system or applied to it (in case of obtained volumes, recycle rates, etc.). Long-term influent time series were used to feed the WWTP models, in order to consider the process influent disturbances at different time scales, from minutes in “first flush” effect to months in infiltration. Influent time series were generated using a simple phenomenological model of the sewer catchment, which was able to adequately simulate influent disturbances at different time-scales. The traditional approach of assigning a steady-state influent is not sufficient to evaluate the performance of different WWTP designs which are modelled by means of dynamic models, and which in reality are subject to such disturbances.

The evaluation of the options is divided in emission-based criteria (considering the quality of the plant effluent), water quality-based criteria (judging on the basis of the receiving water quality, in this case a river stretch that also needs to be included in the model) and economic criteria (capital and operational costs).

One of the remaining issues when dealing with these deterministic models is the degree of uncertainty linked to their predictions (Beck, 1987; Belia *et al.*, 2009). Probabilistic analysis is introduced to assess how model input uncertainties are propagated to model outputs, in order to evaluate the reliability of processes under uncertain conditions. In general, with the same average behaviour and the same input uncertainties, a process which has a more stable output is preferable to another one with large uncertainty in its output. Probabilistic design, which is the combination of probabilistic modelling techniques with the currently available deterministic models, provides a solution to this issue. This concept has already a history of three decades in electronics and structural design, while for the first applications for water quality another decade had to pass (e.g. see Tchobanoglous *et al.*, 1996). Some recent applications have been reported concerning WWTP design (Rousseau *et al.*, 2001; Bixio *et al.*, 2002b; McCormick *et al.*, 2007). By building a probabilistic shell around the deterministic models one can quantify the uncertainty of the model predictions. For example, a goal can be to determine the probability of exceeding the legal effluent standards of a WWTP. This percentage of exceedance should be accompanied by confidence intervals indicating the uncertainty due to the variability of influent characteristics and to the uncertainty in model parameters. This probabilistic analysis can be carried out by means of Monte Carlo simulation (Saltelli *et al.*, 2005), which implies that large numbers of simulations and of output data need to be interpreted and summarised.

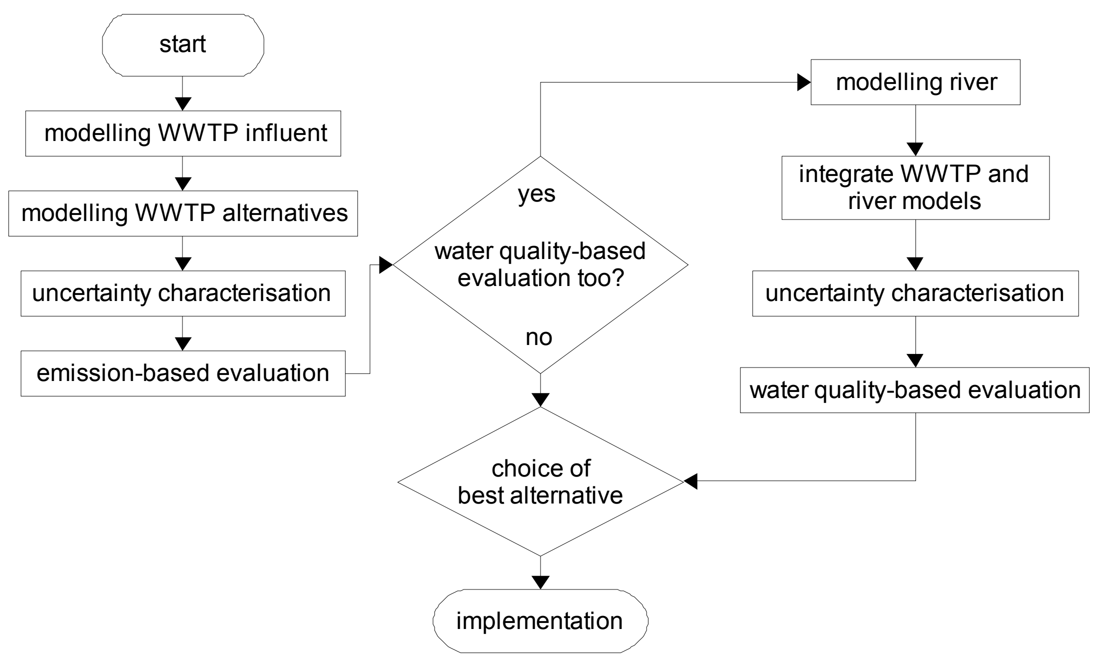
The developed methodology is first illustrated by a case of WWTP design, with comparisons between ten process options on the basis of emission-based criteria. Traditionally, treatment plants have been designed using empirical steady-state equations or “rules of thumb”, introducing conservative safety factors, e.g. in the German ATV guidelines (ATV, 2000). Such approach has led to the construction of over-dimensioned, expensive, and not always properly functioning plants, especially in cases where guidelines developed to fulfil strict national legislations (like in Germany) are applied in countries with less demanding regulations. It is recognised that traditional design procedures are not sufficient to produce WWTP designs which

can deal with uncertainties about the factors influencing the plant performance, like increase and decrease of connections or changing regulations (Dominguez and Gujer, 2006).
The methodology is further illustrated by means of an example of WWTP upgrade. Twelve options are compared on emission-based criteria, and two options also on water quality-based criteria. Probabilistic approaches are becoming quite popular in particular regarding decision support in river management (de Kort and Booji, 2007; Reichert *et al.*, 2007). Previous works which partly contributed to the development of integrated modelling (i.e. WWTP and river), especially dealing with transient events, can be found in Bauwens *et al.* (1996), Vanrolleghem *et al.* (1996a), Meirlaen *et al.* (2001) and Vanrolleghem *et al.* (2005a). However, such works do not aim at establishing a methodology to fully exploit the capabilities of the developed models and software tools and do not include probabilistic aspects, as this paper does.

MATERIALS AND METHODS

The evaluation methodology proposed requires to first provide a sufficiently long and representative influent time series to the WWTP, to implement the WWTP upgrades and the river models, to integrate them, to characterise the model uncertainties and to propagate them to the model outputs by means of Monte Carlo simulations (since uncertainty in WWTP model predictions is considered to be large, therefore it should always be quantified), to evaluate the probabilistic simulation results from economic and environmental points of view, and finally to decide which option should be implemented (see Figure 1). Each step is described in this section.

Figure 1: Methodology flow chart



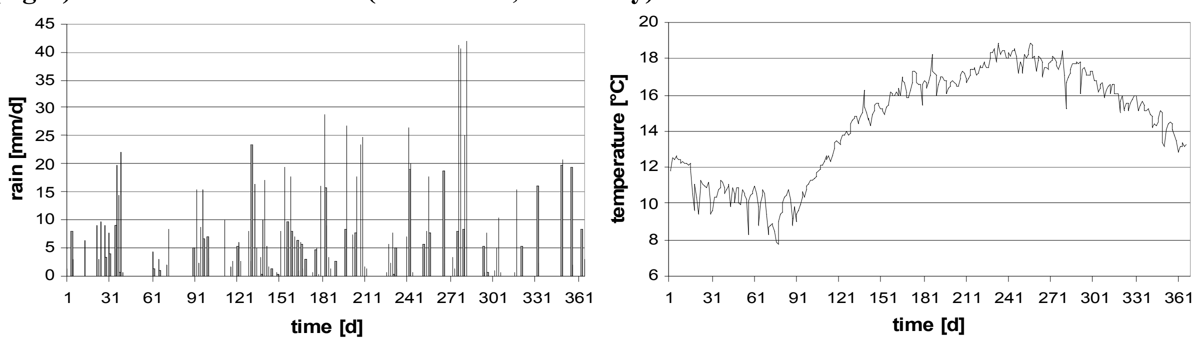
All the modelling and simulation was done with the WEST software (MOSTforWATER NV, Kortrijk, Belgium) (Vanhooren *et al.*, 2003).

Modelling the WWTP Influent

The methodology begins by modelling the WWTP influent, since the length and frequency of (possibly) existing influent data is typically not sufficient to appropriately feed the WWTP models with the desired dynamics, which express the natural variability of the influent characteristics. Such available data usually consist in grab or composite samples collected with a sampling interval varying between days and months, while the systems dynamics have time constants that vary from one month – e.g. sludge age – to a few minutes – e.g. dissolved oxygen variations in the activated sludge tanks, hydraulic and pollutant peaks in rain events – (Jeppsson *et al.*, 2006). Therefore, the modelled WWTP influent has a sampling interval of 15 minutes and a length of one year to cover both short-term effects and seasonal variations.

A simple phenomenological model was developed and implemented, producing dynamic influent flow rate and pollutant concentration trajectories in function of the number of inhabitants, the presence of industry, the loads per capita of households and industry, the size of the catchment, the length of the sewer system, rainfall data, and the interactions with groundwater (infiltration). For a description of this dynamic influent generation model, see Gernaey *et al.* (2006). See Figure 2 for an example of actual rainfall and influent temperature data in Continental climate (in Dresden, Germany).

Figure 2: One year time series (January to December) of rainfall (left) and influent temperature (right) in Continental climate (in Dresden, Germany)



Modelling the WWTP

Dynamic mechanistic models are required in this methodology to be able to predict the dynamic behaviour of the design options. To model the activated sludge units (aerobic, anoxic and anaerobic tanks) in the 10 WWTP process configurations of the case study, ASM2d (Henze *et al.*, 2000) was chosen, in its modified version which takes into account different values for the decay rates of biomass according to the electron acceptor available in the tank (Gernaey *et al.*, 2004). The default parameters with temperature correction as reported in Henze *et al.* (2000) have been used. This model is applied in all process configurations tested, even the ones not removing phosphorous, since in this way it was easier to perform a thorough comparison of effluent quality. The parameters most sensitive to temperature (growth rates and decays), and the concentration of dissolved oxygen at saturation, were considered as temperature-dependent. Based on the work of Gillot and Vanrolleghem (2003), a heat balance model in the WWTP was added to calculate the temperature in the tanks as function of incoming water and ambient air temperature, tank characteristics and aeration intensity, again applying the default parameters. For the primary settlers (where present) the model of Otterpohl and Freund (1992) was used with its standard parameter values. The model of Takacs *et al.* (1991) was adopted for the secondary settlers, with a modification to express the settling characteristics in function of the sludge volume index (SVI) as modelled by Daigger and Roper (1985), with standard parameter values and assuming an SVI of 100mL/g.

Modelling the River

To show 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 for 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 the one adopted in this study had been 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 function of water temperature, which is therefore of great importance and should be adequately calculated. To this end, a simple heat balance model was implemented in the river model to consider the effect of atmospheric changes on water temperature. Based on the model of Talati and Stenstrom (1990), the model includes the effects of solar radiation, atmospheric radiation, surface evaporation and surface convection in function of water surface and time series of daily incoming water temperature, radiation intensity, air temperature, wind speed and relative humidity. An addition to that model was made to include in a very basic way the contribution of base flow coming from groundwater, by just defining the quantity and temperature of the incoming groundwater.

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 to consistently connect any model expressed in the Petersen matrix format (Vanrolleghem *et al.*, 2005b), and the whole integrated model was implemented in WEST. The interface consists of a set 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).

An example of WWTP Design

To illustrate the proposed methodology, ten process configurations were selected to represent the most common plant layouts in use in Europe (see Table 1). The process volumes for single stage processes were dimensioned for 300,000PE according to the ATV guidelines (ATV, 2000). The other processes were dimensioned according to Vesilind (2003). The requirements for nitrification volumes, anaerobic retention time and hydraulic load to the secondary clarifier were followed. In particular for nitrification, the suggested solids retention time (SRT) for different plant sizes at the temperature of 10°C was calculated using the sludge production figures obtained from steady state simulations of the processes using a constant input created following the influent characterisation suggested in the ATV guidelines. The ten configurations were compared on their yearly performance for EQI and TC, and for exceedance frequencies of some effluent quality variables on a yearly basis and looking at the cold season only. In addition, for the LLAS configuration (see Table 1) different activated sludge volumes were compared.

Table 1: Plant configurations tested for WWTP design

Short name	Long name	Description
A2O	anaerobic-anoxic-oxic	low loaded system, performs biological N and P removal
AO	anaerobic-oxic	high loaded system, performs biological P removal
BDNP	Biodenipho	low loaded system, performs biological N and P removal
BDN	Biodenitro	low loaded system, performs biological N removal
HLAS	high loaded activated sludge	high loaded system
LLAS	low loaded activated sludge	performs biological N and chemical P removal
LLAS_PS	LLAS with primary settler	performs biological N and chemical P removal
OD_bioP	oxidation ditch with bio-P removal	low loaded system, performs biological N and P removal
OD_simP	oxidation ditch with simultaneous P precipitation	low loaded system, performs biological N and chemical P removal
UCT	modified University of Cape Town	low loaded system, performs biological N and P removal

An example of WWTP Upgrade

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. Figure 3 shows the general WWTP layout in WEST, which includes a CSO splitter and a by-pass with a storm tank. The specific configuration for "PROCESS" (see Figure 3 and Table 2) in the LLAS layout consists of one anoxic tank for pre-denitrification, followed by a P-precipitant addition point and six aerated tanks in series.

The upgrades were implemented for a 300,000PE plant size treating typical municipal sewage from a combined system. The upgrade scenarios were simulated for the Continental climate type, characterised by specific influent characteristics driven by temperature and rainfall (see Figure 2), which are fed to the influent generation model introduced above. An increase in loads with 33% (from 300,000PE to 400,000PE) has been applied to the influent of the plant to justify the need for upgrading. For all upgrades, the solids concentration in the activated sludge tanks was set to 3.5gTSS/l in summer and 4.5gTSS/l otherwise, with summer defined as the period with mixed liquor temperature above 16°C.

Figure 3: General plant layout in WEST; for node numbers explanation see Table 2

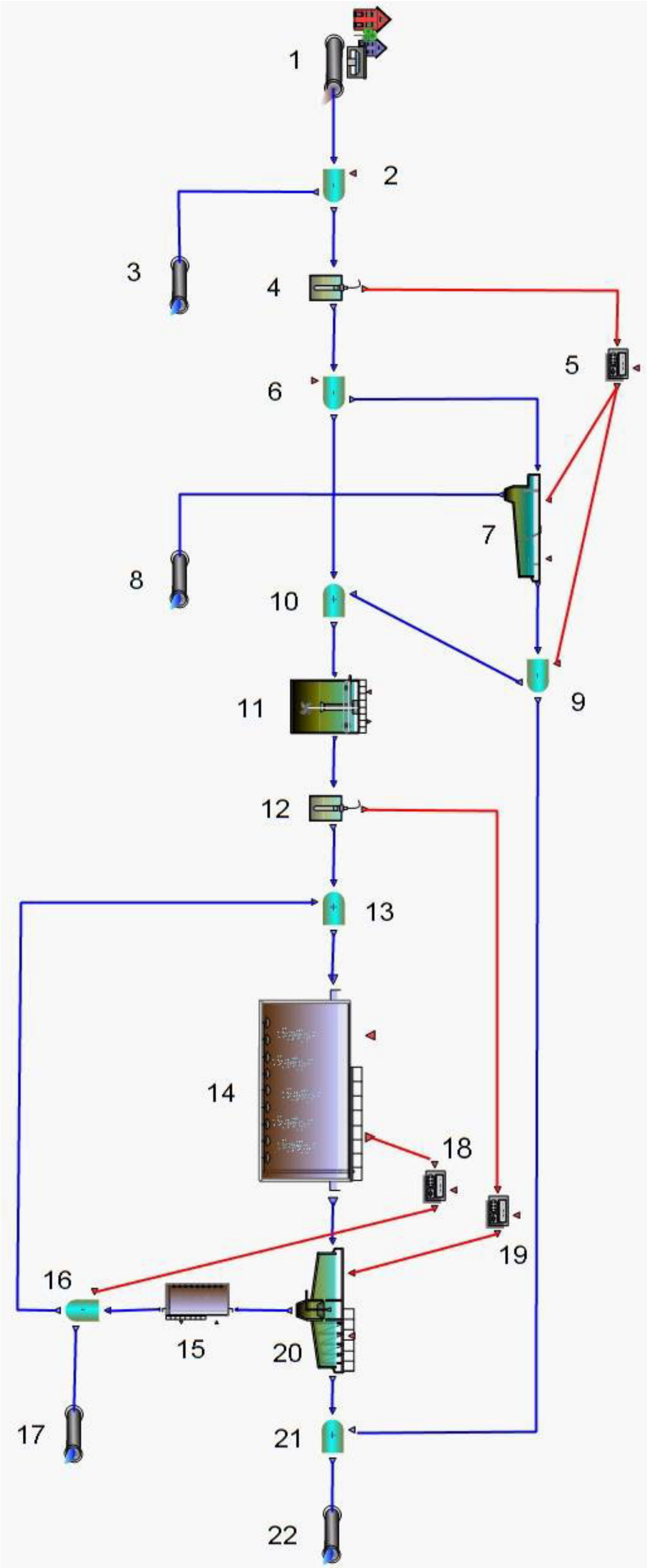


Table 2: Legend for nodes of Figure 3

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

Compared to the original 300,000PE LLAS plant dimensioned (Benedetti, 2006) according to the German ATV-131 guidelines (ATV, 2000), some changes were made to obtain the basic configuration called U0 ("upgrade zero", not upgraded), in order to mimic a situation where upgrades are needed due to load increase. The safety margins built 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 300,000PE, but not complying with the influent for 400,000PE (+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 is to 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, while some (e.g. Germany) have applied in their regulations stricter limits and/or limits based on effluents concentrations measured over short periods (e.g. 2-h composite samples) which 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 selected also 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, cables and controllers. The other seven upgrades also require constructions and equipment like pumping, piping and building of new reactor volumes.

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 called U0.

In RTC options, controller tuning is extremely important because an ill-tuned controller can be the cause for suboptimal results, while the same controller with well-tuned parameter values could allow savings in operational costs 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 2mgNO_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 2.5mgNO_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 2mgNO_3-N/l would better be lowered to 1mgNO_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 case the plant already complied with the standards, the aim was to reduce operating costs without exceeding the yearly average effluent quality limits with the 95th percentile.

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		

The river stretch model – connected to the WWTP models to perform the water quality-based evaluation – consists of 5 tanks in series, each representing a river stretch 1000m long and 30m wide, for a total length of 5000m. 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 treatment plant model, which includes the biological treatment effluent, the storm tank effluent and the combined sewer overflow (CSO) effluent.

Evaluation of Alternatives

The comparison of alternative scenarios is based on performance criteria that are grouped into two categories: environmental and economic criteria. The weight attributed to them in the decision making process depends on the specific situation of the case at hand, and should be left to the decision maker (e.g. the manager), not to the decision facilitator (e.g. the engineer).

Environmental Criteria

The proposed evaluation methodology is partly based on the approach set out by IWA and the EU COST-Action (Spanjers *et al.*, 1998). It consists of the evaluation of the effluent quality index (EQI) and of the COD, TN, TP and NH_4 effluents independently. The EQI is meant to quantify the effluent pollution load to a receiving water body in a single variable. The EQI is the weighted sum over one complete year of the pollution loads due to total suspended solids (TSS), chemical oxygen demand (COD), biological oxygen demand after 5 days (BOD_5), total nitrogen

(TN) and total phosphorus (TP). The weights used in the case study – 2 for TSS, 1 for COD, 2 for BOD_5, 20 for TN and 100 for TP – are based on Vanrolleghem *et al.* (1996b) that cited a Flanders' effluent quality formula for calculating effluent fines. Other weights can be used according to the specific sensitivity of the receiving water or to local effluent fine coefficients. Effluent violations – which are case-specific for the variables and thresholds to evaluate – were calculated for COD, TN, TP and NH_4; the thresholds for COD and NH_4 were chosen in order to be able to appreciate the different behaviour of the design options:

- COD: limit to 80mg/l for 3,000PE and 30,000PE and 65mg/L for 300,000PE;
- TN: limit to 15mg/l for 3,000PE and 30,000PE and 10mg/L for 300,000PE (CEC, 1991);
- TP: limit to 2mg/l for 3,000PE and 30,000PE and 1mg/L for 300,000PE (CEC, 1991)
- NH_4: limit to 3mg/l for 3,000PE and 30,000PE and 2mg/L for 300,000PE.

The percentage of time that the constraints are not met is calculated from the output data generated at 15-minute intervals.

<u>Economic Criteria</u>

The evaluation of costs for wastewater treatment is complex. In a European context, costs can differ among countries or regions because of different specific conditions and also because of differences in planning and building procedures (Bode and Lemmel, 2001). This complexity makes the approach to calculate costs in order to compare different plant configurations and operational strategies difficult. Detailed cost calculations should in general be preferred over the use of cost functions, which can only be useful for rough estimations. Most WWTPs are tailored to specific conditions/needs, i.e. plants with the same treatment performances do not inevitably lead to the same costs. The use of cost functions is feasible only for process options screening (Gillot *et al.*, 1999), i.e. as it is the case here. A detailed description of the way calculations were performed makes the assessment more transparent and comparable with other studies or available data. The main focus of this case study is the water treatment line, while sludge treatment was considered with less detail. The cost categories used in this case study are:

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

All the cost figures provided below and not referenced were received from Aquafin NV (Aartselar, Belgium). Since capital costs information was available for Germany, also the operational costs were given for the same country. Capital costs for the construction of tanks and for the associated mechanical equipment were calculated as function of the volume and of purpose (aeration, settling, etc.), using cost functions valid for Germany (Bohn, 1993; ATV, 1995; Günthert and Reicherter, 2001). Such capital costs were annualised using a service life of 30 years for the civil works and 15 years for the mechanical equipment, and an interest rate of

4%. Associated to capital costs are annual maintenance costs for civil works and mechanical equipment, respectively estimated as 0.5% and 3% per year.

The personnel requirement was estimated to be 8 people for a 300,000PE plant (ATV, 1995), with an associated cost of 50,000€ per person per year. Personnel costs are the same for all configurations, since they were assumed to be function only of plant size and not on plant type. For sludge production some assumptions have been made: for sludge treatment, a thickening table plus centrifuge, at a VC of 0.6 €/m^3 of sludge pumped out of the water line; for sludge disposal, incineration at a VC of 100 €/ton of dry solids. Capital costs for sludge treatment are not considered. The aeration energy (AE) in kWh/y was calculated as:

$$AE = k_L a \cdot S^* \cdot V / AEff / 1000 \cdot 365$$

where $k_L a$ is the oxygen transfer rate (obtained from the simulations) in d^{-1}, S^* is the difference between the oxygen concentration at saturation and the one in the aerated tank (both obtained from the simulations) in gDO/m^3, V is the tank volume in m^3 and *AEff* is the transfer efficiency of the aeration equipment, assumed to be 1.5kgDO/kWh (for fine bubble aeration); it is known that the latter parameter varies as function of other quantities (e.g. temperature), but it was decided to keep it constant throughout the year to simplify the evaluation and to be consistent with the overall level of complexity in the cost calculations. Pumping energy resulted from the simulated flows to be pumped, assuming a head loss of 0.8m for mixed liquor recirculation and 2m for secondary sludge recirculation. Mixing energy was assumed to be 2W/m^3 of volume to be mixed. The cost of energy has been fixed to 0.1€/kWh. As for chemicals, the cost associated to P-precipitant ($FeCl_3$ at 14% concentration) was assumed to be 100€/ton for 3,000PE and 65€/ton for 30,000 and 300,000PE; for C-source (acetic acid at 9% concentration) the cost was 70€/ton.

For upgrades, personnel costs result to be zero in all comparisons, since it was assumed that no extra or more specialised personnel was required in the upgraded plant, given the large size of the plant. The assessment of the effect of different WWTP upgrades on the receiving water quality (immission-based evaluation) was done by analysing quality variables in one or more points of the river. In this study, the yearly averages and exceedance periods of concentration thresholds were taken in the last tank of the river model (5.000m downstream the WWTP effluent) for dissolved oxygen (DO) and in the first tank (1.000m downstream the WWTP effluent) for NH_4, NO_3, PO_4 and COD. The choice of location followed after evaluation of the critical sections for those water quality parameters. The values of the thresholds for the exceedance analysis are 0.5mgNH_4/l and 5mgDO/l.

Three WWTP options were compared for the immission-based evaluation, two already present in the emission-based evaluation (U0 and U2) and one additional upgrade (U13) that was added only here since it would have a positive effect only in case 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.

Probabilistic Aspects

In principle, all model parameters should be considered uncertain (all biochemical model parameters and operational parameters), but in practice only a limited set of parameters are not assigned a deterministic value but a probability density function (PDF). The selection of the parameters to be assigned a PDF can be done by performing a sensitivity analysis of the model, and then assigning a PDF to the most sensitive parameters only, or by referring to literature results for the same type of model. In this work, the modified ASM2d parameters considered as uncertain were chosen according to (Rousseau *et al.*, 2001) and to expert knowledge. Also, some parameters of the influent fractionation model are uncertain since the influent composition is considered as uncertain. The parameters (see Henze *et al.*, 2000) with their statistical properties are listed in Table 4. The parameters $\mu_H\_b_H$, $\mu_{AUT}\_b_{AUT}$ and $\mu_{PAO}\_b_{PAO}$ have been introduced to take into account the correlation which is known to exist between the biomass maximum growth rate and the decay rate. Each Monte Carlo shot for the *b* parameter is calculated by dividing the shot's μ parameters by the shot's μ_b parameter. The *b* parameters are also uncertain, but strictly correlated to the μ parameters. No other parameter correlations have been considered in this study.

Table 4: Uncertain parameters listed with their statistical properties

Name	Probability density function	Mean (median)	Minimum	Maximum	Standard deviation	Unit
f_{S_F}	triangular	0.375	0.3	0.45	-	-
f_{X_S}	triangular	0.68	0.544	0.816	-	-
μ_H	truncated normal	6	4.8	7.2	0.4	d^{-1}
μ_{AUT}	truncated normal	1	0.8	1.2	0.067	d^{-1}
μ_{PAO}	truncated normal	1	0.8	1.2	0.067	d^{-1}
$\mu_{H}\_b_H$	uniform	-	9.2	11.4	-	-
$\mu_{AUT}\_b_{AUT}$	uniform	-	4.6	5.7	-	-
$\mu_{PAO}\_b_{PAO}$	uniform	-	4.6	5.7	-	-
η_{NO3_Hyd}	triangular	0.6	0.48	0.72	-	-
η_{NO3_Het}	triangular	0.8	0.64	0.96	-	-
η_{NO3_PAO}	triangular	0.6	0.48	0.72	-	-
K_{O_A}	triangular	0.5	0.25	0.75	-	$gO_2\ m^{-3}$
Y_{PO}	triangular	0.4	0.32	0.48	-	$gP\ gCOD^{-1}$
$\eta_{NO3\_Het\_d}$	triangular	0.5	0.4	0.6	-	-
$\eta_{NO3\_P\_d}$	triangular	0.33	0.264	0.396	-	-
$\eta_{NO3\_Aut\_d}$	triangular	0.33	0.264	0.396	-	-

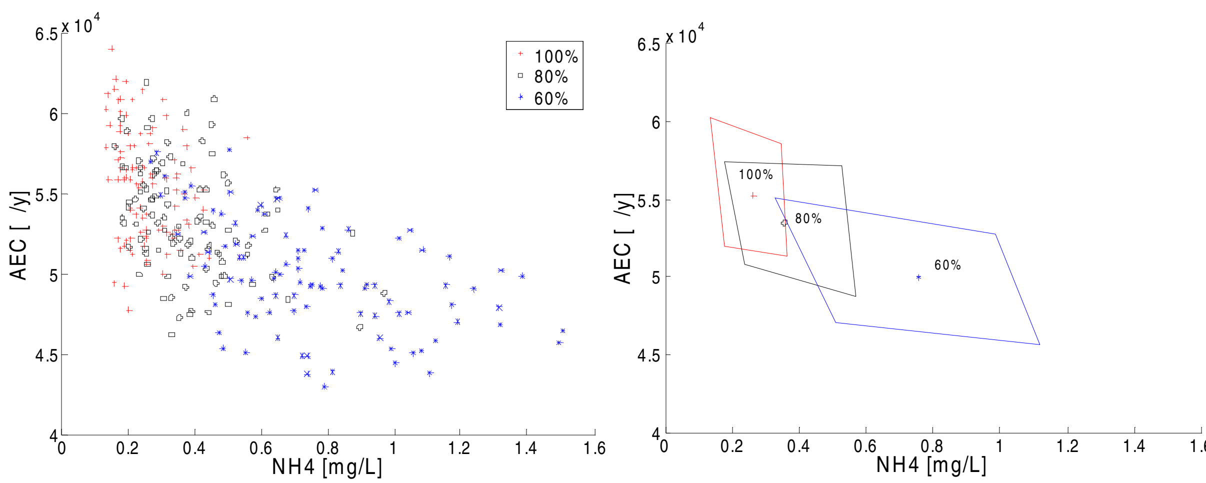
For each combination of plant configuration, size and climate, 100 parameter combinations were sampled from the parameter space using Latin Hypercube Sampling (LHS) (McKay, 1988) to perform the MC uncertainty assessment. This number of simulations was found to be sufficient to reach convergence of the simulation output distributions, being more than 7 times larger than the number of uncertain parameters (Benedetti *et al.*, 2009). The minimum number of simulations to perform LHS can be as small as 4/3 of the number of parameters (McKay, 1988).

Data Analysis

The most immediate way to visualise the effluent dynamics is by means of time series. Because of the very pronounced dynamic behaviour of the simulation output time series – data every 15 minutes for one year (35.040 data points) – the picture can be difficult to interpret. Simple and effective extraction of information can be done by averaging or the creation of concentration-duration curves. In the presence of not a single time series but of a large number of individual time series as in the case of MC simulations, better ways to interpret and summarise the large amount of data have been developed and applied. Two options are here presented for data analysis: (1) averaging of time series and (2) analysis of threshold exceedances.

In the first option the long time series are summarised by calculating for each simulation the average (in this case over one year) for the variables of interest. For example, in Figure 4 (left side) the yearly averages of NH_4 and of aeration energy cost (AEC) are plotted for each of the 100 MC simulations executed for each of the three different configurations tested, in this case LLAS with three different aerated sludge volumes. Since the comparison between the three design options does not appear straightforward due to the overlapping of the three clouds of 100 dots, figures like Figure 4 (right side) have been developed to express the uncertainty characteristics of the MC simulation results. Each of the polygons has been created by joining the 5th and 95th percentiles of the 100 data points calculated along the two principal axes found by using principal component analysis (PCA). The markers in each of the polygons represent the 50th percentiles of the 100 averaged time series. The stability of the process configuration is inversely proportional to the perimeter of the polygon.

Figure 4: Two options to visualise Monte Carlo simulation results: all results as a cloud of markers (left) and polygons joining the 5th and 95th percentiles for the two variables and the 50th percentile as a marker (right); the data show average effluent NH_4 and average aeration energy costs for different tank volumes for LLAS (for 60%, 80% and 100% of ATV dimensioning volume)



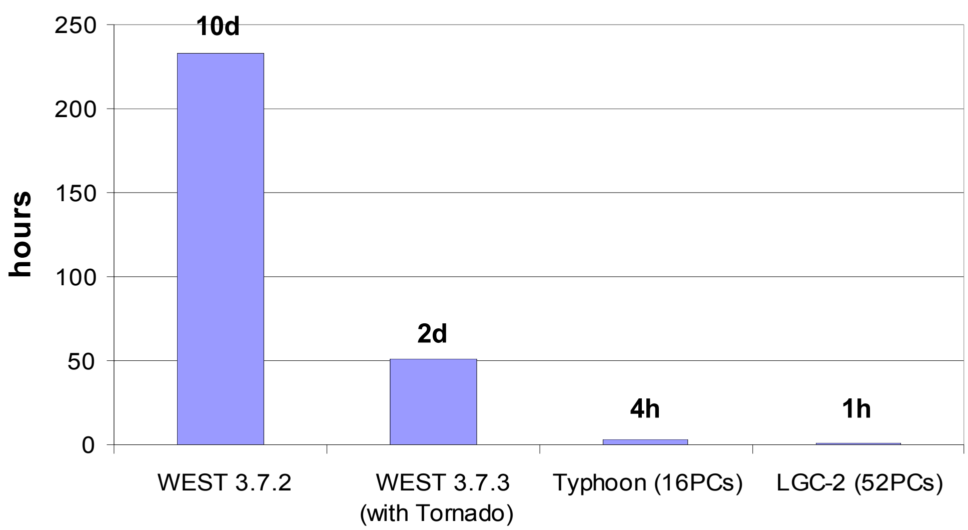
The second option to summarise the large amount of data is the concentration-duration box-plot (for examples see Figures 7, 8, 9 and 12), an instrument to also evaluate the difference among the options in their dynamic behaviour (rather than only its average). The box has lines at the lower quartile, median and upper quartile values; whiskers extend from each end of the box to the adjacent values in the data, the most extreme values within 1.5 times the inter-quartile range from the ends of the box; outliers are data with values beyond the ends of the whiskers. It allows to evaluate, for any given concentration value, the duration (in percentage of the total simulation period) for which that value has been exceeded, and the variability of this duration due to the parameters uncertainty.

Software Tools

In order to make the proposed methodology feasible, new software tools were developed for practical use. These tools are now available as part of the WEST (MOSTforWATER, Kortrjik, Belgium) product suite. A new modelling and virtual experimentation kernel for water quality systems (named "Tornado") has been developed in order to be able to cope with the large computational load implied by Monte Carlo simulation of complex WWTP layouts over one year (Claeys *et al.* 2006b; Benedetti *et al.* 2008a).

The number of simulations that are needed when performing Monte Carlo-based uncertainty assessment tends to be large, and each simulation of a treatment plant over one year under highly dynamic conditions may take considerable computation time. To reduce this computational burden, a piece of software (named "Typhoon") that distributes simulations over idling PCs available in a local network has been developed for this study (Benedetti *et al.* 2008a; Claeys *et al.*, 2006a). A cluster of 16 Linux machines with 3GHz processors at BIOMATH with Typhoon, and the Ghent University Grid with 52 nodes – with the LGC-2 software used to automatically distribute the simulations – were available for this work. To show how the feasibility of the proposed methodology of probabilistic analysis is dramatically increased by the development and use of Tornado and Typhoon, Figure 5 shows the comparison of the execution of a batch of 100 Monte Carlo simulations of the LLAS model with the previous version of WEST (3.7.2) used in this study, with WEST 3.7.3 which includes Tornado, with the BIOMATH cluster using Typhoon and on the Ghent University grid using LGC-2.

Figure 5: Execution time for 100 simulations of the LLAS configuration using different tools

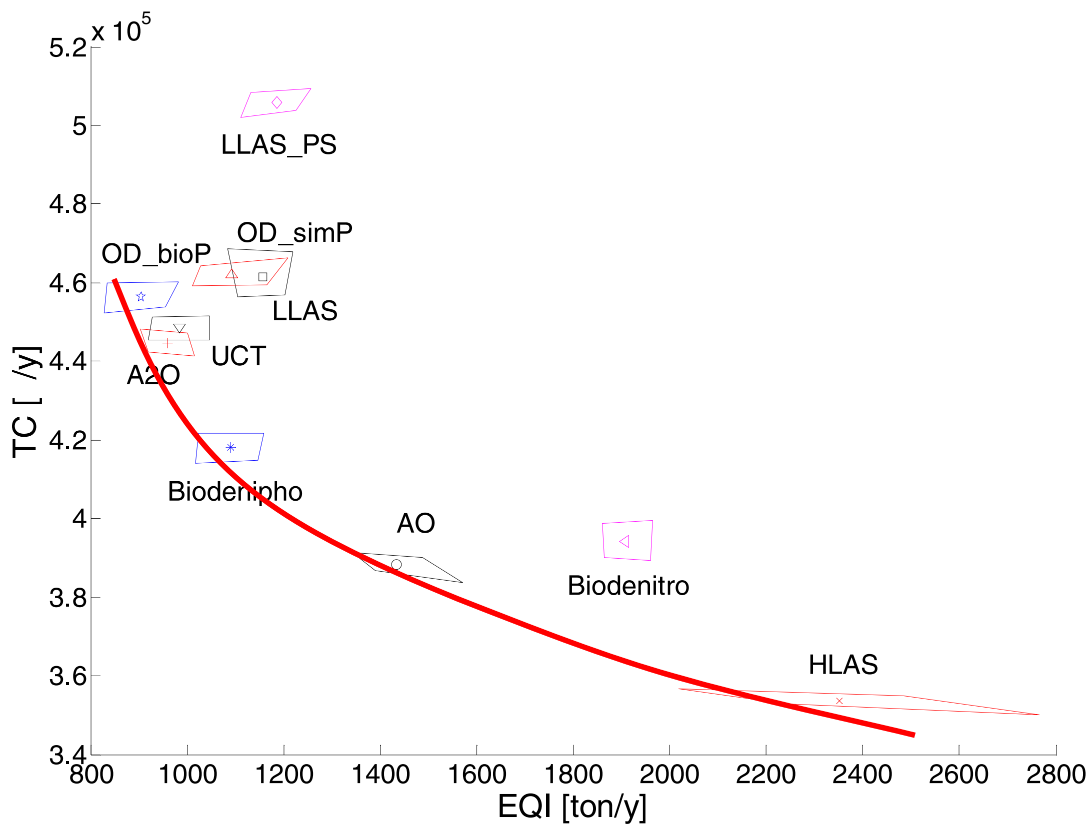


RESULTS

Comparison of WWTP Design Options

The performance of ten configurations was studied to quantify which process types perform better for the different purposes, and which is their performance reliability (uncertainty). Figure 6 shows the comparison of the 10 WWTP design options on the basis of their EQI and of the total costs. The line of Pareto-optimality (an option is Pareto-optimal when there is no other option which is performing better for all criteria) is shown in the figure – comprising the HLAS, AO, Biodenipho, A2O and OD_bioP configurations – which is very useful for the multi-criteria decision-making process, for its ability to guide the analysis of trade-off between environmental and economic objectives.

Figure 6: Comparison of the 10 WWTP design options for 300,000PE on the basis of their EQI and of the total costs (TC); Pareto front indicated by the red line



To evaluate the performance of process configurations in critical periods, the case of limited nitrification with cold temperature was chosen, since classic design procedures focus on nitrification at winter temperatures. The time window for cold temperature was defined as the period with influent wastewater temperature lower than 12°C (see right side of Figure 2). The analysis is restricted to the exceedance of certain effluent concentration thresholds since they are the most important indicators in this case.

For each indicator the whole year analysis is compared to the winter period. For COD (Figure 7) there is a clear improvement of effluent COD concentrations in the cold period for all configurations. This is due to the larger influent dilution in winter time caused by the higher infiltration in the sewer system. This is confirmed by looking at the COD removal in terms of load (Figure 7) for which, as expected, a slightly lower removal is found for all configurations with cold temperature, with equal incoming pollution load in the two periods.

An example of results for different activated sludge volumes for the LLAS is given in Figure 8, showing that with decreasing process volume the aeration costs decrease but the effluent ammonia and the instability of the process (proportional to the polygon perimeter) are increasing. Figure 9 shows the exceedance of the threshold of 3mgNH_4/L for different volumes, interestingly pointing out that without safety factor (i.e. with safety factor = 1 at 60%, as $1.6 \cdot 0.6 \approx 1$) the process is at the limit of stability.

Figure 7: Exceedance time of 65mgCOD/L (top) and COD load removal (bottom) for ten 300,000PE configurations; full year (left) and cold period (right)

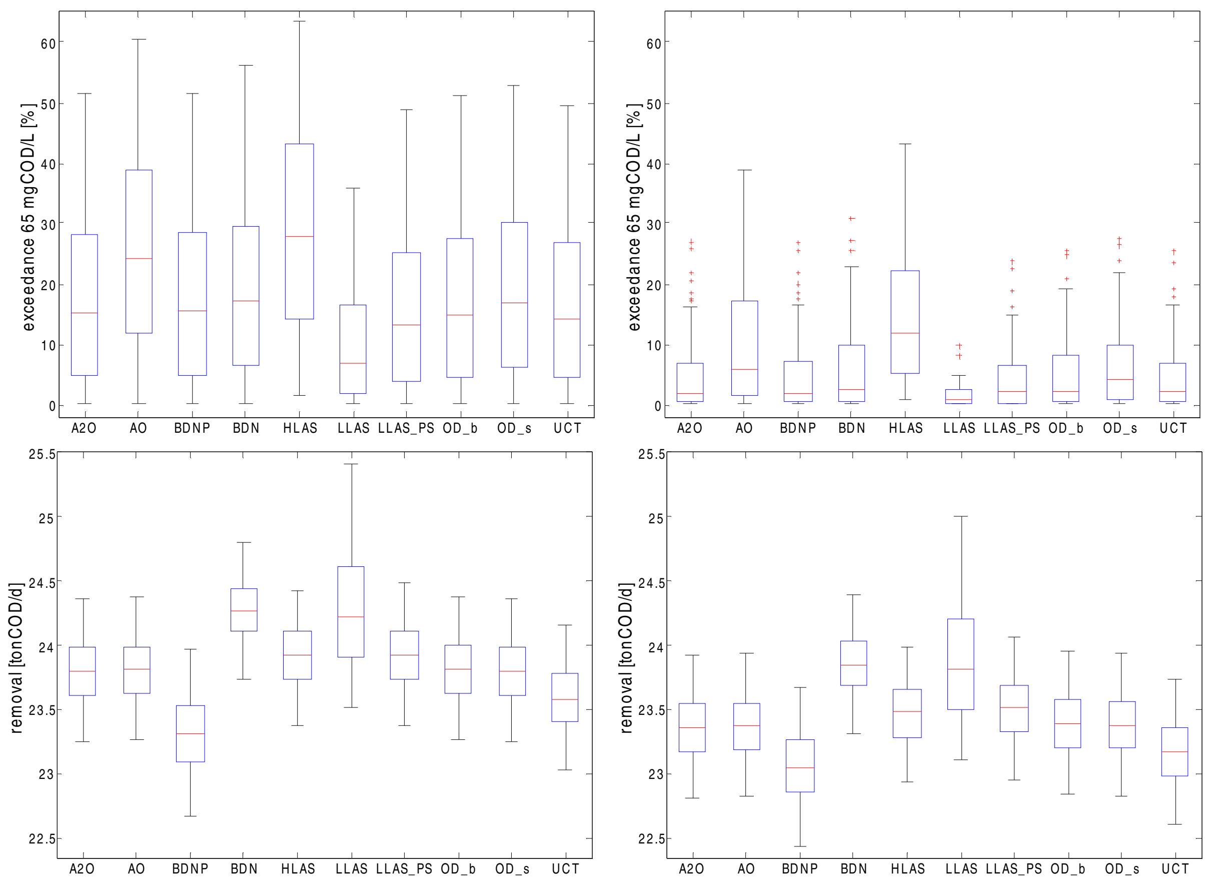


Figure 8: Exceedance time of 2mgNH_4/L for ten 300,000PE configurations; full year (left) and cold period (right)

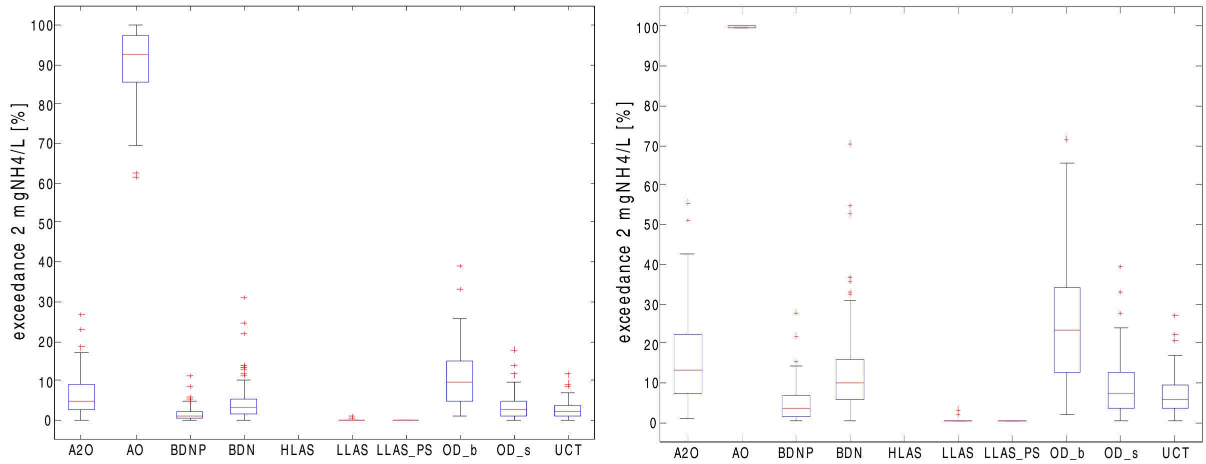
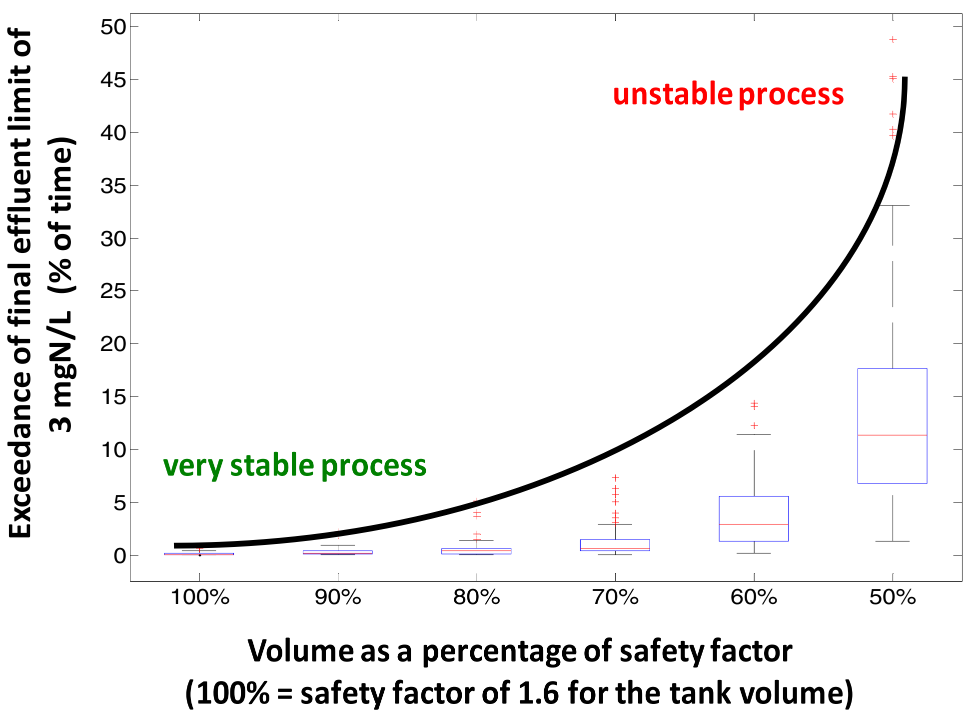


Figure 9: Exceedance time of 3mgNH_4/L for LLAS configuration in function of activated sludge volume



Comparison of WWTP Upgrade Options

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. U11 was excluded since in the assumed climate condition (Continental weather, see Figure 2) 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 the one of U0.

Emission-based Evaluation

The emission-based evaluation is performed by an economic assessment and by an environmental assessment of the options to be evaluated. Figure 10 shows 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, which helps in finding the option with the preferred trade-off between the two plotted variables.

The economic performance was evaluated on the basis of the difference in costs of the upgrade (including U0) fed by the 400.000PE influent minus the costs of U0 fed by the 300.000PE influent. In terms of additional total costs (Figure 11), the “hard” upgrades U1, U2 and U3, which involve mainly constructional intervention, are clearly more expensive than the RTC upgrades. The larger volumes of “hard upgrades” also entail higher additional energy costs mostly due to higher aeration costs, where the general trend can be noticed that lower NH_4 effluent concentrations go together with higher aeration costs.

Figure 11 illustrates that the majority of the additional total costs for upgrade is due to variable costs, and that 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 C-source dosage.

Although it might seem from these figures that all upgrade options have total additional 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, which means that in absolute terms there is certainly a difference that is worth consideration.

It can be noticed that U2 shows the best environmental performance, and that all upgrades have a 50th percentile EQI that is lower than that of U0. Concerning the effluent concentrations, it can be seen that almost all upgrades have better nitrogen removal than U0. 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 has better denitrification performance. This can partly be attributed to the larger final clarifier, in which it is assumed that anoxic processes 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 with 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 a biological phosphorus removal by adding extra anaerobic tank volume and dosage of external carbon source. In spite of those extra investments, the figures show that the environmental performance of U3 is worse than that of U1 and U2. The higher effluent ammonia and TN concentrations in U3 can be attributed respectively to the lower DO set-point used – an attempt to lower the aeration costs – and to the introduction of biological phosphorus removal before the denitrification tank, which leads to the use of most of the carbon source by the phosphorous accumulating organisms, in this way decreasing the denitrification performance.

The poor performance and process instability (large polygon) of U10 concerning nitrogen removal (Figure 10), 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.

Figure 10: EQI and additional TC (left) and TN and additional EC (right) for LLAS 300,000PE upgrades in; U0 with bold polygon; Pareto front with bold line

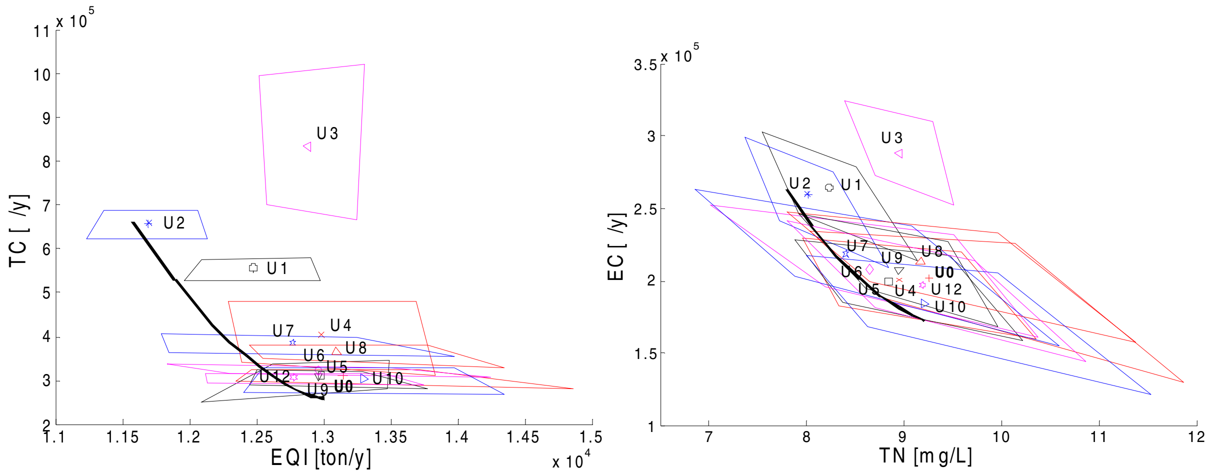


Figure 11: Additional TC (left) and VC (right) for LLAS upgrades; CCCA=capital cost for construction annualised, CCEA=capital cost for equipment annualised, CCSeA=capital cost for sensors annualised, MC=maintenance cost, VC=variable cost, PrC=personnel cost; AEC= aeration energy cost, PEC=pumping energy cost, MEC=mixing energy cost, PPC=P-precipitant cost, CSC=C-source cost, SC=sludge cost

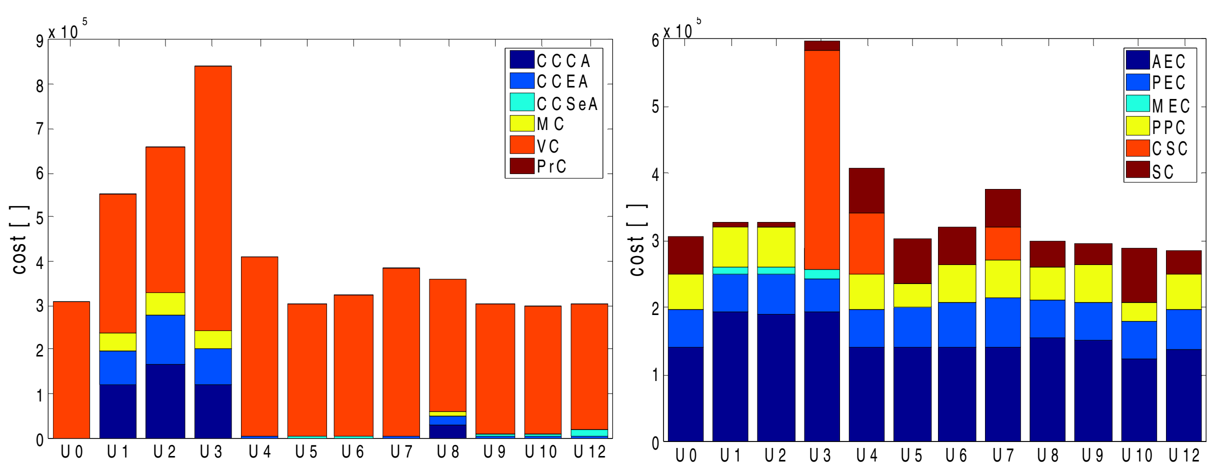
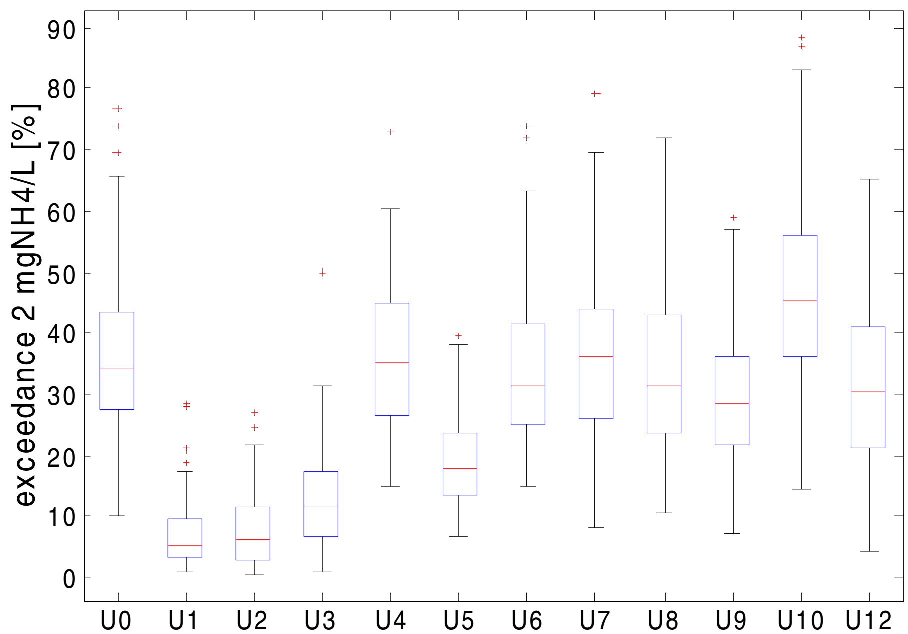


Figure 12: Exceedance time of 2mgNH$_4$/L for LLAS upgrades



Immission-based Evaluation

First of all, a basic emission-based evaluation of the three considered alternatives (U0, U2 and U13) is performed. From Figure 13 it can be deduced that U2 implies higher costs (in particular capital cost) and that U13 has lower costs than U0. Further analysis reveals that the higher hydraulic load through the treatment plant under U13 leads to a lower MLSS concentration in the aerated tanks – due to larger effluent TSS in wet weather – which entails lower aeration requirements and also lower sludge production (Figure 14). 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 explaining the good performance of U13 is the increased maximum pumping capacity, which allows to recirculate more nitrates to the anoxic tank, allowing for a better utilisation of oxygen in the form of nitrates during the denitrification step. No sludge losses happen 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 (100mL/g) was assumed in all simulations.

On the other hand, the EQI (pollutant loads) of U13 is not far from the one of U0, and both are around 20% worse than U2.

Figure 13: Yearly average EQI and TC for LLAS

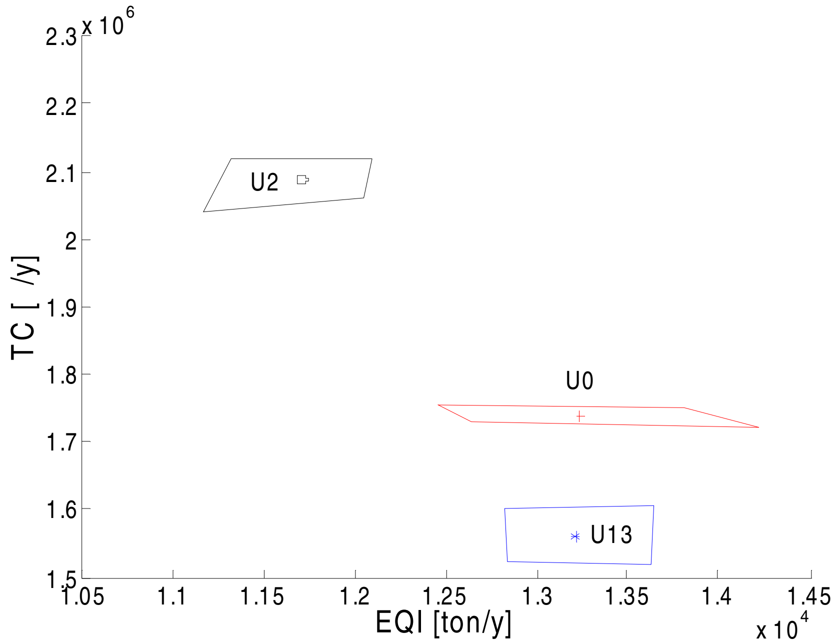
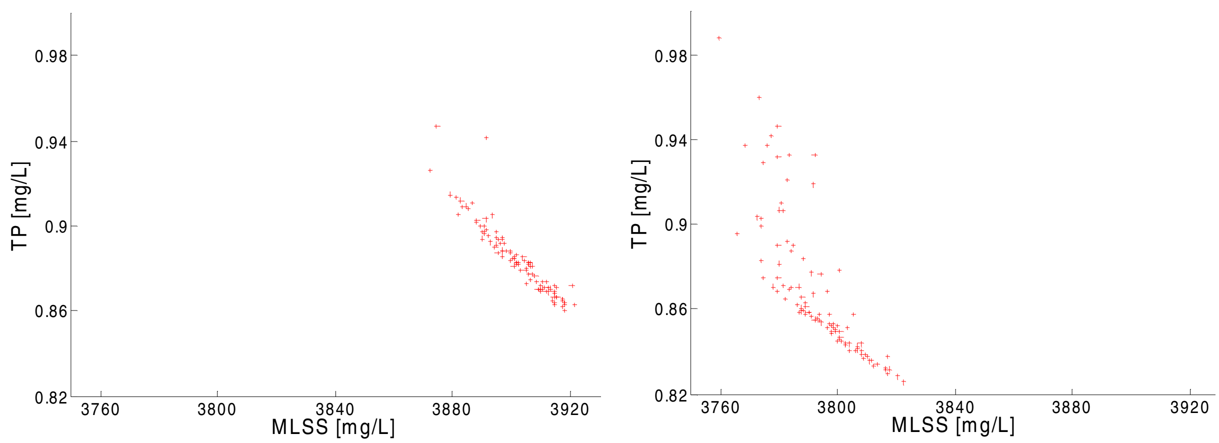


Figure 14: MLSS in the tanks and TP in the effluent for U0 (left) and U13 (right)



Passing to the immission-based evaluation, one can notice a clearly better situation with U13 when analysing the average concentrations in the river (Figure 15). For NH_4, the winter period penalises U0 for its difficult nitrification – both in terms of 50th percentile and of process stability. U13 achieves lower NH_4 in the river than U2, while NO_3 is lower with U2 but only very slightly. Also for DO and COD the pattern is similar, with U13 performing slightly better than U2 and with U0 clearly showing its deficiencies. Concerning the exceedance periods for NH_4 and DO (Figure 16), they all show the same behaviour, with U0 clearly having larger exceedance periods than U2 and U13, which perform very 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.

Figure 15: Yearly average NH_4 and NO_3 (left) and PO_4 and COD (right) in the river 1.000m downstream the WWTP effluent

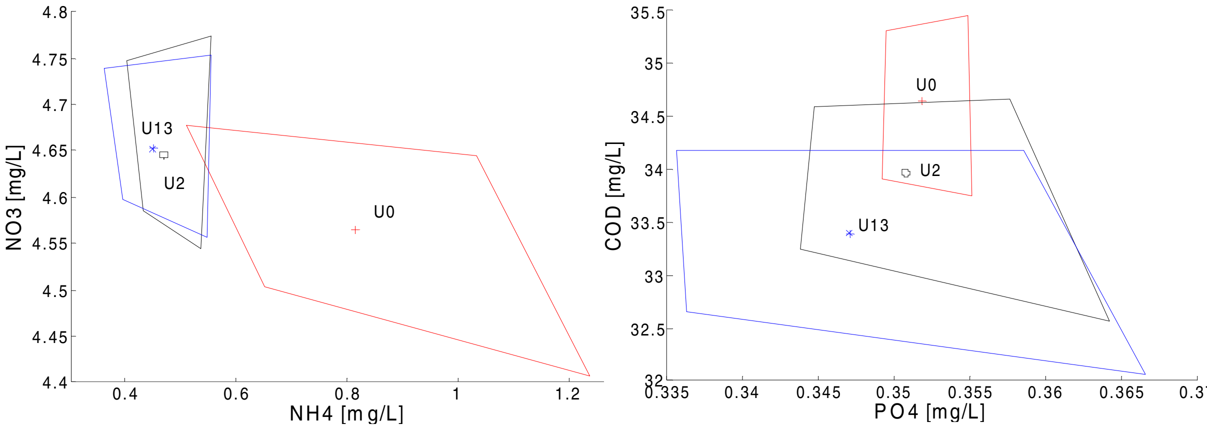
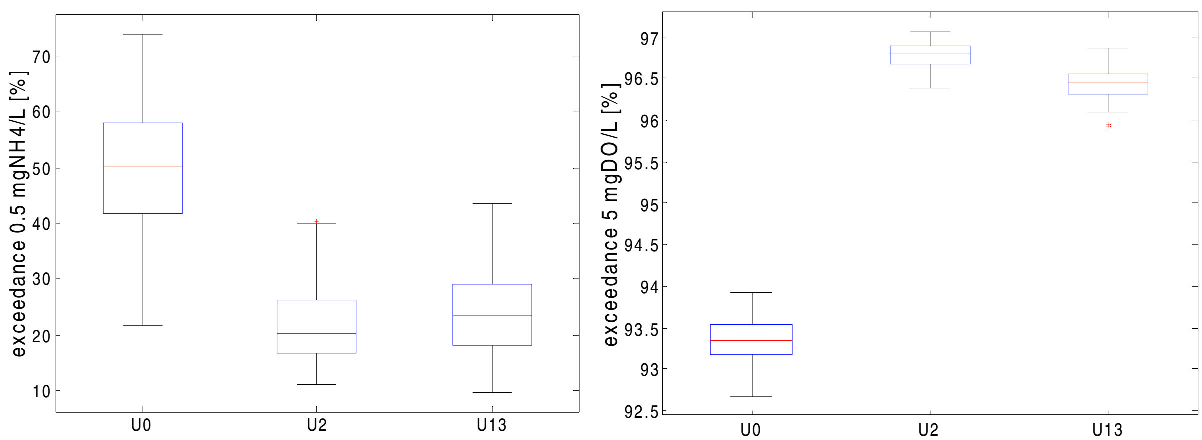


Figure 16: Exceedance of 0.5mgNH_4/L in the river 1.000m downstream the WWTP effluent (left) and exceedance of 5mgDO/L in the river 5.000m downstream the WWTP effluent (right)



CONCLUSIONS

The proposed methodology for benefits, costs and risk of failure analysis of wastewater treatment systems under uncertainty, was illustrated for the case of WWTP design.

First of all, long time influent time series were generated, then the alternative process configurations were designed and implemented in the modelling and simulation software WEST, the uncertainties were characterised, and finally, after performing Monte Carlo simulations, the alternatives were compared and evaluated. To that purpose, the following instruments had to be developed:

- a phenomenological model of the waste- and rainwater generation to feed the WWTP model
- a software to distribute simulations on a computer cluster
- graphical representations of uncertainty, in particular the percentile polygons
- a data processing tool to perform complex statistical analyses of environmental and economic performance on the output data of the MC simulations

When comparing ten different process configurations, alternating systems show the best cost-benefit performance while high loaded systems show the lowest.

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 first generally providing more process stability (less spread of the Monte Carlo simulations, i.e. less output uncertainty) at high cost, and the second delivering good performance improvement at low cost but with more risk of compliance failure.

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

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

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