Model-based evaluation of a full-scale wastewater treatment plant for future influent and operational scenarios

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Abstract: With the need to accommodate a rising population in cities and increasingly stricter effluent limits, wastewater treatment plants undergo major extensions. While traditional design approaches are the first step in determining the new required volumes and design, several other questions in terms of the operation of the future wastewater treatment plant cannot be answered using a steady-state design model, e.g. changes in infiltration rates, impervious area, rainfall patterns, operational and control strategies. This paper presents a model-based toolbox to generate various dynamic influent scenarios (using an influent generator model) that are further used to evaluate the performance of the wastewater treatment plant (using a dynamic process model). The approach is demonstrated for the Henriksdal wastewater treatment plant in Stockholm, Sweden. The results highlight the usefulness of the tool in comparing changes in influent flow rate due to decrease in impervious area and infiltration to sewers. Simulation results show a stronger impact of reduction in infiltration rate on wastewater treatment plant performance when compared to reduction in impervious area. Also, a modified control strategy for dissolved oxygen control has shown significantly lower time in violation (25% of the baseline case) of the effluent ammonia limit with only 4% increase in operational costs. While simple examples are used in this paper to illustrate the applicability of this toolbox, numerous other more complex influent and operational scenarios can be studied using the model.

Keywords: influent generator; future scenario analysis; dynamic modelling; wastewater treatment plant

Introduction

Wastewater treatment plant (WWTP) effluent limits are expected to become even stricter in the future. This will lead to additional pressure on the operation of WWTPs. Many WWTPs will have to expand their treatment capacity in order to meet these effluent limits under predicted future loads. These expansions are traditionally designed based on estimates of the future increase in population equivalents. While there is uncertainty in these estimates, several other aspects, e.g. changes in impervious area, infiltration, rainfall patterns, expansion of the city to include new areas, new industrial zones etc., are difficult to incorporate during the design of new facilities. Model-based simulation studies can be highly useful to analyse the preparedness of a WWTP for such scenarios. There are two essential elements required for performing a model-based study: i. process model of the future WWTP; and ii. influent generator model that is capable of simulating multiple potential scenarios.

Process models are extensively used for evaluating operational strategies in WWTPs (Gernaey et al., 2014; Sharma et al., 2013). However, the lack of dynamic influent data is considered a major bottleneck for such investigations (Martin and Vanrolleghem, 2014). A combination of: i. influent generator model that can predict realistic influent profiles based on future scenarios; and ii. a process model to determine the performance of the WWTP for these influent profiles is presented in this paper using a case study from Henriksdal WWTP, Stockholm, Sweden.

The catchment model from BSM-UWS (Benchmark Simulation Model for Integrated Urban Wastewater Systems) (Saagi et al., 2016) is used to generate dynamic influent data. Model calibration is performed using data from the year 2012. A dynamic process model using the Benchmark Simulation Model No. 2 (BSM2) (Jeppsson et al., 2007; Nopens et al., 2010) model library is developed for the future Henriksdal WWTP configuration. The paper highlights two different influent scenarios (decreased impervious area and infiltration) and a modified control strategy. These scenarios are compared with the expected baseline performance for the year 2040. The results highlight some key findings for the Henriksdal WWTP and more importantly illustrate the potential of the toolbox for evaluating multiple scenarios creating changes related to the influent wastewater as well as WWTP operation.

Case study - Henriksdal WWTP, Stockholm

Influent characteristics

A brief overview of the influent characteristics (year 2012) for Henriksdal WWTP is provided in Table 1. In addition, the details for the combined sewer flows from the sub-catchments Bromma and Eolshäll are also included. This is necessary as the future influent to Henriksdal will include all wastewater from Bromma and Eolshäll as well. A 14 km long sewer tunnel will connect these two sub-catchments to the Henriksdal WWTP. A 32% increase in population equivalents is expected until year 2040 for the sub-catchments Henriksdal, Bromma and Eolshäll (Grundestam and Reinius, 2014).

Table 1. Key influent characteristics for the different WWTPs in 2012.

Catchment	Daily average flow rate ($m^3.d^{-1}$)	Population equivalents (PE)	Industrial contribution ($m^3.d^{-1}$)	Total catchment area (ha)
Henriksdal	283 000	784 000	28 000	19 200
Bromma	141 000	328 000	14 000	9 900
Eolshäll	40 800	119 000	0	2 000

While calibration data is available for two of the influents (Henriksdal and Bromma), influent flow data from Eolshäll is not available. Given that, it is only a small sub-catchment (7% of the total catchment area), the lack of detailed data for calibration is not considered a major issue. Flow rate data is available at 15 min intervals and pollutant load data is accessible as weekly composite values and as daily composite values (once every week). Temperature data is also available at high frequency. Other essential input data required for the model are rainfall intensity, evaporation and ambient air temperature.

Future WWTP configuration

The current Henriksdal WWTP facility is built in a rock cavern. The future WWTP configuration is guided by three main factors: i. usage of the existing space in the rock for the new facility without having to expand the area; ii. increased pollutant load due to the closure of Bromma WWTP and diversion of this flow to the Henriksdal WWTP; and iii. stricter effluent requirements. Given the space constraints, membrane bioreactor (MBR) technology is chosen to achieve improved treatment without expanding the volumes significantly. Figure 1 provides an overview of the WWTP process at Henriksdal for one line (Andersson et al., 2016). Seven such lines will be built in the future. Key design parameters are presented in Table 2. The future WWTP is designed for 1 600 000 PE with a design flow of 527 000 $m^3.d^{-1}$.

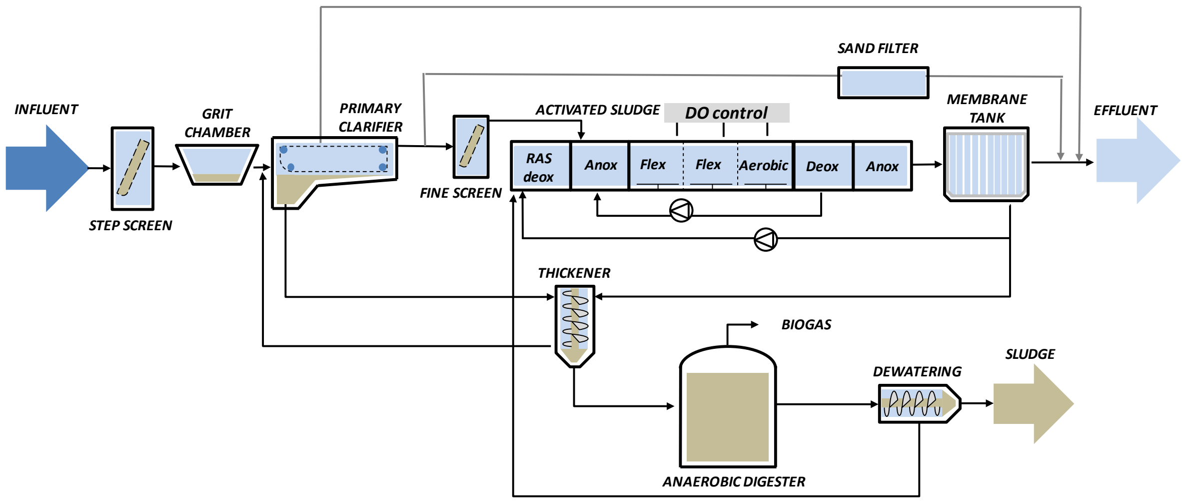

Figure 1. Process configuration for the future Henriksdal WWTP.

Primary treatment includes primary clarification after step screens and grit chambers. The primary clarification is essential not only to reduce the biological load on the following processes but also to enhance biogas production. Additionally, fine screens are used to further remove hair and fibers in order to protect the membranes. A bypass for high flow rates is present. Secondary treatment uses a pre-denitrification/nitrification/post-denitrification layout since only pre-denitrification will not be sufficient to meet the future effluent total nitrogen (TN) limit (6 $g.m^{-3}$). A deoxygenation zone at the beginning of the biological treatment is used to remove oxygen present in the return sludge due to membrane scouring. This is followed by a pre-denitrification zone where internal recirculation flow is also added. Thereafter comes flex zones (flex tank 1 and flex tank 2), possible to use either as anoxic or aerobic reactors. A nitrification zone with oxygen supply is followed by a post-dentrification zone where external carbon source is added for further removal of nitrate. Finally, membrane tank separates the sludge from the effluent. While the sludge is recirculated, effluent is finally discharged into the Baltic sea. Additionally, sand filters are used to remove some organics and particulate contents from the bypass flow prior to the biological reactors, which is then combined with the effluent. Phosphorus removal is achieved by chemical precipitation. Iron salts are dosed in the primary clarifier, biological reactors and sand filters. Anaerobic digesters receive thickened sludge from the primary clarifier and membrane tank to produce biogas, the remaining sludge is sent to landfill while the reject water is returned to the RAS deoxygenation zone.

Table 2. Key design parameters for the future Henriksdal WWTP (modified from Andersson et al. (2016)).

Parameter	**Unit**	**Value**
Design load	PE	1 621 000
Average daily flow	$m^3.d^{-1}$	542 000
Max flow to biological treatment	$m^3.d^{-1}$	864 000
Sludge conc. biology	$g.m^{-3}$	8 000
Sludge conc. membrane tanks	$g.m^{-3}$	10 000
Volume of biological treatment	m^3	204 000
Volume of membrane tank	m^3	40 000
Installed membrane area	m^2	1 600 000
Anaerobic digester volume	m^3	39 000
Total sludge age	d	28

Model description

Influent generation

The influent generator model (Flores-Alsina et al., 2014) is modified into the BSM-UWS catchment model (Saagi et al., 2016) that can describe different sources of wastewater generation (domestic, industrial, stormwater and infiltration to sewers). Dry weather dynamics are modelled using time varying profiles (daily, weekly and yearly) multiplied by the average flow rate and pollutant loads from domestic and industrial sources. The stormwater model considers rainfall runoff from impervious and pervious surfaces separately. An infiltration model describes the variation in infiltration to sewers based on groundwater levels (varying on an annual basis) and pervious area runoff. Additionally, wastewater temperature is modelled using daily and seasonal variations along with reduction in temperature due to precipitation events.

WWTP process model

The BSM2 model library is used to build the process model for the future Henriksdal WWTP. The primary clarifier is modelled using empirical equations for settleability of solubles and particulates (Otterpohl & Freund, 1992). The Activated Sludge Model No. 1 (ASM1) (Henze et al., 2000) is used to describe the biological processes. The membrane tank is modelled using a combination of a biological reactor and a thickener where solids separation takes place. An ideal thickener with predefined efficiency is used to mimic the membrane performance. The anaerobic digester is modelled using the Anaerobic Digester Model No. 1 (ADM1) model (Batstone et al., 2002). Additionally, an empirical model (with pre-defined efficiency) is used to model dewatering of primary and secondary sludge. Sand filters to treat bypass flow are modelled with fixed removal efficiencies for the different soluble and particulate fractions. The return activated sludge flow rate is controlled to maintain a suspended solids concentration of 8 000 $g.m^{-3}$ and 10 000 $g.m^{-3}$ in the bioreactors and membrane tank, respectively. The internal recirculation flow rate is 3.2 times the design flow rate, 1 702 000 $m^3.d^{-1}$. The bypass limits before and after the primary clarifier are 1 642 000 $m^3.d^{-1}$ and 864 000 $m^3.d^{-1}$, respectively. Addition of iron salts is included in the model to account for the increase in total suspended solids concentration and sludge production. However, chemical precipitation is not modelled in detail. Dissolved oxygen controllers for the aerobic tanks are included with individual set points for each tank.

Results and discussion

Calibration of the influent generator model

The calibration of the influent generator model is performed systematically starting with generating the dry weather flow (domestic, industrial and infiltration to sewers) followed by wet weather flow (stormwater and infiltration to sewers). Additionally, the temperature of the influent wastewater is also calibrated.

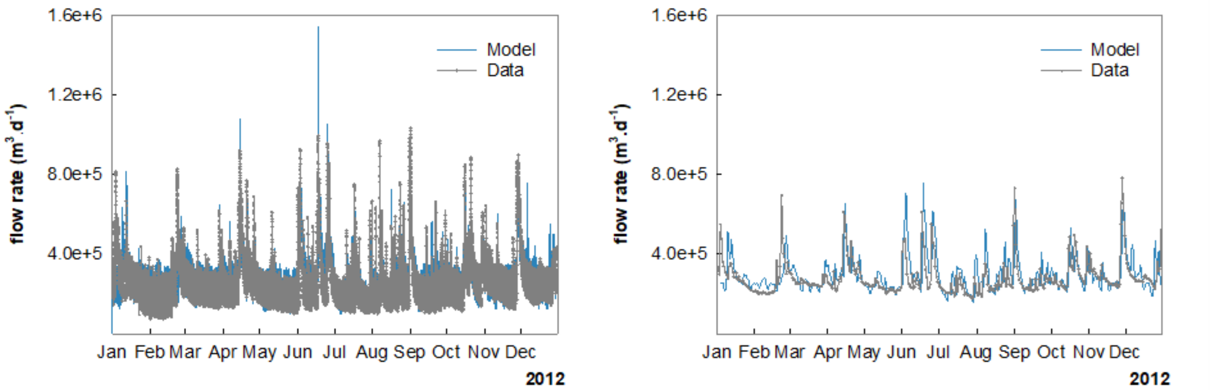

Figure 2. Calibration results for the influent flow rate at Henriksdal WWTP at 15 min intervals (left) and daily average values (right) for the year 2012.

Figure 2 demonstrates a good match between data and model results for influent flow rate at high frequency (15 min intervals) and daily average values. The model can successfully simulate the flow rate dynamics, which include: i. diurnal variation during dry weather; ii. peak flows during rain events; and iii. infiltration to sewers. Figure 3 presents the weekly composite concentrations for chemical oxygen demand (COD) and ammonia (NH_4-N) from data and model. The model can successfully reproduce the daily variation in pollutant concentration. Although more high frequency data for pollutant concentrations (daily, hourly, weekend effects) is usually not available, the model outputs can be used to fill this gap.

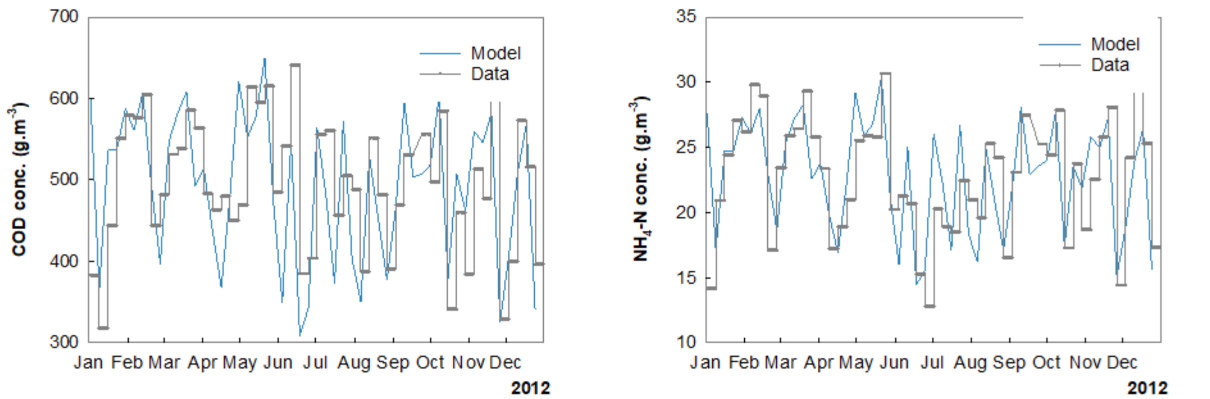

Figure 3. Weekly average influent COD (left) and NH_4-N (right) concentrations predicted by the model (blue) compared to the weekly composite measurements (grey) at Henriksdal WWTP for the year 2012.

WWTP performance for different scenarios

Influent scenarios: Two different influent flow rate scenarios (Figure 4) are generated with the calibrated influent generator model to illustrate the application of the toolbox to study various modifications in wastewater generation and their consequent impacts on WWTP performance. The example scenarios are: i. 5% decrease in impervious area (11% to 6%) (e.g. due to increased green areas) (S1); and ii. 5% decrease in percentage infiltration (44% to 39%) (e.g. due to rehabilitation of the existing sewer network) (S2). The influent profiles for both these scenarios are compared to the baseline situation with a 32% projected population increase for the year 2040. In addition, two scenarios (S1-no SWT, S2-no SWT), where stormwater treatment is removed for the bypass flow from the activated sludge tanks, are also evaluated to understand the significance of stormwater treatment in avoiding effluent limit violations during high flow conditions. The difference in flow rate between baseline and S1 is very limited due to the relatively low impervious area in the baseline case (11%). As the contribution of impervious area to influent flow rate is small in the baseline case, further reduction does not lead to major changes in influent flow rate dynamics. Scenario S2 leads to a lower influent flow rate to the WWTP both during dry weather and rain events compared to the baseline case. This is due to the higher contribution of infiltration to the influent flow rate. Average daily influent flow rate in the baseline case is 542 000 $m^3.d^{-1}$, which is reduced by 0.2% in S1 and 6.8% in S2 indicating that changes to infiltration flow rate will lead to stronger impacts on WWTP performance than a similar change in impervious area.

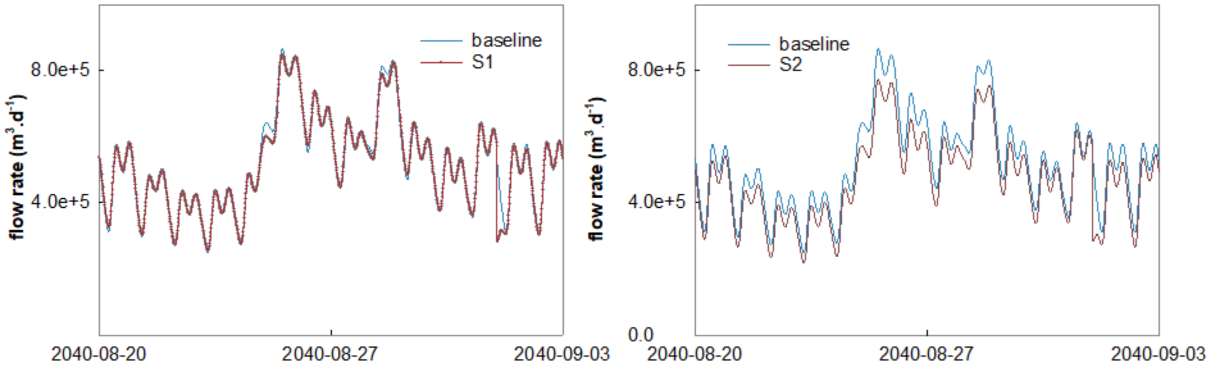

Figure 4. Variation in influent flow rate with reduction in impervious area (S1) (left) and decrease in percentage infiltration flow rate (S2) (right).

The performance of the WWTP in terms of effluent NH_4-N, TN, 7-day biochemical oxygen demand (BOD_7) and total suspended solids (TSS) concentrations is presented in Figure 5. Time in violation of the effluent limit for each of these pollutants is determined. The effluent limits for NH_4-N, TN, BOD_7 and TSS are assumed as 2 $g.m^{-3}$, 6 $g.m^{-3}$, 5 $g.m^{-3}$ and 30 $g.m^{-3}$, respectively. The major difference is the lower effluent TN violations in S1 and S2 compared to the baseline case. This is attributed to lower nitrous/nitric oxide (NO_x) as well as organic nitrogen in the effluent in both the scenarios. The total bypass volume is reduced by 15% in S1 compared to baseline. This reduction also decreases effluent TSS violations. However, S2 leads to better reduction in effluent violations due to 48% drop in total bypass volume. It can be surprising that effluent violations do not reduce significantly inspite of the major drop in bypass volume (15% - S1 and 48% - S2). This is due to the presence of sandfilters for the removal of phosphorus and organic material in the bypass flows. The significance of the sand filters is depicted in scenarios S1-no SWT and S2-no SWT. Higher BOD_7 and TSS violation times are noticed in these cases when compared to S1 and S2, respectively. There is no change in relation to NH_4-N and only marginal change in TN (due to change in organic nitrogen) when no stormwater treatment is available. This is due to the absence of NH_4-N removal in the sand filters.

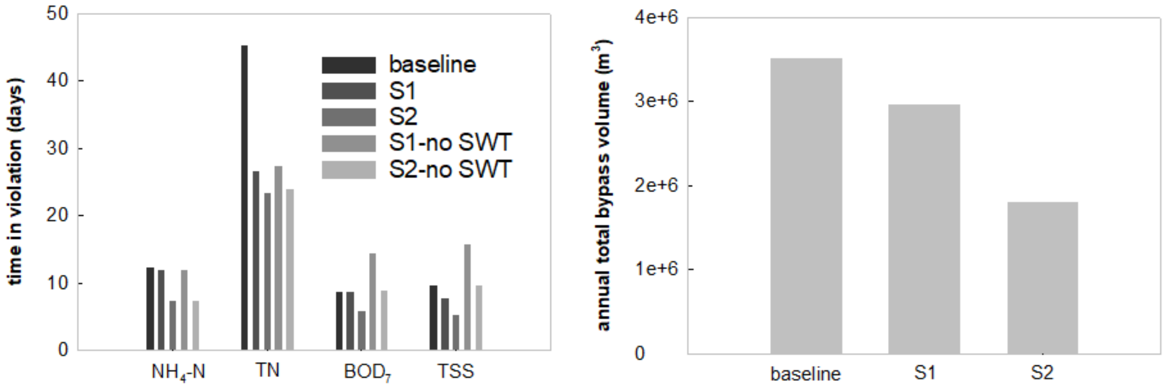

Figure 5. Time in violation for effluent NH_4-N, TN, BOD_7 and TSS (left) and total bypass volume (right) for the different scenarios.

Control strategies: With dynamic influent data available for the future WWTP, evaluation of the performance of various control strategies under dynamic influent loads becomes a possibility. Two different options for dissolved oxygen (DO) control are considered to demonstrate the evaluation of control strategies. The flex tanks 1 and 2, can either be used as aerobic or anoxic zones depending on the control strategy. i. aeration is turned ON in flex tank 2, when temperature is lower than 17 °C. Similarly, flex tank 1 is aerated when temperature is below 10 °C. The set point for DO is 2 $g.m^{-3}$ when aeration is ON (baseline); and ii. flex tank 2 is always aerated (DO set point - 2 $g.m^{-3}$) while aeration in flex tank 1 is turned ON only below 10 °C (S3). The baseline scenario allows flexibility in the operation of both tanks, whereas S3 allows flexible operation only in one of them. A comparison between the number of days in violation for TN, BOD_7 and TSS in the effluent together with operational costs, represented as an operational cost index (OCI) (see evaluation criteria in BSM2) is given in Figure 6. S3 leads to a major reduction in violations for effluent TN (18 days lower) while only marginally increasing the violations for BOD_7 and TSS (less than 2 day increase) for a 1-year evaluation period. According to this scenario evaluation, that significant improvement in performance can be achieved with only 4% increase in operational cost.

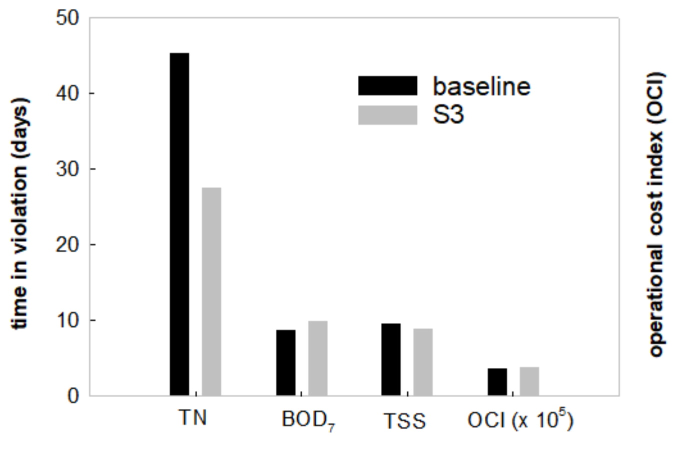

Figure 6. Comparison between baseline and S3 control strategies in terms of time in violation (for TN, BOD_7 and TSS) and operational cost index (OCI).

The above scenarios are small examples used to illustrate the potential of the modelling toolbox. Various other and more complex scenarios can be generated. These include changes to population equivalent projections, additional areas connected to the sewer network, new industrial connections, changes in rainfall patterns etc. In terms of evaluation of the results, the paper has focussed on effluent quality. The model allows for investigationg several other factors, e.g. chemical usage, energy consumption, biogas production.To conclude, it is also important to understand the limitations of the toolbox before attempting to interpret and use such a tool in decision-making. The influent generator model currently does not incorporate changes to influent temperature with changes in infiltration patterns. This may be important if the wastewater temperature and infiltration temperature are very different. Any further improvements in the WWTP especially during wet weather conditions should consider the interactions between the sewer system and WWTP. From a future research perspective, the influent generator model can be a good starting point to develop the model further and integrate sewer system and sewer overflow modelling together with the WWTP model in order to perform a system-wide assessment.

Conclusions

The paper presents a modelling toolbox for evaluating the future performance of a WWTP under varying influent flows, operational and control strategies. The toolbox is demonstrated for a case study at the Henriksdal WWTP in Stockholm, Sweden. Different influent scenarios are evaluated and the results show that decrease in infiltration leads to larger changes in effluent quality compared to reduced impervious area. Also, the model is used to evaluate two different DO control strategies. Time in violation for NH_4-N can be significantly reduced from 12.4 days to 3.75 days without causing any major increase in violation periods for BOD_7 and TSS and with only 4% higher operational costs.

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