Pol. J. Environ. Stud. Vol. 21, No. 4 (2012), 845-853

Original Research

Mathematical Modeling of an Oil Refinery WWTP

Ján Derco[1]*, Lenka Černochová[1], Jana Ivančinová[2], Antonio Lalai[3], Ivana Pijáková[1]

[1]Faculty of Chemical and Food Technology, Institute of Chemical and Environmental Engineering, Department of Environmental Engineering, Slovak University of Technology, Radlinského 9, 812 37 Bratislava, Slovakia
[2]Slovnaft, a. s., Vlčie hrdlo 1, 824 12 Bratislava, Slovakia
[3]Department of Chemical Engineering and Materials, Piazza d'Armi 19, 091 23 Cagliari, Italy

Received: 26 July 2011
Accepted: 17 January 2012

Abstract

Activated Sludge Model No. 1 (ASM1) was used in the modeling of an activated sludge system treating effluents from an oil refinery. The measurements of the diurnal variation in wastewater flow and composition at the wastewater treatment plant inlet and outlet were carried out.

The calibrated model predicting the influence of changes in the wastewater composition and the operational parameters on the effluent wastewater quality and the related operational costs is available. A calibration technique based on the heuristic method was applied.

The model was used to analyze the influence of changes in wastewater flow and composition as well as different aeration systems on the process performance. The results of dynamic simulations indicate that the verified dynamic mathematical model is a useful supporting tool for optimizing operational parameter values and operational costs.

The presented results are for illustration purposes only and are not intended as instructions for the operation of a wastewater treatment plant.

Keywords: ASM1, dynamic simulation, industrial WWTP, operational costs, oxygen supply

Introduction

The dynamic simulation of activated sludge plants has grown in popularity during recent decades. Activated Sludge Model No. 1 (ASM1) [2] is generally accepted as state-of-the-art. Since this model was introduced in the late 1980s, the dynamic simulation of activated sludge plants has become more widespread. ASM1 was primarily developed for municipal activated sludge wastewater treatment plants, describing the removal of organic carbon substances and nitrogen with simultaneous consumption of oxygen and nitrate as electron acceptors. The model provides a good description of sludge production [3].

*e-mail: jan.derco@stuba.sk

The ASM1 was used for modeling industrial wastewater treatment processes in this study. A database of experimental values for the influent and effluent wastewater flow and variation in the composition of wastewater with time for the WWTP was created. A simplified calibration of the model was applied due to very similar biodegradability, i.e. BOD_5/COD ratio of the studied industrial wastewater in comparison with municipal wastewater. The ASM1 calibration technique was based on the trial and error method.

The goals of this study were:

(i) to evaluate the capability of ASM1 to describe the processes carried out at an industrial WWTP
(ii) to investigate different scenarios for influent concentration values of chemical oxygen demand (COD) and ammonium nitrogen ($N\text{-}NH_4^+$)

Table 1. Sampling schedule.

	Q	COD_{tot}	COD_{filt}	N_{tot}	N_{org}	$N\text{-}NH_4^+$	$N\text{-}NO_3^-$	T	DO	pH
Influent	*	2	4	2	12	2	2			
Reactors								*	*	*
Effluent		2		12	12	2	2			

*on-line measurements, 2-, 4- or 12-hour intervals of sampling

(iii) to investigate different wastewater flow scenarios
(iv) to study different aeration modes and options for operational cost savings
(v) to compare the effect of different aeration systems on operational costs

Case Study

An industrial WWTP treating effluents from an oil refinery process is the object of our study. The first stage of the WWTP represents mechanical treatment. This is followed by chemical and biological treatment stages. Afterward, the biologicaly treated wastewater enters the lagoon, where the properties of treated water are modified naturally.

Fig. 1 gives a schematic overview of the biological unit layout. The biological stage consists of an activated sludge system designed for the biological removal of organic matter and nitrogen from the wastewater. The aeration tank, with total volume of 12,636 m^3, is divided into 6 sections in series (each of volume 2,106 m^3). The depth of the tanks is 6.5 m. six blowers, each with 5,000 $m^3 \cdot h^{-1}$ capacity, are available. Five blowers are regularly operated. The circular secondary settling tank has a total volume of 8,000 m^3 and a diameter of 60 m.

Materials and Methods

Sampling

The two-day sampling period took place from 6 to 8 August 2008. Samples for diurnal variation were taken at the influent and effluent of biological stage of the WWTP. The schedule of sampling is provided in Table 1. Samples for analyses of sludge characteristics were taken every 4 hours.

Analytical measurements of the influent and effluent wastewater and monitoring of the treatment procedures included biochemical oxygen demand (BOD), chemical oxygen demand (COD_{tot} and COD_{sol}), $N\text{-}NH_4^+$, $N\text{-}NO_3^-$, N_{org}, suspended solids (SS), and mixed liquor suspended solids (MLSS) [1].

Modeling Tools

ASM1 [2] was used for the mathematical modelling of the WWTP. This model is mostly used for the modeling of municipal wastewater treatment plants, but some papers [3-7] show that it can be used for industrial WWTP. Simulations of the WWTP were carried out using ASIM 4.0 software (activated sludge simulation program) [8].

ASM1 [2] allows for simulation of the removal of carbonaceous pollutants, nitrification, and denitrification processes. Thirteen process components, eight biochemical processes and 19 kinetic parameters are included in the model. Basically, Monod type reaction kinetics is applied to describe the transformation of process components by the biochemical processes included in this concept.

An example of the Monod type reaction rate for oxygen consumption by aerobic growth of heterotrophs and autotrophs can be expressed as follows:

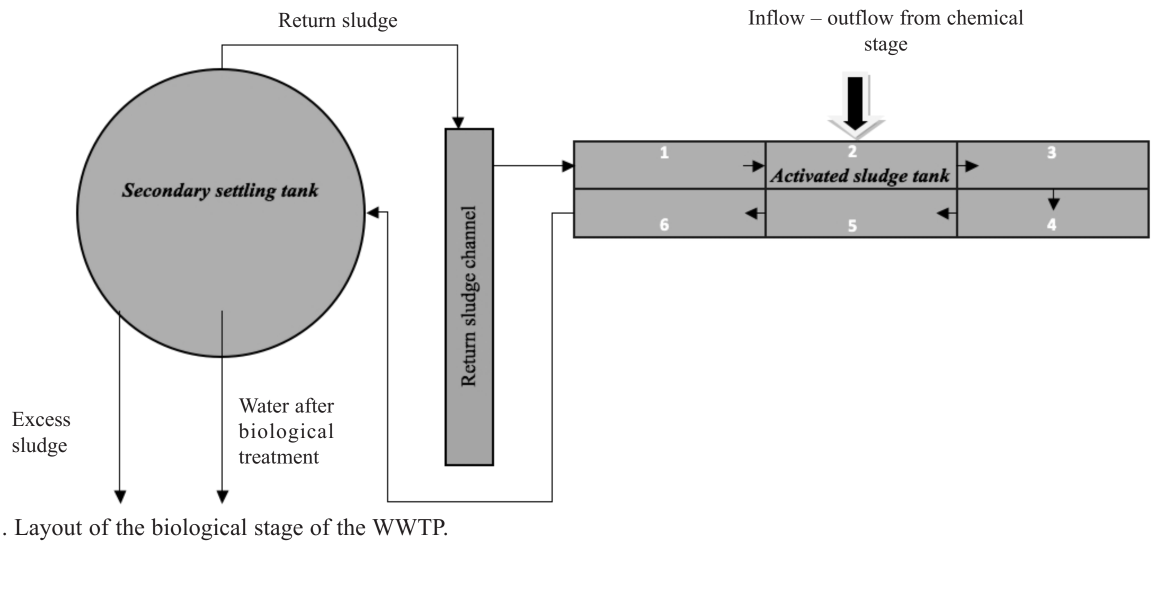


Fig. 1. Layout of the biological stage of the WWTP.

$$r_O = -\frac{1-Y_H}{Y_H}\mu_H\left(\frac{\rho(S_S)}{K_S+\rho(S_S)}\right)\left(\frac{\rho(S_O)}{K_{O,H}+\rho(S_O)}\right)\rho(X_{B,H}) - \quad \text{heterotrophs} \tag{1}$$

$$-\frac{4.57-Y_A}{Y_A}\mu_A\left(\frac{\rho(S_{NH})}{K_{NH}+\rho(S_{NH})}\right)\left(\frac{\rho(S_O)}{K_{O,A}+\rho(S_O)}\right)\rho(X_{B,A}) \quad \text{autotrophs}$$

...where K_{NH} is ammonium saturation constant (mg·l^{-1}), $K_{O,A}$ is oxygen saturation constant for autotrophs (mg·l^{-1}), $K_{O,H}$ is oxygen saturation constant for heterotrophs (mg·l^{-1}), r_O is oxygen consumption rate (mg·l^{-1}·d^{-1}), K_S is readily biodegradable organics saturation constant (mg·l^{-1}), S_{NH} is ammonium nitrogen concentration (mg·l^{-1}), S_O is dissolved oxygen concentration (mg·l^{-1}), S_S is readily biodegradable substrate (mg·l^{-1}), $X_{B,A}$ is concentration of autotrophic biomass (mg·l^{-1}), $X_{B,H}$ is concentration of heterotrophic biomass (mg·l^{-1}), Y_A is yield coefficient of autotrophic biomass (-), Y_H is yield coefficient of heterotrophic biomass (-), μ_A is maximum growth rate of autotrophic biomass (d^{-1}), and μ_H is maximum growth rate of heterotrophic biomass (d^{-1}).

The kinetics and stoichiometry of the model is typically presented in matrix form, e.g. by Henze et al. [2].

The ASIM 4.0 system is an interactive, user-friendly program for simulating biological transformations proceeding in the activated sludge in a WWTP that performs simultaneous carbon oxidation, nitrification, and denitrification. The process rate variables were incorporated into 11 mass balances for heterotrophic and autotrophic biomass, soluble substrate, nitrate nitrogen, ammonia nitrogen, and other constituents significant in the process analysis of single-sludge waste water treatment plants. Applying numerical techniques, this program determines the solution to these material balances for both constant and time-dependent inputs [8]. The activated sludge system was modelled with a 6-tank-in-serries mode. The volume of the return sludge channel was considered part of the volume of the regeneration tank. The secondary settling tank represents one reactor and is ideally separating solids from the liquid with the solid retention time equal to zero. The amount of excess sludge was determined based on SRT value maintained in the activated sludge system.

Results and Discussion

Influent Characterization

Influent wastewater flow and composition depend on the production of the oil refinery. There is variability in both wastewater flow and composition. Diurnal loads do not follow a regular pattern due to production changes. The average value of the wastewater flow was 27,709 m^3·d^{-1}, flow rate fluctuations were approximately 10% during the two-day measurement period (Fig. 2). The average return sludge flow was approximately 23,235 m^3·d^{-1}, the amount of excess sludge was approximately 448 m^3·d^{-1}. The solid retention time was 12.5 days. The average temperature during the measurements was 29.6ºC, with a minimum 25.5ºC and a maximum 30.1ºC. The hydraulic retention time in the aeration tank was 10.9 hours.

Table 2. Average concentration [mg·l^{-1}] in the influent and effluent of the biological stage during the sampling period.

Variable	Influent	Effluent
BOD_5	338.30	7.20
COD	685.00	29.4
$N\text{-}NH_4^+$	19.90	2.23
$N\text{-}NO_3^-$	0.50	0.05
N_{org}	3.20	0.20

Table 3. Oxygen concentration [mg·l^{-1}] in the aeration tank sections during the sampling period.

Aeration tank section	Average	Minimum	Maximum
1	0.7	0.2	1.4
2	0.9	0.4	2.4
3	1.4	0.5	3.6
4	3.3	2.3	4.8
5	5.0	3.9	6.0
6	3.1	1.8	4.7

Table 2 summarizes the average composition of the wastewater in the influent and effluent during the sampling period. The oxygen profiles in aeration tank-in-series during sampling are shown in Table 3 and in Fig. 3. High variations of oxygen concentration in the sections of aeration tank during measurement periods are obvious.

Table 4 summarizes the influent wastewater characteristics required as input for the ASM1 [2]. The fractionation

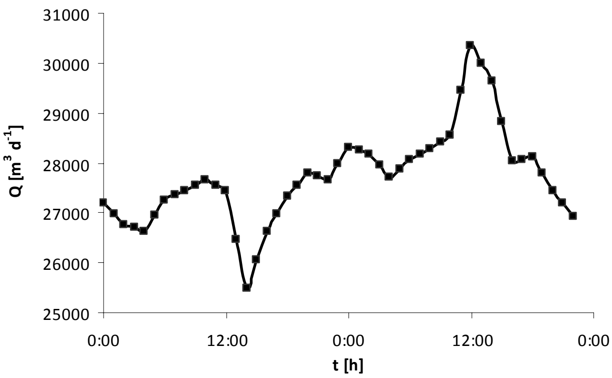


Fig. 2. Variation of the influent wastewater flow rate at the WWTP.

Table 4. Influent characteristics for ASM1.

Variable	Unit	Influent
Dissolved species		
S_O	g COD·m^{-3}	2.0
S_I	g COD·m^{-3}	13.7
S_S	g COD·m^{-3}	548.0
S_{NH}	g N·m^{-3}	19.9
S_{NO}	g N·m^{-3}	0.5
S_{ND}	g N·m^{-3}	1.9
S_{ALK}	mmol·l^{-1}	5.0
Particulate species		
X_I	g COD·m^{-3}	54.8
X_S	g COD·m^{-3}	68.5
X_{BH}	g COD·m^{-3}	0.0
X_{BA}	g COD·m^{-3}	0.0
X_P	g COD·m^{-3}	0.0
X_{ND}	g N·m^{-3}	1.3

COD was performed using recommendations from the literature [6, 7, 9].

Model Calibration

A precondition for the succesful simulation and optimisation of the wastewater treatment process is a calibration of the model in order to achieve a reasonable agreement between the measured data and simulation results. The calibrated model can be used to predict the influence of changes in the wastewater composition and operational parameter values on the quality of the effluent wastewater quality and on operational costs.

A simplified calibration of the model was applied due to a very similar biodegradability (BOD_5/COD=0.49) of the studied industrial wastewater in comparison with typical municipal wastewater. Calibration of the applied ASM1 model was carried out, employing diurnal measurements of wastewater flow and composition at the input and output of the biological stage of the WWTP (Fig. 1). No specific experiments were carried out for the determination of kinetic and stoichiometric coefficients.

The values of technological and operational parameters (solid retention time, recirculation flows, concentration of dissolved oxygen, temperature) valid during the diurnal measurements were also applied in the model calibration.

The model calibration technique was based on a heuristic trial and error method and the results were evaluated statistically. The effluent concentration values for individual pollutants over a 36-hours period were calculated as a flow-weighted average. For example, the COD value in the effluent composite sample was calculated as follows [10]:

$$\rho(COD)_{36\text{-h}} = \frac{\int_{t=1}^{37} Q(t)\,.\,\rho(COD,t)\;\mathrm{d}t}{\int_{t=1}^{37} Q(t)\;\mathrm{d}t} \qquad (2)$$

These values were minimized with regard to the concentration values measured in the composite sample of the effluent wastewater. The collection of individual samples was obtained at regular two-hour intervals during a sampling time span. The resulting mixture (composite sample) forms a representative sample and was analyzed to determine the average conditions during the sampling period. The differences between the experimental and calculated values of the investigated pollutants (ammonium and nitrate nitrogen, COD, MLSS) were minimized by variation of the kinetic and stochiometric parameter values (Table 5).

Experimental values from a 48-hour sampling period were used for model calibration.

The simulation program has immediate response in the effluent concentration to changes in the influent concentration as a consequence of the implementation of the ASM1 in this system, i.e. a compartmental model. This means that the effluent calculated concentration values correspond to the influent concentration values at the same time. On the other hand, there is a shift between influent and effluent experimental concentration values in the real activated sludge system. We compared the experimental and calculated data without shift and with shorter and longer shifts, but the best agreement was obtained with the shift equal to the mean hydraulic residence time from the results of statistical evaluation. Thus, the shift between influent and effluent experimental concentration values equal to average hydraulic retention time in aeration tank (10.9 hours) was considered (Fig. 4), when minimizing the differences between the experimental and calculated effluent concentration values. In other words, it was assumed that the response of the real activated sludge system to the experimental input values is measured at the reactor output after 10.9 hours (hydraulic retention time). Consequently, it was possible to use 36-hours of experimental values from the 48-hours of sampling.

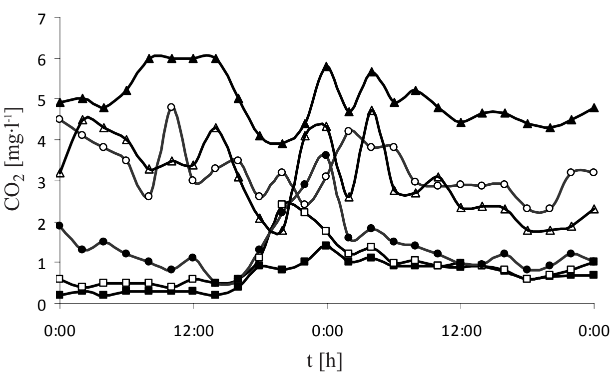
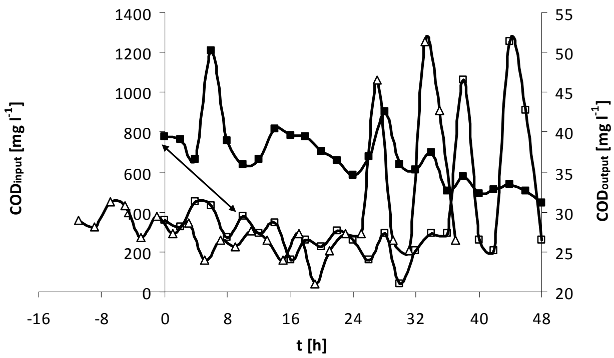


Fig. 3. Variations of oxygen profiles in the sections of the aeration tank during the sampling period.



■ – COD influent, □ – COD effluent, Δ – COD effluent, time-shift

Fig. 4. Visualization of time-shift between the WWTP input and output.

Kinetic parameter values before and after the calibration are provided in Table 5. Values of other kinetic parameters were the recommended values for municipal wastewater [2] and they are also given in Table 5.

There is very good agreement between the experimental and calculated effluent concentrations as shown in Table 6. Considering the quality of the fit of experimental data, the ASM1 model can be considered as suitable for simulations of the wastewater treatment plant performance.

Dynamic Simulations

Dynamic simulations with the calibrated ASM1 were focused on investigations of the influence of the wastewater characteristics and process variables on process efficiency. The influence of wastewater flow and composition and oxygen profiles in the aeration tank sections on effluent concentration (indirectly also on operational costs) and on oxygen supply requirements as the major constituent of operational costs were studied.

Intensity of aeration is dependent on oxygen absorption efficiency. These values were calculated as the multiple of utilization factor, the water depth in activated sludge system and mixed liquor volumetric mass transfer coefficient. Utilization factor is characteristic for each type of aeration: coarse-bubble aeration – 2% per meter of water column, and fine-bubble aeration 5.5% [11, 12]. The water depth was 6.5 m. The mixed liquor volumetric mass transfer coefficient for wastewater is in range of 0.6 and 0.7. The value of 0.65 was used for calculations. Thus, the efficiencies of oxygen absorption from air into mixed liquor are 8.5 % for coarse-bubble aeration and 23.2 % for fine-bubble aeration. These values were used to convert oxygen consumption rate into intensity of aeration, and consequently also for calculation of aeration costs.

The composition of the influent industrial wastewater significantly fluctuates because of the variations in schedules of the production facilities. The next dynamic simulations were focused on assessment of maximal influent organic (COD) and ammonium nitrogen pollution of the WWTP.

The influence of organic and ammonium loads on oxygen requirement and related required intensity of aerations

Table 5. Parameter values of ASM1 at 20ºC.

Parameter	Unit	ASM1 default value	Results from calibration
f_P	-	0.08	0.29
$K_{O,H}$	$gO_2 \cdot m^{-3}$	0.20	1.90
K_{NO}	$gNO_3\text{-}N \cdot m^{-3}$	0.50	0.12
b_H	d^{-1}	0.62	0.39
k_H	$gCOD_{XS} \cdot (gCOD_{XB} \cdot d)^{-1}$	3.00	3.20
K_x	$gCOD_{XS} \cdot (gCOD_{XB})^{-1}$	0.03	-
μ_A	d^{-1}	0.80	1.15
K_{NH}	$gNH_4\text{-}N \cdot m^{-3}$	1.00	0.80
$K_{O,A}$	$gO_2 \cdot m^{-3}$	0.40	0.60
b_A	d^{-1}	0.72	0.66
Y_A	$gCOD_{XB}$ $(gNox)^{-1}$	0.240	-
Y_H	$gCOD_{XB}$ $(gCODox)^{-1}$	0.670	-
i_{XB}	gN $(gCOD_{XB})^{-1}$	0.086	-
i_{XP}	gN $(gCOD_{XP})^{-1}$	0.060	-
μ_H	d^{-1}	6.000	-
K_S	$gCOD \cdot m^{-3}$	20.000	-
η_g	-	0.800	-
η_h	-	0.400	-

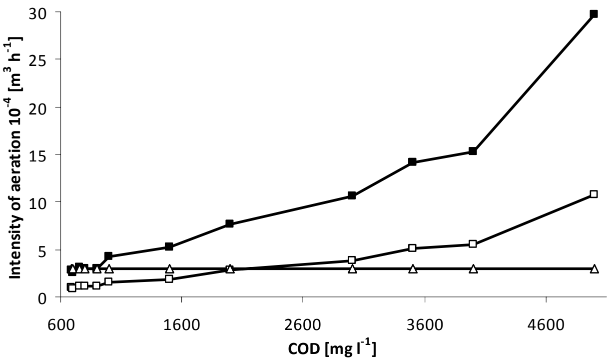
Table 6. Experimental and calculated concentration values [$mg \cdot l^{-1}$] in the composite effluent sample of the plant (1 to 37 hours) and MLSS [$g \cdot l^{-1}$].

	Measurement	Simulation	WWTP effluent limits
S_{NH}	2.00	2.00	20.00
S_{NO}	0.05	0.04	-
COD	30.00	30.00	150.00
MLSS	7.1	7.1	-

for both fine-bubble and coarse bubble aeration systems were studied using dynamic simulations. The aim was to find out the maximal feasible influent COD values to maintain the performance of the reactor for both fine-bubble and coarse-bubble aeration systems with available aeration capacity of existing blowers. Two sets of dynamic simulations were performed to study the influence of influent wastewater composition on process efficiency. The average COD value at the influent was varying while the average concentration of ammonium nitrogen was maintained constant and equal to the average experimental value (19.9 $mg \cdot l^{-1}$, Table 4) for the first set of dynamic simulations. On the other hand, the influent average concentration of ammonium nitrogen varied and the average COD value was maintained constant and equal to the average measured value (685 mg·l-1, Table 4) for the second set of dynamic simulations. An idea was to investigate behavior of the plant by different loads of organic pollution and on the other hand of different loads of nitrogen pollution. All calculations within these two sets were performed with the experimental time varying values of wastewater flow rate that correspond to the average value of 27,709 m3·d-1.

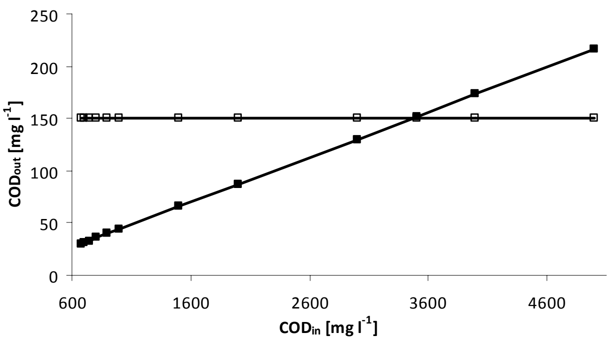
The results for assessment of maximal influent organic pollution (COD) are shown in Figs. 5 and 6 and for ammonium pollution in Figs. 7 and 8. It is obvious that the maximal pollution at the influent is limited by the capacity of the aeration system. As can be seen from Fig. 5, the coarse bubble-aeration system can treat a lower organic load than the fine-bubble aeration system when considering the air blower's maximal aeration capacity of 30,000 m3·h-1. The maximum COD in the influent wastewater is about 750 mg·l-1 for coarse-bubble aeration and 2,400 mg·l-1 for fine-bubble aeration while complying with the WWTP effluent standards. Discharges from the wastewater treatment plant are controlled and monitored according to the Regulation of the Government of the Slovak Republic No. 296/2005 [13], which harmonizes with EU legislation, particularly with Directive 91/271/EEC [14].

As shown in Fig. 6, the COD value in the effluent is approximately linear in the influent pollution, independent



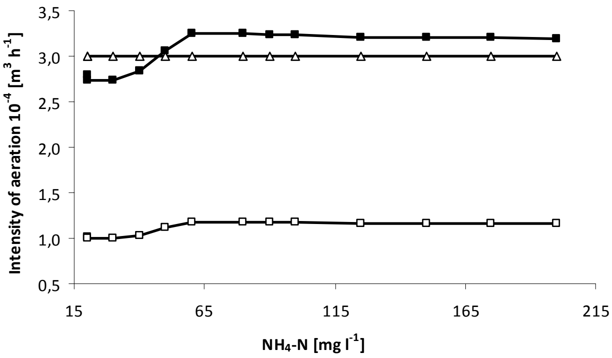
Δ – capacity of the aeration system □ – fine-bubble aeration, ■ – coarse-bubble aeration

Fig. 5. Influence of influent COD value on the intensity of aeration for coarse-bubble aeration and fine-bubble aeration systems.



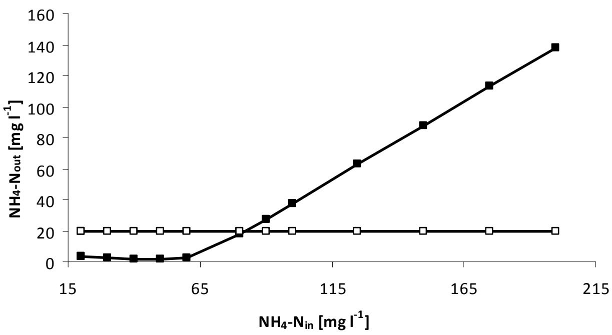
□ – WWTP effluent limit, ■ – influence of values

Fig. 6. Influence of influent COD value on effluent COD value.



Δ – capacity of the aeration system □ – fine-bubble aeration, ■ – coarse-bubble aeration

Fig. 7. Influence of influent ammonium nitrogen content on the intensity of aeration for coarse-bubble aeration and fine-bubble aeration systems.



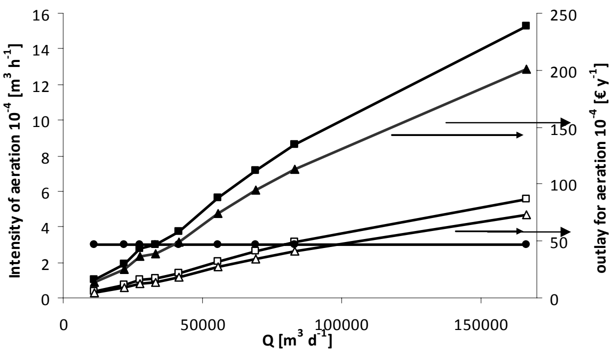
□ – WWTP effluent limit, ■ – influence of values

Fig. 8. Influence of infuent ammonium nitrogen concentration on effluent ammonium nitrogen concentration.

of the aeration system. With an increase of the COD values in the influent, the required aeration intensity (Fig. 5) and, consequently, aeration costs also rise.

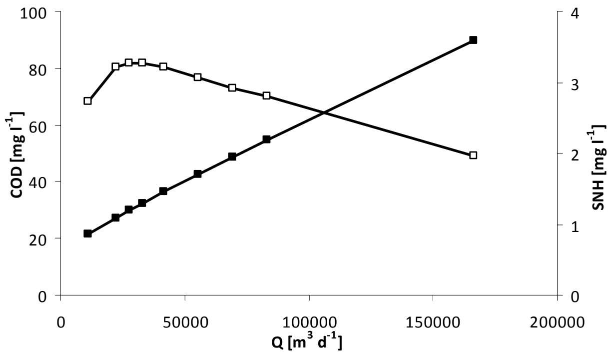
The changes in input ammonium nitrogen concentration do not significantly influence the required performance of the aeration system. On the other hand, the concentration of ammonium nitrogen in the effluent significantly increases with higher ammonium concentrations in the influent (Fig. 8) and exceeds the effluent standard for ammonium nitrogen at influent ammonium concentrations over 80 mg·l-1. The maximum input concentration of ammonium nitrogen is about 47 mg·l-1 for coarse-bubble aeration and 82 mg·l-1 for fine-bubble aeration while complying with the WWTP effluent standards.

The pollution load of a WWTP depends on both wastewater pollution and flow rate. Thus, different scenarios of wastewater flow were investigated. In Fig. 9, the influence of wastewater flow rate on the required aeration intensity and on the oxygen supply costs are presented. The effluent COD and the ammonium nitrogen concentrations from dynamic simulations carried out at various wastewater flows are plotted in Fig. 10 (influent COD and NH_4-N concentration values were equal to experimental ones – Table 4). As mentioned earlier, the principal limitation factor of the WWTP load is the capacity of air blowers. The maximum applicable wastewater flow rate is equal to the aver-



● – capacity of the aeration system, ■ – coarse-bubble aeration,
□ – fine-bubble aeration, ▲ – outlay coarse-bubble aeration,
Δ – outlay fine-bubble aeration

Fig. 9. The intensity of aeration and costs for oxygen supply for coarse- and fine-bubble aeration on different wastewater flow.



□ – S_{NH}, ■ – COD

Fig. 10. Influence of wastewater flow rate on effluent COD and S_{NH} concentrations.

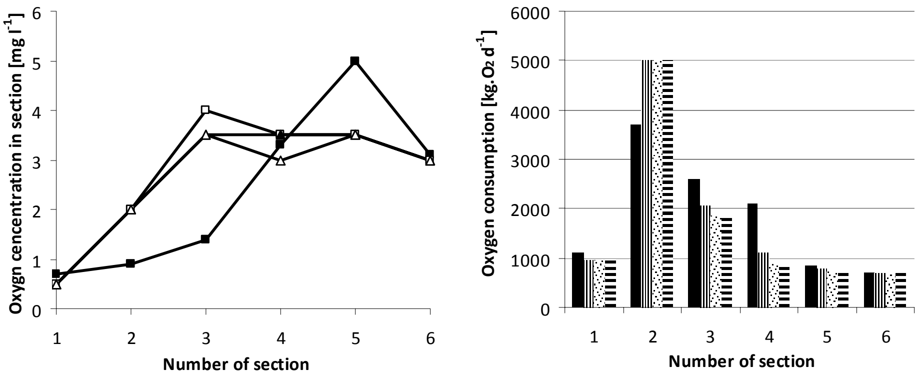
Table 7. Effluent concentration values [mg·l^{-1}] for different aeration modes.

	COD	S_{NH}
Operational modes	29.8	3.3
Simulation A	29.5	3.7
Simulation B	29.6	3.8
Simulation C	29.6	3.8

age experimental wastewater flow (27,709 m^3·d^{-1}) at the air blower's capacity of 30,000 m^3·h^{-1} and coarse-bubble aeration in operation (Fig. 9).

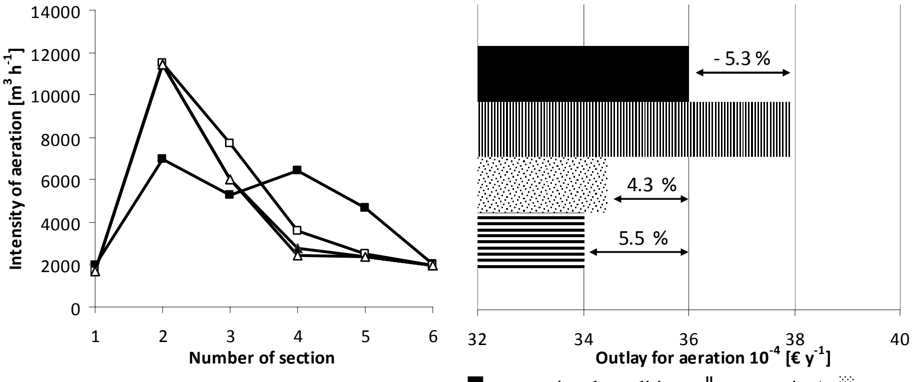
If fine-bubble aeration is used, then approximately two-times higher wastewater flow rate can be treated when complying with the effluent standards (Table 6). It is obvious from Fig. 10 that significantly higher wastewater flows can be treated with regard to the effluent limits. It can be concluded from the results that in the period of measurements the influent wastewater flow rate was significantly lower than the maximal capacity of the WWTP with fine bubble aeration.

The oxygen profiles (Fig. 2) in individual sections of the aeration tank showed high variability during sampling. The following dynamic simulations were focused on an investigation of different aeration modes. The average oxygen profiles (left) and corresponding oxygen consumption rates (right) at operational conditions during sampling, as



■ – operational conditions, □ – scenario A, ▲ – scenario B, Δ – scenario C ▌– operational conditions, ║ – scenario A, ░ – scenario B, ═ – scenario C

Fig. 11. Oxygen profiles and corresponding oxygen consumption for three scenarios.



■ – operational conditions, □ – scenario A, ▲ – scenario B, Δ – scenario C ▌– operational conditions, ║ – scenario A, ░ – scenario B, ═ – scenario C

Fig. 12. Intensity of aeration and oxygen supply costs for aeration for three scenarios.

Table 8. Effluent concentration values [mg·l^{-1}] for different aeration modes.

	COD	SNH
Operational conditions	29.8	3.3
Simulation D	51.2	4.7
Simulation E	149.0	6.1
Simulation F	104.2	6.4
WWTP effluent limits	*150.0*	*20.0*

well as the results corresponded to three chosen scenarios (A, B, C) with different oxygen profiles, are presented in Fig. 11. The intensity of aeration and the aeration costs for different aeration scenarios are shown in Fig. 12. Aeration costs were assumed to be 0.0015 € per m^3 of supplied air.

As can be seen from Fig. 12, some options to save operational costs (5.5%, i.e. approximately 20,000 EUR per year) are to change the aeration mode from coarse- to fine-bubble. From these results (Fig. 12 – right) it follows that option A is less convenient than aeration maintained at operational conditions. The effluent concentration values for each scenario are summarized in Table 7. In each case the effluent concentrations are much lower than the effluent standards (Table 6).

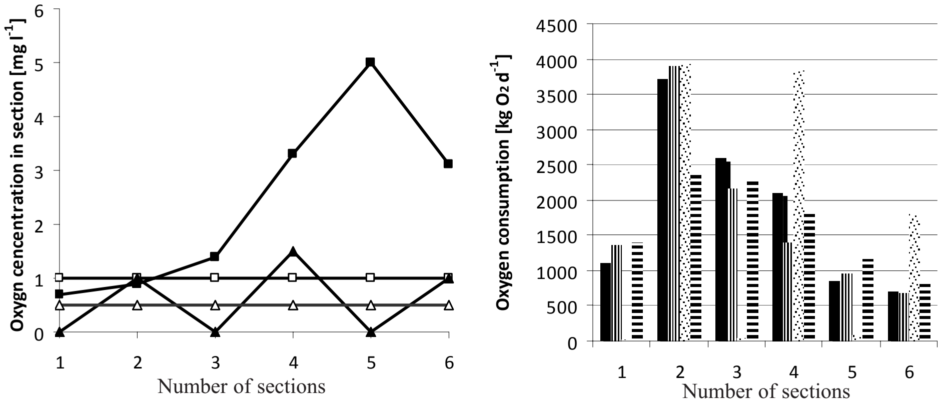
The next simulations were aimed at evaluating operational costs for the effluent concentration values between the experimental values and effluent standards. Three scenarios, i.e. D, E, and F, which correspond to different aeration modes, are presented (Fig. 13). In each case the effluent COD and ammonium nitrogen values were lower than the effluent standards (Table 8) for the biological stage. From the results of dynamic simulations (Fig. 14) it can be concluded that oxygen supply costs could be reduced by 36% (approximately 130,000 EUR per year).

Conclusions

Our paper illustrates some possibilities for the utilization of simulation programs to the operation of the oil refinery WWTP.

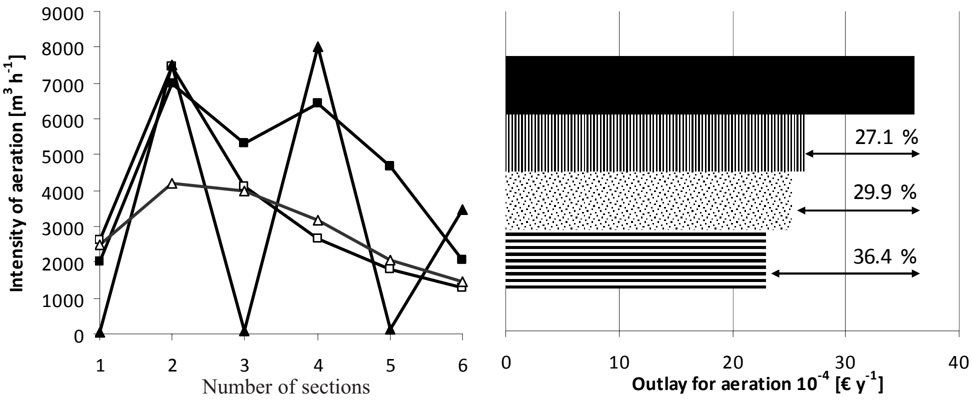
Simplified calibration of the applied ASM1 model was carried out employing two-day measurements of wastewater flow and composition variability at the input and output of the biological stage and corresponding technological and operational parameter values. It can be concluded that ASM1 proved to be capable of describing the processes performed at the oil refinery WWTP.

Influent wastewater flow rate and composition are limited by the capacities of the WWTP and air blowers. From the results of dynamic simulations it follows that influent



■ – operational conditions, □ – scenario D, ▲- scenario E, Δ – scenario F ▌- operational conditions, ‖ - scenario D, ░ - scenario E, ═ - scenario F

Fig. 13. Oxygen profiles and corresponding oxygen consumption for three aeration scenarios.



■ – operational conditions, □ – scenario D, ▲- scenario E, Δ – scenario F ▌- operational conditions, ‖ - scenario D, ░ - scenario E, ═ - scenario F

Fig. 14. Intensity of aeration and outlay for aeration for three scenarios.

wastewater flow was significantly lower than the maximal treatment capacity during the measurement period.

The maximum wastewater flow rate is more or less equal to average operational value when coarse-bubble aeration is used. Approximately two-times higher wastewater flow can be treated at the WWTP when fine-bubble aeration is applied. Maximal influent COD and ammonium concentrations while complying with effluent standards were also obtained by dynamic calculations.

From results of dynamic simulations follow some options to save aeration costs by approximately 5.5% when changing aeration mode from coarse- to fine-bubble, while maintaining actual effluent concentrations/operational conditions. On the other hand, reducing oxygen supply costs by approximately 36% can be expected if the effluent concentrations are close to effluent standard values.

The presented results are illustrative and they are not intended to be instructions for the wastewater treatment plant operation. A more accurate description of the actual processes under study and more detailed calibration should be performed for the process performance evaluation and optimization.

Acknowledgements

The authors wish to thank the Slovak Research and Development Agency for the financial support of the bilateral Slovak-Slovene project SK-SI-0019-10.

References

1. GREENBERG A.E., CLESCERI L. S., EATON A. (Eds.) Standard Methods for the examination of water and wastewater. 22nd Edition. John Wiley & Sons: Washington DC, **2005**.
2. HENZE M., GRADY Jr. C.P.L., GUJER W., MARIS G.R., MATSUO T. Activated sludge model No. 1. Scientific and technical report No. 1. IAWPRC: London, **1987**.
3. PETERSEN B., GERNAEY K., HENZE M., VANROLLEGHEM P.A. Evaluation of an ASM1 model calibration procedure on a municipal-industrial wastewater treatment plant. Journal of Hydroinformatics. **04**, 1, **2002**.
4. BURY S.J., GROOT C.K., HURTH C., HARDT N. Dynamic simulation of chemical industry wastewater treatment plants, Wat. Sci. Tech. **45**, (4-5), 355, **2002**.
5. COEN F., VANDERHAEGEN B., BOONEN, I., VANROLLEGHEM P.A., VAN MEENEN P. Improved design and control of industrial and municipal nutrient removal plants using dynamic models. Wat. Sci. Tech **35**, (10), 53, **1997**.
6. MELCER H., DOLD P.L., JONES R.M., BYE Ch.M., STENSEL H.D., WILSON A.W., SUN P., BURY S. Methods for wastewater characterization in activated sludge modelling. IWA Publishing: London, **2003**.
7. PINZÓN PARDO A.L., BRDJANOVIC D., MOUSSA M.S., LÓPEZ-VÁZQUEZ C.M., MEIJER S.C.F., VAN STRATEN H.H.A., JANSEN A.J.H., AMY G., VAN LOOSDRECHT M.C.M. Modelling of an oil refinery wastewater treatment plant. Environ. Technol. **28**, 1273, **2007**.
8. EAWAG. Dynamic simulation software for biological wastewater treatment modelling. ASIM 4.0, Holinger: Berne, **2006**.
9. HENZE M., HARREMOËS P., JANSEN J. C., ARVIN E. Wastewater treatment. Biological and chemical processes. Springer-Verlag: Berlin, **1995**.
10. DERCO J., KOVÁCS A. Utilization of simulation programs for design of biological units for nitrogen removal. Chem. Pap. **56**, (2), 117, **2002**.
11. HLAVÍNEK P., MIČÍN J, PRAX P. Handbook of wastewater collection and treatment, Vyd. NOEL: Brno, **2001** [In Czech].
12. DRTIL M., HUTŇAN M. Technological project. Wastewater treatment processes and technologies. Vyd. NOI: Bratislava, **2007** [In Slovak].
13. Regulation of the Government of the Slovak Republic N° 296/2005: Quality objectives of surface waters and on limit values for pollution indicators of waste waters and special waters. **2005** [In Slovak].
14. Council Directive 91/271/EEC on Urban Waste Water Treatment. **1991**.