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Wastewater Treatment Processing Simulation Technology Using "Activated Sludge Model"†

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Abstract:

In developing design support software and operation support software for advanced wastewater treatment plants, JFE Engineering uses the "Activated Sludge Model" advocated by International Water Association (IWA). As examples of development, this report describes model construction and verification for design support of oxidation ditch (OD) facilities and model construction for operation support of an advanced treatment process with a microbiol carrier, together with an example of practical application.

1. Introduction

Sewage/wastewater treatment plants generally perform biological treatment using microorganisms called activated sludge. Activated sludge contains diverse microorganisms which form a complex ecological system involving reproduction, death, and predation. Consequently, microorganism treatment performance naturally changes with the change in the operating condition of the plant, but also with one in influent quality. At sites where plant design and operation control are performed, it had been considered difficult to adapt simulation technologies to sewage/wastewater treatment plants, and until now, simulation technologies have not reached practical application.

In 1986, International Water Association (IWA) proposed an Activated Sludge Model with the aim of creating a world standard for numerical models of activated sludge systems, and in 1995, the IWA announced Activated Sludge Model No. 2, which is a model for advanced wastewater treatment. Various innovations were adopted in the new model to facilitate practical application, including (1) limitation of the number of types of microorganism to the minimum necessary for predicting treatment performance, (2) definition of influent fractionation suited to the model, and (3) introduction of the concept of "calibration," which allows control of the range of parameters to be fixed as a means of adjusting for external factors. In Japan, with the trend toward advanced wastewater treatment processes (treatment for removal of N and P as nutrient), it had become necessary to establish a rational method to cope with the increasing complexity of processes and increased number of operational control factors accompanying the adoption of advanced treatment technologies. Large expectations were placed on the Activated Sludge Model as a technology which responds to these needs.

JFE Engineering began research on practical application of the Activated Sludge Model in 1998 and has developed design support software and operation support software which use the Activated Sludge Model. At present, the company is conducting research and development on the oxidation ditch (OD; for small-scale treatment processes) as part of joint development with Japan Sewage Works Agency (since 2001), and is also involved in joint development of operation support software for microbiol carrier-type advanced wastewater treatment plants with Kawasaki City.

As wastewater treatment simulation technologies using the Activated Sludge Model, this paper describes (1) model construction and verification for design support of OD facilities and (2) model construction for operation support in a carrier-type advanced treatment process and an example of practical application.

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2. Construction of Process Model for OD Facilities: Development of Design Support Software

As part of a project called "Joint Research on Practical Use Methods for Activated Sludge Model" with Japan Sewage Works Agency in progress since 2001, JFE Engineering has been engaged in the development of design support software for the OD method, which has a substantial record of use in small-scale wastewater treatment facilities. This chapter describes the method of constructing a model for the biological reaction section and sedimentation section of OD facilities in Japan, a method of representing aeration equipment of different types, and the results of simulations based on water quality data obtained from actual treatment centers.

2.1 Composition of Constructed Model

2.1.1 Composition of model

Activated Sludge Model No. 2d (improved No. 2) was used. The composition of the simulation model consisted of a biological reaction tank, which was divided into multiple stages based on a report by Miyata et al.,[1] and a secondary clarifier, which was divided into a reaction zone and sedimentation zone.

Concretely, as shown in **Fig. 1**, the reaction tank forms an endless water channel divided into 8 assumed perfect mixing tanks, in which a mixed liquor of activated sludge is circulated at a specified flow rate. In Fig. 1, the influent section ①, effluent section ⑧, and aerator sections ②, ⑦ are arranged corresponding the structure of an actual facility, and their capacities are set at smaller values than those of the other sections ③–⑥.

2.1.2 Expression of oxygen supply in aeration equipment

As aeration equipment, a vertical shaft-type system and a submerged propeller system are assumed. In expressing the oxygen supply rate, K_{La} (total oxygen transfer capacity coefficient) was used, as shown below. R_{O_2} is a value which depends on aeration intensity and was calculated from materials showing the performance of the corresponding aeration equipment.

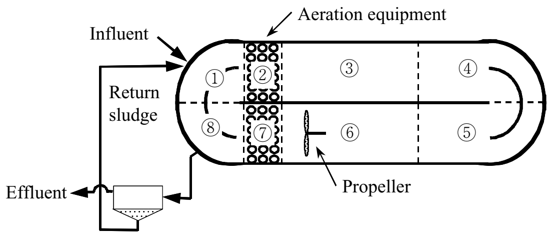

Fig. 1 Composition of a simulation model (Propeller OD)

$$K_{La} = \frac{R_{O_2} \times 10^3}{V_{air} \times S_{O_2}}$$

where,

K_{La}: Total oxygen transfer capacity coefficient (d^{-1})
R_{O_2}: Oxygen supply rate (kg-O_2/d)
V_{air}: Volume of aerator section (m^3)
S_{O_2}: Saturation dissolved oxygen concentration (g/m^3)

2.2 Verification of Simulation

2.2.1 Water sampling at actual treatment centers and analysis results

Water was sampled at three actual treatment centers which employ different aeration systems and tank configurations (Center A: vertical shaft-type/horseshoe-shaped tank, B: vertical shaft-type/elliptical tank, C: submerged propeller aeration equipment/elliptical tank). Operating and treatment conditions at the respective centers are shown in **Table 1**. Here, the water quality analysis values for the reaction tank influent (called influent in the following) and secondary clarifier overflow (treated water, called OD effluent in the following) are weighted average values (proportional to flow rate) of the analysis values of samples taken once an hour for a period of

Table 1 Operating conditions of treatment plant

		Wastewater treatment center			
		Center A	Center B	Center C	
				Run 1	Run 2
OD	Capacity (m^3/d)	1 600	1 600	2 500	
	Inflow rate	0.80	0.78	0.82	0.97
	HRT (h)	30	31	30	25
Aeration	Operation	Intermittent	High-Low	Intermittent	
	Aeration time (h/d)	12	High:11, Low:13	14	20
Inflow load	BOD (kg/d)	176	231	200	596
	COD_{Cr} (kg/d)	359	483	527	1 040
	T-N (kg/d)	42	46	54	124
Sludge in OD	MLSS (mg/*l*)	2 670	2 050	3 800	4 400
	SRT (d)	25	14	46	19
Temperature (°C)	Influent	16	16	16	20
COD_{Cr} (mg/*l*)	Influent	279	388	256	430
D-COD_{Cr} (mg/*l*)	Effluent	11	12	14	21
BOD (mg/*l*)	Influent	137	186	97	246
T-N (mg/*l*)	Influent	32	37	26	51
NH_4-N (mg/*l*)	Influent	21	26	15	14
	Effluent	0.48	0.49	0.00	5.3
NO_X-N (mg/*l*)	Effluent	1.6	1.2	0.91	0.01
T-P (mg/*l*)	Influent	3.4	4.0	3.9	4.0
PO_4-P (mg/*l*)	Effluent	0.74	0.15	0.90	0.06

Table 2 Influent and effluent data of secondary clarifier

		Center A	Center B	Center C Run 1	Center C Run 2
NO_X-N (mg/*l*)	OD effluent	–	–	–	0.27
	Effluent	1.6	1.2	0.91	0.01
	Return sludge	1.9	0.13	0.06	0.00
PO_4-P (mg/*l*)	OD effluent	–	–	–	0.00
	Effluent	0.74	0.15	0.90	0.06
	Return sludge	20	0.97	3.2	0.16

24 h. Because water is sampled in two runs at Center C, the results are listed as Runs 1 and 2.

With the exception of Center C, the influent flow rate was approximately 75–80% of the capacity (planned flow rate), and hydraulic retention time (HRT) in the reaction tank was around 30 h. Where the treatment condition is concerned, the quality of the OD effluent was comparatively stable in spite of large daily fluctuations in the inflow rate and inflow load. Except for Run 2 at Center C, the N and P contents of the OD effluent were under 0.5 mg/*l* for ammonia-N (NH_4-N), under 2 mg/*l* for the sum of nitric acid-N and nitrous acid-N (NOx-N), and under 1 mg/*l* for phosphoric acid-P (PO_4-P). Although the main purpose of these processes is BOD removal, all three treatment centers studied here also achieved high N and P removal efficiency as a result. As all of the centers operate with intermittent aeration, this appears to be a suitable treatment condition for N and P removal. **Table 2** shows the values of NOx-N and PO_4-P before and after inflow (OD effluent, effluent, and return sludge) at the secondary clarifier at each center. In comparison with the OD effluent, the NOx-N value of the return sludge is generally low, while return sludge PO_4-P tends to increase. From this, it was apparent that biological reactions which include denitrification and P release occur in the sludge accumulation zone of the secondary clarifier.

2.2.2 Verification of secondary clarifier model construction

To confirm the accuracy of the secondary clarifier model used in this work, a flow property test using a tracer was performed at the secondary clarifier at Center C to verify whether the tracer flow condition can be expressed satisfactorily by calculations using a simulation model for the object facility.

The tracer test was performed by the standard method, in which an LiCl tracer is introduced instantaneously into the reaction tank effluent and changes in the Li concentration of the secondary clarifier overflow are observed. To check the influence of sludge return, the Li concentration of the reaction tank effluent was also measured simultaneously. **Figure 2** shows the simulated and measured values of changes over time in the Li concentration of the secondary clarifier effluent and reactor effluent. Calculation was also repeated with arbitrary changes in the secondary clarifier tank composition. The calculated values indicated by A in the figure show a case where the effluent zone is assumed to consist of 2 perfect mixing tanks in series (effluent zone represents the part where the purified top portion of the reactor effluent introduced into the secondary clarifier overflows as OD effluent, i.e. treated water); B shows the case of 1 perfect mixing tank. Comparing the measured and simulated results, when the effluent zone was assumed to comprise one tank, the simulated results expressed changes in the Li concentration of the secondary clarifier effluent relatively satisfactorily. Moreover, under this condition, the simulation also expressed changes in the Li concentration of the reactor effluent relatively well.

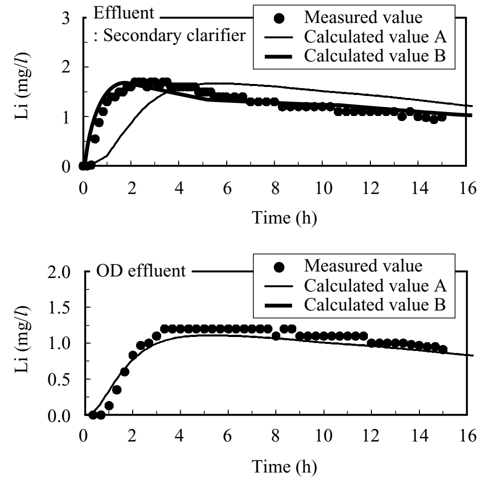

Fig. 2 Simulated and measured values of Li concentration

2.2.3 Composition of organic matter in influent

The composition of organic matter in the influent was investigated based on oxygen utilization rate (OUR) measurements, referring to Kappeler and Gujer's method for the biodegradable substrate.[2] However, slowly biodegradable substrate (X_S) was calculated by subtracting the oxygen demand of readily biodegradable substrate (S_S) and equivalent endotrophic absorption (measured by adding the mixed liquor to distilled water) from long-term oxygen demand when the mixed liquor in the tank is introduced in the influent. **Table 3** shows the results of fractionation of the organic components of influents at each of the treatment centers.

2.2.4 Verification of simulation results

Using the model described above, simulations were performed based on a total of 4 sets of measured results from the three treatment centers. Basically, the

Table 3 Oraganic components of influent

(mg/*l*)

		Center A	Center B	Center C	
				Run 1	Run 2
COD_{Cr}	Total	279	388	256	430
	Dissolved	134	189	81	266
Fermentation products (acetate)	S_A	4.7	6.9	0.9	34
Readily biodegradable substrate	S_F	11	8.2	7.2	2.0
Inert, non-biodegradable organics (dissolved)	S_I	11	12	14	21
Slowly biodegradable substrate	X_S	203	294	190	322
Inert, non-biodegradable organics (particulate)	X_I	8.9	12	8.2	10
Heterotrophic biomass	X_H	37	51	34	38

default parameters were used for quantitative coefficients, kinetic constants, and similar. However, some parameters, such as the reproduction rate, death rate, and half-saturation constant, were modified. The simulated and measured values of the effluent at Center B are shown in **Fig. 3**; the same results for Centers A and C are shown in **Fig. 4**.

Comparing the simulated and measured values of effluent D-COD_{Cr}, although the range of fluctuation in the calculated values was smaller than that of the measured results, the average values were virtually the same. The simulated results expressed the trends in NH_4-N and NOx-N relatively well, including time changes in effluent water quality. However, where PO_4-P was concerned, the results for Center B expressed the measured values with relatively well, but the calculated values for Centers A and C deviated considerably from the measured values.

Table 4 shows the simulated and measured values and average error in simulated values vs. measured values for the NH_4-N, NOx-N, and PO_4-P concentrations of the effluent at Center B. Here, calculated value ① shows the simulated results using Activated Sludge Model No. 2d default values for parameters, while calculated value ② uses parameters after calibration (parameters used in the aforementioned simulation). Using the default values, the average errors for NH_4-N, NOx-N, and PO_4-P were 1.2, 0.1, and 2.55 mg/*l*, respectively. In contrast, after calibration, the same values were 0.3, 0.2, and 0.13 mg/*l*, demonstrating that the concentrations

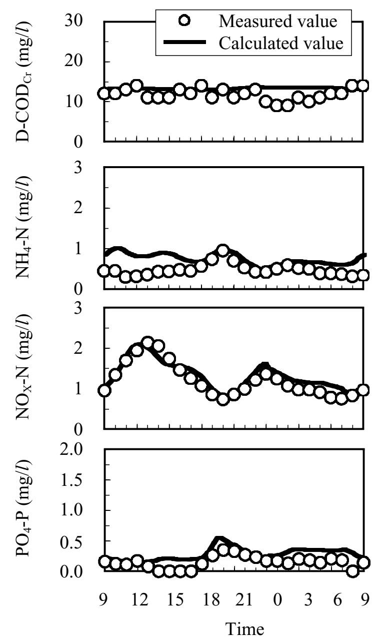

Fig. 3 Simulated and measured values of effluent in B treatment center

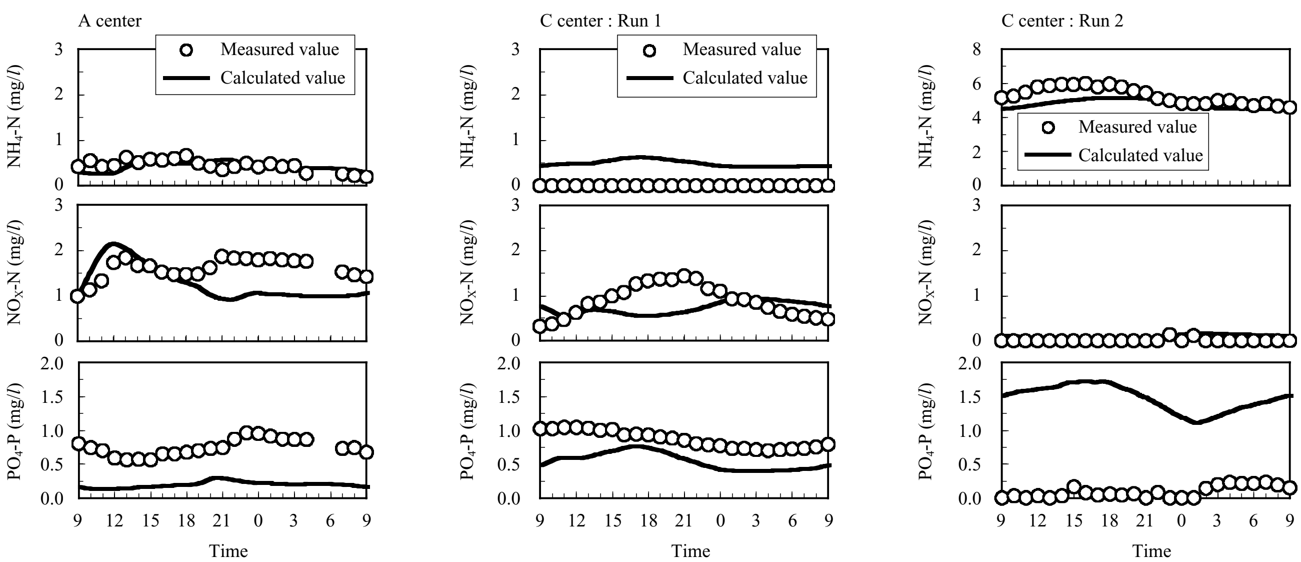

Fig. 4 Simulated and measured values of effluent in A and C treatment center

Table 4 Simulated and measured values of effluent, error of simulated and measured values

			NH_4-H	NOx-N	PO_4-P
Average values of effluent	(mg/*l*)	Measured	0.5	1.2	0.15
		Calculated ①	1.7	1.1	2.6
		Calculated ②	0.8	1.3	0.28
Average values of error	(mg/*l*)	Calculated ①	1.2	0.1	2.4
		Calculated ②	0.3	0.2	0.13
	(%)	Calculated ①	256	11	1 610
		Calculated ②	57	13	8.7

Calculated ①: Default palameters of ASM2d are used.
Calculated ②: Calibrated palameters are used.

in the measured data can be reproduced satisfactorily. However, as can be seen in the above-mentioned results for PO_4-P at Centers A and C, there is room for reconsideration in calibration for the reactions related to P.

2.3 Summary

It was possible to express the treatment condition at actual treatment centers which employ different aeration methods, OD configurations, and operating conditions using the same simulation method. Therefore, the present model is considered to function satisfactorily as a process model presuming design support for OD wastewater treatment facilities. As a remaining problem, it is still not possible to predict the measured values of P removal at a practical level. Thus, it is necessary to construct a model which is capable of highly accurate simulation through additional calibration even under different conditions. The authors also intend to construct a process model which assumes operation support for OD by studying long-term changes in treatment conditions and improvement in treatment results at actual facilities.

3. Construction of Model for Carrier-type Advanced Treatment Facilities: Development of Operation Support System

Because a simultaneous N and P removal process involves numerous control factors, a complex study on proper controls of the related factors is necessary to ensure stable, satisfactory removal performance. Thus, in developing a simulation software using the Activated Sludge Model as an operation support system, the authors constructed a model of the entire process which conforms to the equipment and treatment features of an actual treatment plant and verified the model using actual operating results from the plant. An outline of model construction for operation support at an actual plant and the results of verification are presented in this chapter.

3.1 Outline and Operating Conditions of Facility

A flow chart of the advanced treatment plant taken as an object is shown in **Fig. 5**. A bonding-type carrier manufactured by PEG is charged in an oxic tank at the carrier filling rate of 16% to immobilize nitrifiers. The design details of this treatment plant are shown in **Table 5**.

3.2 Modeling of Process in Actual Facility

A simulation model was constructed using Activated Sludge Model No. 2d. Calibration of the kinetic constants and other factors was performed using data surveyed at the actual plant on a regular basis after con-

Table 5 Design detail of the treatment plant

Design flowrate	Mean flowrate	11 363 m^3/d	
	Maximum flowrate	In summer	13 738 m^3/d
		In winter	13 050 m^3/d
Primary sedimantion tank		Water surface load 50 $m^3/m^2 \cdot d^{-1}$	
MLSS concentration in reactor		2 600 mg/*l*	
Recirculation ratio		2.5	
Return sludge ratio		0.5	
Reactor	Anaerobic tank	HRT 1.5h	
	Anoxic tank	HRT 4.5h	
	Oxic tank	HRT 3.6h Carrier filling factor 16% Microbiol carrier PEG	
Secondary sedimantion tank		Water surface load 25 $m^3/m^2 \cdot d^{-1}$	

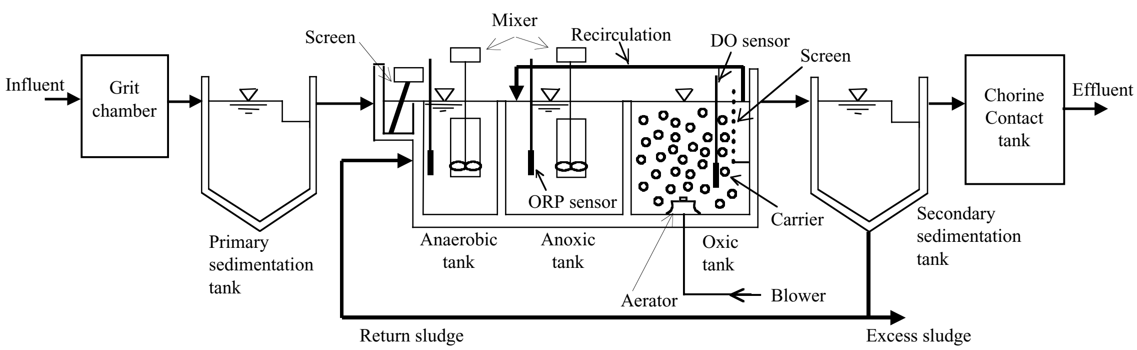

Fig. 5 Schematic diagram of anaerobic-anoxic-oxic plant with carrier

struction of the model.

3.2.1 Modeling of biofilm on carrier

Because the treatment plant is a carrier-type advanced one, properties related to the biofilm on the carrier surface were incorporated in the simulation model. As shown in **Fig. 6**, the area is divided into three zones, the biofilm layer (I), the boundary film layer (II), and the completely mixed bulk layer (III). Mass transfer between these layers occurs by (a) diffusion of substances and (b) separation of the biofilm. The thickness of the biofilm (L) is expressed by a value which changes depending on microorganism multiplication and separation.

Construction of this model made it possible to reproduce a condition in which an equivalent amount of nitrifiers is retained in the biofilm on the carrier while the nitrifiers bacteria in the biofilm are also supplied to the tank (bulk layer) by separation.

3.2.2 Oxygen supply

Oxygen supply was incorporated in the biological reaction (structure) model by treating Eq. (1) shown below as a velocity equation for the oxygen supply process, using the total oxygen transfer coefficient, K_{La}, for the equipment.

$$\partial S_{O_2}/\partial t = K_{La} \times (S_{O_2}{}^* - S_{O_2}) \cdots\cdots\cdots\cdots\cdots (1)$$

where,

K_{La}: Total oxygen transfer coefficient (d^{-1})

S_{O_2}: Concentration of dissolved oxygen in oxic tank ($g\text{-}O_2/m^3$)

$S_{O_2}{}^*$: Concentration of dissolved oxygen in saturation ($g\text{-}O_2/m^3$)

Because K_{La} differs depending on the aeration air flow rate, it was expressed as a function of the aeration flow rate based on measured results at this plant. From the measured results of K_{La} at the facility, it can be assumed that K_{La} of the submerged aerator is basically proportional to the aeration flow rate. Therefore, K_{La} can be expressed as in Eq. (2).

$$K_{La}\,(20) = 7.78 \times 10^{-3} \times Q_{air} \cdots\cdots\cdots\cdots (2)$$

where,

Q_{air}: Aeration flow rate of submerged aerator (m^3/h)

K_{La} (20): K_{La} at water temperature of 20°C (h^{-1})

After adding a temperature compensation term to K_{La}, K_{La} in Eq. (3) was used in the simulation.

$$K_{La} = 7.78 \times 10^{-3} \times 1.024^{(t-20)} \times Q_{air} \cdots\cdots (3)$$

3.2.3 Secondary sedimentation tank

Figure 7 shows an example of the N balance of this treatment plant calculated based on the results of daily tests. Here, the total of denitrification in the anaerobic tank and secondary sedimentation tank accounts for a large percentage of total influent N load, at approximately 20% (during other periods, there were cases when the value was approximately 10%), and thus cannot be ignored when reproducing the treatment conditions at this plant. Therefore, the treatment conditions at this plant were reproduced by a flow in which the secondary sedimentation tank is expressed by separation into simple solid-liquid separation and a reaction zone. After solid-liquid separation, the solid component is concentrated and discharged outside the system as excess sludge at the same rate as the excess sludge extraction rate at the actual plant (approx. 50–100 m^3/d/sedimentation tank). All of the remainder is reacted for a specified time in the reaction zone, and is then returned to the anaerobic tank as return sludge.

3.2.4 Model of tank series

As shown previously in Fig. 5, the flow in this treatment plant comprises three tanks, an anaerobic tank, anoxic tank, and oxic tank. Because the anoxic tank, which is the largest of the units, has a capacity of 1 230 m^3 and length of 25 m in the flow direction, it is not reasonable to express the reaction tank as a single perfect mixing tank. The results of a tracer test performed during trial operation of the plant indicated that

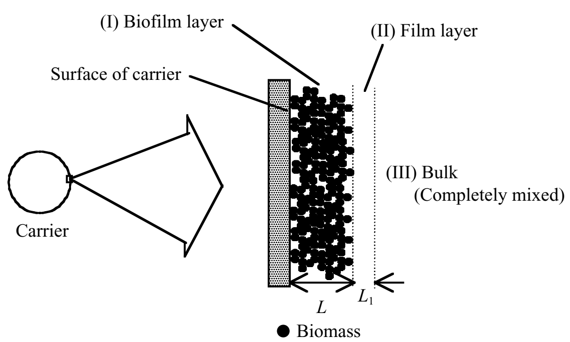

Fig. 6 Schematic of biofilm model

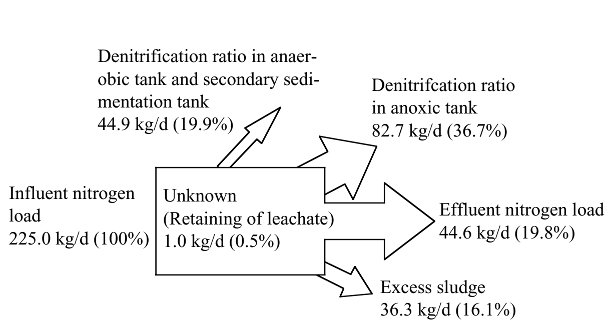

Fig. 7 Nitrogen balance at the treatment plant

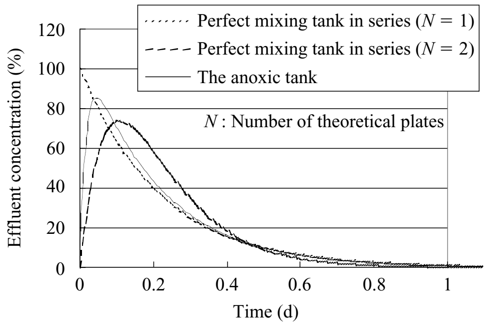

Fig. 8 Response curve (δ response) of the anoxic tank

it is equivalent to 1.4 theoretical plates. Assuming hypothetically a case where the anoxic tank comprises 1 perfect mixing tank and a case of 2 perfect mixing tanks (with equally divided capacity), and comparing denitrification using data obtained at the actual plant (Oct. and Nov.), denitrification can be calculated at 54.1 kg-N/d with 1 tank and 60.4 kg-N/d with 2 tanks.

Therefore, to reproduce the flow characteristics of this plant, modeling of the reaction tank was performed considering division of the tank and back-mixing. **Figure 8** shows the flow characteristics (response curve) of the anoxic tank at the actual plant.

3.3 Treatment Conditions at Actual Plant and Simulation Results

After calibration of the parameters, a long-term (approx. 2 months) dynamic simulation was performed, and the adaptability of this simulation model was verified by comparison with the effluent analysis data from the actual plant. The results for effluent N are shown in **Fig. 9**, with the measured values plotted on the abscissa and the calculated values obtained by simulation on the ordinate. From Fig. 9, it can be understood that simulation adequately reproduces treatment conditions at the actual plant.

3.4 Study of Operating Conditions at Actual Plant

After verifying this simulation model, a study was conducted by simulation to determine the permissible concentration of MLSS, which is one operation control item.

Based on the maximum flow rate (in winter) and influent quality (in winter) at this plant, a setting value of 2 600 mg/*l* was adopted for the MLSS concentration in the reactor. The designed N concentration for influent at this plant is 40 mg-N/*l*. However, in contrast to this, an investigation of the actual values of influent quality in the previous year during the period when the water temperature was 20°C or higher (Apr.–Dec.) showed a value of only 32 mg-N/*l*, suggesting that adequate N removal is possible with the MLSS concentration set to a lower level. Therefore, MLSS concentration setting conditions were evaluated to determine a suitable level for actual influent water quality conditions. The evaluation results are shown in **Fig. 10**.

As simulation conditions, this evaluation was carried out using a water temperature of 20°C and actual measured results at the facility from the previous year for influent water quality. As shown in Fig. 10, the effluent P concentration (T-P) satisfies the target value in all cases when the MLSS concentration is in the range of 1 200–1 800 mg/*l*, while effluent N (T-N) satisfies the target value when the MLSS concentration is 1 220 mg/*l* or more. Therefore, satisfactory treatment can be expected if the plant is operated with an MLSS concentration of 1 220 mg/*l* or higher.

Although not as remarkable as in summer, because the plant was designed based on the planned winter maximum load, the actual influent N concentration also showed a lower value than planned in winter, at 37 mg-N/*l*. Therefore, the same evaluation was performed for the setting MLSS concentration in winter, with the results shown in **Fig. 11**.

From Fig. 11, satisfactory treatment can be expected if the plant is operated in winter with an MLSS concentration of 1 450 mg/*l* or higher. However, in comparison with the simulation results for summer, the change in the effluent N concentration relative to the MLSS concentration is more rapid in winter, and the microorganism reproduction rate is also slower due to the low water

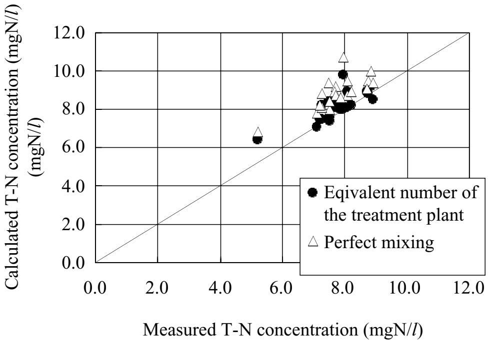

Fig. 9 Simulated and measured values of effluent T-N

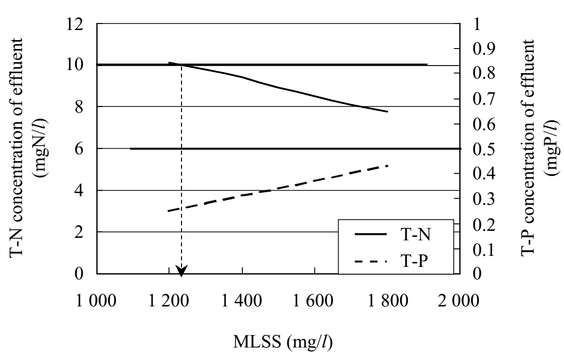

Fig. 10 Reconsideration of MLSS (in summer)

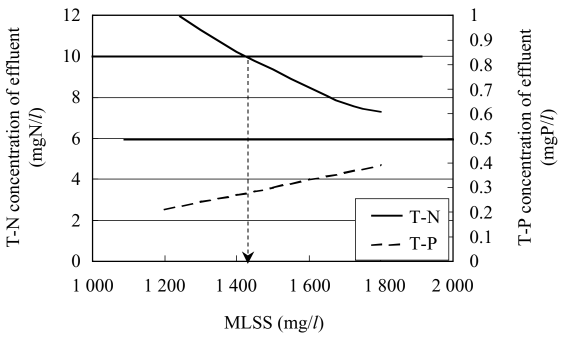

Fig. 11 Reconsideration of MLSS (in winter)

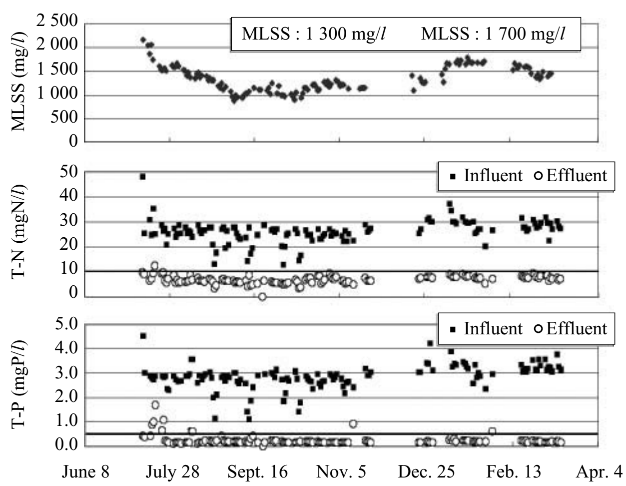

Fig. 12 T-N and T-P of effluent in the treatment plant

temperature. As a result, there may be cases in which treatment performance deteriorates and time is required for recovery. Considering this, an MLSS concentration of 1 600 mg/*l* or higher was judged necessary.

Based on the results described above, the plant was operated with the MLSS concentration set at 1 300 mg/*l* in summer and 1 700 mg/*l* in winter. As shown in **Fig. 12**, the effluent N concentration did not exceed the target value at any time during the study period.

3.5 Summary

A model was constructed for the purpose of developing simulation software as an operation support system for advanced wastewater treatment plants (microbiol carrier-type anaerobic-anoxic-oxic process). The simulation model was constructed based on study data from an actual treatment plant, with special attention to the oxygen supply, effluent from the reaction tank, the secondary sedimentation tank, and the carrier biofilm as features of the plant. The measured data and simulation results showed good agreement, confirming the appropriateness of the simulation model. A study was also carried out in connection with the MLSS concentration in the reaction tank, which is one operation control item, indicating that operation is possible with an MLSS concentration of 1 300 mg/*l* in summer (1/2 of the design specification) and 1 600 mg/*l* in winter. Satisfactory treatment results were obtained during the period when the actual plant was operated with the MLSS concentration set based on these simulation results.

4. Conclusion

This paper has described the construction of a model for design support of oxidation ditch (OD) wastewater treatment facilities and the construction of a model for operation support of microbiol carrier-type advanced treatment plants based on an Activated Sludge Model. In particular, a practical simulation technology for carrier-type, advanced wastewater treatment plants was developed by constructing a process model which expresses the biological reaction at the biofilm and plant mixing characteristics simply and with sufficient accuracy for practical application.

JFE Engineering conducts development with the aim of creating practical tools such as design support software and operation support software, as described above, and also plans to establish simulation technologies based on data from actual plants.

In this paper, Chapter 2, "Construction of Process Model for OD Facilities: Development of Design Support Software" includes some results of joint research with Japan Sewage Works Agency. In preparing Chapter 3, "Construction of Model for Carrier-type Advanced Treatment Facilities: Development of Operation Support System," data from research reports by Kawasaki City were used with permission. The authors wish to thank Japan Sewage Works Agency and Kawasaki City for their cooperation.

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