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Optimum municipal wastewater treatment plant design with consideration of uncertainty

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Abstract: A newly developed model for the optimum municipal wastewater treatment plant (MWTP) design is presented. Through introducing the interval variables, the model attempts to consider the effects of uncertainties caused by the fluctuation of the wastewater quality and quantity during the design of MWTP. The model solution procedure is illustrated in detail, and a numerical example is given to verify the feasibility and advantage of the model. Furthermore, the possibility of the model application is briefly outlined.

Keywords: uncertainty; municipal wastewater treatment plant; optimum design; model

Introduction

In general, a preferable MWTP design can be acquired based on designer's experience. However, sometimes the preferable MWTP design may not be optimal because of many inherent uncertainties. As a result, many unexpected problems will occur during the operation of the MWTP(Berthouex, 1970) and many manpower, material and financial resources will be wasted. How to deal with these uncertainties and find a most suitable design scheme in practical operation is one of the main concerns of optimum MWTP design and also one of the main goals that many designers want to attain(Middleton, 1976).

The optimization theory of the MWTP design, which began in 1960'(Lynn, 1962), has been developed for several decades. Although the model structures are becoming quite mature, the difference is still large between theoretical model and practical applications for less consideration of uncertainties. Therefore, it is of significance to go on the research on the optimization theory of the MWTP design to close the gap.

Uncertain system optimization models consist of random, fuzzy and interval optimization models(Zou, 1999). However, the application of the random optimization model is greatly restricted due to the data requirements on the probability distribution of parameters during the process of modeling, as well as the creation of some insurmountable intermediate models. The fuzzy optimization model is confined to solve the uncertain problems on the right terms of constrains, and it is incapable of solving the uncertainties of technological coefficients. In addition, the data information of the membership function is needed to establish the fuzzy models, which also increases difficulties in practical applications. Interval optimization model can directly reflect the uncertainties that exist in actual systems, and a group of result intervals can be obtained from the solution of the model. According to personal or collective experience and prejudice, the decision-makers could determine detailed schemes in the result intervals combining with some other actual conditions(Zou, 1999; Huang, 1993; 1995). Obviously, interval optimization model is more scientific, applicable and operable than traditional models.

Based on above considerations, this paper imports interval variables into the MWTP design for partly considering the fluctuation of influent wastewater quality and quantity. The interval optimization model is established and the solution procedure is also illustrated. Furthermore, a numerical example is shown to verify the feasibility and advantage of the model.

1 Sources of uncertainties in the MWTP design

The various uncertainties inherent in the design of the MWTP can be generalized in three aspects as follows(Uber, 1991; Tang, 1987): (1) uncertainty caused by human activity such as the expansion and movement of population; (2) uncertainty caused by natural phenomena such as the air temperature and rainfall; (3) the limitation of human being's capacity on understanding objective world will result in lots of fuzziness in MWTP design which may behave as some uncertainties in practical works.

There are many uncertainties that need to be considered in the MWTP design while many traditional or deterministic models have not done so or little. Therefore, the interval variables are introduced to develop the model and theory of the optimum MWTP design with the consideration of uncertainty.

2 Mathematical model for optimum MWTP design

It is impossible to take all uncertainties into consideration because of so many uncertainties existing in the MWTP design. For MWTP, the waste sources mainly come from domestic sewage, and partly from precipitations and some treated industrial wastewater etc. Generally, domestic and industrial wastewaters vary little with different seasons and regions. While in terms of precipitations, the change has great uncertainties and the general situation is that the water quantity is larger in summer and smaller in winter. In addition, the changes of wastewater quality and quantity are related with each other: if the flow rate is larger, the water quality will be comparatively better to the contrary, if smaller, then worse. Whereas, happening of some accidents such as industrial leakage will seriously affect water quality. Besides, human activities(such as construction works) will more or less influence the water quality of wastewater treatment plant. All in all, the change of the wastewater quality and quantity has many uncertainties because of the

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influence of numerous uncertainties, which brings on many corresponding uncertain effects on the MWTP design.

The quality and quantity of the influent wastewater are the main factors in the MWTP design, so it is a concerned problem that how to express the uncertainties caused by the fluctuation of the wastewater quality and quantity during the process of modeling. Normally, it is impossible to consider all these uncertainties in practical design. However, although the fluctuation of wastewater quality and quantity has some uncertainties, they should have varying ranges, i.e. a comparatively certain intervals while the average or maximum values cannot effectively represent the fluctuation of water quality and quantity. Based on above analysis, some of the parameters are converted into interval variables to consider some related uncertainties. The influent wastewater quality includes many indexes such as BOD_5 (biological oxygen demand) and TSS (total suspended solids, including four parts: the active biomass, the aerobically biodegradable volatile SS, the inert volatile SS and the fixed SS), which are the main pollutant indexes of wastewater treatment degree and have dominant influences on the design parameters of every part of the MWTP (Academy of Beijing for Municipal Design and Research, 1985). Therefore, three deterministic variables, namely the quantity of the influent wastewater, BOD_5 and TSS, are changed into non-deterministic interval variables to partly deal with some uncertainties caused by the variety of influent wastewater quality and quantity in the MWTP design. The flow chart of the traditional activated sludge process is selected as the basic one for the research (Fig.1).

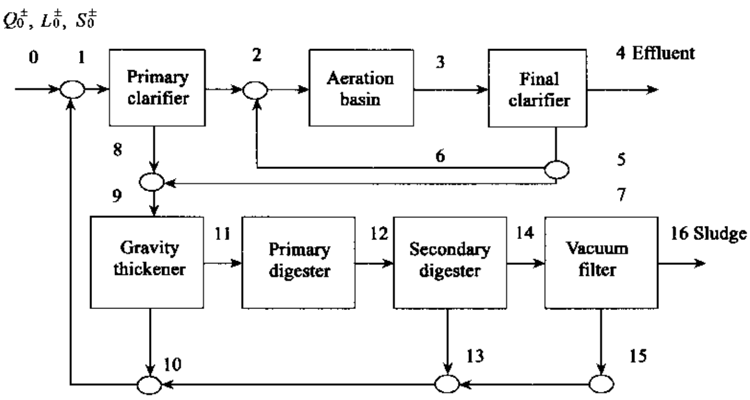

Fig.1 Flow chart

2.1 State variables

The state variables of the MWTP model include the variables of the wastewater and sludge streams at the control point j ($j = 0, 1, \cdots, 16$) in Fig.1, which are: $Q_j^{\pm}$ is the flow rate (m^3/d); $L_j^{\pm}$ is the concentration of soluble BOD_5 (mg/L); $S_{aj}^{\pm}$ is the active biomass (assumed to be a species of suspended solids; mg/L); $S_{dj}^{\pm}$ is the aerobically biodegradable volatile suspended solids (mg/L); $S_{ij}^{\pm}$ is the inert volatile suspended solids (stable relative to aerobic ones; mg/L); $S_{fj}^{\pm}$ is the fixed (inorganic) suspended solids (mg/L); and $S_j^{\pm}$ is the total suspended solids (mg/L). The plus and minus signs of each variable mean that the variable is an interval variable.

2.2 Mathematical model for the MWTP design

The researched mathematical model for the MWTP design includes the primary clarifier, the aeration basin, the final clarifier, the gravity thickener, the two anaerobic digesters, and the vacuum filter. The mathematical relationship for the influent and effluent of each unit process is described as follows.

2.2.1 Primary clarifier

The surface overflow rate and the influent solid concentration are two critical design parameters which affect the SS removal efficiency of the primary clarifier. The fraction of the influent solids remaining in the primary effluent is calculated by the Voshel-sak model as (Voshel, 1968):

$$S_2^{\pm} = S_1^{\pm} - v_1 (S_1^{\pm})^{v_2} (q_p^{\pm})^{-v_3}, \tag{1}$$

where $v_1 = 0.897$, $v_2 = 1.27$ and $v_3 = 0.22$, are constants. $q_p^{\pm}$ is the surface overflow rate, is a decision variable defined by $Q_2^{\pm}/A_p^{\pm}$; and $A_p^{\pm}$ is the surface area of the primary clarifier in m^2.

Based on the limiting flux theory, the concentration of the primary sludge is calculated by different thickening techniques. Thickening constants for primary sludge can be obtained from batch settling tests. The primary sludge concentration can be calculated as (Dick, 1975):

$$S_8^{\pm} = [a_p(n_p - 1)]^{1/n_p} \left(\frac{n_p}{n_p - 1}\right) \left(\frac{A_p^{\pm}}{Q_8^{\pm}}\right)^{1/n_p}, \tag{2}$$

where $a_p = 198.7$ and $n_p = 2.803$, are settling constants for the primary clarifier.

The total BOD_5 of the effluent from the primary clarifier was divided into soluble and suspended portions. The soluble BOD_5 was assumed unaffected by the primary sedimentation. The concentration of the suspended solids in the primary effluent is calculated assuming that the compositions of suspended solids in the primary effluent are the same as that in the primary influent.

2.2.2 Aeration basin

Standard biokinetic models are widely accepted in practice for the design of the activated sludge process. To use these models, the aeration basin must be modeled as a complete-mixing reactor. Waste stabilization is assumed to occur only in the aeration basin, and the biodegradable volatile solids are assumed to be completely consumed in the process. The soluble BOD_5 in the effluent of the aeration basin can be calculated as (Lawence, 1970):

$$L_3^{\mp} = \frac{K_s(1 + b\theta_c^{\pm})}{\theta_c^{\pm}(yk - b) - 1}, \tag{3}$$

where $K_s = 60$, half velocity coefficient in grams of BOD_5 per cubic meter; $k = 5.0$, maximum specific utilization rate in d^{-1}; $y = 0.4$, growth yield coefficient in kg cells/kg BOD_5; $b = 0.04$, endogenous decay coefficient in d^{-1}; $\theta_c^{\pm}$ mean cell residence time in days, is defined as:

$$\theta_r^{\pm} = \left(\frac{1}{24}\right)\frac{S_{a3}^{\pm} V_t^{\pm}}{Q_4^{\pm} S_{a4}^{\pm} + Q_7^{\pm} S_{a7}^{\pm} - Q_2^{\pm} S_{a2}^{\pm}}, \tag{4}$$

where $V_t^{\pm}$ is the aeration basin volume in m^3. And the active biomass concentration in the aeration basin can be calculated by (Tyteca, 1981):

$$S_{a3}^{\pm} = \frac{y}{1 + b\theta_r^{\pm}}\left(\frac{\theta_r^{\pm}}{\theta^{\pm}}\right)\left[L_2^{\pm} + \left(\frac{1.42gBOD_l}{gVSS}\right)\cdot \left(\frac{gBOD_5}{1.5gBOD_l}\right)S_{d2}^{\pm} - L_3^{\pm}\right], \tag{5}$$

where BOD_l is the final BOD_5 product; and $\theta^{\pm}$, hydraulic retention time in days, is defined by $V_t^{\pm}/24Q_2^{\pm}$.

The volatile inert suspended solid concentration ($S_{i3}^{\pm}$) and fixed suspended solid concentration ($S_{f3}^{\pm}$) in the mixed liquor can be calculated from the mass balance relationship with the assumption that the solid compositions remain unchanged after passing the final clarifier (Lin, 2000a).

$S_{a5}^{\pm}$ for the aerator can be obtained according to the mass balance relationship of the active biomass (Lawrence, 1970; Tang, 1987a):

$$S_{a5}^{\pm} = \left(1 + \frac{1}{r^{\pm}} - \frac{\theta^{\pm}}{r^{\pm}\theta_r^{\pm}}\right)S_{a3}^{\pm} - \frac{1}{r^{\pm}}S_{a2}^{\pm}, \tag{6}$$

where $r^{\pm}$ $Q_6^{\pm}/Q_2^{\pm}$, is defined as the sludge recycle ratio.

The oxygen requirement for the aeration basin may be estimated from the Lawrence-Mc Carty model by (Lawrence, 1970):

$$OR^{\pm} = 0.024Q_2^{\pm}(L')^{\pm}\left[\left(\frac{1.5gBOD_l}{gBOD_5}\right) - \frac{1.42gBOD_l}{gVSS}\left(\frac{y}{1 + b\theta_r}\right)\right], \tag{7}$$

where $OR^{\pm}$ is the oxygen requirement (kg/d), while

$$L' = L_2^{\pm} + \left(\frac{1.42gBOD_l}{gVSS}\right)\left(\frac{gBOD_5}{1.5gBOD_l}\right)S_{d2}^{\pm} - L_3^{\pm}. \tag{8}$$

The air flow rate is

$$AFR^{\pm} = \frac{C_s \cdot OR^{\pm}}{\gamma\alpha(\beta C_s - C)(1.024)^{T_t - 20}\zeta\rho}, \tag{9}$$

where $AFR^{\pm}$ is air flow rate in m^3/d; $\alpha = 0.8$, $\beta = 0.95$ are two correction coefficients; γ is the weight fraction of oxygen in air; C_s is the saturation concentration of the dissolved oxygen (DO) at 20℃ in g/m^3; C is the DO concentration maintained in the aeration basin in g/m^3; $\zeta = 0.08$ is the oxygen transfer efficiency; T_t is the temperature of the aeration basin in ℃; ρ is the density of air in kg/m^3. Assumed that the DO concentration maintained in the aeration basin is 1.5 g/m^3. Thus, the biological activity of a non – nitrifying activated sludge system would not be inhabited. And the air flow rate should satisfy the following equation:

$$AFR^{\pm}/V_t^{\pm} \geqslant \eta, \tag{10}$$

where $\eta = 28.8$ (referenced value) is the minimum mixing requirement of oxygen($(m^3\cdot d)/m^3$).

2.2.3 Final clarifier

Clarification efficiency of the final clarifier is a critical factor in determining the efficiency of the entire wastewater treatment system for both BOD_5 and TSS removal. The effluent BOD_5 from the final clarifier consists of both soluble organic materials and biodegradable suspended solids.

Both the design conditions of the aeration basin and the final clarifier have the influence on the clarification efficiency. The model developed by Chapman is selected to calculate the clarification efficiency of the final clarifier (Chapman, 1983):

$$S_4^{\pm} = -c_1 + c_2 S_3^{\pm} + c_3\frac{Q_3^{\pm}}{A_f^{\pm}} + SWD\left(90.2 - 62.5\frac{Q_3^{\pm}}{A_f^{\pm}}\right), \tag{11}$$

where $A_f^{\pm}$ is the surface area of the final clarifier in m^2; $c_1 = 180.6$, $c_2 = 4.0$, and $c_3 = 135.6$ are all constants, SWD is the side water depth (m).

The sludge compositions and the soluble BOD_5 are assumed to be unaffected by the final clarifier. The required effluent BOD_5 concentration and TSS can be formulated as:

$$L_3^{\pm} + \left(\frac{1.42gBOD_l}{gcell}\right)\left(\frac{gBOD_5}{1.5gBOD_l}\right)f\cdot S_{a4}^{\pm} \leqslant BOD_5\text{, standard}; \tag{12}$$

$$S_4^{\pm} \leqslant TSS\text{, standard}. \tag{13}$$

The sludge thickening is an important part for the final clarifier design, and different thickening models can be used for the modeling. The underflow solid concentration from the clarifier can be calculated by

$$S_5^{\pm} = [a_w(n_w - 1)]^{1/n_w}\left(\frac{n_w}{n_w - 1}\right)\left(\frac{A_f^{\pm}}{Q_5^{\pm}}\right)^{1/n_w}, \tag{14}$$

where $a_w = 24.2$ and $n_w = 2.375$ are two settling constants for the activated sludge. During the activated sludge process, the mean cell residence time($\theta_r^{\pm}$), the sludge recycle ratio($r^{\pm}$) and the hydraulic retention time($\theta^{\pm}$) are termed as decision variables.

2.2.4 Gravity thickener

The primary sludge and the activated sludge are assumed to be mixed before being thickened. Therefore, a set of mass balance relationships can be used to calculate the characteristics of the combined sludge. The underflow solid concentration from the thickener is calculated by (Academy of Beijing for Municipal Design and Research, 1985):

$$S_{11}^{\pm} = [a_r^{\pm}(n_r^{\pm} - 1)]^{1/(n_r^{\pm}-1)}\left(\frac{n_r^{\pm}}{n_r^{\pm} - 1}\right)^{n_r^{\pm}/(n_r^{\pm}-1)}(q_g^{\pm})^{-1/(n_r^{\pm}-1)}, \tag{15}$$

where $q_g^{\pm} = Q_{11}^{\pm}S_{11}^{\pm}/A_g^{\pm}$ is the decision variable; $A_g^{\pm}$ is the surface area of the gravity thickener in m^2; and $a_r^{\pm}$, $n_r^{\pm}$ are the thickening constants of the mixed sludge (Lin, 2000a).

The overflow concentration of the gravity thickener is assumed to be a deterministic parameter in the model due to its small range of change. The solid compositions for the overflow and the underflow in the gravity thickener can be calculated from the mass balance relationships with the assumption that the thickening process does not affect the solid compositions.

2.2.5 Anaerobic digester

Conventional designs of an anaerobic digester use two-stage systems. The primary sludge is generally mixed and heated to the fermentation temperature; most sludge stabilization occurs in this unit. The hydrolysis of the sludge is the rate-limiting step. It was assumed here that the stabilization rate is the second-order reaction: the first-order rate coefficient is mainly affected by the temperature of digestions and the second-order one is affected by other factors. The best digestion temperature is ranging from 33 to 35℃ (Voshel, 1968), herein it is assumed to be 35℃. Thus, the rate coefficient of the digestion reaction is (Pfeffer, 1974):

$$K = K_1 \cdot K_2 = 0.632\exp\left(23.408 - \frac{7675}{T_d + 273}\right)K_2, \tag{16}$$

where K_1 is the first-order rate coefficient in d^{-1}; $K_2 = 3.003$ (Uber, 1991), the second-order rate coefficient in d^{-1}; and T_d is the fermentation temperature in℃.

The compositions of the digested sludge could be calculated by assuming: (1) the volatile suspended solids in the digester are nondegradable when they are recycled back to the aerobic environment of the activated sludge process; (2) the volatile solids consist of no microorganisms that are capable of the aerobic degradation of organic material; (3) the fixed suspended solids are unaffected by the anaerobic digestion(Lin, 2000a).

It is inevitable that the methane will be produced during the anaerobic digestion process. Methane can be used in many different ways. However, economic benefits coming from methane are not considered in this model because of the little influence on the optimization of the total system.

The secondary digester is modeled as a gravity thickener and the underflow solid concentration can be calculated as:

$$S_{14}^{\pm} = [\delta a_d(n_d - 1)]^{-1/(n_d-1)}\left(\frac{n_d}{n_d - 1}\right)^{n_d/(n_d-1)}(q_d^{\pm})^{-1/(n_d-1)}, \tag{17}$$

where $a_d = 292.6$ and $n_d = 2.9$ are two settling constants for the completely digested sludge; $\delta = 0.167$ is a factor to discount the setting velocity of the digested sludge since the gas production in the second digester may be sufficiently high to cause enough turbulence to reduce the settling velocity of the digested sludge. $q_d^{\pm} = Q_{14}^{\pm} S_{14}^{\pm} / A_d^{\pm}$, which is the solid loading of the second digester, is a decision variable; $A_d^{\pm}$ is the surface area of the secondary digester in m^2. Both the effluent BOD_5 of the primary digester and the effluent concentration of the secondary digester are assumed to be constants for their very small variety. At the same time, the compositions of the soluble BOD_5 and the suspended solids are unchanged because it is assumed that the secondary digester has no effect on them.

2.2.6 Vacuum filter

Solids yield is the primary design variable for vacuum filter and can be calculated as (Tang, 1987):

$$q_r^{\pm} = 657.3\left(\frac{\chi P}{\mu R T}\right)^{0.5}(W^{\pm})^{0.5}, \tag{18}$$

where $q_r^{\pm}$ is solids yield in kg/($m^3 \cdot h$); $\chi = 0.33$ is form time per cycle; $P = 83300$, vacuum pressure in N/m^2; $\mu = 0.00089$, viscosity of filtrate in $(N \cdot s)/m^2$; $R = 10^{12}$, specific resistance in m/kg; $t = 6.0$, the cycle time in minutes; and

$$W^{\pm} = Q_{16}^{\pm} S_{16}^{\pm} / Q_{15}^{\pm} \tag{19}$$

is the mass of solids filtered per volume of filtrate in kg/m^3.

The solid of filter is selected as a decision variable for this unit. The surface area of vacuum filter is

$$A_r^{\pm} = Q_{16}^{\pm} S_{16}^{\pm} / q_r^{\pm}. \tag{20}$$

The effluent concentration of vacuum filter is assumed to be a constant. And soluble BOD_5 and the solid compositions are unchanged through the unit.

2.2.7 Sludge disposal

Sludge disposal is a part of the municipal wastewater treatment plant. There are many ways to dispose the sludge such as landfill, composting, and fertilizing. The sludge disposal is not considered in the model due to its little effect.

2.2.8 Objective function

Formulation of the optimization model for specific activated sludge systems is described herein. The objective of the total system is to find the minimum total cost. The total cost includes capital, operation and maintenance cost(O&M) which are associated with the capacity(square or volume) of the treatment units. The specific cost function can be expressed as(Huang, 1993):

$$C = aX^b, \tag{21}$$

where C is the total capital or O&M cost of a given unit process; X is the design variable of the given unit which significantly influences the cost; and a and b are two estimated constants.

Then, the objective function of the overall system can be formed as:

$$TC = \sum_{k=1}^{N} CC_k + \tau \sum_{k=1}^{N} OC_k, \tag{22}$$

where TC is the total annual cost for the system; CC_k is the capital cost for unit k; OC_k is the annual O&M cost (assumed to be constant over the plant lifespan) for unit k; CC_k and OC_k are the optional cost functions expressed by Eq. (21); N is the total numbers of cost units to be considered; and τ, a discount factor(Lin, 2000a). It is noted that X in Eq.(21) is an interval variable and hence TC in Eq.(22) is also an interval variable.

Therefore, the overall system model can be expressed as an interval nonlinear programming model:

$$\text{Min } TC^{\pm} = \frac{F^{\pm}(x^{\pm}, y^{\pm}, z)}{x^{\pm}, y^{\pm}}. \tag{23}$$

Subject to:

$$f^{\pm}(x^{\pm}, y^{\pm}, z) = 0, \tag{24}$$

$$BOD^{\pm}(x^{\pm}, y^{\pm}, z) \leqslant BOD \text{ standard}, \tag{25}$$

$$TSS^{\pm}(x^{\pm}, y^{\pm}, z) \leqslant TSS \text{ standard}, \tag{26}$$

where $TC^{\pm}$ is the total cost (include the capital and O&M cost); $x^{\pm}$ are the state variables including $Q_0^{\pm}$, $L_0^{\pm}$, $S_0^{\pm}$; $y^{\pm}$ are the decision variables (or design variables) such as recycle ratio, solid loading; and Z are the model parameters such as v_1, a_w; and BOD standard and TSS standard are equal to relative effluent standards.

The model has 51 equations(including 23 flow and mass balance equations), 3 inequality constraints and 8 decision variables. To obtain the optimal solution, the characteristic of the recycle flow has to be determined

3 Model solutions

The influent wastewater concentration is hard to determine if the characteristic of the recycle part is unknown during the solution process. In the control points 10, 13, 15, there are 12 unknown variables, which are $Q_{10}^{\pm}$, $L_{10}^{\pm}$, $S_{a10}^{\pm}$, $S_{d10}^{\pm}$, $S_{i10}^{\pm}$, $S_{f10}^{\pm}$, $Q_{13}^{\pm}$, $S_{i13}^{\pm}$, $S_{f13}^{\pm}$, $Q_{15}^{\pm}$, $S_{i15}^{\pm}$, $S_{f15}^{\pm}$. Among them, $Q_{10}^{\pm}$, $Q_{13}^{\pm}$, $Q_{15}^{\pm}$ can be estimated by the following expressions:

$$Q_{10}^{\pm} = 4.667 Q_0^{\pm} \cdot \left(\frac{L_0^{\pm}}{100}\right)^{0.5} \cdot \left(\frac{S_0^{\pm}}{200}\right)^{0.3} \times 10^{-3}, \tag{27}$$

$$Q_{13}^{\pm} = 6.667 Q_0^{\pm} \left(\frac{S_0^{\pm}}{200}\right)^{1.0} \times 10^{-4}, \tag{28}$$

$$Q_{15}^{\pm} = 2.0 Q_0^{\pm} \left(\frac{L_0^{\pm}}{100}\right)^{0.5} \left(\frac{S_0^{\pm}}{200}\right)^{0.6} \times 10^{-3}, \tag{29}$$

$L_{10}^{\pm}$, $L_{13}^{\pm}$ and $L_{15}^{\pm}$ are assumed to be constants in the design, and so are $S_{10}^{\pm}$, $S_{13}^{\pm}$ and $S_{15}^{\pm}$ because of the small changing range for these variables.

Because thickening and filtration have little influence on the solids compositions, the effluent compositions of the digester will be the same as that of the vacuum filter based on the assumptions that suspended solids in the digester consist of only the inert volatile suspended solids (S_{i13}^{*}) and the fixed suspended solids(S_{f13}^{*}). Hence, there is:

$$\frac{S_{i13}^{*}}{S_{f13}^{*}} = \frac{S_{i15}^{*}}{S_{f15}^{*}} = m^{*}. \qquad (30)$$

If m^{*} is determined, the underflow compositions from the secondary digester and the filter will be obtained. The result will be more satisfactory if m^{*} has the range between 0.2—0.4. When solving the models, we could obtain a certain solution if we assumed that the wastewater quality and quantity was fixed first(Lin, 2000b). Of course, each solution represents one design scheme. However, the random project(any single solution) could meet the effluent standard, but maybe have enormous cost of the whole system, which is unacceptable in practical application due to the close relationships between structure size of each unit and the total cost(including capital, operation and maintenance cost etc.). Therefore, it is necessary to evaluate these schemes, which is an optimization process.

It would be quite difficult to apply the traditional optimal method to solve the complex interval nonlinear programming model described by Equations (23)—(26). The effective step searching method of the genetic algorithm was utilized in this paper. The principles are as follows: Within the range of error, a continuous interval of each interval variable can be separated into some definite discrete intervals, and every possible value of these discrete intervals can be combined to form many certain programming models, and then the solution of these models will be acquired according to the rule of solving a certain solution; after any given combination of these intervals is obtained, the results will be accepted as the solution intervals. Obviously, more intervals are divided; more precious results will be obtained. The final optimal solution will be got by using the random searching ability of the genetic algorithm with greatly improved efficiency. Elaborate solution procedure is not listed here, and the reader can refer to literature(Qin, 2002) if needed.

At present, there are still no definite ways to solve the interval nonlinear programming model. Though the searching method itself has its disadvantage in precision, it is practical within the error allowance. It is important to note that the result obtained from the step searching method of the genetic algorithm is not the proper optimal interval solution but a relatively better interval solution in certain conditions.

4 Numerical example

Table 1 lists the statistic of the influent wastewater quality and quantity in one year of a given MWTP in North China.

Table 1 Influent wastewater quality and quantity of a given MWTP in China

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.	Ave.
Q	32413	33516	34812	35289	42758	43406	45238	37083	45116	40002	37873	35654	38618
BOD_5	222.6	276.0	160.4	165.2	217.6	195.9	177.4	153.0	160.0	172.7	183.8	202.1	190.0
TSS	200.4	191.5	144.1	148.1	310.4	279.8	253.6	241.9	247.0	220.5	209.5	165.4	217.9
S_{a0}	3.1	2.9	3.3	4.1	5.3	6.4	8.2	8.4	7.5	5.4	4.9	3.8	5.3
S_{d0}	102.3	93.2	70.6	71.4	170.8	150.9	148.9	115.9	150.8	120.4	110.4	79.6	115.6
S_{i0}	44.6	40.3	34.8	37.1	59.0	55.8	43.6	52.3	38.6	44.8	40.3	38.2	44.2
S_{f0}	50.4	55.1	35.4	35.5	75.3	66.7	52.9	65.3	50.1	49.9	53.9	43.8	52.8

Notes: influent flow, Q, m^3/d; influent BOD_5 concentration, mg/L; influent TSS concentration, mg/L; S_{a0}, active biomass; S_{d0}, aerobically biodegradable volatile suspended solids; S_{i0}, inert volatile suspended solids; S_{f0}, fixed suspended solids

It can be seen in Table 1 that the influent wastewater quality and quantity vary in a certain range. The wastewater is larger and comparatively better in summer than that in winter. As a whole, the fluctuation of the influent wastewater quality and quantity means some uncertainties.

The numerical intervals of the change of the influent quality and quantity can be obtained based on Table 1(Table 2). These intervals can partly reflect the characteristic of the uncertain system and their ranges of change. Therefore, the establishment of the MWTP model can take these uncertainties into consideration directly.

Table 2 Numerical interval of fluctuation of the influent wastewater quality and quantity

State variable	Mensal average minimum	Annual average	Mensal average maximum
Influent flow, Q_0, m^3/d	32413	38618	45238
BOD_5 concentration, L_0, mg/L	153.0	190.0	276.0
Active biomass, S_{a0}, mg/L	2.9	5.3	8.4
Aerobically biodegradable volatile, S_{d0}, mg/L	71.4	115.6	170.8
Inert suspended solids, S_{i0}, mg/L	34.8	44.2	59.0
Fixed suspended solids, S_{f0}, mg/L	35.4	52.8	75.3
Total suspended solids, S_0, mg/L	144.1	217.9	310.4

Generally, the average values of the wastewater quality and quantity are supposed to be the basic data for the MWTP design. In this research, the numerical intervals were formed by the average values and the maximum values. The calculated results are obtained in Table 3. The interval values for the relevant decision variables are listed in Table 4.

Table 3 The calculated results

Design variable	Numerical interval
Primary clarifier: area, m^2	579.9—1362.3
Aeration basin: volume, m^3	11366.0—15448.5
Final clarifier: area, m^2	872.6—1059.3
Gravity thickener: area, m^2	485.6—994.3
Primary digester: volume, m^3	4052.8—7303.4
Secondary digester: area, m^2	81.4—163.5
Vacuum filter: area, m^2	13.64—24.63
Oxygen requirement, kg/d	9607.1—15522.9
Total cost, 10^4 RMB Yuan/a, $n = 20$	906.2—1246.3

As far as the MWTP design is concerned, the basic design can be made if the above design parameters are known. Of course, many details need to be figured out in the design such as the grid design, options of the tank type. Any single group of parameters from the above intervals of the solution is related to some optimal design that is based on some influent wastewater quality and quantity. In practical design, the lower bounds of the intervals can be used for the sake of canonization while the upper bounds of the intervals can be used for the guarantee of the reach of the water quality for the effluent wastewater from the MWTP. It is convenient for the decision-makers to get many choices in the intervals according to their personal experiences, preferences and other practical conditions. Any random combination of parameters in the intervals can meet the requirement of the discharge standard as the low bounds of the solution interval are based on the average quality and quantity of wastewater(assuming the standards are BOD_5 < 30 mg/L, TSS < 30 mg/L).

Table 4 Decision variables

Design variable	Numerical interval
Surface overflow rate, $m^3/(m^2 \cdot h)$	2.8—1.4
Sludge age, d	4.4—4.3
Hydraulic retention time, h	7.0—8.1
Sludge recycle ratio, %	23.0—25.0
Solids loading of thickener, $kg/(m^2 \cdot h)$	0.68—0.64
Sludge digestion time, d	26.4—27.2
Solids loading of secondary digester, $kg/(m^2 \cdot h)$	1.20—1.11
Solids loading of filter, $kg/(m^2 \cdot h)$	6.50—6.68

If no interval variables were introduced, the results would be normally a definite solution, which was the only choice for decision-makers. If so, the decision makers would not know the effects of a little change of the calculation result to the final design, which would result in the functions of the decision-makers experiences or other practical conditions were not exerted. Conversely, the interval solutions could provide a large space for decision makers to easily make a choice in the intervals according to actual situations, so some related uncertainties in the MWTP design could be considered.

The model is more reasonable than the traditional one for it has tried to incorporate some uncertainties into the MWTP design. At present, this model is supposed to evaluate the schemes of MWTP design with activated sludge process or provide some references for decision-makers. Moreover, the model can also be developed to control the treatment process by adjusting the parameters(such as hydraulic retention time, sludge recycle ratio) with the wastewater quality and quantity. As a result, the MWTP could be operated with a relatively high efficiency and low cost.

5 Conclusions

The model and theory of the optimal MWTP design have been greatly improved in the past decades. However, there still exists a large gap between the theoretic model and the practical application because of a lot of complexities or uncertainties related to the MWTP design. In this paper, a newly developed model for optimizing municipal wastewater treatment plant (MWTP) design is presented to accommodate some uncertainties. Interval variables are introduced to consider the effect of uncertainties caused by the fluctuation of the wastewater quality and quantity into MWTP design. The model solution procedure is illustrated, and a numerical example is given to verify the feasibility and advantage of the model and the possibility of the model application is also outlined briefly.

References:

Academy of Beijing for Municipal Design and Research, 1985. Feedwater and drainage design manual (5): Municipal drainage [M]. Beijing: China Construction Industry Publishing House. 47—88.

Berthouex P M, Polkowski L B, 1970. Waste treatment plant design under uncertainty[J]. Jour Water Poll Control Fed, 42(9): 1589—1613.

Chapman D T, 1983. The influence of process variables on secondary clarification [J]. Journal of the Water Pollution Control Federation, 55(12, Part 2): 1425—1434.

Dick R I, Suidan M T, 1975. Modeling and simulation of clarification and tricking process in mathematical modeling for water pollution control process[M]. Ann Arbor, Mith: Ann Arbor Science Publishers Inc. 147—191.

Huang G H, Beatz B W, 1995. Grey fuzzy integer programming: An application to regional solid waste management planning[J]. Socio-Econ Plan Sci, 29: 17—38.

Huang G H Moore, 1993. Grey linear programming: its solving approach, and its application[J]. International Journal of Systems Sciences, 24(1): 159—172.

Lawrence A W, Mc Carty P L, 1970. A unified basis for biological treatment design and operation[J]. Journal of the Sanitary Engineering Division ASCE, 96(3): 757—778.

Lin Y P, Zeng G M, 2000a. Optimum municipal wastewater treatment plant design model under uncertainty(a)(model)[J]. Journal of Hunan University, 27(2):56—62.

Lin Y P, Zeng G M, 2000b. Optimum municipal wastewater treatment plant design model under uncertainty(b)(solution and application)[J]. Journal of Hunan University, 27(4):73—82.

Lynn W R, Logan J A, Charnes A, 1962. Systems analysis for planning wastewater treatment plants[J]. J Water Poll Control Fed, 34(1): 565—581.

Middleton A C, Lawrence A W, 1976. Least cost design of activated sludge systems[J]. J Water Poll Control Fed, 48(1): 889—905.

Pfeffer J T, 1974. Temperature effects on anaerobic fermentation of domestic refuse [J]. Biotechnology and Bioengineering, 16: 771—787.

Qin X S, Zeng G M, 2002. The application of genetic-algorithms on grey nonlinear programming in water environment[J]. Advances in Water Science, 13(1):31—36.

Tang C C, Brill E D Jr, Pfetter J T, 1987. Comprehensive models of an activated sludge wastewater treatment system [J]. Journal of the Environmental Engineering Division, ASCE, 113(10): 952—969.

Tyteca D, 1981. Nonlinear programming model of wastewater treatment plant[J]. Journal of the Environmental Engineering Division, ASCE, 107(4): 747—766.

Uber J G, Pfetter J T, Brill E D Jr, 1991. Robust optimal design for wastewater treatment. II: Application [J]. Journal of the Environmental Engineering Division, ASCE, 117(4): 438—456.

Voshel D, Sak J G, 1968. Effect of primary effluent suspended solids and BOD on activated sludge production [J]. Journal of the Water Pollution Control Federation, 40(5, Part 2): 203—212.

Zou R, Guo H C, 1999. An environ-economic harmonized inexact system programming for agriculture forestry landuse in the Lake Erhai Basin[J]. Acta Scientiae Circumstantiae, 19(2): 186—193.

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