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Optimal Sizing of Activated Sludge Domestic Wastewater Treatment Processes Considering Uncertainties

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OPTIMAL SIZING OF
ACTIVATED SLUDGE DOMESTIC
WASTEWATER TREATMENT PROCESSES
CONSIDERING UNCERTAINTIES

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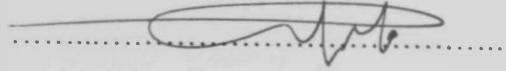
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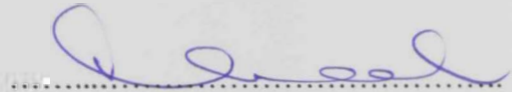
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ABSTRACT

A mathematical framework is developed for use in the optimal sizing of a wastewater treatment system comprises an activated sludge system preceded by a primary clarifier. Mathematical models predicting the performance of various unit processes are used to construct the system model. ASM3, used in the developed framework, is among the most recent and comprehensive models that closely describe the biological reactions taking place in the activated sludge system. Cost information functions including capital and operational costs of different system units are also modeled. An optimization problem is formulated with the objective to produce optimal sizes of different units with least cost and meeting the effluent requirements. The problem is a nonlinear programming problem that is solved using the General Algebraic Modeling Systems software "GAMS". The optimization model is applied to an illustrative problem producing valuable and practical results. The model is also used as an analysis tool to reveal the influence of various involved parameters and inputs upon the system performance and relevant results. Uncertainty consideration is also highlighted with an example showing an expected-value problem. Important insights about process design, modeling, and integration were gained by exercising the model. Such include the effectiveness of each unit operation, the importance and effect of sludge retention time, the effect of temperature on model performance and cost, and the effect of influent characteristics variability. Huge cost savings can be achieved by controlling the system at different temperatures. Influent characteristics variability is of great importance and considering such at the design stage contributes significantly to the designed system optimality and reliability.

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ABBREVIATIONS AND SYMBOLS

GAMS   General Algebraic Modeling Systems software

ASM1   Activated Sludge Model No. 1

ASM2   Activated Sludge Model No. 2

ASM3   Activated Sludge Model No. 3

IWA   International Water Association

q Primary clarifier overflow rate

Q Wastewater flow rate

A_P Primary clarifier surface area

MLSS   Mixed liquor suspended solids

μ_{max} The maximum specific growth rate

Y The growth yield

X_I Inert particulate organic matter

X_S Slowly biodegradable substrate

$X_{B,H}$ Active heterotrophic biomass

$X_{B,A}$ Active autotrophic biomass

X_D Debris from biomass death and lysis

S_I Inert soluble organic matter

S_S Readily biodegradable substrate

S_O Oxygen

S_{NO} Nitrate nitrogen

S_{NH} Ammonia nitrogen

S_{NS} Soluble biodegradable organic nitrogen

X_{NS}	Particulate biodegradable organic nitrogen
S_{ALK}	Alkalinity molar units
X_H	Heterotrophic organisms
X_A	Nitrifying organisms
X_{STO}	A cell internal storage product of heterotrophic organism
X_{SS}	Suspended solids
S_{NOX}	Nitrate plus nitrite nitrogen
S_{NH4}	Ammonium plus Ammonia nitrogen
S_{N2}	Dinitrogen
SRT	Sludge retention time
HRT	Hydraulic retention time
SVI	Sludge volume index
V	Volume of the aeration tank
r	Sludge recycle ratio
w	Wastage ratio
SR	Overflow rate of final clarifier
A_f	Surface area of the final clarifier
RO_H	Oxygen Requirement for removal of organic matter
RO_A	Oxygen Requirement associated with nitrification
AFR	Air flow rate
H	Side water depth of final clarifier
X1	Removal efficiency in the primary clarifier

CHAPTER 1

INTRODUCTION

Pollution of water has great implications since water is considered as life for humans. This pollution is mainly attributed to domestic and industrial usage. The polluted water contains among others large quantities of organic and nitrogenous compounds. For many reasons (regulation constraints, health, environment, water reuse, etc.), the concentration of these compounds must be reduced. This is achieved by means of physical, biological, and chemical treatment methodologies. Wastewater originated from domestic water use is usually treated in treatment plants of various types and configurations. The activated sludge process is one of the most widespread biological wastewater purification technologies for domestic wastewater treatment purposes. Activated sludge wastewater treatment plants are widely used for various treatment purposes.

The activated sludge process as the most widely used biological wastewater treatment process has gained a great attention from researchers. This is also attributed to its complexity as a biological operation where it has triggered various efforts to understand and model the various biochemical activities form the process. Modeling and design of the process have faced several challenges until the current modeling and design practice is formulated. Especially during the last two decades modeling of biological degradation processes in activated sludge plants has been an important research topic. Recent developments in process modeling have resulted in the inauguration of advanced dynamic general-purpose models. The most common recent applied activated sludge models are ASM models of the International Water Association (IWA).

The main objective of present wastewater treatment plan design, in general, is to provide a cost effective processing system for a given wastewater. In the design of activated sludge wastewater treatment plants, several design decisions are to be undertaken. These include for example: selecting the appropriate unit processes,

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choosing configuration options, and determining operational conditions of selected unit processes. Configuration options may include aspects such as capacity, shape, and placement, while operational conditions may include minimum and maximum design temperatures, flow rate, aeration rate, and quantity of return activated sludge. Ideally, the resulting design should consider minimizing costs while meeting constraints such as treatment capacity and current and expected effluent regulations. These considerations are combined to form a complex design problem. Additionally, each of the unit processes is designed to achieve a specific goal, and only limited consideration is given during the design procedure to interactions among the unit processes.

In practice, the most commonly used resources for making design decisions are past experiences, published guidelines, and trial and error. While these procedures generally produce designs of acceptable performance, most often there is remaining considerable space for improvement in terms of performance as well as cost. As an alternative, activated sludge models, such as ASM models, are becoming increasingly important in assisting design engineers in making good decisions for wastewater treatment design and operational conditions. These models allow for the testing of design performance in a virtual environment and for reducing the need for expensive pilot studies. In addition, modeling experience increases the engineers' understanding of the performance of treatment process options and their confidence that innovative designs will reliably meet current and future effluent limits.

Generally, activated sludge models are used in a trial-and-error fashion. First, a designer proposes a tentative design based on treatment objectives and other criteria. The design is modeled and effluent characteristics are predicted using simulation. If possible, capital and operations and maintenance costs are also estimated. Based on its performance, the design is then iteratively modified and tested until a satisfactory design is obtained. This description of the design process does not adequately convey the potential difficulties faced by the design engineer, however. For example, even with a small number of design decisions, the problem can be very large. A simple example illustrates this point. If there are 10 design decisions, and each of these can take one of two values, the result is 2^{10}, or 1024, possible design combinations. The combinatorial nature of such problems generally means that not all design possibilities can be tested. Instead, guided by experience, the designer must choose which design

directions to pursue. Such decisions are made more difficult by the complex biokinetics of treatment operations and by the potential interaction between treatment processes, which may lead in some cases to counterintuitive performance. The "mental model" used by the designer to consider these issues and direct the design process may be very different from reality and may lead to designs that have considerable room for improvement, as mentioned earlier.

An alternative to this design paradigm is one in which the design process, essentially a search through design possibilities, is automated using optimization. In an optimization-driven design context, the designer supplies mathematical descriptions of design objectives and constraints (e.g., minimize total cost while meeting effluent targets). An optimization algorithm is then used to identify one or more design alternatives that best meet these criteria. This paradigm has the advantage that it is able to consider design objectives, constraints, and performance comprehensively and simultaneously. In addition, such an approach can be extended to provide system-wide optimization wherein all of the plant's processes are optimized together. The literature review in Section 1.1 provides an overview of the considerable progress that has been made since Lynn et al. reported the first work in this area in 1962.

This research applies recent developments in modeling and understanding of activated sludge process to optimally size units in a wastewater treatment system. It combines unit processes models within an overall optimization framework as an analysis and design tool. It is important to stress that wastewater treatment plant design, as mentioned, is a complex process and that good designs cannot be generally achieved using only mathematical computerized model. The best system models are designed for use by designers, who ultimately have the responsibility for taking into account factors not considered in the model. System optimization models, like the one developed in this research, can be very useful design tools. An optimal solution for a given input data and effluent requirements can be obtained based on prescribed constraints and assumptions. By varying the specified conditions, the designer can use such models to facilitate the evaluation of options and tradeoffs.

Research in developing a comprehensive design procedure is important because the need for wastewater treatment will clearly continue to require the commitment of significant resources at the national and international level. It is also

important to improve the understanding of various treatment processes so that innovative regulatory approaches to water quality management can be better evaluated. In general, as more cost effective regulatory approaches are developed it will be even more important to better understand the options and tradeoffs in wastewater treatment. Perfect understanding (e.g. of costs) cannot be expected, but relative performances, costs, trends, etc. provide fundamental insights.

In the reminder of this chapter, a thorough literature review of past research efforts related to optimization of wastewater treatment facilities is presented. Sections 1.2 and 1.3 outline research objectives and procedure. Section 1.4 describes the organization of the thesis.

1.1   Literature Review

A number of studies have been devoted to the cost-effective design "Optimization" of wastewater treatment plants, in general, and activated sludge treatment plants in particular. Three important works are considered the basis for this literature review. The comprehensive literature review given by Tang et al. (1984), the similar literature review presented by Tyteca (1985), and the series of literature review papers published in Water Environment Research Journal by different researchers. This series covers almost all aspects of wastewater treatment in the last decade.

Mathematical models for optimization of wastewater treatment appear in the literature as early as 1962 (Lynn et al., 1962) and since then various studies have been devoted to this problem. Optimization in the field of wastewater treatment has been utilized for more than obtaining the most cost-effective design. Optimization studies covered almost all aspects of wastewater treatment including: optimal process combination, optimal process design and operation, in addition to cost-effective design. This thesis can be considered as of the third category as it is concerned with finding an optimal sizing of activated sludge system that minimizes the total cost and fulfills the effluent requirement. Hence, only studies lying under this category are emphasized in this literature review. Other studies that cover the aspects of optimization of various design parameters for certain purposes are out the scope of this work.

As mentioned, Lynn et al. (1962) pioneered the studies on cost-effective wastewater treatment. In their study a network linear programming model was formulated to represent the BOD removal in a treatment plant that consists only of liquid waste treatment. The model was solved for the combination of unit processes that would remove a given amount of BOD at the least treatment cost.

After this early inception, in the next twenty years, several studies have been devoted to this field. Among such, the work of Evenson et al. (1969), Shih and Krishnan (1969), Naito et al. (1969), Berthouex and Polkowski (1970), Schulter and Loehr (1971), Middleton and Lawrence (1976), Adams and Panagiotakopoulos (1977), Rossman (1980), Suidan et al. (1983), Tang et al. (1984), and Tyteca (1985) can be noticed. Many other valuable studies also exist but are not mentioned here. Most of these studies have considered treatment systems that involve a primary settler, biological treatment unit mainly an aerator, and a final settler. Few consider only the activated sludge system, i.e. aerator and final settler, while others consider trickling filters and other unit operations. Moreover, only a few studies have considered the operation costs of plant beside the capital costs although an optimal design in terms of capital cost is most likely not optimal when operational costs are considered.

The aforementioned studies have been developed for different purposes and utilized various optimization algorithms. Tyteca (1985) has listed and compared all research efforts spent in this field up to 1982 including the aforementioned studies and others. He differentiated clearly between two research trends. The first trend, which he called the "biological – mathematician" trend, is characterized by an attempt to describe the operation of the plant using accurate mathematical models. The optimization phase is performed by simplistic techniques, which in some cases are even replaced by simple enumeration. The second trend, the "Economist – Operation Researcher" trend, is opposite where the main effort is on developing and demonstrating the use of powerful and/or sophisticated optimization techniques, while the operation of the plant is modeled through oversimplified relationships.

This distinction remains true for only the early studies. Later on (middle of eighties and beginning of nineties), the gap between these two trends was progressively attenuated, by putting more accent on the necessary adjustment between an efficient optimization technique and a sufficiently realistic mathematical

description of the plant. Two key studies bridging the gap between the two trends are mentioned herein.

Suidan et al. (1983) developed a comprehensive wastewater treatment plant mathematical model. The model includes primary sedimentation followed by a single-stage activated sludge system for both BOD reduction and nitrification. Primary and waste activated sludges are assumed to be mixed, chemically-conditioned prior to vacuum filtration and cake incineration. Capital, materials, operating and maintenance costs are utilized in arriving at a least cost system design. The sensitivity of the optimum design to the values of various model constants and effluent constraints is evaluated to establish which variables most significantly affecting the system design.

Tang et al. (1987a) presented a comprehensive model of a typical activated sludge wastewater system operated in steady state. It includes both the liquid and solids portions of the treatment system and recycle stream. The biokinetics was modeled according to Lawrence and McCarty (1970) model for activated sludge. They used this comprehensive model for analysis and optimization. The same authors in another publication in the same year (Tang et al., 1987b) developed an optimization technique to optimize the same comprehensive model they presented. They introduced a decomposition approach based on dividing the comprehensive system into a liquid subsystem and a sludge subsystem, optimize each and integrate them together to obtain the comprehensive system optimized. This approach reduced the elapsed time and efforts usually associated with complex integrated optimization models.

During the last decade several studies treated the problem of wastewater systems optimization in a way and another. The following is a thorough review of such studies.

Fujiwara (1990) introduced a preliminary approach for optimal design of a wastewater treatment plant. For a given influent characteristics and effluent requirements one can find a least cost combination of unit processes and a treatment efficiency of each process. The author stated that more elaborate mathematical models should be used to reach a final decision with regard to the optimal arrangement of the plant.

Akca et al. (1993) presented an activated sludge optimization model that addresses primary settling, aeration, and secondary settling. The model was evaluated

with varying dissolved oxygen levels and relationships were constructed relating sludge age and the solids flux process to sludge volume index. The model was solved by using the Box-Complex Algorithm. With the addition of a pre-aeration and a pre-precipitation stage to a conventional activated sludge process, process stability was observed to increase (Echeverria et al., 1993).

A simple and efficient heuristic screening methodology was presented by Voutchkov and Boulos (1993) for use in regional wastewater treatment system planning. This tool could aid in the decision-making process regarding optimal locations of regional wastewater collection and treatment facilities. The criterion for eliminating non-optimal plant sites is distance as derived from transportation cost and regionalization efficiency. The method significantly reduces the number of candidate locations of shared facilities for regionalized treatment. The final solution is selected using conventional mathematical programming procedures.

Optimization of regional wastewater system may be generally formulated to define the transport and treatment in a region or water basin so as to assure compliance with given pollution control criteria at minimum cost. According to de Melo and Camara (1994), the two main problems that make the optimization of this solution difficult are the dimensionality and the concavity of cost functions. They discussed solution strategies in three major areas: definition of the objective function and constraints, optimization method, and practical applicability of the models.

Tench (1994) reported on an equation linking the activity of a sludge with its concentration. It was shown that the fraction of active mass decreased as sludge concentration increased. This approach was extended to develop equations that define an optimum sludge concentration as a function of the square root of the load to the plant, thereby giving desired effluent quality with minimum power consumption. The effects of changing active biomass, dissolved oxygen, and loading were studied, and evidence was presented to show that, as predicted by a second-order reaction rate hypothesis, the oxygen concentration necessary to maintain effluent quality increased as the aeration period decreased.

Columbo and Nelson (1994) suggested that strategies and process management techniques used in manufacturing can be applied at treatment plants to improve bio-solids quality and operational cost-effectiveness. Strategies were

reviewed with regard to optimizing or enhancing feedstock and product quality, production, monitoring, energy and chemical usage, effective maintenance, and operational flexibility.

An activated sludge secondary clarifier inlet design for improving settling efficiency was based on two-dimensional numerical modeling (Krebs et al., 1995).

Pincince et al. (1995) presented analytical relationships and sensitivity analysis diagrams to aid in the selection of minimum-cost steady-state activated sludge designs.

Kurata et al. (1996) used ASM2 to simulate and optimize a full-scale anaerobic/aerobic biological phosphorus removal process with low influent carbon and found that injection of primary sludge and shortening of the anaerobic mean sludge retention time would improve total-P removal efficiency to 90%. Full-scale implementation resulted in 95% P removal. Bischof et al. (1996) presented design guidelines that facilitated the minimization of activated sludge plant (ASP) aeration operational cost.

Another type of research studies investigate and verify the models used in simulating various processing units using pilot-scale and full scale measurements. Some of these studies are presented below.

Pincince et al. (1997) argued that activated sludge plant-aerated biomass affected the MLSS concentration that minimized aeration and clarification concentration cost, those plants with higher biomass should have higher design MLSS, and plants anticipating poor settling should have lower design MLSS. Construction cost sensitivity increased with increasing mean solids retention time (MSRT) and sludge volume index (SVI) values.

Ueberl and Hager (1997) presented a series of ASP rectangular secondary clarifier design recommendations based on full-scale testing results. Rensink and Rulkens (1997) reported that sludge production in a pilot-scale ASP treating settled domestic wastewater was reduced from 0.40 to 0.15 g MLSS/g COD removed when metazoa were added to the system. Novak (1997) demonstrated by inhibiting nitrification in bench-scale ASP that increased ammonium ion concentration results in deteriorating solids settling and dewatering characteristics.

Potter et al. (1998) used ASM1 simulations to compare a partial nitrification/complete denitrification (PN/CD) process to the Lutzack-Ettinger process. They concluded that the PN/CD process required 26 to 44% lower total reactor volume and consumed 15 to 18% less energy, mainly due to reduced aeration requirement. Hermanowicz (1998) presented activated-sludge clarification diagrams based on the solids flux theory that related the MLSS and RAS concentrations with the recycle ratio and hydraulic loading rate. Yuan et al. (1998) used steady-state analysis and dynamic simulation to demonstrate that including an online settled sludge storage tank in a nitrifying ASP, from where sludge can be occasionally returned to the aeration tank to counter nitrogen shock loads will typically reduce tankage by 20% while maintaining the same nitrification capacity.

Petrides et al. (1998) have discussed the role of process simulation in designing, evaluating, and optimizing wastewater treatment facilities. They have utilized the commercial program EnviroPro Designer to track the fate of VOCs and other chemicals.

Ayesa et al. (1998) have presented a new optimization algorithm for the selection of design and operation parameters in a complex activated sludge process that is the Alpha process. The algorithm estimates automatically the dimensions and operating point of the plant that minimize a global penalty function combining effluent requirements and costs. They have illustrated some examples concerning the design and operation of the Alpha process. Results obtained generate useful guidelines for the design and operation and suggest a great potential in the application of optimization models.

Chachuat et al. (2001) studied the dynamic optimization of small size wastewater treatment plants. An optimal aeration policy which minimizes the energy consumption and satisfies effluent and technical constraints was found. The model considered consists of small single plug flow aeration tank with mechanical aerators and rectangular settler.

Scuras et al. (2001) have presented a procedure to determine the optimum reactor configuration for a range of influent and effluent substrate concentrations, half saturation coefficients, and number of tanks in series for both inhibitory and non-inhibitory substrates. They have developed dimensionless plots that show the

minimum biomass requirement of the series relative to that for a single CSTR and the optimal relative sizes of the tanks. The plots may be used directly for staged system design. They concluded that three tanks in series is generally best, high influent substrate concentrations and stringent discharge requirements increase the benefit of staging, and optimal tank sizing is significantly better than using equal sized tanks.

Recently, Rivas et al. (2001) presented a mathematical formulation for the optimum design of a new activated sludge wastewater treatment plant. The optimum design problem has been formulated as a mathematical programming problem, which is solved through a nonlinear optimization method. The plant model has been based on ASM1. The minimum volume of the biological reactors and the minimum total cost (including construction and operation costs) has been considered as optimization criteria. Some practical results are also included, using as a case study the design of the second stage of the Galindo-Bilbao wastewater treatment plant.

Doby et al. (2002) have described a framework in which a genetic algorithm and a static activated sludge treatment plant design model (WRC model) are used to identify low-cost activated sludge designs that meet specified effluent limits (e.g., for BOD, N, and P). The performance of genetic algorithms has compared to that of classical non-linear optimization approach. The results suggest that the approach is computationally practical for use in activated sludge system design, and that it outperforms a classical nonlinear programming routine, both with respect to solution quality and robustness of the search process. However, ability of such framework to accommodate advance dynamic models such as ASM models is still under question.

As shown in the literature review, the state of the art has evolved considerably over the last twenty years in the application of optimization concepts to wastewater treatment systems analysis and design. There are still, however, areas where additional improvements can be made (e.g., uncertainty based optimal design). The following summary of guidelines is from the aforementioned discussion of literature. These guidelines serve as the initiative for the development of the optimization model described in this thesis.

1- Exploring the aforementioned literature shows that several studies are of the same type of this thesis (optimization of activated sludge process). Some are as old as 1983 (Suidan et al., 1983) and some are very recent as Rivas et al.

(2001). However, these studies differ significantly in many aspects. One main issue is the mathematical performance models utilized to describe the behavior of system units. Old studies utilized simple approximate models which are now dominated by advanced detailed models reflecting better understanding of the activated sludge process. Although some recent studies consider these advanced models. They include many particularities which make them suitable for a situation and not another. Moreover, performance models of units accompanied to bioreactor usually in activated sludge processes (primary clarifier and secondary clarifier) are considered with less concern. In some studies, performance of such is approximated with rough assumptions.

2- Two obvious directions are clear in literature when dealing with wastewater systems optimization. Early studies include comprehensive systems while recent ones tend towards more specialties and ignore comprehensive systems. Comprehensive systems, even though desired, usually have rough assumptions that reduce the value of the optimum solution obtained, while more specific models are more detailed and more practical. For example, studies like Suidan et al. (1983) and Tang et al. (1987) considered with many assumptions a comprehensive system while the recent study of Rivas et al. (2001) considered one activated sludge system.

3- Costs are of major concern in optimization studies. However, such costs are a major source of uncertainties. An agreement between most of the studies that construction, operation and maintenance costs should be included is clear. This is reasonable since a construction cost-effective design will not necessarily be an operation cost-effective design.

4- Efficient optimization techniques are important issues in optimization problems. Many optimization methods used previously can be applied only to a special and limited process scheme or only when simplifying assumptions about process design are made. Moreover, old studies have been considering low robust optimization algorithms that are highly affected by initial and boundary conditions. The use of an efficient state of the art technique is a crucial option in producing robust optimal designs.

A major contribution of this thesis is to make additional progress toward developing an efficient optimization method for use in sizing an activated sludge treatment system. Attempts are made to incorporate recent mathematical performance models with an efficient optimization technique considering reasonable practical assumptions and constraints.

A treatment plant optimization model has been perceived by a number of researchers as a mean to obtain the least cost system design. This role of an optimization model is suggested by the very nature of the optimization concept. The planning and design of wastewater treatment systems, however, is a complex problem. Many important issues such as energy requirements and real efficiency of mechanical units may not be captured in a cost optimization model. As a result, the optimal design obtained from solving such a model may only be meaningful mathematically. However, with engineering intuition, it can become a valuable design tool. Another view suggests that the most appropriate role of this type of optimization models is as a decision-making aid. This role is more appropriate because of the importance issues and the uncertainties associated with planning a wastewater treatment system. The other major role is the use of this type of models for the analysis of process performance. An optimization model can lead to the examination of the validity of process models form the cost-effective point of view. Useful insights about process performance, integration, or limitations are gained as valuable by-products from exercising an optimization model. Examining such issues beside other design issues form the main objective of this thesis.

It might be noticed that the literature review presented above discussed only the research efforts related to applying optimization techniques in designing the activated sludge system or the entire biological wastewater treatment plants. Other literature will be cited with respect to each individual modeling component presented in following chapters. Research related to modeling of primary clarifiers, secondary clarifiers, and activated sludge processes is covered in Chapters 2, 3, and 4. Studies addressing the cost functions that can be used in optimal mathematical formulations are elaborated on in Chapter 5.

1.2   Research Objectives

The overall objective of this thesis is to implement the recent advanced models of various unit processes in activated sludge wastewater treatment plant along with cost information functions in a comprehensive optimization framework that can be used to investigate the problem of sizing various units optimally. Such optimization framework is supposed to serve new and existing treatment plants by incorporating operational conditions along with other design parameters. It is also supposed to take into consideration many sources of uncertainty that affect the final sought optimal design. Consequently, this thesis seeks the development of a rational optimization model for sizing various units in activated sludge wastewater treatment plants. This overall objective comprises several sub-objectives include:

1- Introduce latest advanced modeling techniques of the activated sludge process and their use in design and analysis.

2- Introduce a method of building a comprehensive overall mathematical model of activated sludge wastewater treatment plant by incorporating various performance models and mass balance concepts.

3- Discuss the method of formulating an optimization problem in terms of governing equations, constraints, bounds and objective function.

4- Evaluate the introduced optimization model for various operating conditions to build a better understanding of activated sludge process behavior.

Such objectives are sought for the purpose of introducing a rational tool for designers to help them in taking correct decisions when designing or upgrading activated sludge wastewater treatment plants. The specific steps taken to achieve the mentioned objectives are discussed in the following section.

1.3   Methodology

It is well known that activated sludge systems involve two processes: biological treatment of organics in an aeration tank, and separation of solids in a sedimentation tank. Whether to consider optimizing the system alone or incorporating it in a comprehensive treatment system is a controversial research issue (as mentioned earlier). Every research direction has its supporters. Researchers defend the first

direction believe that as smaller the optimization problem, more details can be incorporated and less assumptions and uncertainties are involved. In addition, optimized individual system units can be coupled to other optimized system units to produce a comprehensive optimized system by means of mathematical formulation. Such has the advantage of reducing the problem mathematically which results in a more robust solution. On the other hand, supporters of the second direction claim that without the interaction between various unit processes incorporated in one comprehensive model, no one can declare an optimum design of individual unit process.

In fact, no one can deny that the two directions have their own advantages and disadvantages. However, it has been noticed as obvious in the aforementioned literature review, that recently researchers tend to prefer the first approach. Optimization of individual unit processes for various design purposes (not only economical purposes) is obvious in recent research efforts. This can be attributed to two main reasons; focusing on a unit process allows researchers to incorporate more detailed design variables as well as reduces approximation assumptions and solution uncertainty. In addition, several researchers have pointed out the possibility of combining optimized individual unit processes to produce an optimal comprehensive system (e.g. Tang et al., 1984). Those researches claimed several advantages of this approach over the approach of optimizing the comprehensive system model in one shot. As an example, Tang et al. (1984) have introduced a decomposition approach that divides the wastewater treatment system into liquid treatment train and sludge treatment train. The liquid train is optimized first then by means of mass balance is coupled to the sludge train to produce an overall comprehensive optimal system. Such approach compared to a comprehensive approach yielded same results but with less computational effort and more robust solution.

Moreover, Tang et al. (1984) have proved that the effect of sludge train optimization on the liquid train can be considered minor. They showed that the cost of the liquid subsystem is not very sensitive to the recycle flowrates from the sludge subsystem, and it is the design of the sludge subsystem that determines the most cost-effective overall system design. Moreover, the design of the liquid subsystem when optimized as a part of the comprehensive subsystem was very similar to the design when optimized alone in the decomposition approach.

As a result, this thesis considers optimizing the activated sludge system (aeration tank and secondary settler) without coupling it to other unit processes that usually appear in treatment plants except for the primary clarifier. This is because an activated sludge treatment system without a primary clarifier is rarely found. Moreover, its performance affects directly the performance of activated sludge system. Hence it is meant by the system in this study a treatment system comprises primary clarifier, aeration tank, and secondary settling tank.

Any cost-effective optimization study involves three main parts; mathematical performance models, cost information models, and an optimization technique (see Figure 1.1). Hence the main tasks taken to achieve the objectives mentioned in the previous section and fulfill the main three portion of an optimization model are as follows:

1- Evaluate current unit process performance models to determine their suitability for use in a system model and design procedure, and construct a conventional activated sludge wastewater treatment system model that can be used to describe the performance of the system with given influent and effluent conditions.
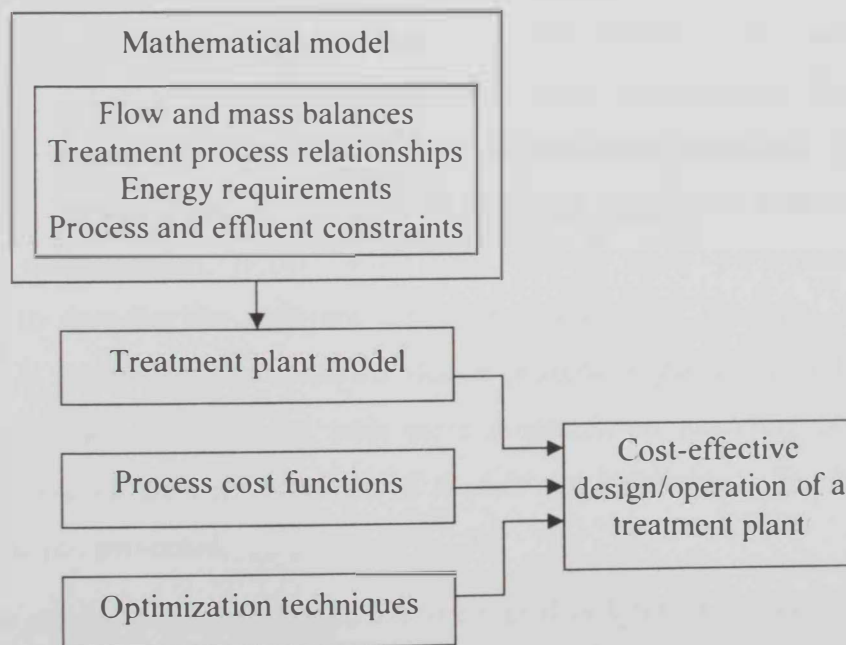
Figure 1.1: Main portions of any cost-effective wastewater treatment study (Tyteca et al., 1985)

2- Evaluate available cost information models that relate the cost of various components of activated sludge plant to design and operation conditions. This includes capital and operational/maintenance costs.

3- Combine the developed system model in (1) and the cost information equations chosen in (2) in one comprehensive optimization problem with the total system cost as the objective function.

4- Utilize well known optimization software that is General Algebraic Modeling Systems software (GAMS) to solve the optimization problem developed in (3).

5- Illustrate the use of the developed optimization model as a tool for optimum sizing of various activated sludge system units. The developed model is also used as a tool for the analysis of performance, integration, limitation of unit processes considered in the study. Analyze several variations of the base treatment system to verify insights obtained from the design optimization of the base system.

1.4   Thesis Outline

An optimization activated sludge model development is the main objective of this thesis. As shown in Figure 1.1, any cost-effective optimization model development involves three main portions. Chapters of this thesis cover these portions systematically. Starting with the treatment plant model development, Chapters 2, 3, and 4 are devoted to present the modeling of the three main operations in the system considered. Chapters 2 and 4 introduce the modeling of primary sedimentation and secondary sedimentation, respectively. They discuss many performance models developed to describe the different aspects of these units behavior. In contrast, Chapter 3 is devoted to the activated sludge process in details. In this chapter a historical overview is presented with more emphasis on modeling issues of the process. Simplified as well as advanced models are highlighted. The basic design methods are also presented.

The second portion of Figure 1.1 is covered in Chapter 5. Cost functions are discussed in this chapter comprehensively. The third part is presented in Chapter 6 along with a full formulation of the model combining the models presented in Chapters 2, 3, 4, and 5. The model formulation include in details all the models,

variables, and parameters incorporated in the model with an optimization problem statement listing the objective function along with applied constraints. At the end of Chapter 6, the optimization software used to solve the formulated optimization model given a prescribed data set is introduced with emphasis on the available solving algorithm.

This completes the three preliminary tasks required to develop an optimization model to produce a cost effective design of activated sludge process. The use of this model is illustrated through the solving of an illustrative problem in Chapter 7. In Chapter 8, the performance of the developed model for various design scenarios is investigated. Several design insights can be drawn from the investigations in this chapter. Chapter 9 highlights the uncertainty involved in the system model. A comprehensive review of dealing with uncertainty in such type of models is presented there. A simple uncertainty study is presented along with sensitivity analysis of various parameters. The last chapter, Chapter 10, is devoted to the discussion of conclusions and future research.

CHAPTER 2

PRIMARY SEDIMENTATION

The primary clarifier is the first main unit operation in most wastewater treatment systems. In general, sedimentation is the separation from water, by gravitational settling, of suspended particles that are heavier than water. It is used for grit removal, particulate matter removal in the primary sedimentation basin, biological-floc removal in the activated sludge settling basin (secondary settling basin), and chemical-floc removal when the chemical coagulation process is used. It is also used for solids concentration in sludge thickeners.

Primary sedimentation tanks in the activated sludge process are used before aeration tanks with the main objective to remove particulate matter from the influent raw sewage. These tanks are designed to reduce the velocity of the wastewater flow for solids (called raw sewage) to settle. The purpose is to produce a clarified effluent, but it is also necessary to produce sludge with a solids concentration that can be easily handled and treated. In designing primary clarifiers, considerations must be given to both functions, the production of both clarified effluent (clarification) and concentrated sludge (thickening) (Metcalf and Eddy, 1991).

Primary sedimentation is an important part of a treatment plant where primary settling tanks in the activated sludge process are used to reduce the load on the subsequent biological treatment units. Efficiently designed and operated primary sedimentation tanks should remove from 50 to 70 percent of the suspended solids and from 25 to 40 percent of the BOD_5 (Metcalf and Eddy, 1991). Primary sedimentation has many advantages, including: (1) minimizing operational problems in subsequent biological treatment processes; (2) lowering the demand for oxygen, resulting in a reduction of the rate of energy consumption in the oxidation of particulate matter; (3) promoting a high rate of soluble substrate removal during aeration; and (4) reducing the volume of waste activated sludge (Lessard and Beck, 1988). All these advantages and others make the primary sedimentation a crucial unit operation in any treatment

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plant where its efficiency is directly linked to the efficient operation of the secondary treatment units and to the performance of sludge treatment. It has been shown that operating a treatment plant without primary sedimentation units might result in a total treatment cost increase of about 30 percent (Suidan et al, 1983).

Indeed, the understanding of the primary clarifiers operation is important to the overall effectiveness of the treatment plant. Several researchers have recognized this fact and long efforts have been spent in developing models describing the behavior of primary clarifiers. Modeling the behavior of primary clarifier involves modeling the main two functions it fulfills, namely, clarification and thickening.

2.1 Modeling of Primary Sedimentation

Broadly, there are three types of models that have been proposed for the description of primary clarifier behavior (Lessard and Beck, 1988):

1- Simple, steady-state relationships derived from the statistical analysis of field data, quantifying removal efficiencies, and used largely for the purpose of design. These models, known widely as empirical models, usually correlate removal efficiency with the overflow rate and/or the influent SS concentration, and have found some applications in simulation studies.

2- Relatively simple, lumped-parameter, dynamic models, expressing component mass balances across a constant-volume, continuously stirred tank reactor (CSTR) approximation of the clarifier.

3- More complex, distributed-parameter models, based on the convectional assumptions for advection and dispersion of materials passing through the clarifier. These models have also been used mostly for design purposes, with special reference to the description of the fluid velocity field and solids concentration within the clarifier.

Many models of the three types have been proposed to describe the behavior of primary clarifiers which is often considered as being not "very sensitive", resulting in the use of simplified models (i.e., type one) to represent its behavior (Gernaey et al., 2001). Furthermore, its performance has usually being assessed from a steady state point of view (Lessard and Beck, 1988). Consequently, models of the first type

will be considered in this research as the other two types are more tailored to dynamic analysis and online control operation.

The overflow rate and influent suspended solids concentration have been identified as two important parameters that affect the performance of primary sedimentation tanks and appear in almost all types of primary clarifiers' performance models. The overflow rate can be defined as follows;

$$q = Q/A_p \tag{2.1}$$

Where q is the overflow rate, Q is the effluent flow rate, and A_p is the surface area of the clarifier. The temperature also was found to be an important factor that affects settling velocities as well as the velocity gradients in the liquid that in turn affect flocculation (Christoulas et al., 1998).

2.1.1 Clarification Behavior Modeling

Considering the clarification function of the primary clarifier, several theoretical and empirical models have been proposed over the last two decades. The theoretical mathematical models, though helpful to the understanding of the sedimentation process, are still far from being reliable and effective design tools. They have failed until now to predict the behavior of sedimentation tanks under actual operating conditions due to the difficulties in simulating the effect of the density currents and the complex phenomenon of flocculation (Christoulas et al, 1998). In the absence of more valid theoretical models, empirical models, sometimes called "regression models", are more suitable for the design of primary sedimentation tanks, either directly or after calibration with pilot plant performance data. They are developed by gathering sets of experimental data and identifying the linear relationships between process variables by regression analysis. Then, empirical constants can be determined by dimensional analysis.

Empirical models have generally been restricted to relate the suspended solids removal efficiency to overflow rate and influent suspended solids. An excellent review of the historical development of such models for the clarification function of primary clarifiers is given in Tang et al. (1984) and in Christoulas et al. (1998), a short summary of which is presented below.

In 1954, Fair and Geyer proposed a graph relating the removal efficiency to retention time and influent suspended solids concentration. A similar diagram was proposed by Steel (1960). Smith (1968) proposed that solids removal efficiency is a function of the surface overflow rate. He developed a model using data from the WPCF Manual of Practice (1959). Voshel and Sak (1968) developed two models relating the solids removal efficiency to both the influent solids concentration and the overflow rate based on their plant-scale study performed in Michigan. Berthouex and Polkowski (1970) developed a linear model with respect to the overflow rate based on the same data of Smith (1968). This model is mathematically simple, but it is not an adequate representation of the observed data.

Based on data from a number of full-scale plants, Escritt (1972) derived a relationship that relates the effluent suspended solids to influent suspended solids and retention time. The CIRIA (1973) model used retention time instead of overflow rate to represent the hydraulic features of the settling tank. This model was developed from the analysis of data obtained from a number of large sewage works in London area.

Tebbutt and Christoulas (1975) proposed another empirical relation relates the removal efficiency to overflow rate and influent suspended solids. They developed that relation based on data from a pilot-plant of the university of Birmingham, U.K. and it was shown to describe plant operating data adequately with correlation coefficient r=0.94. Later on, Tebbutt (1979) observed that the same model provided a good description (r=0.95) of the full-scale data of White and Allos (1976). Dick et al. (1978) fitted the WPCF (1959) data to a model of the form proposed by Tebbutt and Christoulas. Annesnni et al. (1979) proposed a regression model based on published performance data from various pilot and full-scale primary sedimentation tanks. Anderson and Mun (1981) studied the performance data of several primary settling tanks and concluded that the suspended solids removal was proportional to the concentration of the so-called "settleable" solids.

Christoulas et al. (1985) carried out a pilot-plant study of the treatability of Athens sewage. Regression analysis of the data obtained gave a relationship with r=0.98. One of the conclusions is that temperature and suspended solids concentration are possibly the only sewage parameters that can cause significant variations in the average performance of sedimentation tanks treating municipal wastewater. In 1998,

Christoulas et al. developed a model based on the model presented by Tebbutt and Christoulas (1975). They studied the effect of temperature on settleability, along with the other typical sewage characteristics. A new relation that comprises the effect of temperature was introduced. It has been shown that all the correlation coefficients are significant at probability levels higher than 99%. Table 2.1 summarizes the aforementioned models.

Tang et al. (1987a) utilized the model of Voshel and Sak (1968) to model the suspended solids removal of the primary clarifier in a comprehensive model of activated sludge wastewater system. The same model has been introduced by Tyteca (1985) to predict the suspended solids removal as a measure for the primary clarifier performance.

In the present study, the model of Christoulas et al. (1998) has been chosen to describe the clarification process in the primary clarifier since it is recent in addition to its capability of showing good performance in fitting the observed data.

Table 2.1: Models for suspended solids removal (clarification) in primary settling tanks

Authors	Models	Source of Data	Remarks
Smith (1968)	$E_s = 0.82\exp(-.2112q)$	WPCF (1959)	$0.42<q<3.75$ m hr^{-1}
Voshel and Sak (1968)	$\dfrac{S_e}{S_i} = 1 - 0.139\dfrac{S_i^{0.27}}{q^{0.22}}$	Voshel and Sak (1968)	$70<S_i<160$ mg L^{-1} $1.71<q<1.88$ m hr^{-1}
Berthouex & Polkowski (1970)	$E_s = 0.82 - 0.142q$	WPCF (1959)	$0.42<q<3.75$ m hr^{-1}
Escritt (1972)	$S_e = \dfrac{S_i}{C_1\theta^n \log S_i}, n = 1/C_2 \log S_i$	Escritt (1972)	$C_1=1.1, C_2=10$
CIRIA (1973)	$E_s = [0.00043S_i + 0.51][1 - \exp(-0.7\theta)]$	CIRIA (1973)	$6<q<33$ m d^{-1}
Tebbutt & Christoulas (1975)	$E_s = 1.12\exp(-358/S_i - 0.0020q)$	Tebbutt & Christoulas (1975)	$S_i>200$ mg L^{-1} $25<q<150$ m d^{-1}
Dick et al. (1978)	$E_s = 0.84\exp(-40/S_i - 0.177q)$	WPCF (1959)	$0.42<q<3.75$ m hr^{-1} $S_i=230$ mg L^{-1}
Annesnni et al (1979)	$\dfrac{S_e}{S_i} = 0.4q^{-009}S_i^{-0.042}(D/H)^{-0.26}$	Annesnni et al (1979)	
Christoulas et al. (1985)	$E_s = B\exp(-352/S_i),$ $B = a\exp(-cq) = 1.141$	Christoulas et al. (1985)	
Christoulas et al. (1998)	$E_s = a\exp(-b/S_i - cq)$ $b = 683.6 - 21.13T$ $a = 1.71 - 0.03T, c = 0.0035m^{-1}d$	Christoulas et al. (1998)	Effect of temperature considered

Note: E_s: solids removal efficiency, q: overflow rate (m d^{-1}) or as mentioned, S_i: influent solids concentration (mg L^{-1}), S_e: effluent solids concentration (mg L^{-1}), θ: retention time (h), D: tank diameter (m), T: Temperature

It is worth-mentioning that most of the models indicate that the solids removal efficiency increases with decreasing overflow rate and with increasing influent solids concentration. Parameters in the models represent the degree of dependence of the solids removal on influent solids concentration and overflow rate. These parameters are related to the characteristics of the influent to the primary settling tank. Therefore, they are to be determined for the wastewater treatment plant under consideration considering the characteristics of the influent and the ambient environmental conditions (e.g. temperature). In this research, ideal (typical) parameters values have been used for generality purposes.

2.1.2   Thickening Behavior Modeling

Primary sludge concentration (i.e. thickening behavior) has been modeled by two approaches (Tang et al., 1984). The first approach assumes that this concentration is controlled by the hydraulic limitations of the sludge withdrawal mechanisms. As a result, a constant concentration is assigned to the primary sludge. The second approach uses the deferential thickening technique which is based on the limiting flux theory to calculate the primary sludge concentration. The limiting flux theory depends on modeling the settling velocity of which several models have been proposed. Explanation of such theory and the settling velocity models is given in Chapter 4.

Although most of the researchers have used the solids flux theory to model the thickening function of secondary settling tanks, some have also used it to explain the same process in primary settling tanks (e.g., Tang et al., 1987a). They proposed that the primary sludge concentration can be calculated as:

$$X_u = [k(n-1)]^{1/n} \left(\frac{n}{n-1} \right) \left(\frac{A}{Q_u} \right)^{1/n} \tag{2.2}$$

in which k and n are settling constants of the primary sludge. Q_u is the underflow rate (m^3/h) and A is the surface area of the clarifier (m^2).

Cho et al. (1996) showed the derivation of Equation (2.2) from the solids flux theory considering the power model to describe the settling velocity as a function of solids concentration. They used the same equation to describe the thickening function of the secondary settler, but with different secondary sludge settling constants ($n = 2.3$ and $k = 375$ m/d when the velocity is represented in m/d and the concentration in g/L).

This model is widely accepted and is used herein to describe the thickening function of the primary clarifier. Settling constants are the most important part when using this type of models. They should be determined for the sludge under consideration specifically. Cho et al. (1996) mentioned that in a measurement conducted in a wastewater treatment plant, k values were between 65 and 460 m/d while the index n was between 1 and 5.

There exist also some attempts to model the two functions, clarification and thickening, together in one model. Lessard and Beck (1988) developed a relatively simple, lumped parameter model of primary sedimentation dynamics. The model quantifies successfully the interactions among essentially five state variables: total suspended solids, volatile suspended solids (VSS), total chemical oxygen demand (COD), soluble chemical oxygen demand, and ammonium-N concentrations. This model is a dynamic model and can be better utilized in online operation and control. Gernaey et al. (2001) utilized the solids flux theory and the Takacs et al. (1991) model of settling velocity to develop a reactive primary clarifier model that can be used in a wastewater treatment plant simulator (WEST). The model simulates, in addition to clarification and thickening, COD behavior. The model was tested with full scale data where particulate COD was well described. However, problems occurred in predicting the underflow suspended solids concentration (Gernaey et al., 2001).

Although, most of the primary clarifier models do not consider any biological reactions and simulating only the suspended solids behavior, there exist in the literature some empirical models predicting the removal of organic matter. Tang et al. (1984) list samples of such models. A common feature of these models is the lack of fit of the data to the proposed model, generally with R^2 less than 0.6 (Tang et al., 1984). As a result, Tang et al. (1987a) have not recommended the use of such models and instead assumed that the species distribution of the suspended solids in the primary effluent is the same as in the primary influent. The soluble organic matter was assumed to be unaffected by primary sedimentation.

In contrast, Christoulas et al. (1998) developed a regression relationship relating the removal efficiency of suspended solids and COD removal efficiency. They proposed that the following relation provides a good estimate.

$$E_c = 0.733E_s - 0.08 \hspace{5cm} (2.3)$$

in which E_c is the COD removal efficiency and E_s is the suspended solids removal efficiency.

In this study, no certain model is considered to predict the removal of organic matter. Instead, the species distribution of the suspended solids in the primary effluent is assumed to be the same as in the primary influent.

In summary, the Christoulas et al. (1998) model (Table 2.1) is utilized to simulate the clarification process and the Cho et al. (1996) model (Equation 2.2) to simulate the thickening function of the primary clarifier. The next chapter describes the activated sludge process in more detail. Modeling and design are highlighted.

CHAPTER 3

ACTIVATED SLUDGE PROCESS

Activated sludge processes are among the most widespread biological wastewater treatment techniques. It is an aerobic suspended growth process in which microorganisms are grown in a variety of bioreactor configurations. Like all biological treatment processes for wastewater, the major objective is the removal of pollutants. Besides removal of organic carbon substances, an activated sludge wastewater treatment plant can achieve biological nitrogen removal and biological phosphorus removal. It is a flexible, reliable process capable of producing a high quality effluent.

The basic idea of the process is that a mass of "activated sludge" is kept in suspension by stirring or aeration. Operationally, biological waste treatment with the activated sludge process is typically accomplished using two distinct operations usually performed in two separate basins: aeration and settling. A typical flow diagram of the process is shown in Figure 3.1. Organic waste is introduced into a reactor where aerobic bacterial culture is maintained in suspension. The suspension contains not only living biomass, but also inorganic and organic particles. The reactor contents are referred to as the "mixed liquor suspended solids" (MLSS). In the reactor, the biomass metabolizes the easily biodegradable organic material (substrate) in the presence of oxygen and nutrients, that is, the organic material will be removed from the wastewater while more biomass and other end products are produced. This is often called bacterial growth and it is associated with substrate utilization. At the same time, decay of biomass exists because of endogenous respiration to produce inert material. Some of the organic particles in the suspension are slowly biodegradable and may be broken down into simpler components (easily biodegradable) by a process known as hydrolysis, while other organic particles are not affected (inert material). As the case in all biological wastewater treatment processes, three main biological processes exist in the activated sludge process: growth,

hydrolysis, and decay. New models include other processes, for example, storage of internal products in microorganisms (Henze et al., 2002). Modeling of such biological processes forms the base of activated sludge process modeling and is going to be discussed hereafter.

The aerobic environment required for the process is achieved by means of diffused or mechanical aeration, which also serves to maintain the activated sludge in suspension. After a specified period of time, the mixed liquor passed into the second reactor in the process: the secondary settling tank. In this tank, solids including biomass and inorganic particles are separated from the treated wastewater (secondary settling is discussed in Chapter 4). A portion of the settled biomass (activated sludge) is recycled to the aeration reactor to maintain the desired concentration of organisms in the reactor, and the other portion is wasted.
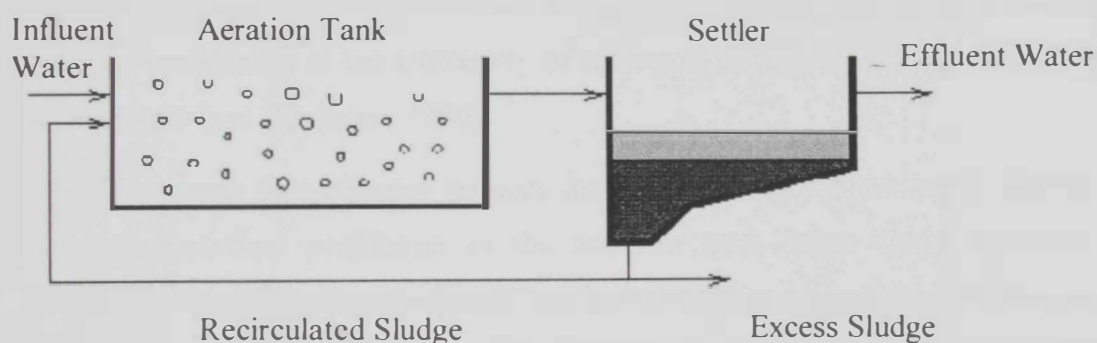
Figure 3.1: A typical activated sludge process

The above description introduces the activated sludge process in its basic form. However, what really happens in an operating activated sludge plant is far more complicated. Furthermore, since its inception in 1914, the process has gone through several modifications and variations to improve its performance and widen its application. Such variations are briefly discussed hereafter after a short historical overview.

3.1 Historical Overview:

The concept of using aeration, as a mean of sewage purification, dates back to the beginning of the last century. Before the inception of the activated sludge process, the

fill-and-draw approach was the common treatment technique. That is, a wastewater was put into a reactor and aerated, after a period of time the wastewater was released, the deposit of solids was removed and the process was repeated.

In 1914, Arden and Lockett in England (Metcalf and Eddy, 1991) studied the effect of saving the flocculent solids and using them repeatedly. They proved an increase in the purification capacity of simple aeration. The first uses were on a batch basis. At the end of each aeration period, suspended solids (referred to as sludge) were present and they were left in the bioreactor when the clear wastewater was withdrawn after settling. As this batch procedure was repeated the quantity of suspended solids increased, giving more complete removal of organic matter within the allocated time. Although this increase in suspended solids with the associated improvement in removal activity was due to the growth of a viable microbial culture, the reason was unknown to the early researchers, who characterized the sludge as being "activated", thereby giving the process its name (Grady et al., 1999). During the same year, similar studies were undertaken at the University of Illinois in the United States, leading to the same conclusions (Jeppsson, 1996).

Efforts were then directed towards the adaptation of the process to operate under continuous-flow conditions as the need to treat larger flows increased. Continuous flow reactors were designed shortly thereafter and regularly used because of problems in controlling a number of batch reactors throughout fill-react-settle-draw cycles with variable influent flow rates. By 1917, two small-scale continuous-flow plants in England and a larger plant in Houston, Texas, were put into operation. Successful experience with these plants and the establishment of the diffused air process as a feasible means of air provision encouraged the construction of other major plants that were soon placed in operation. All were based on the continuous-flow principle, which had proven itself as the major practical method for activated sludge operation. Although batch treatment was ignored for over 50 years, interestingly, batch treatment systems have been reestablished as a viable treatment alternative in modern times (Droste, 1997).

The early success of the activated sludge process did not persist for long. Rapid population expansion and industrial development greatly altered the magnitude and nature of sewage loads to existing wastewater treatment plants, and the effect of flow and organic load variations became more pronounced. Serious problems, as

sludge bulking and shortage of oxygen, started to arise in the 1930s, which encouraged extensive studies and triggered the development of modified processes that would permit treating larger flows and greater loads while maintaining high effluent quality.

3.2 Process Variations

For various applications, eight different variations (types) of activated sludge process exist nowadays. The common characteristic of all of them, however, is that they use a flocculent suspended growth culture of microorganisms in an aerobic bioreactor and employ some means of biomass recycle. Furthermore, the primary objective is the removal of soluble organic matter and oxidation of the carbon contained in it. Under appropriate conditions, nitrification might also occur, and thus it is listed as an objective for those systems in which it is most likely.

The eight well known process variations include conventional activated sludge process, step feed, completely mixed, contact stabilization, extended aeration, high-purity oxygen, selector activated sludge process, and sequencing batch reactor processes. Full comparison among these variations of activated sludge process can be found in Grady et al. (1999). Advantages and disadvantages in addition to flow diagrams are presented there. Listing them is beyond the scope of this brief review. The history of the activated sludge process is very interesting and the reader is encouraged to learn more about it by referring to Jeppsson (1996), Grady et al. (1999), Henze et al. (2002), and Metcalf and Eddy (1991).

During the last thirty years, nutrient removal has become a very important factor in wastewater treatment due to restrict disposal and use regulations. Consequently, new systems, biological nutrient removal systems, were derived from the ordinary activated sludge system. Unlike the aforementioned systems, which were developed primarily for the removal of the organic material, biological nutrient removal systems consider the removal of nitrogen and phosphorus as well. Like the activated sludge systems, they also come in a number of configurations. One basic characteristic is that the microorganisms performing nitrification, denitrification and enhanced biological phosphorus removal require very different environments to function effectively, that is, a combination of aerobic, anoxic and anaerobic conditions. The term anoxic is frequently used to define a condition when oxygen is

absent and nitrate or nitrite is present. More details about various activated sludge process types can be found in Grady et al. (1999), Droste (1997), Henze et al. (2002), and Metcalf and Eddy (1991).

3.3   Modeling of Activated Sludge Systems

Modeling is an intrinsic element for the design and understanding of biological wastewater treatment systems. Its use has proven to be invaluable in the design, analysis, and optimization of activated sludge wastewater treatment plants (Yuan et al., 1997). Although the activated sludge process was developed in the early 1900s, there has been a long transition between its development and the establishment of a theoretical framework that describes the process and provides a basis for its design and control. Reasons behind that slow evolution are the conflicting nature of the mechanistic explanation hypotheses of the process, the difficulty of expressing them in precise mathematical models, and the contrived nature of the systems on which the models were developed. From the 1920s to 1960s different hypotheses explaining mechanisms of organic matter removal by activated sludge, were proposed. After the 1960s models started to jump from hypotheses to practically applicable models (Jeppsson, 1996).

Principally modeling of any system is usually commenced by a conceptual model, which is reducing the complex system to a conceptual image of how it functions. Usually such a model does not provide sufficient information alone. So accompanied with a physical model, such as lab-scale or pilot plant, upon which different design ideas can be tested. However, time and money consuming is a main disadvantage of such models. The solution is often a mathematical model. The primary function of a mathematical model is to reduce a complex system to the minimum terms essential for its description so that those terms can be manipulated. Thereby helping in understanding how the system will respond under a variety of conditions. The modeling of biological wastewater treatment systems has passed through the above sequence (Henze et al., 2000).

Mathematical models have numerous advantages over conceptual and physical models. In general, mathematical modeling is a useful and powerful tool for optimum design and control of any process, mainly because the effects of adjusting operating variables can be studied far more quickly on a computer than by doing experiments.

This is particularly true for a process like activated sludge biological wastewater treatment. which may take days to reach a new steady-state condition after one of the process variables is altered (Padukone and Andrews, 1989). In addition, designers of biological wastewater treatment systems can investigate easily, using mathematical models, the performance of a number of possible systems under a variety of conditions (Henze et al., 1987).

An activated sludge system mathematical model is usually composed of two main parts (see Figure 3.2) (Dunn et al., 1992). The first is a biological process model, which is the kinetic modeling of the main biological events identified the system under consideration. This part consists of three main components: a description of biological "structure", that is the components into which substrate and biomass are to be divided; a set of rate equations describing how rapidly one component is converted into another by various physical and biological processes; and finally a description of the stoichiometry showing how much of a component is consumed or generated by a process.
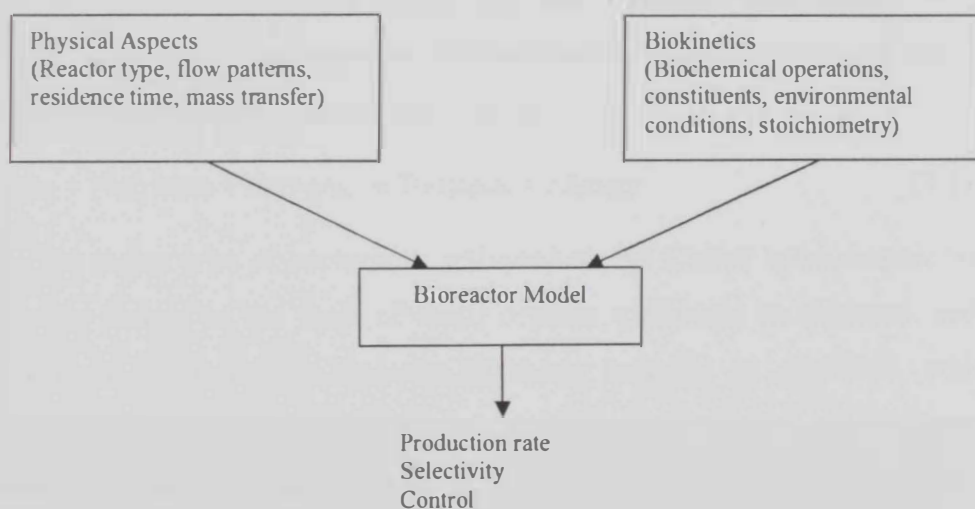
Figure 3.2: Information for bioreactor modeling (Dunn et al., 1992)

The second part is the "physical" modeling of the system, and this is concerned with the type of reactor used, flow pattern considered, and mass balance equations utilized. It consists of a set of conservation equations describing the mass balance of each component in the particular type of reactor chosen. Both parts when

incorporated together produce an activated sludge bioreactor (system) model. Our concern, in fact, is the first part; the modeling of the process from the biological point of view. However, we will show how this biological model can be imbedded in a "physical" model to produce a bioreactor model.

The starting point in modeling the activated sludge process, from the biological modeling point of view, is to understand the biological operations that occur during the process. As mentioned briefly earlier, the main objective of the process, as with all other biological wastewater treatment processes, is to remove organic matter (soluble or insoluble) through biochemical operations (transformations) and produce an effluent with acceptable characteristics. Regardless of the nature and complexity of the system involved, there are certain fundamental processes that occur universally in biochemical operations. In their barest essential, biological treatment systems are systems in which microorganisms are allowed to grow by using pollutants as their carbon and/or energy source, thereby removing the pollutants from the wastewater and converting them to new biomass and carbon dioxide, or other innocuous forms. Bacteria, which are the most important microorganisms in the activated sludge process, constantly need energy in order to grow. Growing cells utilize substrate and nutrients for growth and energy in a process, which can be described in a simplified form as:

$$\text{Substrate} + \text{Nutrients} + \text{Oxygen} \rightarrow \text{Biomass} + \text{Energy} \qquad (3.1)$$
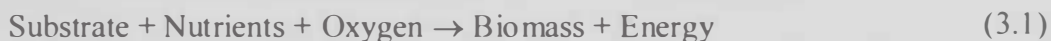
The major type of bacteria in activated sludge (called heterotrophic bacteria) use organic carbon in the form of small organic molecules as substrate, and some bacteria (called autotrophic bacteria, nitrifying bacteria, or nitrifiers), which are essential to biological nutrient removal, use inorganic carbon as substrate. This biochemical operation is called microbial growth.

At the same time biomass degradation (decay) occurs, coupled to oxidation of part of the organic matter in the biomass. Another process, which is hydrolysis, takes care of insoluble (particulate) and soluble large molecules occur. Hydrolysis converts larger molecules into small, directly degradable molecules. Other processes are soluble microbial product formation, ammonification, and phosphorus uptake and release. The later three processes are of less applicability and increase significantly

the complexity of the model when incorporated. Thus, they are not detailed in the following discussion.

Models range from simple to complex based on considering some or all of the aforementioned processes. The most basic model considers only the removal of soluble organic matter in which only biomass growth and decay is considered with the substrate as soluble organic matter. While more general models consider in more details most of the above mentioned processes. The following is a discussion of the modeling of the most common biological operations (i.e. biological growth, hydrolysis, and decay).

3.3.1 Microbial Growth

It is the main process through which the organic matter is removed. It is usually coupled with the substrate utilization (if growth is balanced). As a consequence the removal of one unit of substrate results in the production of Y units of biomass, where Y is called the growth yield, or simply the yield. Because of the coupling between biomass growth and substrate utilization, the rates of the two activities are proportional, with Y as the proportionality factor. Thus, selecting one as the primary event (or cause) and the other as the secondary event (or effect) is arbitrary. Both selections are equally correct and benchmark papers have been published using both substrate utilization and biomass growth as the primary event.

The process of bacterial growth can be modeled using the following expression:

$$r_{XB} = \mu X_B \tag{3.2}$$

r_{XB} is the biomass growth rate (also, dX_B/dt), $(ML^{-3}T^{-1})$.

μ is the specific growth rate, (T^{-1}).

X_B is the concentration of biomass, (ML^{-3}).

The specific growth is referred to as a specific rate coefficient because it defines the rate of biomass growth in terms of the concentration of active biomass present, i.e., the mass of biomass COD formed per unit time per unit of active biomass COD present. Equation (3.2) holds for any type of bacterial growth.

A major concern was how to mathematically describe the specific growth rate. The most commonly recognized rate expression with historical precedence and wide acceptance is the hyperbolic expression proposed by Monod:

$$\mu = \mu_{max}\left(\frac{S}{K_s + S}\right)$$
(3.3)

μ_{max} is the maximum specific growth rate, (T^{-1}).

S is the concentration of substrate, (ML^{-3}).

K_s is the half saturation coefficient, (ML^{-3}).

Although Monod's original work was done in batch reactors and was developed basically for pure cultures of microorganisms growing on single organic substrates, many researchers have shown that this equation is applicable for continuous cultures, and can be used to express removal of substrate that is really a mixture of hundreds of organic compounds, and can describe the growth of the heterogeneous assemblage of many bacterial species by simply "biomass" (Grady et al., 1999).

Examination of Equation (3.3) reveals that two simplifications can be made, and this is often done in the modeling of wastewater treatment systems. First, if S is much larger than K_s, then the equation may be approximated as:

$$\mu \approx \mu_{max}$$
(3.4)

This is called the zero-order approximation. Second, if S is much smaller than K_s, the term in the denominator may be approximated as K_s and the equation becomes:

$$\mu \approx \frac{\mu_{max}}{K_s}S$$
(3.5)

This is called the first-order approximation.

Besides the common Monod expression, there exist other expressions that describe the specific growth rate. However compared to Monod they are of less applicability. These models include the following (all K and θ coefficients represent different model parameters) (Dunn et al., 1992 and Jeppsson, 1996):

- The Tiessier model (Tiessier, 1936):

$$\mu = \theta_1\left(1 - e^{\theta_2 S}\right)$$
(3.6)

- The Blackman model (Blackman, 1905; Garrett and Sawyer, 1952):

$$\mu = \begin{cases} \dfrac{\mu_{max} \cdot S}{K_B} & \text{if } S < K_B \\ \mu_{max} & \text{if } S \geq K_B \end{cases}$$ (3.7)

- The Contois model:

$$\mu = \frac{\mu_{max} \cdot S}{K_C X + S}$$ (3.8)

- The Powell model (Powell, 1967):

$$\mu = \theta_1 \left(\frac{\theta_2 + \theta_3 + S}{2\theta_3} \right) \left(1 - \sqrt{1 - \frac{4\theta_3 S}{(\theta_2 + \theta_3 + S)^2}} \right)$$ (3.9)

- The Haldane model for inhibition kinetics:

$$\mu = \frac{\mu_{max} S}{K_S + S + \dfrac{S^2}{K_I}}$$ (3.10)

Substrate utilization, which is coupled to microbial growth, as mentioned, can be related to microbial growth through process stoichiometry. Its rate can be given as:

$$r_{XB} = -Y \cdot r_s$$ (3.11)

r_s is the substrate utilization rate (also, dS/dt), $(ML^{-3}T^{-1})$.

As mentioned earlier, many investigators have selected substrate utilization, rather than microbial growth, as their basic event and have written their rate equation accordingly. Several investigators have used the Monod expression to derive design equations for the activated sludge process. Lawrence and McCarty (1970) and the recent International Water Association models (Activated Sludge Models No. 1, No. 2, and No. 3) are good examples (Reynolds and Richards, 1996, and Henze et al., 2000).

Beside the above mentioned Monod approach in describing the biological growth kinetics, there exist other approaches. However, these approaches have less applicability in research and design. Among such approaches is the one that uses a modification of chemical kinetics and considers the substrate removal as the basic

event. Its equations are based on the Michaelis-Menten equation in enzyme kinetics. This approach suggests the following relationship for substrate removal rate:

$$\frac{1}{X}\frac{dS}{dt} = k_S\left(\frac{S}{K_m + S}\right) \tag{3.12}$$

$(1/X)(dS/dt)$ is the specific rate of substrate utilization, $(M.M^{-1}(microbes).T^{-1})$.

dS/dt is the rate of substrate utilization, $(ML^{-3}T^{-1})$

k_S is the maximum rate of substrate utilization, $(M.M^{-1}(microbes).T^{-1})$.

K_m is the substrate concentration when the rate of utilization is half the maximum rate, (ML^{-3})

S is the substrate concentration, (ML^{-3})

This approach has been used by Professor W. W. Eckenfelder, Jr. (1966, 1970, 1980, 1989) for the design of various biological treatment processes (Reynolds and Richards, 1996).

Because of the similarity of Equation (3.12) to Equation (3.3), many people have erroneously concluded that Monod proposed his equation (Equation (3.3)) on mechanistic grounds. While the Michaelis-Menten equation can be derived from consideration of the rates of chemical reactions catalyzed by enzymes, and has a mechanistic basis, the Monod equation is strictly empirical (Grady et al., 1999).

Another approach is the one that has been used by Grady and Williams (1975). They have presented data, which suggest that neither Monod nor Michaelis-Menten approaches adequately describe the effects of a varying influent substrate concentration on the substrate utilization rate. For such a situation, the relationship proposed by Grau et al. (1975) appears to more accurately describe the rate of substrate utilization (Benefield and Randall, 1980). They proposed that:

$$\frac{dS}{dt} = K_1 X\left(\frac{S}{S_o}\right)^n \tag{3.13}$$

n is the reaction order and is generally assumed to have a value of 1

S_o is the initial substrate concentration, (ML^{-3})

S is the substrate concentration surrounding the biomass at any time, (ML^{-3})

K_I is the specific substrate utilization rate constant. (T^{-1})

X is the biomass concentration. (ML^{-3})

Eckenfelder (2000) uses the same equation but suggests using the concentration of active biomass instead of the concentration of biomass in general (Eckenfelder, 2000).

The aforementioned expressions to model the kinetics of biological growth are presented in their simplest form. There exist more advanced expressions to model more complicated phenomenon happen in biological systems. As an example, the expression to describe the specific growth rate in the presence of inhibitory substrates will not be as simple as Equation (3.3). A modified Monod relation is used instead:

$$\mu = \frac{\mu_{max} S}{K_S + S + \dfrac{S^2}{K_I}} \tag{3.14}$$

Another worthy note, is that the abovementioned expressions apply in a situation where only the substrate, S, is a limiting factor for growth. Alternatively, μ_{max} can be seen as the maximum specific growth rate under given environmental conditions (temperature, pH, oxygen, nutrients, and toxic substances). For example, in the case of oxygen, the specific growth rate can be in the form of double Monod expression:

$$\mu = \mu_{max} \left(\frac{S}{K_s + S} \right) \left(\frac{S_{O2}}{K_{O2} + S_{O2}} \right) \tag{3.15}$$

Adding another environmental condition will add another Monod form term in the expression. Most of the common design models consider only one limiting factor of growth that is substrate (example is Lawrence and McCarty, 1970) (Metcalf and Eddy, 1991). Whereas, these extended rate expressions are common for dynamic analysis and operation and common in recent advanced general models (examples are Activated Sludge Models 1, 2, and 3).

3.3.2 Biomass Decay

There are two approaches to describe the reduction in yield and viability in biological wastewater treatment systems: traditional approach, that has been in use for many years and has found many applications, and is called lysis-regrowth approach, which

is an advanced approach utilized in most of the latest developed models for activated sludge systems.

A slight difference between the two approaches can be noticed. In the traditional approach, loss of biomass happens as a result of decay, which produces biomass debris, which is considered inert to further biological attack. In the lysis-regrowth approach, the loss of biomass happens through a death and lysis process which produces in addition to biomass debris a particulate substrate. This produced particulate substrate is hydrolyzed to soluble substrate, which is then oxidized to produce a new biomass.

In both approaches, the rate expressions have the same form except that for the lysis-regrowth approach a rate for the production of particulate substrate is added. Moreover, the rate coefficients are conceptually and numerically different. The rate expression for decay of biomass is first order with respect to biomass concentration:

$$r_{XB} = -b \cdot X_B \tag{3.16}$$

b is the decay coefficient (T^{-1}).

The rate of production of biomass debris can be seen to be:

$$r_{XD} = b \cdot f_D \cdot X_B \tag{3.17}$$

f_D is the fraction of active biomass contributing to biomass debris, X_D.

and for the lysis-regrowth approach, the rate of production of particulate substrate (Xs) is:

$$r_{XS} = b \cdot (1 - f_D) \cdot X_B \tag{3.18}$$

This particulate substrate is converted to soluble substrate, ready for uptake by biomass, through the hydrolysis process.

3.3.3 Hydrolysis

The conversion of particulate and high molecular weight organic matter into forms small enough for bacteria to take up and degrade is an important step in biochemical operations for wastewater treatment because such materials are commonly present in wastewater and also arise from lysis reactions as discussed previously. In spite of that,

relatively few studies have focused on those reactions and few researchers have considered them in their models.

The stoichiometry of hydrolysis is as simple as:

Particulate substrate COD → soluble substrate COD

This means that the rate of formation of soluble substrate COD is equal to the rate of loss of particulate substrate COD. It is common for engineers to choose the simplest possible reaction rate, and that is what a number of investigators have done, assuming that hydrolysis is a first-order with respect to the concentration of particulate substrate, Xs (Grady et al., 1999). However, in advanced models a more complex reaction rate expression is introduced. The following kinetic expression has been adopted by the IWA group in the model ASM No. 1 (Henze et al., 2000).

$$r_{XS} = -k_h \left[\frac{X_s / X_B}{K_X + (X_s / X_B)} \right] X_B \qquad (3.19)$$

k_h is the hydrolysis coefficient (h^{-1}).

K_X is the half saturation coefficient (mg particulate substrate COD/mg active biomass COD)

Hydrolysis as a phenomenon is not common in simple models, and appears only in advanced general models.

This is only a brief description of the three main biochemical transformations happening in all biological wastewater treatment systems. Once these biochemical transformations are defined and the models describing them are identified, the biological model is formed. The next step now is to form the physical information. As mentioned previously, the physical information comprises the reactor type and the flow pattern considered. From such information, mass balance equations can be derived, in which biological models can be imbedded to form the full model of an activated sludge system. In the following section, a basic model is developed to show how the biokinetic information is integrated with the physical information. It should be noted from the beginning that the developed model is the simplest possible model for activated sludge process and there exist more advanced models that account for most of biological events. These advanced models still simpler than what is there in reality although they provide good approximation of real systems.

3.3.4 A Basic Model

The basic expression of a mass balance of a given constituent takes the form:

Accumulation = input – output + generation $\qquad$ (3.20)

Each term in the mass balance equation has the units of mass/time. The generation term represents the sum of all reactions in which the constituent of interest participates, and incorporates the reaction rate terms mentioned above [e.g., Equation (3.2) of biomass growth rate]. If it is positive, the constituent is being produced; if it is negative, the constituent is being destroyed. Mass balance equations depend on the reactor (control volume) considered. Activated sludge processes employ a number of different types of reactors. Most of them are continuous flow, which means that the liquid flows through them continuously, bringing in reactants and carrying away products. Batch reactors, which have no flow through them while reaction is occurring, are also employed. The single continuous stirred tank reactor (CSTR) is the simplest reactor configuration used in biological treatment, finding application in activated sludge, aerated lagoons, aerobic digestion, anaerobic digestion, and biological nutrient removal.

For the sake of illustration, a basic CSTR (shown in Figure 3.3) with the very basic and simple assumptions is considered hereafter. A bioreactor with volume V receives a flow at rate Q containing only soluble, non-inhibitory, biodegradable organic substrate at concentration S_{So} and sufficient organic nutrients to make the organic substrate the growth limiting material. The influent flow and concentrations are constant, as per pH, temperature, and other environmental conditions. Within the bioreactor, the biomass (assumed only heterotrophic biomass) uses the substrate as its food source, thereby growing to concentration X_B while reducing the substrate concentration to S_S. Biomass decay accompanies the growth so that microbial debris at concentration X_D is also present.

Two essential terms are important to the performance of CSTRs. That is the hydraulic residence time (HRT) and solids retention time (SRT). A residence time defines the average amount of time a constituent stays in a system. Two types of constituents are present in the CSTR in Figure 3.3: (1) soluble, denoted by the symbol S, and (2) particulate, denoted by the symbol X. Consequently, two residence times can be defined. Dissolved constituents are intimately associated with the fluid and

cannot be easily separated from it. Thus, their residence time in a reactor is equal to the *HRT*, which is defined by:

$$HRT = V/Q \qquad (3.21)$$

The second residence time, called the SRT, represents the average length of time a particulate constituent stays in a bioreactor. The *SRT* is defined as the mass of particulate constituent contained in the bioreactor divided by the mass discharged from the bioreactor per unit time:

$$SRT = V.X_B/[Q_w.X_{Bw} + Q_e\,X_{Be}] \qquad (3.22)$$
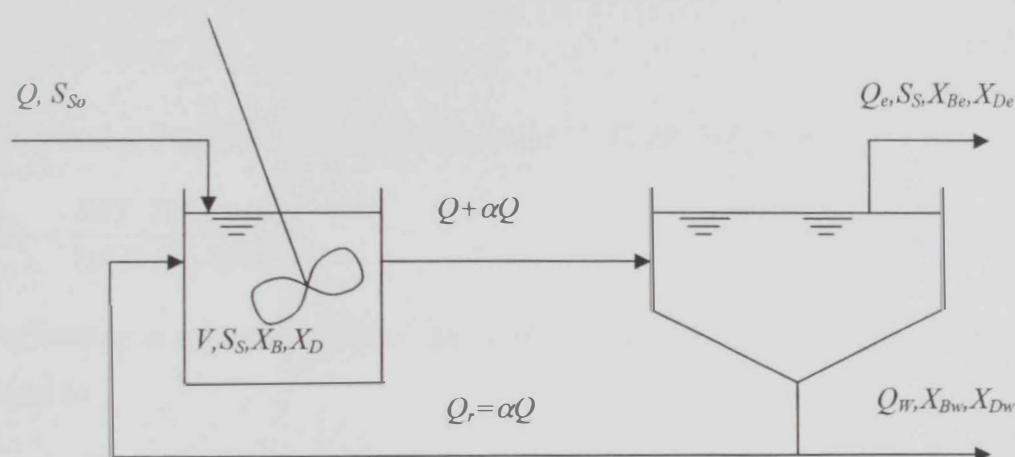
Figure 3.3: Schematic diagram of CSTR with biomass recycle from sedimentation

For the situation depicted in Figure 3.3, mass balance equations must be written for at least three constituents: S_S, X_B, and X_D. Table 3.1 gives the biological transformations incorporated into this model.

Table 3.1: Process kinetics and stoichiometry for simplified model

j	Component → i Process ↓	1 X_B	2 X_D	3 S_S	Process rate, r_i, $ML^{-3}\,T^{-1}$
1	Aerobic growth of heterotrophs	1		$-\dfrac{1}{Y}$	$\mu_{max}\left(\dfrac{S_S}{K_S + S_S}\right)X_B$
2	"Decay" of heterotrophs	-1	f_D		$b \cdot X_B$

As an example, referring to Figure 3.3, a mass balance for the biomass in the entire system can be written as:

Accumulation = Inflow − Outflow + Net growth (3.23)

$(dX_B/dt)V = Q. X_{Bo} - [Q_w X_{Bw} + Q_e X_{Be}] + r_{XB} V$ (3.24)

Using Table 3.1 to substitute for the rate of growth of biomass and assuming that the cell concentration in the influent is zero and steady-state conditions prevail ($dX_B/dt = 0$) yields:

$[Q_w X_{Bw} + Q_e X_{Be}] / V X_B = r_{XB} / X_B = (-Y r_s / X_B) - b$ (3.25)

The left-hand side of Equation (3.25) represents the inverse of the SRT as defined in Equation (3.22). Knowing that the term r_s is the rate of substrate utilization which equals:

$$r_s = -\frac{Q}{V}(S_{So} - S_S)$$ (3.26)

Substituting Equation (3.26) into Equation (3.25) and solving for X_B:

$$X_B = \frac{SRT}{HRT} \frac{Y(S_{So} - S_S)}{(1 + bSRT)}$$ (3.27)

Performing a substrate balance, the effluent substrate concentration is found to be equal to:

$$S_S = \frac{K_s(1 + SRT \cdot b)}{SRT(\mu_{max} - b) - 1}$$ (3.28)

In similar manner, all models describing activated sludge systems in different reactor configurations are formulated. Whether utilizing a simplified biological model or advanced, using a CSTR reactor or a plug flow one, the procedure is the same.

The model shown is a very basic model. It considers a system receiving only soluble substrate, although, most of wastewater contain soluble organic matter that is non-biodegradable. Furthermore, all domestic and many industrial wastewaters contain suspended matter that escapes removal by sedimentation prior to entrance of the wastewater into the biochemical operation. Hence, advanced models were also developed and will be discussed in the coming section. Most common simplified models include Lawrence and McCarty (1970), Eckenfelder (1966), Goodman and Englande (1974), Gaudy and Kincannon (1977), and Chen and Hashimoto (1980) (Padukone and Andrews, 1989; and Eckenfelder, 2000). The Lawrence and McCarty (1970) model has found a wide acceptance and has been the base for many design

equations and procedures. Metcalf and Eddy (1991) used such a model to formulate design equations for the activated sludge process. Other applications of this model will be listed later when talking about applications of activated sludge models.

Over the past two decades, the research group at the University of Cape Town, South Africa, has developed steady state design models based on a simplified conceptualization of the behavior of the activated sludge system. These models have progressively included aerobic COD removal and nitrification, anoxic denitrification, and anerobic-anoxic-aerobic biological excess phosphorus removal (Wentzel and Ekama, 1997). Some of these models have provided the basis for most of the advanced models developed later, for example, for the IWA Activated Sludge Model No. 1. Advanced models are discussed in the next section.

3.3.5 Advanced Models

Simplified models in general have two characteristics that restrict their applicability in many wastewater treatment situations. One is that they are limited to soluble, readily biodegradable substrates, whereas most wastewaters contain particulate contaminants and soluble constituents of large molecular weight that must be reduced in size before they can be taken into bacteria for degradation. If a model is to depict accurately the response of bioreactors receiving such wastewaters, it must include hydrolysis reactions. The other restriction is that the biomass is assumed to be in a constant biochemical environment with no limitations while in real situations many environmental and nutrient limitations occur.

Advanced models or sometimes called general models for activated sludge systems are developed including most of the possible biochemical transformations discussed previously with taking environmental conditions into consideration (temperature, pH, oxygen, nutrients). Recent advanced models include the family Activated Sludge Model No.1, ASM No. 2, and ASM No. 3, which have been developed by the IWA task group on mathematical modeling for design and operation of biological wastewater treatment (Henze et al. 1987, 1995, and Gujer et al. 1999) in addition to other models like Barker and Dold models (1997a and 1997b). However, ASM models are considered the most famous and recent general models. ASM models, like all advanced models, have the capability to depict the performance of

wastewater treatment systems receiving both soluble and particulate substrates in which organic substrate removal, nitrification, and denitrification are all occurring.

ASM1 was published in 1987 including nitrogen removal processes. In 1995, biological phosphorus removal has been added to nitrogen removal to form a new model called ASM2. In 1998, the task group decided to develop a new modeling platform, the ASM3, in order to create a tool for use in the next generation of activated sludge models. ASM3, since it is the recent, is considered in this work and will be discussed in detail hereafter. However, we will start by introducing ASM1 because it is the base for most advanced models including ASM3.

Activated Sludge Model No. 1 (ASM1)

The IWA task group has introduced ASM1 in a matrix format as shown in Table 3.2, where it can be seen to incorporate 8 processes and 13 components. The matrix representation was initiated to overcome the difficulties in representing more complex systems incorporating multiple parallel reactions acting on several components. Definitions of each component along with the units are given in Table 3.3. Detailed description of the model components and the main processes considered can be found in Henze et al. (2000), Grady et al. (1999), and Jeppsson (1996). Kinetic and stoichiometric parameters incorporated, typical ranges of such parameters, and assumptions, restrictions, and constraints can also be found in the same references. Listing such is beyond the scope of this work and the reader is encouraged to read about it in the mentioned references. However, it is essential to give a brief description of the processes in the model before showing how the model is integrated with the physical information to formulate the full model.

The fundamental processes incorporated into the model are listed in the leftmost column of Table 3.2, while their rate expressions are listed in the rightmost column. Basically, four processes are considered: growth of biomass, decay of biomass, ammonification of organic nitrogen, and hydrolysis of particulate organics. Both common types of biomass in activated sludge systems are considered: heterotrophs and autotrophs. The model depicts the performance of activated sludge systems under aerobic and anoxic conditions. Decay of the same two biomass groups is modeled following the lysis-regrowth approach. In the basic model presented earlier only growth and decay of heterotrophs were considered. The following is a

Component → i	1	2	3	4	5	6	7	8	9	10	11	12	13	
j Process ↓	X_I	X_S	$X_{B,H}$	$X_{B,A}$	X_D	S_I	S_S	S_O	S_{NO}	S_{NH}	S_{NS}	X_{NS}	S_{ALK}	Process rate, ρ_j, $ML^{-3} T^{-1}$
1 Aerobic growth of heterotrophs			1				$-\dfrac{1}{Y_H}$	$-\dfrac{1-Y_H}{Y_H}$		$-i_{XB}$			$-\dfrac{i_{XB}}{14}$	$\hat{\mu}_H\left(\dfrac{S_s}{K_s+S_s}\right)\left(\dfrac{S_O}{K_{O,H}+S_O}\right)X_{B,H}$
2 Anoxic growth of heterotrophs			1				$-\dfrac{1}{Y_H}$		$-\dfrac{1-Y_H}{2.86 Y_H}$	$-i_{XB}$			$\dfrac{1-Y_H}{14(2.86 Y_H)}$ $-\dfrac{i_{XB}}{14}$	$\hat{\mu}_H\left(\dfrac{S_s}{K_s+S_s}\right)\left(\dfrac{K_{O,H}}{K_{O,H}+S_O}\right)$ $\cdot\left(\dfrac{S_{NO}}{K_{NO}+S_{NO}}\right)\eta_g X_{B,H}$
3 Aerobic growth of autotrophs				1				$-\dfrac{4.57-Y_A}{Y_A}$	$\dfrac{1}{Y_A}$	$=i_{XB}-\dfrac{1}{Y_A}$			$-\dfrac{i_{XB}}{14}-\dfrac{1}{7Y_A}$	$\hat{\mu}_A\left(\dfrac{S_{NH}}{K_{NH}+S_{KH}}\right)\left(\dfrac{S_O}{K_{O,A}+S_O}\right)X_{B,A}$
4 "Decay" of heterotrophs	$1-f_D'$	-1			f_D'							$i_{XB}-f_D' i_{XB}$		$b_H X_{B,H}$
5 "Decay" of autotrophs	$1-f_D'$		-1	f_D'								$i_{XB}-f_D' i_{XB}$		$b_A X_{B,A}$
6 Ammonification of soluble organic nitrogen										1	-1		$\dfrac{1}{14}$	$k_a S_{NS} X_{B,H}$
7 "Hydrolysis" of particulate organics		-1					1							$k_h \dfrac{X_S/X_{B,H}}{K_x+(X_S/X_{B,H})}\left[\left(\dfrac{S_O}{K_{O,H}+S_O}\right)\right.$ $\left.+\eta_h\left(\dfrac{K_{O,H}}{K_{O,H}+S_O}\right)\left(\dfrac{S_{NO}}{K_{NO}+S_{NO}}\right)\right]X_{B,H}$
8 "Hydrolysis" of particulate organic nitrogen											1	-1		$r_7(X_{NS}/X_S)$
Observed conversion rates, $ML^{-3} T^{-1}$								$r_i=\displaystyle\sum_{j=1}^{n}\psi_{ij}\rho_j$						

brief description of the processes (Henze et al., 2000 and Jeppsson, 1996):

- Aerobic growth of heterotrophs: Examination of row 1 in Table 3.2 shows that growth occurs at the expense of soluble substrate and results in the production of heterotrophic biomass. Associated with this is the utilization of oxygen. Ammonia nitrogen will be removed from the solution and incorporated into cell mass. The kinetics are assumed to be subject to double nutrient limitation, with the concentrations of S_S and S_O being rate determining. The primary purpose of the oxygen term is as a switching function, which stops aerobic growth at low DO concentrations. The growth is modeled using Monod kinetics. This process is generally the main contributor to the production of new biomass and removal of COD. It is also associated with an alkalinity change.

- Anoxic growth of heterotrophs: In the absence of oxygen, the heterotrophic organisms are capable of using nitrate as the terminal electron (row 2 in Table 3.2). Like aerobic growth it occurs at the expense of readily biodegradable substrate and results in heterotrophic biomass. The process will lead to a production of heterotrophic biomass and nitrogen gas (denitrification). The nitrogen gas is a result of the reduction of nitrate with an associated alkalinity change. The same Monod kinetics as used for the aerobic growth is applied except that the kinetic rate expression is multiplied by a factor η_g (<1). Ammonia serves as the nitrogen source for cell synthesis, which in turn changes the alkalinity.

Table 3.3: Definitions of components in ASM1 model (Symbols in Table 3.2)

Component number	Component symbol	Definition
1	X_I	Inert particulate organic matter, mg/L as COD
2	X_S	Slowly biodegradable substrate, mg/L as COD
3	$X_{B,H}$	Active heterotrophic biomass, mg/L as COD
4	$X_{B,A}$	Active autotrophic biomass, mg/L as COD
5	X_D	Debris from biomass death and lysis, mg/L as COD
6	S_I	Inert soluble organic matter, mg/L as COD
7	S_S	Readily biodegradable substrate, mg/L as COD
8	S_O	Oxygen, mg/L as COD
9	S_{NO}	Nitrate nitrogen, mg/L as N
10	S_{NH}	Ammonia nitrogen, mg/L as N
11	S_{NS}	Soluble biodegradable organic nitrogen, mg/L as N
12	X_{NS}	Particulate biodegradable organic nitrogen, mg/L as N
13	S_{ALK}	Alkalinity molar units

- Aerobic growth of autotrophs: As shown in row 3 in Table 3.2, soluble ammonia is oxidized to nitrate via a single-step process (nitrification) resulting in production of autotrophic biomass. Ammonia is also used as the nitrogen source for synthesis and incorporated into the cell mass. Once again the growth rate is modeled using Monod kinetics. A double saturation function is used to express the dependency upon the soluble concentration of both ammonia and oxygen, with the latter serving as a switching function.

- Decay of heterotrophs: The process is modeled according to the death regeneration hypothesis as depicted in row 4. The adopted rate expression is first-order with respect to heterotrophic biomass concentration. However, the rate coefficient is different from the traditional decay coefficient. In this case, decay converts biomass to particulate products and slowly biodegradable substrate. No loss of COD is involved and no electron acceptor is utilized. The process is assumed to continue with the same rate under aerobic, anoxic and anaerobic conditions.

- Decay of autotrophs: The process, given in row 5, is modeled in the same way as used to describe decay of heterotrophs.

- Ammonification of soluble organic nitrogen: Biodegradable soluble organic nitrogen is converted to ammonia nitrogen in a first-order process. The reaction is depicted in row 6 of Table 3.2.

- Hydrolysis of entrapped organics: Slowly biodegradable substrate enmeshed in the sludge mass is broken down, producing readily biodegradable substrate available to the organisms for growth. The process is modeled on the basis of surface reaction kinetics and occurs only under aerobic and anoxic conditions. The rate of hydrolysis is reduced under anoxic conditions compared with aerobic conditions by a factor η_h (<1.0). The rate is also first-order with respect to the heterotrophic biomass present but saturates as the amount of entrapped substrate becomes large in proportion to the biomass. Row 7 depicts the process.

- Hydrolysis of entrapped organic nitrogen: Biodegradable particulate organic nitrogen is broken down to soluble organic nitrogen at a rate defined by the hydrolysis reaction for entrapped organics described above. Row 8 in Table 3.2 shows this process.

Based on the above description and Table 3.2, one can formulate the differential equations of rate expressions. As an example, if heterotrophic biomass concentration is considered, r_3 should be found as follows:

$$r_3 = \sum_{j=1}^{8} \psi_{3j} \rho_j = 1 \times \rho_1 + 1 \times \rho_2 + \bullet \times \rho_3 + 1 \times \rho_4 + \bullet \times \rho_5 + \ldots + \bullet \times \rho_8 \qquad (3.29)$$

then,

$$\frac{dX_{B,H}}{dt} = \left[\hat{\mu}_H \left(\frac{S_S}{K_S + S_S} \right) \left\{ \left(\frac{S_O}{K_{O,H} + S_O} \right) + \right. \right.$$
$$\left. \left. \eta_g \left(\frac{K_{O,H}}{K_{O,H} + S_O} \right) \left(\frac{S_{NO}}{K_{NO} + S_{NO}} \right) \right\} - b_H \right] X_{B,H}$$

$$(3.30)$$

In the same manner, the same can be developed for all the 13 components incorporated in the model. Equation (3.30) shows the rate of reaction of $X_{B,H}$ and can be substituted in Equation (3.24) (replaces r_{XB}) to formulate the mass balance equation of $X_{B,H}$. It is obvious that the resulting mass balance equation will be much more complicated than the one for the basic model. Similarly, the mass balance equation for S_S will also be much more complicated. This, in fact, shows how the advanced activated sludge models produce complex mathematical models. This complexity affects the application of such models in design and operation and in many cases limits their applications to research and analysis. Moreover, it is clear that analytical solutions to find the concentration of any component cannot be obtained as shown for the simplified model. Instead, numerical techniques and computer programs are being used to solve systems with such complex models

Among the six common biological transformations mentioned previously, two were not considered in ASM1: (1) soluble microbial product formation, and (2) phosphorus uptake and release. This is because of the minor impact of soluble microbial product formation on the process (Grady et al., 1999). The phosphorus uptake and release will occur only when anaerobic zones are included in the system. This process was incorporated in the model ASM2 of the same task group of IWA. ASM2 is beyond the scope of this work since only aerobic systems are considered. As mentioned earlier, the third model in the family of ASM models is ASM3, which will be discussed in the following section.

Activated Sludge Model No. 3 (ASM3)

ASM3 was developed to correct for some defects noticed in ASM1 and to incorporate latest advances in the modeling of activated sludge systems. Figure 3.4 shows a comparison between ASM1 and ASM3. In ASM1, the flow of COD is rather complex. The major difference between ASM1 and ASM3 models is that the latter recognizes the importance of storage polymers in the heterotrophic conversions in the activated sludge processes. In ASM3 model, it is assumed that all readily biodegradable substrate (S_S) is first taken up and stored in an internal cell component (X_{STO}) prior to growth. The biomass is thus modeled with an internal cell structure. The internal component X_{STO} is subsequently used for biomass growth in ASM3 model. Biomass growth directly on external substrate as described in ASM1 is not considered in ASM3. Furthermore, the death regeneration concept of ASM1 is replaced in ASM3 by endogenous respiration, which is believed to be closer to the phenomena observed in reality. As a result, the conversion processes of both groups of organisms (autotrophs and heterotrophs) are clearly separated in ASM3, whereas the decay regeneration cycles of the autotrophs and heterotrophs are strongly interrelated in ASM1. Finally, ASM3 allows a differentiation between aerobic and anoxic biomass decay whereas ASM1 does not.
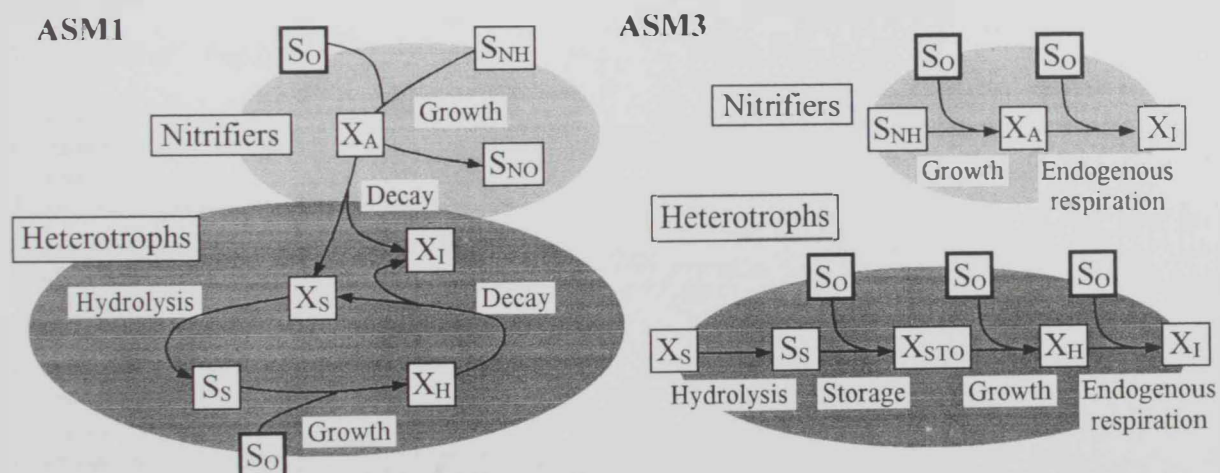
Figure 3.4: Flow of COD in ASM1 and ASM3 (Henze et al., 2000)

ASM3 includes 12 processes and 13 components. New components have been introduced while others were in ASM1 had been disregarded. Table 3.4 shows the main processes incorporated in ASM3 and their associated rate expressions. Table 3.5 shows a stoichiometric matrix for ASM3 based on typical stoichiometric parameters values suggested in Henze et al. (2000). Table 3.6 defines the components considered in the model. For definitions and typical values of the parameters appear in Table 3.4, the reader should refer to Henze et al. (2000).

Table 3.4: Processes incorporated in ASM3 and their kinetic expressions (Henze et al., 2000)

j Process	Process rate equation ρ_j, all $\rho_j \geq 0$.
1 Hydrolysis	$k_H \cdot \dfrac{X_S/X_H}{K_X + X_S/X_H} X_H$

Heterotrophic organisms, aerobic and denitrifying activity

2	Aerobic storage of S_S	$k_{STO} \cdot \dfrac{S_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_S}{K_S + S_S} \cdot X_H$
3	Anoxic storage of S_S	$k_{STO} \cdot \eta_{NOX} \cdot \dfrac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot \dfrac{S_S}{K_S + S_S} \cdot X_H$
4	Aerobic growth	$\mu_H \cdot \dfrac{S_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \cdot \dfrac{S_{ALK}}{K_{ALK} + S_{ALK}} \cdot \dfrac{X_{STO}/X_H}{K_{STO} + X_{STO}/X_H} \cdot X_H$
5	Anoxic growth (denitrification)	$\mu_H \cdot \eta_{NOX} \cdot \dfrac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot \dfrac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \cdot \dfrac{S_{ALK}}{K_{ALK} + S_{ALK}} \cdot \dfrac{X_{STO}/X_H}{K_{STO} + X_{STO}/X_H} \cdot X_H$
6	Aerobic endogenous respiration	$b_{H,O_2} \cdot \dfrac{S_{O_2}}{K_{O_2} + S_{O_2}} \cdot X_H$
7	Anoxic endogenous respiration	$b_{H,NOX} \cdot \dfrac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot X_H$
8	Aerobic respiration of X_{STO}	$b_{STO,O_2} \cdot \dfrac{S_{O_2}}{K_{O_2} + S_{O_2}} \cdot X_{STO}$
9	Anoxic respiration of X_{STO}	$b_{STO,NOX} \cdot \dfrac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot \dfrac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot X_{STO}$

Autotrophic organisms, nitrifying activity

10	Aerobic growth of X_A, nitrification	$\mu_A \cdot \dfrac{S_{O_2}}{K_{A,O_2} + S_{O_2}} \cdot \dfrac{S_{NH_4}}{K_{A,NH_4} + S_{NH_4}} \cdot \dfrac{S_{ALK}}{K_{A,ALK} + S_{ALK}} \cdot X_A$
11	Aerobic endogenous respiration	$b_{A,O_2} \cdot \dfrac{S_{O_2}}{K_{A,O_2} + S_{O_2}} \cdot X_A$
12	Anoxic endogenous respiration	$b_{A,NOX} \cdot \dfrac{K_{A,O_2}}{K_{A,O_2} + S_{O_2}} \cdot \dfrac{S_{NOX}}{K_{A,NOX} + S_{NOX}} \cdot X_A$

Similar to ASM1, the rate expressions can be incorporated into mass balance equations to develop the required mathematical model of a certain bioreactor. Again,

the resulting mathematical model would be complex and highly mechanistic compared to the basic model developed previously. As a consequence, for such complex models, it is impossible to attain analytical solutions for the concentrations of the various constituents in a bioreactor, as was done in Equations (3.27) and (3.28) for the concentration of active heterotrophic biomass $X_{B,H}$. Rather, matrix solutions and numerical techniques must be used, depending on the complexity of the system under consideration. Such complexity and the need for numerical methods have triggered two research directions: (1) Development of computer programs to handle such complex models, and (2) The revelation of the concept of reduced order models. This concept will be discussed later on.

Table 3.5: Stoichiometric matrix of ASM3 based on parameters suggested in Henze et al. (2000)

Component → i	1	2	3	4	5	6	7	8	9	10	11	12	13
j Process	S_{O2}	S_I	S_S	S_{NH4}	S_{N2}	S_{NOX}	S_{ALK}	X_I	X_S	X_H	X_{STO}	X_A	X_{SS}
↓ Expressed as →	O_2	COD	COD	N	N	N	Mole	COD	COD	COD	COD	COD	SS
1 Hydrolysis		0	1	0.01			0.001		-1				-0.75
Heterotrophic organisms, aerobic and denitrifying activity													
2 Aerobic storage of S_S	-0.15		-1	0.03			0.002				0.85		0.51
3 Anoxic storage of S_S			-1	0.03	0.07	-0.07	0.007				0.80		0.48
4 Aerobic growth of X_H	-0.60			-0.07			-0.005			1	-1.60		-0.06
5 Anoxic growth (denitrific.)				-0.07	0.30	-0.30	0.016			1	-1.85		-0.21
6 Aerobic endog. respiration	-0.80			0.066			0.005	0.2		-1			-0.75
7 Anoxic endog. respiration				0.066	0.28	-0.28	0.025	0.2		-1			-0.75
8 Aerobic respiration of X_{STO}	-1										-1		-0.60
9 Anoxic respiration of X_{STO}					0.35	-0.35	0.025				-1		-0.60
Autotrophic organisms, nitrifying activity													
10 Aerobic growth of X_A, nitrific.	-18.04			-4.24		4.17	-0.600					1	0.90
11 Aerobic endog. respiration	-0.80			0.066			0.005	0.2				-1	-0.75
12 Anoxic endog. respiration				0.066	0.28	-0.28	0.025	0.2				-1	-0.75

Many organizations have developed computer codes for solving the simultaneous mass balance equations for the constituents in the models. This allows many researches and organizations to apply such advanced models to a variety of bioreactor configuration. Table 3.7 lists several computer codes that are available for using and implementing ASM models. As an example, the SSSP code, which was developed for implementation of ASM1 on microcomputers, has been utilized

effectively by Grady et al. (1999) to perform simulations for single CSTRs and for multiple bioreactor systems. It is menu driven and may be used for both steady state and dynamic simulations.

Table 3.6: Definitions of components incorporated in ASM3

Component number	Component symbol	Definition
1	X_I	Inert particulate organic matter, $[M(COD) L^{-3}]$
2	X_S	Slowly biodegradable substrate, $[M(COD) L^{-3}]$
3	X_H	Heterotrophic organisms, $[M(COD) L^{-3}]$
4	X_A	Nitrifying organisms, $[M(COD) L^{-3}]$
5	X_{STO}	A cell internal storage product of heterotrophic organism, $[M(COD) L^{-3}]$
6	X_{SS}	Suspended solids, $[M(SS) L^{-3}]$
7	S_I	Inert soluble organic matter, $[M(COD) L^{-3}]$
8	S_S	Readily biodegradable substrate, $[M(COD) L^{-3}]$
9	S_O	Dissolved oxygen, $[M(O_2) L^{-3}]$
10	S_{NOX}	Nitrate plus nitrite nitrogen, $[M(N) L^{-3}]$
11	S_{NH4}	Ammonium plus Ammonia nitrogen, $[M(N) L^{-3}]$
12	S_{N2}	Dinitrogen, $[M(N) L^{-3}]$
13	S_{ALK}	Alkalinity of the wastewater $[mole(HCO_3) L^{-3}]$

Table 3.7: Computer codes implementing IWA Activated Sludge Models

Code name	Features	Contact information
SSSP	Implements Model No. 1	C. P. Leslie Grady Jr., Environmental System Engineering, Rich Environmental Research Lab, Clemson University, Clemson, SC 29634-0919 USA
EFOR	Implements ASM1, ASM2, and ASM3 plus settler models	Jan Peterson, I. Kruger AS, Gladsaxevej 363, DK-2860 Soborg, Denmark
ASIM	A flexible modeling tool that implements both Model No. 1 and No. 2, as well as several others	Willi Gujer, EAWAG, Swiss Federal Institute for Environ. Science and Technology, CH-8600 Dübendorf, Switzerland
GPS-X	A general purpose simulator that implements ASM1, ASM2, and ASM3 plus other unit operations	Hydromantis, Inc., 1685 Main St. West, Suite 302, Hamilton. Ontario L8S 1G5 Canada
STOAT	A modeling tool incorporates ASM1, ASM2, ASM3, as well as other models like ASAL models	WRc plc, Frankland Road, Blagrove, Swindon, UK
SIMBA	A WWTP simulator that implements ASM1, ASM2, ASM3, as well as modified versions of them.	ifak system GmbH, Schleinufer 11, D-391104 Magdeburg, Germany (www.ifak-system.com)
SBRSIM	Implements Model No. 1 for a sequencing batch reactor	Jürgen Oles, Technical University Hamburg-Hamburg. Eissendorfer Strasse 42, 2100 Hamburg 90, Germany

Samuelsson et al. (2001) have developed a JAVA based simulator for activated sludge processes. The simulator has a user-friendly graphical interface and can be reached over Internet and operated from a web browser. The simulator has

been used for educational purposes both for university students and personnel from wastewater treatment plants. A demo version of the simulator is located at the URL address: www.syscon.uu.se/JASS/. ASM1 model is implemented to model the activated sludge process with a settler modeled as a traditional one-dimensional layer model (Samuelsson et al., 2001).

3.3.6 Reduced Order Models

The highly complex mechanistic models presented previously (i.e., ASM models) have initiated research to develop simpler, reduced order models for activated sludge processes, more suited for design and operation. The main issue in developing reduced order models is to find a compromise between model simplicity and accuracy. This requires basically a well understanding of the physical and biological concepts behind the original model. One of the main difficulties when developing a model is often to determine which reactions are the most significant ones and to describe these in a simple, yet comprehensive manner. A good physical model should realistically mimic the true dynamics of the process in question but still contain a minimum number of variables and parameters while maintaining the physical interpretation of those. As shown previously, for example, ASM models contain thirteen state variables and more than twenty parameters, resulting in a highly complex representation. Researchers' efforts concentrated on reducing the number of such variables and parameters to a minimum number based on reasonable simplifying assumptions.

Simplification assumptions are usually based on how dissolved oxygen, organic matter, nitrogen, and microorganisms are treated in the model (Jeppsson, 1996). It is common to assume that the dissolved oxygen (DO) is controlled separately. Hence the corresponding growth expressions become independent of DO variations and the oxygen concentration can be excluded as a state variable. Description of the organic matter is a significant reduction factor as well. Excluding one or more variables (e.g., particulate organic matter) can reduce the number of state variables describing the organic matter. The same can be applied on nitrogen and microorganisms.

Researchers have recognized early the importance of reduction of complex models based on physical intuition and several have been given to reduce wastewater

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models complexity. Gomez-Quintero et al. (1999) and Lee et al. (2001) list some recent efforts in this regard. One of the early attempts is the model of Zhao et al. (1994), who proposed a reduced order model describing only the nitrogen dynamics (ammonification and nitrate concentrations) of the alternating sludge process. Jeppsson (1996) has developed a reduced order model based on assumptions and simplifications applied to ASM1. He reduced the 13 state variables to only 5 variables: heterotrophic and autotrophic biomasses, biodegradable organic substrate, ammonia nitrogen and nitrate nitrogen. DO concentration was assumed to be 2 mg/L. He showed that the reduced order model was able to mimic the behavior of the original model (ASM1) with reasonable accuracy. Steffens et al. (1997) have proposed an algorithm for eliminating state variables from model based on variables affection over the process depending of the time scales dynamics of interest; oxygen dynamics were not taken into account. Gomez-Quintero et al. (1999) proposed a reduced nonlinear model based on ASM1. Their reduction was based on a certain number of considerations about time scales process dynamics, simplification of biomass dynamics, and utilization of available on-line measurements. Process behavior was evaluated with respect to both experimental data and computer simulations of the reference model. The proposed model has shown good representation of nitrogen dynamics and does take into account the dissolved oxygen (unlike most of reduced order models). Recently, Jannssen et al. (2000) developed similar assumption-based reduced-order model for controller tuning which resulted in a decreased simulation time by a factor of three.

Exploring the aforementioned efforts reveals the following common reduction assumptions:

- DO is controlled (considered > 2 mg/L), hence DO concentration is excluded as a state variable and any anoxic (anaerobic) reaction is omitted.

- Neglecting alkalinity dynamics resulting in reducing number of state variables.

- Grouping of state variables describing organic matter (and/or nitrogen) into one state variable.

In recent work, Koch et al. (2001) have utilized reduction assumptions to reduce the highly complex model ASM3 to a steady state model. They proposed the following assumptions (Koch et al., 2001):

- DO in the aerated volume is always > 2 g O_2 m^{-3} and simultaneous denitrification therefore neglected.

- Internal storage products are negligible because all particulate organic substrate from the influent is hydrolyzed.

- All the readily degradable substrate from the influent is used for pre-denitrification.

- Autotrophic biomass is negligible (only 2-3% of the total activated sludge mass for municipal wastewater).

Another common reduction approach is to use zero-order or first-order kinetics instead of using Monod kinetics in describing the growth kinetics. Henze et al. (2000) in their report about their ASM1 model, they assumed for modeling a steady state single CSTR using ASM1 that all processes may be described by first-order kinetics instead of Monod kinetics. This assumption besides the assumption of setting the DO concentration at a desired positive value so that denitrification is eliminated has yielded a simple linear model.

In this work, a reduced order version of ASM3, based on the assumptions mentioned earlier will be developed and utilized in the analysis. Development of the reduced order ASM3 based model is explained in the Chapter 6.

3.3.7 Application of Activated Sludge Models

In previous sections, several models have been discussed ranging form simple models utilized in design to advanced highly complex models utilized in dynamic analysis and control. The concept of reduced order models was also explained in the previous section. Most of these models have found applications in research and industry. Researchers, engineers, and operators now are utilizing models for design and operations including on-line measurements and control. The following is a literature survey of some of the recent research applying activated sludge models for different purposes. Works listed in the literature review given in Chapter 1 are not repeated here although they are good examples of activated sludge models application.

In 1993, Kao et al., based on the model of Tang et al. (1987), have developed a prototype computer based design environment for wastewater treatment plant design. Such a system can be expected to shorten the time for producing a feasible

design and to provide functions to assist in the exploration of better designs. A design engineer could readily perform sensitivity analysis.

A steady-state model for activated sludge with or without nitrogen control is proposed in Argaman (1995). The model addresses both soluble and particulate organics in the influent beside distinguish between various fractions of the MLSS. The pseudo first-order kinetic law is used for soluble BOD removal and first-order kinetic for the hydrolysis of particulate organics. Monod is used for nitrification while denitrification is controlled by BOD removal under anoxic conditions. An iterative procedure was developed to solve the model equations. The proposed model is most applicable in the preliminary phases of a system design, when various process alternatives are evaluated. Compared to dynamic models like ASM1, the model contains a smaller number of parameters and coefficients. Although most of the recent models are based on COD, this model uses BOD because, as the author stated, it is still the main parameter used by authorities. In Argaman and Papkov (1995), the proposed model has been applied. Bench scale experiments using domestic wastewater were carried out under a constant flowrate. A good agreement between the experimental results and the predictions of the proposed model were noticed. The authors carried a sensitivity analysis that indicated a high sensitivity of the system's size to the nitrifiers' growth and decay rates and the denitrification nitrate utilization rate.

Koch et al. (2001) applied a steady state model to the stoichiometry and kinetics of ASM3 for the prediction of denitrification efficiency and sludge production. The model is calibrated and validated with data from long-term full-scale and pilot-plant experiments for Swiss municipal wastewater. They concluded from sensitivity analyses that the total COD and suspended solids from the primary effluent are the most sensitive parameters for predicting both the sludge production and the denitrification efficiency. They applied also Monte-Carlo simulations and they showed that with increasing to nitrogen ratios, the uncertainty of the predicted denitrification rate decreases significantly (58%) while the prediction is more uncertain (about 20%) for substrate-limited conditions, which are often found in denitrifying plants.

In another work, Koch et al. (2001) developed an ASM3-based steady-state model which can be used for estimating the average nitrogen removal, sludge-

production and phosphorus-removal rates of different biological phosphorus-removing systems. It considers the wastewater composition, the oxygen and nitrate input in the anaerobic compartment and the interaction between biological phosphorus removal and denitrification for different operating conditions. The model is calibrated and validated with data from a number of long-term pilot and full-scale experiments for Swiss municipal wastewater. They concluded from a sensitivity analysis that the COD, the suspended solids and the nitrogen load from the primary effluent are the most sensitive parameters for predicting the sludge production, the denitirification efficiency as well as for the phosphorus removal.

In a very recent work, Amano et al. (2002) proposed an automatic calibration method of biological reaction model constants by applying the optimal regulator method in modern control theory. The IWA ASM No. 2 was applied as the biological reaction model. It is expected to overcome the problems that arise when calibrating these constants by experiments. This method was verified using measured values of a small sewage treatment testing facility. Results show that calculated values of component concentrations approach measured values and the method is useful for actual plants.

The author has reviewed plenty of interesting works in this regard; however, it is out of the scope of this thesis to list all such works. Those listed above are good examples of application of activated sludge models for various purposes.

3.4   Design and Analysis of Activated Sludge Systems

The term "design" refers to the process of determining the size and configuration of new facilities needed to provide sufficient treatment capability. In other instances, a facility may already operate, but its performance may not be satisfying. In such case, analysis is the action. The term "analysis" refers to the process of determining the behavior of an existing system or a trial system that is being designed in order to find possibilities of improving or "optimizing" the performance.

The design of biological wastewater treatment systems is typically an iterative process where several levels of refinement are required. The design process usually starts with a preliminary assessment of available data about wastewater characteristics. This assessment in some instances includes treatability studies to

define stoichiometric and kinetic parameters while in other instances such parameters can be obtained from literature or experience with the wastewater to be treated. The design can be more or less detailed. The level of details to be chosen depends on the design purpose. The current design practice includes different approaches (levels). Such range from simple, less detailed, and less accurate design approaches suitable for small wastewater treatment plants to more detailed and more accurate design approaches developed originally for complex systems. The following section emphasizes the current design practice briefly.

3.4.1 Current Design Practice

The most common design approach is based on a simple stoichiometric model incorporating broadening assumptions, such as considering the oxidation of soluble biodegradable substrate and ignoring other forms of substrate. Such design approach is introduced by Metcalf and Eddy (1991). Several examples are also presented there. This approach is based on the very simple model of the process which was explained earlier in this chapter.

Henze et al. (2002) have distinguished three design methods (levels). The design by means of volumetric loading, by means of sludge age, and the computer aided process design. Reasonable results can be obtained from the first simple method if the plant considered is small plant receiving a uniform wastewater composition. The second approach is somewhat more advanced. However, the sludge loading can be used for BOD/COD removal only whereas the sludge age should be used in connection with other processes utilizing slowly growing bacteria like nitrification processes. With the introduction of processes such as biological phosphorus removal, nitrification and denitrification it will be in many cases dangerous, difficult or impossible to use the abovementioned two approaches as the design basis. Instead the computer aided design based on more advanced description of the process (like ASM models) should be considered.

Grady et al. (1999) introduced a slightly different three levels of design and evaluation. The first one is called a preliminary design approach based on basic principles of the process. Such design approach provides an initial assessment of the capacity and capability of a new plant. It also allows development of preliminary scope and cost estimate for the project of concern. The equations used in this design

procedure are approximate and incorporate several assumptions. Consequently, it does not allow precise estimates of process effluent quality. The second approach is stoichiometric based which incorporates more precise estimates of design parameters. This approach requires determining values for the kinetic and stoichiometric parameters that are specific to a particular wastewater. This approach actually is an extension of the common approach presented by Metcalf and Eddy (1991) where more detailed description of the process components is considered. Like, for example, consider particulate substrate (X_S) and soluble substrate (S_S) separately as well as account for inert particulate substrate (X_I) which is not shown in the simple design approach of Metcalf and Eddy (1991). The third approach is more complex than the other two. What is called simulation based design is considered when complex biochemical operation such as nitrification and denitrification is considered. In such cases the use of stoichiometric based design is not adequate, except as a starting point. This approach is computer aided, as called by Henze et al. (2002), because it is usually utilizes advanced models such as ASM1, ASM2, and ASM3. Through this approach, the most feasible design can be achieved with much less uncertainty associated with it. Several commercial programs have been developed incorporating advanced models. They are being used in simulation based design level (see Table 3.7).

Although the approaches presented by Grady et al. (1999) might produce a more accurate design, the design procedure of Metcalf and Eddy (1991) is still widely accepted and almost all of the conventional activated sludge plants are being designed based on it[1].

The reader is referred to the above references for more details about the abovementioned design approaches, the proposed procedure, and design equations implemented. It is not the scope of this work to list such information. However, main principles are elaborated hereafter which are applicable to all design approaches.

It is well known that the *SRT* is the most important design and control parameter. Several important characteristics of bioreactors can be determined from process stoichiometry once the *SRT* has been chosen, regardless of the bioreactor

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[1] Personal communication with a design engineer working with Metito, 2002.

configuration. These are the mass of biomass in the system, the mass rate of solids wastage, the quantity of oxygen that must be supplied, and the amount of nutrients needed.

Consequently, most of design approaches start the design by assuming a desired *SRT*. Such *SRT* must always exceed the minimum *SRT* which below it a particular group of microorganism is unable to grow. The range of typical *SRT* values is already known for many applications. Design approaches consider choosing the *SRT* from these ranges according to the application sought. For systems considering nitrification as well as organic carbon oxidation, *SRT* must be long enough to allow nitrifying bacteria "autotrophic" to grow. Metcalf and Eddy (1991) suggest a typical range of 1 to 15 days for complete mix activated sludge systems.

Beside the *SRT*, another parameter is commonly used in practice as a design and control parameter. That is the food-to-microorganisms ratio (F/M) which is defined as follows:

$$F / M = \frac{S_o}{HRT \cdot X} \tag{3.31}$$

Where S_o is the influent substrate concentration and X is the microorganisms concentration while *HRT* stands for the hydraulic retention time. The F/M ratio ranges from 0.2 to 1.0 d^{-1} for complete mix activated sludge systems (Metcalf and Eddy, 1991).

Another important design parameter is the MLSS concentration. In fact, two factors limit the bioreactor MLSS. (1) Solids thickening which limits maximum economical MLSS concentration to about 5000 mg/L as TSS. And (2) bioflocculation, which is typically, requires a minimum MLSS concentration of 500 to 1000 mg/L as TSS (Grady et al., 1999). Metcalf and Eddy (1991) suggest a typical range of 1000 to 6500 mg/L for complete mix activated sludge systems.

From the *SRT* and MLSS, volume (V) of bioreactor can be determined. According to the procedure presented by Metcalf and Eddy (1991), where a simple model is considered to represent the process, volume can be calculated from the following equation:

$$V \cdot X = SRT \, Q \frac{Y(S_o - S)}{(1 + b \cdot SRT)} \tag{3.32}$$

Where X is the mixed liquor suspended solids, Q is the flow rate, Y is the yield coefficient (defined as the mass of cells formed to the mass of substrate consumed, measured during any finite period of logarithmic growth), b is the endogenous decay coefficient (time^{-1}), S_o is the substrate concentration in influent, and S is the substrate concentration in effluent. It is obvious that a main assumption in Equation (3.32) is the grouping of substrates in one parameter (S) expressed as BOD or COD and ignoring any inert particulate substrate.

On the other hand, considering the stoichiometric based design approach presented in Grady et al. (1999) where a more extended model is considered to represent the process, the volume can be determined as follows:

$$V \cdot X = SRT \, Q \left[X_I + \frac{(1 + f_D \cdot b \cdot SRT) Y (S_{So} + X_{So} - S_S)}{1 + b \cdot SRT} \right] \tag{3.33}$$

It is clear that Equations (3.32) and (3.33) are similar but the latter considers in more detail and precision the components of the activated sludge. In other words, it is clear that inert particulate and particulate substrate are not included in Equation (3.32) while they appear in the other equation. From the above information (SRT, MLSS, V) other design characteristics can be determined. This includes the solids wastage rate and oxygen requirement.

However, specifying such information is not the end of the design cycle. Designers should consider interactions among design parameters. Examples include the relation between volume of bioreactor and the air flow rate. Since the oxygen transfer equipment is used usually both to transfer oxygen and to maintain solids in suspension, the volume of bioreactor is constrained between minimum and maximum values. Consequently a designer should refine his/her preliminary selections to fulfill constraints and effluent requirements. Such, is an example of the iterative nature of the design process as mentioned earlier.

Another worthy point is that the design considering simple model of activated sludge process implies huge approximations that might not be an efficient option when complex systems are being designed. When an activated sludge system is to operate in a fluctuating environment, more challenge is expected in obtaining a

reliable design based on simple models. In such cases, even a small degree of uncertainty can result in large consequences in terms of effluent quality or system cost.

The iterative nature of the design process and the approximations implied in the system design model make it extremely difficult to obtain a claimed optimal design. Moreover, the uncertainty associated with some of the design parameters adds another challenge in the way of obtaining an optimal design.

In this study, the simulation based design or a computer aided design approach is considered where an optimal sizing of a typical complete mix activated sludge system is introduced based on more detailed description of the process and less dangerous approximation. However, it worthy mentioning that this type of approaches could not eliminate the role of traditional design approaches. Such approaches could be the preliminary step before proceeding to the use of the new approach. A designer could start with the simple model based design to obtain an initial estimation of the design parameters. Then implement such initial design into the new model to obtain an optimal solution.

CHAPTER 4

SECONDARY SEDIMENTAION

The separation and concentration of active biomass in an activated sludge process is performed in a settling basin referred to as the secondary clarifier, the secondary settler or the secondary thickener. Secondary clarifier plays a crucial rule in biological wastewater treatment processes where activated sludge is used. From the bioreactor, the mixed liquor enters the secondary clarifier where it should be sufficiently clarified in order to produce an effluent of acceptable quality. Sludge should also be adequately thickened to maintain the desired solids level in the bioreactor through sludge recirculation and to achieve an effective treatment of the wasted activated sludge. This means that the settler combines the functions of clarification and thickening into one unit. Should the settling tank fail with respect to either of these functions, the result would be a rapid increase of suspended solids in the effluent or a deterioration of the activated sludge process. Practical experience has shown that the secondary clarifier is often the main bottleneck of the entire activated sludge process (Jeppsson, 1996 and Carlsson, 1998).

The complex behavior of the secondary clarifier and its great importance for the successful operation of the activated sludge process have made the settling process a major issue for researchers working within the field of design, operation, and mathematical modeling.

Depending on the nature and tendency of solid particles to interact, four modes of settling are normally encountered in a wastewater treatment plant; discrete particle, flocculant, hindered, and compression. Discrete particle settling takes place at low concentrations and characterized by solids which settle as individual entities with little or no interaction with other particles. This behavior is predominant for grit removal and in the upper regions of primary sedimentation tanks. Flocculant particle settling is the typical type of settling found in primary clarifiers, and the upper layers of a secondary settler. It is characterized by the flocculation of solid particles as they

settle through the water column. In hindered settling, the inter-particle forces hinder the settling process and the mass of particles settles as a unit. This type of settling is typical in secondary sedimentation tanks used in conjunction with biological treatment facilities. Compression settling refers to settling in which particles are of such concentration that a structure is formed, and further settling can occur only by compression of the structure. This type occurs at the lower layers of secondary settlers where suspended solids concentration exceeds 3000 mg/L. A secondary settler used to separate flocculent, compressible particles, is usually divided into four zones representing the four types of settling. Figure 4.1 shows such zones.
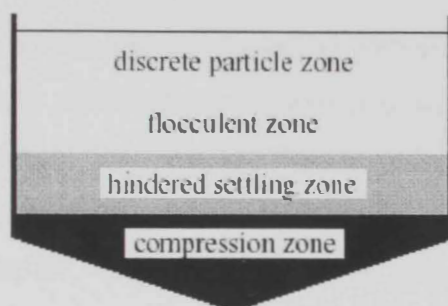
Figure 4.1: Settling zones for secondary clarifier

Design of settling tanks is normally based on the surface overflow rate (which can be defined as the primary clarifier overflow rate, see Equation 2.1). For each type of settling, a usual procedure can be followed to design the settling basin (Metcalf and Eddy, 1991). In discrete type of settling, for example, the usual procedure is to select a terminal velocity and design the settling basin so that all particles that have a terminal velocity equal to or greater than the selected terminal velocity are removed.

4.1 Secondary Sedimentation Models

Researchers working in the field of mathematical modeling have realized since early times the need to model the complex behavior of secondary settling tanks. Models developed range from simple empirical models to sophisticated computational fluid dynamics (CFD) models. One can distinguish between three main modeling schemes; (1) empirical models, (2) solids mass flux models, and (3) computational fluid

dynamics (CFD) models. Empirical models are still widely used today to predict mainly the characteristics of effluent and return sludge but cannot model the flow pattern or solids distribution within the tank. The solids mass flux model is used to perform mass balance on secondary sedimentation tanks for new designs or for the audit of a poor process and to determine the return sludge characteristics. It can also be used to predict the height of the sludge blanket. CFD models are used to predict the flow pattern and suspended solids distribution within the tank and are normally applied in research to find the relationship between the tank hydraulics and process performance. Modeling using CFD in water industry has not been widespread because of the associated high costs and the unfamiliarity of designer with mathematical models (Matko et al., 1996b).

Empirical models are usually developed by gathering sets of full-scale, pilot-scale, or experimental data and find the relationship that best fits these data by regression analysis. Empirical constants can then be found by dimensional analysis. Empirical models are mostly developed to describe the clarification behavior rather than the thickening behavior, although there exist some models describing the thickening.

4.1.1 Empirical Models for Clarification and Thickening

Tang et al. (1984) give an excellent review of a number of empirical models that predict the clarification performance of secondary sedimentation tanks. Among such, is the model developed by Chapman (1983) and utilized by Tang et al. (1987a). Chapman's model relates clarification efficiency to influent flow rate, MLSS concentration, and side water depth (Tang et al., 1987a). Except the efforts of Voutchkov (1992), a shortage in empirical clarification models since the middle of eighties can clearly be noticed. Voutchkov (1992) developed a regression based model describing the clarification efficiency of circular activated sludge secondary clarifiers. The developed model correlated the surface overflow rate to MLSS, sludge volume index, side water depth and effluent suspended solids. The model was developed based on data collected from 3 different activated sludge plants. The accuracy of the model was tested where predicted effluent suspended solids concentration deviated only within 15% of their actual values. The validity was confirmed at other three treatment plants. The data of the three plants scattered within

18% from the model estimates. Table 4.1 summarizes the models reviewed by Tang et al. (1984) and the model presented by Voutchkov (1992).

Table 4.1: Empirical models predicting total suspended solids concentration in secondary clarifier effluent (Tang et al., 1984 and Voutchkov, 1992)

Models	Effluent TSS Concentration (mg/l)	Source of Data
Villier (1967)	$45(Q_4 / A_f)^{.49} M_{t3}^{.82} t_s^{-.439}$	Villier (1967)
Takamatsu and Naito (1967)	$63.2 M_{t3}^{.5} \exp(-.74 t_s)$	Takamatsu and Naito (1967)
Agnew (1972)	1) $18.2 + 8.01(Q_4 / A_f) - 3.3 M_{t3}$ 2) $73.2(Q_4 / A_f)^{.12} FM^{.27} M_{t3}^{-.35} t_s^{1.03}$	Agnew (1972)
Lech (1973)	$1.4(17.6 - .739T)(Q_4 / A_f) M_{t3}$	Pflanz (1969)
Busby and Andrew (1975)	$10.88(Q_3 / A_f) M_{t3}$	Pflanz (1969)
Keinath et al. (1977)	$4.5 + 7.48(Q_4 / A_f) M_{t3}$	Pflanz (1969)
Tuntoolavest et al. (1980)	$-7.83 + 468 Q_a r - 70 r^2 + 14.59 M_{t3} + 13 r M_{t3}$ $-82.8 Q_a M_{t3} - 2.48 t_s M_{t3} + .162 M_{t3}(Q_4 / A_f)$	Tuntoolavest et al. (1980)
Dietz and Keinath (1982)	$5.341 + .506 M_d - 1.406 t_l$	Dietz and Keinath (1982)
Chapman (1983)	$-180.6 + 4.03 M_{t3} + 133.24(Q_3 / A_f)$ $+ \left[90.16 - 62.54(Q_3 / A_f)\right] H$	Chapman (1983)
Cashion and Keinath (1983)	$48.2 - 4.33\theta_c + 3.98\theta - .352\theta_c^2$ $- 248\theta^2 + 28.6\theta_c\theta$	Cashion and Keinath (1983)
Voutchkov (1992)	$\dfrac{6.21 \cdot \ln(MLSS \cdot SVI)}{0.67 \cdot \ln(H) - \ln(SR)} - 26.43$	Voutchkov (1992)

Note: A_f= surface area of secondary clarifier (m²)
FM= food to microorganism ratio in the activated sludge system (g BOD/g MLSS/d)
H= side water depth (m)
Q_a= air flow rate to aeration tank (m³/min)
Q_4= effluent flow rate from secondary clarifier (m³/h)
Q_3= effluent flow rate to secondary clarifier (m³/h)
r = sludge recycle ratio to aeration tank
T= temperature of mixed liquor (°C)
t_s= hydraulic detention time in secondary clarifier (hours)
t_l= detention time in clear zone (hours)
θ= hydraulic retention time in aeration basin (d)
θ_c= sludge age of the activated sludge system (d)
M_{t3}= mixed liquor suspended solids (MLSS) concentration (kg/m³) also MLSS
M_d= dilute blanket solids concentration (kg/m³)
SR= surface overflow rate

The author did not recognize any recent published research addressing the clarification behavior of secondary settlers empirically. This is in spite of the fact that it has been clearly stated in several recent publications that the best way to predict the effluent concentration is the empirical approaches (Wett, 2002). This may be attributed to the wide use of the solid flux theory (to be discussed later) in modeling

both the effluent as well as the underflow concentrations (e.g., Diehl and Jeppsson, 1998 and Cho et al., 1996).

Empirical models for activated sludge thickening in secondary clarifiers are also available in the literature although they are less common than models for clarification. Roche et al. (1995) developed a semi-empirical model to predict the behavior of secondary sedimentation tank thickening. The introduced equation was determined from 23 different experiments from industrial, municipal, and laboratory pilot plants. It shows that different activated sludges have similar thickening behavior. The developed model shows a good agreement with measured data from a full-scale dynamic secondary clarifier. The model is described by the general equation:

$$X_r = a(t + \Delta t)^b \tag{4.1}$$

where X_r is the underflow solids concentration (g/L), a, b, and Δt are empirical parameters described by exponential expression of X (MLSS) or SVI (sludge volume index), in the form:

$$a = 2.065\, X^{0.382} \tag{4.2}$$

$$\Delta t = 0.086\, X^{2.234} \tag{4.3}$$

$$b = 0.545\, SVI^{0.152} \tag{4.4}$$

t in Equation (4.1) represents the HRT (hours) in the bottom section of the clarifiers, i.e.,

$$t = (V_s/Q_r)n = [A(\text{SBH})/\, Q_r]n, \tag{4.5}$$

Where A is the surface area of clarifier (m^2), SBH is the sludge blanket height (m), Q_r is the total return sludge flow rate (m^3/h) and n is the number of clarifiers involved in the process.

Pipes and Kim (1996) highlighted some advantages and defects of this model. They argued that Δt has little influence on the predicted X_r value because it is small in comparison with t. In addition they proposed a new equation to account for the actual time that the sludge solids spend in the settling tanks on each pass. Giokas et al. (2002) considered the discussion of Pipes and Kim (1996). By excluding Δt from the equation of Roche et al. (1995) and replacing the expression of t by the one proposed

by Pipes and Kim (1996). The resulting empirical model integrates the previous models and emphasized the phenomenon of dilution of the incoming activated sludge. Testing with data from full-scale plant proves that it describes fairly well the return sludge concentration, the diluted sludge blanket concentration, the sludge blanket solids concentration, and the sludge blanket height.

Rather than using empirical models, most of the researchers in the field of secondary clarifiers modeling prefer to consider the models developed based on the concept of solids flux theory.

4.1.2 Solids Mass Flux Models

The origin of the solids flux theory dates back to the beginning of last century when Coe and Clevenger (1916) suggested that if a layer in a suspension has a lower total solids-handling capacity than the overlaying layer, it will be unable to discharge solids as rapidly as they are received and will therefore grow in thickness. If a given layer has a higher total solids-handling capacity than the layer above, its thickness will decrease or remain infinitesimal. The layer with the lowest total solids-handling capacity therefore limits the throughput of the thickener. If the thickener is overloaded, this layer (which contains the limiting solids concentration) will ultimately reach the liquid surface (Watts et al., 1996). Although the work of Hazen (1904), Camp (1936), and Dobbins (1944) was also pioneering and forms the foundation of the sedimentation theory, it ignores the thickening phenomenon prevalent in the activated sludge systems (Jepsson, 1996). Nowadays nearly all commonly applied hindered settling models are based on the theory proposed by Kynch (1952) (Wett, 2002). In a fundamental work based on the theory of Coe and Clevenger (1916), Kynch considered several main assumptions. Besides constant horizontal density layer Kynch assumed another fundamental principle: He declared that the settling velocity of particle depends only on the local concentration of the particles.

In general, the total flux [mass/(area × time)] of solids is obtained by:

$$J = Xv \tag{4.6}$$

where X is the solid (sludge) concentration and v is the settling velocity which in general depends on X. In a continuous flow settler, the downward solids flux is the

sum of the gravity settling flux (J_s) and the solids flux due to the bulk movement of the liquid (J_u), namely the underflow flux:

$$J = J_s + J_u \tag{4.7}$$

The gravity settling flux is the product of the average solids concentration, X, and the hindered settling velocity, v_s. The underflow flux is the product of the solids concentration, X, and the downward velocity, v_u, defined as the downward flowrate divided by the settling area, A.

$$J = X v_s + X v_u \tag{4.8}$$

As mentioned earlier, the hindered settling velocity only depends on the local concentration of the solids; hence the total flux can be written as:

$$J = X v_s(X) + X v_u \tag{4.9}$$

The hindered settling velocity is the most important parameter in the solids flux theory. There exist in the literature a number of different models for the settling velocity as a function of the suspended solids concentration. The Vesilind (1974) exponential model is widely accepted as the best model of the settling velocity of the mixed liquor for high suspended solids concentrations but does not consider low solids concentrations such as in the upper region of secondary clarifiers. Vesilind's model states that:

$$v_s = ke^{-nX} \tag{4.10}$$

where k is the maximum settling velocity and n gives a measure on how fast the settling velocity decreases with increasing concentration of particles. In practice, these parameters can be found by multiple batch settling experiments where log v_s is measured for different sludge concentrations. Then, k and n can be found by a simple least squares fit to the data (linear regression).

Inserting Vesilind formula in the total flux yields

$$J = (ke^{-nX} + v_u)X \tag{4.11}$$

An illustration of this relation is given in Figure 4.2.

Notice that the flux curve has a local minimum denoted J-lim. This flux is the maximum allowable flux loading if the settling is to be successful. If the influent flux

to the settler is larger than J-lim, the sludge blanket will increase, resulting in solids (sludge) in the effluent.

For a general total flux model $J(X)$, the limiting flux can be obtained by solving $J(X)/dx = 0$. To find the minimum we have to check which of the extreme points that fulfill $J^2(X)/dx^2 > 0$. These calculations may have to be solved numerically. Cho et al. (1996) show a sample of such calculation. Obviously, graphical solutions are also possible. Yoshioka et al. (1957) presented a simple geometric technique to find the limiting values from solids flux curves (Watts et al., 1996). Other methods also have been developed to determine the steady-state behavior of the secondary clarifier which could be used for design purposes (Jeppsson, 1996).
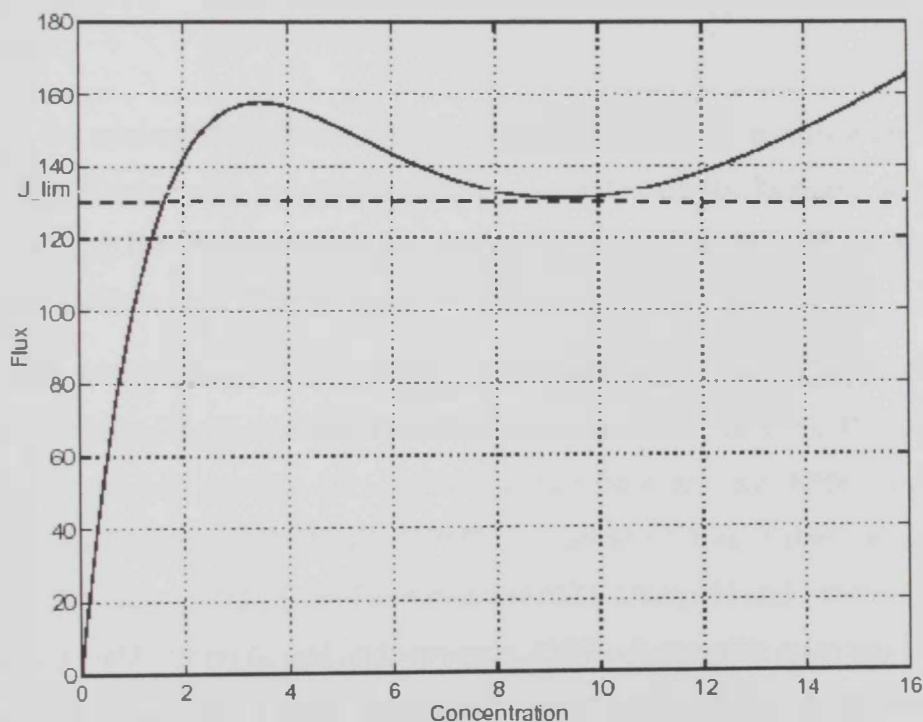
Figure 4.2: Total flux as a function of the solids concentration

The determination of an appropriate settling velocity model is indispensable for modeling the secondary clarifier using the solids flux theory. Therefore, a number of empirical functions of the settling velocity have been proposed. The majority of the

functions are based either on the exponential function (Equation 4.10) or the power function:

$$v_s = kX^{-n} \tag{4.12}$$

Usually, the exponential function is considered to be more accurate but is sometimes considered to require more complex numerical procedures for the mathematical analysis (Cho et al., 1996). A few examples of different settling velocity functions found in the literature are given in Jeppsson (1996). Recent examples of such models include Takacs model (Takacs et al., 1991) and Cho model (Cho et al., 1996), described as follows, respectively:

$$v_s = ke^{-r_h(X-X_{min})} - k'e^{-r_p(X-X_{min})} \tag{4.13}$$

$$v_s = k\frac{e^{-nX}}{X} \tag{4.14}$$

with X_{min} the minimum attainable suspended solids concentration corresponding to the non-settleable suspended solids concentration, r_h and r_p are the hindered settling zone parameter and the flocculent settling zone parameter, respectively. Also, several empirical relations exist relating parameters like SVI to k and n.

It is widely accepted that the design and operation of secondary settling tanks is based on the solids flux theory (Vanderhasselt and Vantolleghem, 1999). Families of one-dimensional models for secondary clarifiers are available in literature. Examples are Vaccari and Uchrin (1989), Vitasovic (1989), Takacs et al. (1991), Dupont and Henze (1992), Hartel and Popel (1992), Otterpohl and Freund (1992) and Dupont and Dahl (1995) (Diehl and Jeppsson, 1998). A common approach for them is to divide the settler into a fixed number of layers, within which the concentration is assumed to be constant. Mass balance is applied to each layer. As the number of layers increases, better approximations of the physically correct solution, are obtained. The settling mass flow from one layer to the other is limited by the minimum flux of both the considered interface and the interface below. The predictions of the effluent and underflow concentration are often made by assuming that these concentrations are the same as the boundary concentrations at the top and bottom within the settler.

Recently several publications discuss the solids flux theory in one way or another, the following is a presentation of some of these recent publications. Takacs et al. (1991) suggested a double exponential expression representing the settling function, i.e., the relation between settling velocity and particle concentration. He adopted a 10-layer model presented by Vitasovic (1989) that considered a reduced settleability of the micorfloc fraction of activated sludge and therefore improved the model validity for low-solid concentrations. Grijspeerdt et al. (1995) compared and evaluated complexity and data fit of six different layer models where Takacs model achieved the top score in the final rating.

Watts et al. (1996) developed a one-dimensional model of activated sludge secondary clarifiers with a dispersion term dependent on concentration and feed velocity. They utilized Takacs et al. (1991) model. Data collected from a full-scale clarifier were used to evaluate the model. Better matches were achieved than with the gravity-flux-constraining model. Chatellier and Audic (2000) studied also Takacs et al. (1991) model. They claimed that the hypothesis of such a model induces a strong underestimation of the sludge blanket level. This underestimation is explained because the sedimentation velocity does not take into account all the physics and hydraulics of the clarifier. They added a complementary hypothesis that the sum of convection flux and sedimentation flux remains constant through all the depth of the clarifier. This hypothesis leads to a new expression of the sedimentation velocity. The use of this expression in the simulation gave better estimations of the sludge blanket level dynamics. One drawback noticed is when the changes in the clarifier hydraulics are too sharp, this results in a slightly overestimated sludge blanket level.

Queinnec and Dochain (2001) emphasized the limitations of the solids flux theory basic models in representing steady-state operating conditions of secondary clarifiers. They introduced a more sophisticated unidimensional model. The model has been calibrated using lab-scale experimental data. It also reflects the diffusivity phenomena of thickening suspended solids without increasing the complexity of the model. Wett (2002) presented a model different from the common 10 layers models. His model consists only of three layers with variable volume, clarification, hindered settling, and compression zone, that are not derived from numerical requirements but from the basic principles of solids flux theory for batch sedimentation. In the model

development, simplicity for on-line control purpose and for coupling with a biokinetic model has been considered. The model has been evaluated against the analytical solution of the flux function and against data from full-scale SBR experiments and from a secondary clarifier under hydraulic overload.

Some researches have noticed a major problem when evaluating the performance of the activated sludge process associated with the difficulty of separating the dynamics of the biological reactor from the dynamics of the settler. Therefore, it is clearly stated that of major importance is to develop models that can handle the reactor-settler interaction (Diehl and Jeppsson, 1998). Watts et al. (1996) concluded that the clarifier model should be integrated with a model of the activated sludge process under investigation, using, for example the ASM1 (Henze et al. 1987). In this way, the integrated model can be employed to simulate the impacts of varying flow rates and feed concentration on both biochemical and sludge thickening performance. Early attempts have been noticed to model the coupling of the aerator with the clarifier in the activated sludge system. Cho et al. (1996) have listed examples of these attempts.

Cho et al. (1996) carried out a steady state analysis of coupling the function of aerator and secondary settling tank in an activated sludge process to obtain appropriate response of output variables and to decide optimum operating parameters. They utilized the Lawrence and McCarty (1979) model of biokinetics and the solids flux theory to model thickening and clarification of the secondary clarifier. They incorporated the velocity function developed by Cho et al. (1993) to find the effluent and underflow concentrations of the clarifier. Diehl and Jeppsson (1998) presented a dynamic simulation model of the entire activated sludge process. For the biological reactor the standard model by IWA, ASM1, was used, and for the continuous sedimentation in the secondary clarifier a new one-dimensional model based on the theory of non-linear partial differential equations was introduced.

Dupont and Henze (1992) described a development of a model for the secondary clarifier based on the general flux theory for zone settling, which can be used in combination with ASM1 for form a dynamic computer model/program for wastewater treatment plant. In addition to the flux model, the developed model includes a simple model for predicting the contents of particulate components in the

effluent. This latter model is a purely empirical model, which connects the effluent quality with the hydraulic load, suspended solids load and the nitrate load.

Most of the aforementioned models are suitable for the use in dynamic simulation and on-line control purposes. In this study, for the thickening performance of the secondary clarifier, the model utilized by Cho et al. (1996) to model the coupling of the reactor and the clarifier is considered. Cho et al. (1996) have utilized the solids flux theory in a coupled model. They utilized the Cho et al. (1993) velocity model which is considered one of the recent models representing settling velocity as a function of the solids concentration. A similar approach was considered by Tang et al. (1984) but with the old Vesilind's velocity model. For clarification performance, generally, a solids flux based model is not the appropriate tool to calculate the solids effluent concentration (Wett, 2002). Instead, empirical approaches are usually utilized. The model of Voutchkov (1992) is considered to model the clarification function of the secondary clarifier. It is worthy to mention that several researchers have assumed a perfect performance of the secondary clarifier i.e. the effluent concentration is set equal to zero and no clarification model is considered. One example is Wett (2002).

CHAPTER 5

COST FUNCTIONS

In the previous chapters mathematical models for the considered unit operations (i.e. primary clarifier and activated sludge) were introduced. Beside these mathematical models, as mentioned in Chapter 1, another two mathematical tools are required for performing an optimization study for the activated sludge process. Cost functions, which are considered in this chapter, and mathematical programming (optimization) techniques are those two mathematical tools. The latter is considered in the coming chapters. Cost functions are those functions describing the cost of a process unit as a function of design and/or operation parameters.

Recent developments in wastewater treatment techniques have complicated and widened the options available to treat a given wastewater. In today's plants, a vast number of plant configurations and operation schemes can be found. When designing a new wastewater treatment plant or when upgrading an existing one, different treatment alternatives and operating strategies may be evaluated. However, what is the base of such evaluation? Commonly such an evaluation is made based on finding the most economical alternative that fulfills the effluent requirements. Nowadays wastewater treatment is able to cope with almost any effluent quality objective. The problem is that cost of such treatment becomes prohibitively large since costs increase rapidly with the effluent requirements. Therefore, objective methods for evaluating the overall design and operation of wastewater treatment plants are of importance, for both economical and environmental reasons (Vanrolleghem et al., 1996).

There is no doubt that reliable methods for wastewater treatment plant cost estimation are needed to guide us in the process design and development activities; to aid in the discussion of operation schemes; and to facilitate the process of upgrading existing plants. In general, design and costing are interlinked (Wright and Woods, 1993). Some technically feasible design options become non-options when they are evaluated against cost. Thus costing is a crucial and ongoing consideration during the

planning phase of new wastewater treatment plants and for the evaluation of new operational strategies versus traditional expansions of plants already in operation.

This importance of costing has been early recognized and several efforts were made to derive cost functions for different unit processes in wastewater treatment.

5.1 Wastewater Treatment Plant Costs

In general wastewater treatment plant costs are subdivided into investment (also capital or construction) costs and operation and maintenance costs. The latter may be fixed (normal operation and maintenance, fixed power, etc.) or variable (power and chemical consumption, and effluent levies).

Construction costs include expenditures for labor and materials to build facilities of a wastewater treatment plant. These include piping, steel, concrete, excavation, buildings, electrical work, heating and ventilation, instrumentation, etc.

Operating costs can be split into fixed operating costs and variable operating costs. Only the latter can be controlled in current operation of a previously designed treatment plant. The fixed operating costs comprise normal maintenance and repair costs, including material and supply, and are usually a function of the size of the treatment units. Once the plant has been designed, these costs are a fixed part of the annual operation and maintenance costs. On the other hand, the variable operating costs originate from expenses, which directly depend on the rate at which the units are operated. They include wages for operating labor, and more importantly energy and chemical costs.

Variable operating costs also include effluent taxes or levies. Such costs started to gain increasing importance in the last decade. It is believed that in the coming years, the focal point in wastewater treatment will be the receiving water. The wastewater treatment of the future is expected to be dependent on the requirements from the local recipient and not some common effluent standards (Vanrolleghem et al., 1996).

A number of costing factors are still under research and are of special importance. These are costs associated to reliable design, as more complex treatment systems may result in better performance at reduced costs, but may be prone to increased risk of failure. Another issue is the plant flexibility, which may increase the

operation costs but at the same time may prove an advantage at later phases when expansions are thought (Vanrolleghem et al., 1996). Reliability and flexibility are of growing importance in wastewater design and operation.

As clear from the above, wastewater treatment costs have many origins and depend highly on the system of treatment and the unit processes incorporated. Quantification, formulation, and analysis of such costs have attracted many researchers over the last decades. Such efforts produced different ways of expressing wastewater treatment plants costs. The following is a literature review of the main efforts in this regard.

5.2   Literature Review

A number of studies were published on the development of cost estimation techniques for different wastewater treatment processes. The earliest study goes back to 1968. Tang et al. (1984) presented the early efforts of researchers in this regard; the following is a brief overview of their presentation.

In 1968, Smith estimated the cost of wastewater treatment facilities from cost data collected by other researchers in 1962. In 1971, Patterson and Banker presented the capital, operation and maintenance costs in graphical forms with respect to the sizes of the unit processes. Cost functions have been developed from such graphical information, as mentioned by Tang et al. (1984), Middleton and Lawrence (1975), the U.S. Army Corps of Engineers (1978), and Rossman (1979). In 1978, Dick et al. also developed a set of cost functions based on data presented by Patterson and Banker, Metcalf and Eddy, Inc. (1975), and Ettlich (1977). Tang et al. (1984) compared these cost functions and came up with new functions (Table 5.1) describing the cost of the most common unit processes considered in a wastewater treatment plant.

Tyteca (1985) also addressed the mentioned "Patterson and Banker" study. He considered it as one of the most authoritative studies in this regard. He used the cost information provided by them to derive cost functions as shown in Table 5.2. In this table, the operating cost has been split into two categories, namely, the fixed operating costs and the variable operating costs.

In 1980, EPA published a report about construction cost estimation of municipal wastewater treatment plants. It can be used for preliminary estimation of

construction costs for individual processes and/or complete facilities. In such a report, construction costs have been analyzed and reported by three levels of details; for complete treatment plants of various types, for specific unit processes such as clarifiers, aerators, etc., and for the costs of various components required: excavation, electrical, etc. The cost relationships are shown in the report as design flow versus cost, which makes them unsuitable for optimization studies where a cost relation need to relate the cost to a sizing or operation parameter. Another report was published in the following year to include the operation and maintenance costs.

Table 5.1: Summary of cost functions derived by Tang et al. (1984)

Process Unit	Capital (1971$)	Operation (manhours/yr)	Maintenance (manhours/yr)	Material and Supply (1971$/yr)	Power (kW hr/yr)
Primary Clarifier	$824A^{.77}$	$17.15A^{.6}(A \geq 279)$ $92.45A^{.3}(A < 279)$	$9.23A^{.6}(A \geq 279)$ $106A^{.14}(A < 279)$	$8.62A^{.76}$	—
Primary Sludge Pumping	$16042Q^{.53}$	$374Q^{.41}$	$166Q^{.43}$	$385Q^{.64}$	$23.85QH/\varepsilon_p$
Aeration Tank	$461V^{.71}$	—	—	—	—
Diffused Aeration	$8533Q_a^{.66}$	$187Q_a^{.48}$	$74.4Q_a^{.55}$	—	—
Secondary Clarifier	$824A^{.77}$	$17.15A^{.6}(A \geq 279)$ $92.45A^{.3}(A < 279)$	$9.23A^{.6}(A \geq 279)$ $106A^{.14}(A < 279)$	$8.62A^{.76}$	—
Return & Waste Sludge Pumping	$2779Q^{.53}$	$.333Q + 390$	$.2375Q + 370$	$300(Q < 63.2)$ $40.57Q^{.52}(Q < 252)$ $5.97Q^{.87}(Q < 632)$ $2.54Q(Q > 632)$	$23.85QH/\varepsilon_p$

A is the surface area in m^2, Q is the flow in m^3/hr, V is the volume in m^3, Q_a is the air flow rate in m^3/min, H is the pumping head in meters, and ε_p is the pumping efficiency.

A group of researchers started in 1975 to publish a series of papers to develop capital cost correlations for different treatment facilities. Their research covers most treatment facilities. In Part 7 of this series, Wright and Woods (1993) introduced capital cost correlations for physical treatment facilities while biological treatment facilities were addressed in Part 8 (Wright and Woods, 1994). These studies, like EPA studies, are good for the preliminary estimation of capital costs of individual process facilities.

Table 5.2: Summary of cost functions given by Tyteca (1985)

Process unit	Investment costs ($)	Fixed operating costs ($/year)	Variable operating costs ($/year)
Activated sludge	$461iV^{.71} + 6530iG_S^{.71}$	$53.9w_{ma}G_S^{.6}$	$121w_{op}G_S^{.55} + 65P_C HP$
Settlers and thickener	$824iA^{.77}$	$9.32w_{ma}A^{.6} + 8.62iA^{.76}$	$17.1w_{op}A^{.6}$
Sludge pumps	$9870iQ^{.53}$	$112w_{ma}Q^{.43} + 214iQ^{.64}$	$257w_{op}Q^{.41} + 65P_C W$
Water pumps	$1710iQ^{.53}$	$.0951w_{ma}Q + 6.11iQ^{.8}$	$.133w_{op}Q + 65P_C W$

A is the surface area in m^2, Q is the flow in m^3/s, V is the volume in m^3, G_s is the air flow rate in m^3/s, HP is the installed power for aeration in kW, w_{ma} and w_{op} are wage parameters for maintenance and operation, P_C is the cost of energy cents/kWh, W is the installed power for pumping in kW, and i is a cost index.

Pincince et al. (1997) studied the effect of MLSS on total capital cost of activated sludge processes. They developed a capital cost equations for aeration tank as a function of its volume and for secondary clarifier as a function of its surface area. In the same year, Fels et al. (1997) studied the design optimization of wastewater treatment systems for the mechanical pulp and paper mill industry. They developed cost relations based on data obtained from the industry. As a consequence, their cost relations are applicable to the pulp and paper mill industry and generalizing them involves considerable errors.

Asfari (2000) listed cost functions of most of treatment unit operations. He acquired them from a study conducted by the Department of the Army, Corps of Engineers, USA in 1971. He modified the costs to 1995 costs assuming a 5% increase in cost.

Vanrolleghem et al. (1996) discussed a general framework for the formulation and analysis of an overall decision support performance index (cost function) to aid in evaluating different design alternatives and operation schemes. This issue was first addressed by COST682 Working Group in 1994 when they introduced for the first time an overall decision support index based on economic cost functions for different aspects of treatment plant construction, maintenance and operation, and including internalization of the value of a river's quality. They also mentioned the importance of including more elaborate aspects of wastewater treatment such as plant flexibility and robustness against failure. The latter aspects play a special role as the time

horizon over which the cost evaluation is made increases towards the life span of a treatment works. The objective performance index (J) is described as

$$J = Input - Output + Conversion \qquad (5.1)$$

Where the Input term is the quality/value associated with incoming wastewater, the output term is the quality/value of the effluent wastewater, solids, gases, and energy produced at the wastewater treatment plant, and the conversion term takes the applied efforts (investment and operational costs) into account. This approach is an innovative approach and researchers are requested to focus on it.

In summary, three main streams regarding the costing of wastewater treatment plants can be distinguished in the literature. The first is efforts to develop capital cost relations and can be used in the preliminary cost estimation of new wastewater treatment plants (e.g. the EPA report). The second is the development of cost relationships (functions) relating the cost (investment and operation) to a sizing/operation parameter. Such cost relations were used primarily for optimization studies. The third is the innovative development of a new performance index that incorporates in addition to traditional investment and operation criteria aspects such as flexibility and reliability. Lack of studies in this stream can be noticed although it is very important and could replace traditional cost functions in optimization studies. As a consequence, in the coming discussion the focus will be on the second stream since such is the one related to optimization studies.

5.3 Cost Functions and Cost Indices

Different ways of expressing costs of treatment plants are found in literature. The general form of cost functions is (Tyteca, 1985):

$$C = \sum_{i=1}^{n} a_i X_i^{b_i} \qquad (5.2)$$

in which C = the total capital or operation and maintenance cost of a given treatment unit (in \$ or \$ per year), X_i = the design and/or operating parameters of that unit which most significantly influence the cost (e.g. Volume, area, etc.), a_i and b_i = estimated parameters, and n = number of terms in the sum.

Equation (5.2) can be expressed in another simpler way (Vanrolleghem et al., 1996):

$$COST = \theta \, (Process \, Size)^n \tag{5.3}$$

The process size is typically chosen to be relevant and easy to measure plant characteristics such as volume or area of a process unit, the design flow rate, installed mechanical power or pumping capacity. The constant n depends on the process unit and ranges between 0.25 and 1 (Wright and Woods, 1993, and 1994).

A crucial point in developing cost functions is data collection. Two possibilities are noticed in literature; data collected based on certain type of treatment at a given location; and data collected to represent as much as possible the cost of a treatment unit in all treatment configurations. The first is more accurate although it is applicable for the situation it is developed for and extending its use to other applications results in high level of errors. The second is more general with less accuracy and can be applied successfully for preliminary estimates. Example of the first type is the work of Fels et al. (1997) where they developed cost relationships for wastewater treatment systems in the mechanical pulp and paper industry.

After collecting a set of cost data, the development of a cost function as Equation (5.2) involves determining the design or operating parameter of the unit considered (X) and the estimation of the parameters (a_i and b_i) through regression analysis.

Since the second approach is general it can be used after years of its development and in places other than the place it is developed in. This usually accomplished with the aid of cost indices. Cost indices relate costs at one time and place to costs at any other time and/or place. For example, if a project were estimated to cost $100,000 in 1982 using an index of 2233, that same project would cost 6580/2233 multiplied by $100,000 in 2002 when the cost index rises to 6580. Geographical adjustments may also be necessary.

Commonly used indices in the US are the U.S. Environmental Protection Agency (EPA) Sewage Treatment Plant (EPA-STP) Cost Index and the Engineering News Record (ENR) Indices (Corps of Engineers, 1987). Other indices include the Marshall and Swift cost index (used primarily in the process industry), Southam Construction Cost Index (developed for Canadian construction conditions), and

Chemical Engineering plant cost index (used primarily in the process industry) (Wright and Woods, 1993).

The overall cost of a treatment configuration will be the summation of the costs of different unit operations considered. The total cost could be expressed either using the present worth method (NPV) or the equivalent annual worth (AW). Using the first, all annual operating costs for each process are converted into their corresponding present value and added to the investment cost of each process to yield the net present value. While the AW implies discounting the investment cost over the life span of the treatment plant and add it to the annual operating cost. Formulation of cost functions as an objective function in the optimization model is given in the subsequent chapter, Chapter 6.

In conclusion, considerable variations in unit process costs were observed among different sources of data. Costs of wastewater treatment systems vary locally and depend on many factors (Tang et al., 1984). Cost functions are indeed developed at a given time for a specific purpose, region, or country and any extrapolation is not without risk. Moreover it is difficult to compare various relationships extracted from different sources, as the description of the components taken into account in the relationships often differs from a relationship to another. Therefore the cost functions considered in this study are only meaningful in the sense that they represent typical relative costs among unit processes. This is justified since the main purpose of the study is not cost estimation but optimization of a typical treatment system where relative costs of different incorporated units is the important not the cost itself as a figure. Most of the optimization studies developed cost functions based on Patterson and Banker (1971) cost estimations by correcting them using the appropriate cost index. The cost functions utilized in this study are derived from the cost functions of Tang et al. (1984) and the cost functions of Tyteca (1985) (Table 5.1 and Table 5.2) after being corrected using the Engineering News Record construction cost index of 2003. Such are listed in the subsequent chapter, Chapter 6.

Obviously, the selected cost functions are not representing the costs of the treatment units in the United Arab Emirates. Such costs can be obtained by developing specific cost functions for the UAE or at least a Cost Index that can relate other costs to costs in the UAE.

CHAPTER 6

MODEL DEVELOPEMENT

Activated sludge systems, as mentioned in previous chapters, constitute two main units, one for biological treatment and the other for sedimentation, namely, aeration tank and the secondary (final) settling tank. In most activated sludge treatment plants, especially conventional and complete mix activated sludge plants, a primary sedimentation unit is installed before the activated sludge system. It is mentioned previously that primary sedimentation is of special importance and contribute significantly in the determination of the total cost of a treatment plant. The use of primary clarifiers is an economic issue, not a process issue (Grady et al., 1999). As a consequence it is considered along with the activated sludge system in this study. The proposed optimal design approach considers the interaction among primary sedimentation, biological treatment, and final sedimentation in terms of process continuity and costs. Such interaction is crucial to obtain an optimal design.

In this chapter, the proposed optimal design approach is introduced mathematically and the problem of optimization is formulated in terms of constraints and the objective cost function.

6.1   System Layout

Figure 6.1 shows a typical layout of an activated sludge system. It includes a primary clarifier and a complete mix activated sludge system, which constitutes an aeration tank and a final settler. All the streams are numbered to facilitate the description of the model. Stream 1 represents the influent while 4 is the effluent. Streams 2 and 3 connect primary clarifier to aeration tank and aeration tank to secondary settler, respectively. Stream 5 is the underflow from the secondary settler which is divided into stream 6 (recirculation of sludge from final settler to aeration tank) and stream 7

which represents along with stream 8 the wastage sludge streams that might be subject for further sludge treatment or disposal according to applicable legislation.

6.2 System Components

The complete description of the system requires the specification of two groups of symbols:

1) Parameters: those quantities that remain constant in the design. Examples are the kinetic and stoichiometric parameters in the activated sludge process, the cost index value, sedimentation constants, etc. A complete list of the parameters used in the system design and optimization is provided in Table 6.8. The stoichiometric and kinetic parameters considered in ASM3 model are shown alone in Table 6.3 and Table 6.5, respectively, and not shown in Table 6.8.
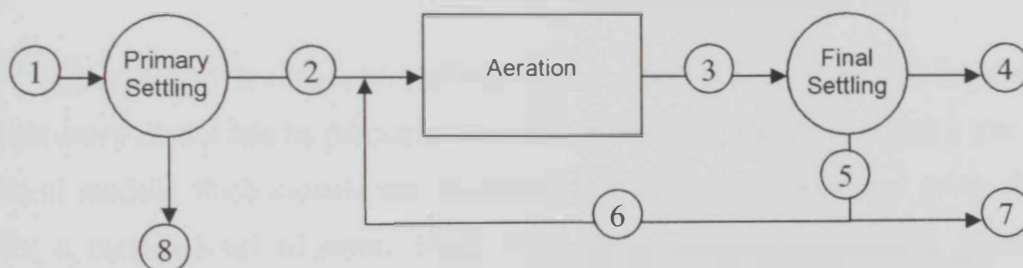
Figure 6.1: System Layout

2) Variables: those quantities which are determined during the design and optimization. They specify the dimensions or the design condition of a unit process or a stream in the model. They represent the wastewater characteristics at a particular stage during the treatment process, and they are defined at the eight control points shown in Figure 6.1. Later on, when we get to solve the model, these variables are separated into decision variables and state variables. Decision variables are needed to be defined to solve the system. Table 6.1 lists state variables that are defined at every stream along with their units. Table 6.9 lists all other variables.

6.3   Model Formulation

As mentioned in the previous chapters, several models exist to describe the operation of primary clarifier, aeration tank, and secondary clarifier. In Chapters 2, 3, and 4 such models were discussed and one of them that best fits the problem under consideration was chosen to be used in this study.

Table 6.1: Model state variables

Symbol	Characterization
Q_j	Flow rate, $[L^3 \, T^{-1}]$
S_{Ij}	Inert soluble organic matter, $[M(COD) \, L^{-3}]$
S_{Sj}	Readily biodegradable substrate, $[M(COD) \, L^{-3}]$
S_{NH4j}	Ammonium plus Ammonia nitrogen, $[M(N) \, L^{-3}]$
S_{NOxj}	Nitrate plus nitrite nitrogen, $[M(N) \, L^{-3}]$
X_{Ij}	Inert particulate organic matter, $[M(COD) \, L^{-3}]$
X_{Sj}	Slowly biodegradable substrate, $[M(COD) \, L^{-3}]$
X_{Hj}	Heterotrophic organisms, $[M(COD) \, L^{-3}]$
X_{STOj}	A cell internal storage product of heterotrophic organism, $[M(COD) \, L^{-3}]$
X_{Aj}	Nitrifying organisms, $[M(COD) \, L^{-3}]$
X_{SSj}	Suspended solids, $[M(SS) \, L^{-3}]$

$j = 1, 2, \ldots, 8$ "control point"

Selected models cannot be claimed to be the best representation of the reality because every model has its particularities and restrictions. This is especially true for empirical models. Such models are developed for certain situations and using them implies a certain level of error. Using them for a particular treatment plant of wastewater requires calibrating their parameters to represent the wastewater/plant under study.

The main objective of this study is to explore the problem of optimizing activated sludge process and not to treat a special case for a certain wastewater at a certain treatment plant. The models chosen are only examples of how the models can be incorporated in such type of problems. They were chosen to best suit the problem and they are recent, simple, and understandable. To use the optimization framework introduced in this study for certain plant or design, models then should be chosen based on the best representation of the wastewater/plant in question. Sometimes developing a specific model would be an option.

In this chapter, the comprehensive system model is introduced. This includes defining the state variables in every stream and the relations variables that define the system (i.e., mathematical models describing the operations).

Stream 1 is the influent to the system. The first unit operation is the primary clarifier. Streams 2 and 8 are the primary effluent and the primary sludge streams, respectively. Values of state variables in those two streams depend on the performance of the primary clarifier, which is described by two mathematical models as follows.

6.3.1 Primary Clarifier

The primary effluent suspended solids concentration (X_{SS2}) is modeled according to the Christoulas et al. (1998) model for clarification in primary clarifiers (see Chapter 2).

$$\frac{X_{SS2}}{X_{SS1}} = 1 - [a\exp(\frac{-b}{X_{SS1}} - cq)] \tag{6.1}$$

Where a, b (mg/L), and c (d/m) are positive parameters. The value of a and b are found to be related to temperature while c is assumed a constant value. q (m/d) is the overflow rate which is defined as:

$$q = \frac{Q_2}{A_p} \tag{6.2}$$

Where A_p is the surface area of primary clarifier (m^2) and Q_2 is primary effluent flow rate (m^3/d).

The concentration of individual solids components is calculated based on the assumption that the portion of each solid component in the primary effluent suspended solids concentration (X_{SS2}) is the same as the portion that component occupies in the influent suspended solids concentration (X_{SS1}). i.e., the solids distribution remains the same as the influent while the quantities become less due to primary sedimentation:

$$X_{j2} = X_{j1}\frac{X_{SS2}}{X_{SS1}} \qquad j = I, S, H, STO, A \tag{6.3}$$

The soluble components are assumed unaffected by primary sedimentation:

$$S_{j2} = S_{j1} \qquad\qquad j = I, S, NH4, NOX \qquad\qquad\qquad (6.4)$$

The underflow solids concentration is calculated according to the solids flux theory as given by Cho et al. (1996):

$$X_{ss8}(g/L) = [k(n-1)]^{1/n}\left(\frac{n}{n-1}\right)\left(\frac{A_p}{Q_8}\right)^{1/n} \qquad\qquad (6.5)$$

Where k (m/d) and n are settling constants of primary sludge and their ranges are (65 – 460 m/d) and (1 – 5), respectively. A_p and Q_8 are in m^2 and m^3/d, respectively. The concentrations of the solids and soluble components in the underflow are calculated as the solids and soluble components have been calculated in the primary effluent.

$$X_{j8} = X_{j1}\frac{X_{ss8}}{X_{ss1}} \qquad\qquad j = I, S, H, STO, A \qquad\qquad\qquad (6.6)$$

$$S_{j8} = S_{j1} \qquad\qquad j = I, S, NH4, NOX \qquad\qquad\qquad (6.7)$$

The flow and mass balance relationships around the primary clarifier are

$$Q_1 = Q_2 + Q_8 \qquad\qquad\qquad\qquad\qquad (6.8)$$

$$Q_1 X_{SS1} = Q_2 X_{SS2} + Q_8 X_{SS8} \qquad\qquad\qquad\qquad (6.9)$$

6.3.2 Activated Sludge

The activated sludge process comprises several operations: aerobic waste stabilization in the aeration tank, clarification of the aeration tank effluent and sludge concentration in the secondary clarifier, and recycle of the thickened sludge to the aeration tank to maintain the microbial population (Figure 6.1).

In Chapter 3, a review of various kinetic models for the design of activated sludge biological process is given. Among the introduced models, ASM3 model has been chosen as the basis for the design of activated sludge process in the introduced optimal approach. ASM3 is the latest advanced model developed by the IWA where it corrects for many defects noticed in the widely acceptable model, ASM1. However, it is noticed in the literature that because of its complexity, ASM3 is rarely utilized in its full version and in most cases a reduced version is developed and adopted. In this study, a reduced ASM3 based model is considered. Such model considers the ASM3 model after applying the following restrictions:

(1) The reactor is considered totally aerobic bioreactor where oxygen is controlled all the time to be ≥ 2 mg/L. Consequently, all anoxic reactions are neglected and the oxygen dynamics are not taken into account (Koch et al., 2001 and Steffens et al., 1997).

(2) Alkalinity dynamics is neglected and hence the state variable describing the total alkalinity is excluded (Jeppsson, 1996 and Chachuat et al., 2001). This assumption is reasonable since the effect of alkalinity on other reactions in the process is minor (see stoichiometric coefficients in Table 3.5)

The resulting model consists of 10 state variables. Table 6.2 shows the processes incorporated in the reduced model along with their kinetic relations. As discussed in previous chapters, ASM3 is introduced originally in the form of a stoichiometric and composition matrix, which is reduced to stoichiometric matrix based on suggested values for the stoichiometric and composition parameters appearing in the original matrix. The original stoichiometric and composition matrix is not presented here. Table 6.3 shows the typical values for the stoichiometric and composition parameters as suggested by Henze et al. (2000) to produce the stoichiometric matrix of the ASM3 model, which is shown in Table 6.4. Table 6.2 and Table 6.4 are only shown for the reduced model.

Table 6.2: Processes in the developed reduced ASM3 based model and their kinetic expressions

j Process	Process rate equation ρ_j, all $\rho_j \geq 0$.
1 Hydrolysis	$k_H \cdot \dfrac{X_S/X_H}{K_X + X_S/X_H} \cdot X_H$

Heterotrophic organisms, aerobic and denitrifying activity

2 Aerobic storage of S_S	$k_{STO} \cdot \dfrac{S_S}{K_S + S_S} \cdot X_H$
3 Aerobic growth	$\mu_H \cdot \dfrac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \cdot \dfrac{X_{STO}/X_H}{K_{STO} + X_{STO}/X_H} \cdot X_H$
4 Aerobic endogenous respiration	$b_{H,O_2} \cdot X_H$
5 Aerobic respiration of X_{STO}	$b_{STO,O_2} \cdot X_{STO}$

Autotrophic organisms, nitrifying activity

6 Aerobic growth of X_A, nitrification	$\mu_A \cdot \dfrac{S_{NH_4}}{K_{A,NH_4} + S_{NH_4}} \cdot X_A$
7 Aerobic endogenous respiration	$b_{A,O_2} \cdot X_A$

Table 6.3: Typical stoichiometric and composition parameters for ASM3 (Henze et al., 2000)

Symbol	Characterization	Value	Units
f_{SI}	Production of S_I in hydrolysis	0	g COD_{SI} (g COD_{XS})$^{-1}$
$Y_{STO,O2}$	Aerobic yield of stored product per S_S	0.85	g COD_{XSTO} (g COD_{SS})$^{-1}$
$Y_{STO,NOX}$	Anoxic yield of stored product per S_S	0.80	g COD_{XSTO} (g COD_{SS})$^{-1}$
$Y_{H,O2}$	Aerobic yield of heterotrophic biomass	0.63	g COD_{XH} (g COD_{XSTO})$^{-1}$
$Y_{H,NOX}$	Anoxic yield of heterotrophic biomass	0.54	g COD_{XH} (g COD_{XSTO})$^{-1}$
Y_A	Yield of autotrophic biomass per NO_3-N	0.24	g COD_{XA} (g N_{SNOX})$^{-1}$
f_{XI}	Production of X_I in endog. Respiration	0.20	g COD_{XI} (g COD_{XBM})$^{-1}$
$i_{N,SI}$	N content of S_I	0.01	g N (g COD_{SI})$^{-1}$
$i_{N,SS}$	N content of S_S	0.03	g N (g COD_{SS})$^{-1}$
$i_{N,XI}$	N content of X_I	0.02	g N (g COD_{XI})$^{-1}$
$i_{N,XS}$	N content of X_S	0.04	g N (g COD_{XS})$^{-1}$
$i_{N,BM}$	N content of biomass, X_H, X_A	0.07	g N (g COD_{XBM})$^{-1}$
$i_{SS,XI}$	SS to COD ratio for X_I	0.75	g SS (g COD_{XI})$^{-1}$
$i_{SS,XS}$	SS to COD ratio for X_S	0.75	g SS (g COD_{XS})$^{-1}$
$i_{SS,BM}$	SS to COD ratio for biomass, X_H, X_A	0.90	g SS (g COD_{XBM})$^{-1}$

Comparing the reduced model tables to the original full model tables (Tables 3.4 and 3.5) show obviously the following: the anoxic processes are eliminated, the terms describing the dynamics of oxygen and alkalinity are excluded from all reactions, and since no denitrification (anoxic process) occurs the state variables describing the nitrogen concentration are also excluded.

Finding the conversion rate of a component depends on Tables 6.2 and 6.4. As discussed previously, the conversion rate of a component can be written as follows:

$$r_i = \sum_{j=1}^{7} \psi_{ij} \rho_j \qquad (6.10)$$

Where r_i is the conversion rate expression for the component i, ψ_{ij} is the stoichiometric coefficient as shown in Table 6.4, and ρ_j is the kinetic expression for the process j as shown in Table 6.2.

As an example, the rate conversion expression of the S_S can be written as:

$$r_S = 1 \times \rho_1 - 1 \times \rho_2 = k_H \frac{X_S / X_H}{K_H + X_S / X_H} X_H - k_{STO} \frac{S_S}{K_S + S_S} X_H \qquad (6.11)$$

In similar way the rate expressions for all the components can be written. One major issue is characterizing of wastewater by defining the kinetic parameters appearing in Table 6.2. Such values should be defined for the wastewater under consideration through standard methods given in literature considering environmental conditions

such as temperature and pH. Theoretically, kinetic parameters are determined at neutral pH while temperature effect can be approximated.

Table 6.4: Stoichiometric matrix of the developed reduced ASM3 based model

Component → 1	1	2	3	4	5	6	7	8	9	10
j Process	S_I	S_S	S_{NH4}	S_{NOX}	X_I	X_S	X_H	X_{STO}	X_A	X_{SS}
↓ Expressed as →	COD	COD	N	N	COD	COD	COD	COD	COD	SS
1 Hydrolysis	0	1	0.01			-1				-0.75
Heterotrophic organisms, aerobic and denitrifying activity										
2 Aerobic storage of S_S		-1	0.03					0.85		0.51
3 Aerobic growth of X_H			-0.07				1	-1.60		-0.06
4 Aerobic endog. Respiration			0.066		0.2		-1			-0.75
5 Aerobic respiration of X_{STO}								-1		-0.60
Autotrophic organisms, nitrifying activity										
6 Aerobic growth of X_A, nitrific.			-4.24	4.17					1	0.90
7 Aerobic endog. Respiration			0.066		0.2				-1	-0.75

A common temperature adjustment technique for kinetic parameters characterizing the activated sludge is the following equation (Grady et al., 1999):

$$k_1 = k_2 \cdot \theta^{(T_1 - T_2)} \tag{6.12}$$

Where k represents any kinetic parameter. Generally, the reference temperature, T_2, is 20°C. θ is the temperature coefficient and its value depends on the kinetic parameter being adjusted.

Henze et al. (2000) recommended interpolating kinetic parameters k to different temperatures T (in °C) with the following temperature equation:

$$k(T) = k(20°C) \cdot \exp\left[\theta_T \cdot (T - 20°C)\right] \tag{6.13}$$

where θ_T (in °C) may be obtained from:

$$\theta_T = \frac{\ln(k(T_1) / k(T_2))}{T_1 - T_2} \tag{6.14}$$

Table 6.5 shows typical values of kinetic parameters of ASM3 at 10 and 20°C (Henze et al., 2000). It is obvious from the table that some parameters are unaffected by temperature change. Using the approach of Henze et al. (2000) that was mentioned above, values at different temperatures have been derived as shown in Table 6.6. Obviously the effect of temperature change differs from one parameter to another. It is evident in some parameters more than others and maximum for μ_A and $b_{A,O2}$.

Table 6.5: Typical values of kinetic parameters of ASM3 as given by Henze et al. (2000)

Symbol	Characterization	Temperature 10°C	20°C	Units
k_H	Hydrolysis rate constant	2	3	g COD_{XS} (g COD_{XH})$^{-1}$d^{-1}
K_X	Hydrolysis saturation constant	1	1	g COD_{XS} (g COD_{XH})$^{-1}$
k_{STO}	Storage rate constant	2.5	5	g COD_{SS} (g COD_{XH})$^{-1}$d^{-1}
K_S	Saturation constant for substrate S_S	2	2	g COD_{SS} m^{-3}
K_{STO}	Saturation constant for X_{STO}	1	1	g COD_{XSTO} (g COD_{XH})$^{-1}$
μ_H	Heterotrophic max. growth rate of X_H	1	2	d^{-1}
K_{NH4}	Saturation constant for ammonium, S_{NH4}	0.01	0.01	g N m^{-3}
$b_{H,O2}$	Aerobic endogenous respiration rate of X_H	0.1	0.2	d^{-1}
$b_{STO,O2}$	Aerobic respiration rate for X_{STO}	0.1	0.2	d^{-1}
μ_A	Autotrophic max. growth rate of X_A	0.35	1	d^{-1}
$K_{A,NH4}$	Ammonium substrate saturation for X_A	1	1	g N m^{-3}
$b_{A,O2}$	Aerobic endogenous respiration rate of X_A	0.05	0.15	d^{-1}

Table 6.6: Values of kinetic parameters at different temperatures

Symbol	Characterization	Temperature 30°C	40°C	50°C	Units
k_H	Hydrolysis rate constant	4.50	6.75	10.13	g COD_{XS} (g COD_{XH})$^{-1}$d^{-1}
K_X	Hydrolysis saturation constant	1.00	1.00	1.00	g COD_{XS} (g COD_{XH})$^{-1}$
k_{STO}	Storage rate constant	10.00	20.00	40.00	g COD_{SS} (g COD_{XH})$^{-1}$d^{-1}
K_S	Saturation constant for substrate S_S	2.00	2.00	2.00	g COD_{SS} m^{-3}
K_{STO}	Saturation constant for X_{STO}	1.00	1.00	1.00	g COD_{XSTO} (g COD_{XH})$^{-1}$
μ_H	Heterotrophic max. growth rate of X_H	4.00	8.00	16.00	d^{-1}
K_{NH4}	Saturation constant for ammonium, S_{NH4}	0.01	0.01	0.01	g N m^{-3}
$b_{H,O2}$	Aerobic endogenous respiration rate of X_H	0.40	0.80	1.60	d^{-1}
$b_{STO,O2}$	Aerobic respiration rate for X_{STO}	0.40	0.80	1.60	d^{-1}
μ_A	Autotrophic max. growth rate of X_A	2.86	8.16	23.32	d^{-1}
$K_{A,NH4}$	Ammonium substrate saturation for X_A	1.00	1.00	1.00	g N m^{-3}
$b_{A,O2}$	Aerobic endogenous respiration rate of X_A	0.45	1.35	4.05	d^{-1}

To calculate the state variables around the aeration tank, steady state mass balances of all state variables should be considered. The mass balance of state variables around the aeration tank and secondary settling tank can be written as follows (see Figure 6.1):

$$(dx_i/dt)V = Q_2. x_{i2} - [Q_4. x_{i4} + Q_7. x_{i7}] + r_{xi} V = 0 \tag{6.15}$$

Where x_i is a vector that components are the state variables [S_I, S_S, S_{NH4}, S_{NOX}, X_I, X_S, X_H, X_{STO}, X_4, X_{SS}]. r_{xi} is the conversion rate of the component x_i and can be calculated as shown previously (see Equations 6.10 and 6.11).

The *SRT* is the most important variable in the system. By definition, it is the mass of organisms in the reactor divided by the mass of organisms removed from the system each day (Metcalf and Eddy, 1991). Commonly the mass of organisms is approximated to the concentration of the VSS. A slight modification is suggested on this common definition that is to relate the sludge age to the concentration of heterotrophic biomass instead of the VSS. This is because there is no variable in the model to quantify the VSS while there is one to quantify the heterotrophic biomass (X_H). Then the *SRT* is defined in the model as:

$$SRT = V. X_{H3}/[Q_7. X_{H7} + Q_4. X_{H4}] \tag{6.16}$$

Other important variables are defined as follows. The *HRT* is defined as:

$$HRT = \frac{V}{Q_2} \tag{6.17}$$

the recycle ratio *r* as:

$$r = \frac{Q_6}{Q_2} \tag{6.18}$$

and the wasting ratio as:

$$w = \frac{Q_7}{Q_2} \tag{6.19}$$

The total oxygen requirement is the sum of the oxygen requirement for the removal of organic matter plus the oxygen requirement associated with nitrification. The oxygen requirement for removal of organic matter can be calculated with the following equation (see Table 6.8 and Table 6.9 for definition of parameters and variables):

$$RO_H = Q_2(S_{S2} + X_{S2} - S_{S3})\left[1 - \frac{(1 + f_{XI} \cdot b_H \cdot \theta_c)Y_H}{1 + b_H \theta_c}\right] \tag{6.20}$$

While the oxygen requirement associated with nitrification is given through the following equation:

$$RO_A = Q_2(S_{NH_42} + S_{NOX2} - S_{NH_43}) \left[4.57 - \frac{(1 + f_{XI} \cdot b_A \cdot \theta_c)Y_A}{1 + b_A\theta_c} \right]$$
(6.21)

For diffused air systems, the process air requirement can be calculated from the following dimensional expression (Grady et al, 1999):

$$AFR = \frac{6.0(RO_H + RO_A)}{n_e}$$
(6.22)

where AFR is the air flow rate in m^3/min, $(RO_H + RO_A)$ is the total oxygen requirement in kg/h, and n_e is the field oxygen transfer efficiency expressed as the percent of the oxygen in the air actually transferred to the liquid. The value of n_e depends on the nature of the diffuser and the depth at which the air is released. It typically lies in the range of 6 to 15% with 10% as an average value.

In activated sludge systems, for economic reasons, the equipment used to transfer oxygen also provides the turbulence necessary to maintain solids in suspension. This results in constraints on process design and operation. The upper feasible bioreactor volume (in m^3) can be related to air flow rate and the minimum air input rate (AIR) as follows:

$$V \le \frac{1000AFR}{AIR_L}$$
(6.23)

where AIR_L value depends on the type of diffusers used. The value of 20 m^3/(min·1000 m^3) is generally applied.

The lower feasible bioreactor volume can be given as:

$$V \ge \frac{1000AFR}{AIR_U}$$
(6.24)

where for diffused aeration systems AIR_U can be approximated to 90 m^3/(min·1000 m^3). For the types of oxygen transfer systems typically used today, the maximum volumetric oxygen transfer rate that can be achieved economically on a sustainable basis is around 0.10 kg O_2/(m^3.h). This imposes another constraint on the volume of

the bioreactor. The lower limit on bioreactor volume based on oxygen transfer can be expressed as follows:

$$V \geq \frac{(RO_H + RO_A)}{0.10 \; [kg \; O_2 \, /(m^3 \cdot h)]}$$

(6.25)

The abovementioned design factors are the main design elements that affect the cost of an activated sludge system. There is other design details should be considered when designing a full system. The purpose of this formulation is to find an optimal sizing of the plant, so that only elements contribute to cost and related to sizing are considered.

The next and final unit operation in the system considered is the secondary sedimentation tank which will be the subject of the subsequent section.

6.3.3 Secondary Sedimentation

Influent of the secondary sedimentation tank (as shown in Figure 6.1) is the effluent of the aeration tank (stream 3) while its effluent is composed of two streams (streams 4 and 5). Stream 4 represents the clarified effluent while stream 5 is the thickened sludge to be circulated to the aeration tank after wasting a portion of it. As mentioned previously, the secondary sedimentation tank performs two functions, clarification and thickening. Clarification is modeled according to Voutchkov (1992) where the effluent suspended solid concentration (X_{SS4}) is given as follows (see Chapter 4):

$$X_{SS4}(mg \, / \, L) = \frac{6.21 \cdot \ln(MLSS \cdot SVI)}{0.67 \cdot \ln(H) - \ln(SR)} - 26.43$$

(6.26)

Where $MLSS$ is equal to X_{SS3} (g/L), SVI is in (mL/g), H is the side water depth in the settling tank (m), and SR is the surface overflow rate (m/h) which is equal to Q_4/A_f. According to Metcalf and Eddy (1991), an SVI greater than 200 ml/g indicates poor settling. For the side water depth, current practice favors a minimum side-water depth of 3.7 m and depths ranging up to 6.1 m have been used. It should be noted, however, that in some cases tanks with relatively shallow side-water depths have been used successfully. For the overflow rate, the typical range is $16 - 32$ m^3/m^2.d (Metcalf and Eddy, 1991).

On the other hand, thickening function is modeled according to the solids flux theory as given by Cho et al. (1996). Cho et al. (1996) suggested that the underflow solids concentration can be calculated as follows:

$$X_{SS5}(g/L) = [k_w(n_w-1)]^{1/n_w} \left(\frac{n_w}{n_w-1}\right)\left(\frac{A_f}{Q_5}\right)^{1/n_w} \tag{6.27}$$

Where k_w (m/d) and n_w are constants representing thickening properties of the waste activated sludge. They are analogous to the settling constants shown in Equation (6.5) and the ranges shown there are applicable here. A_f (surface area of final settler) and Q_5 are in m^2 and m^3/d, respectively.

The ratios between individual solids component and the TSS are assumed to be unaffected by secondary sedimentation. In other words,

$$X_{jk} = X_{j3}\frac{X_{SSk}}{X_{SS3}} \qquad j= I, S, H, STO, A \text{ and } k= 4,5,6,7 \tag{6.28}$$

The soluble components are also assumed to be unchanged through sedimentation and sludge separation. In other words,

$$S_{jk} = S_{j3} \qquad j = I, S, NH4, NOX \text{ and } k= 4,5,6,7 \tag{6.29}$$

The flow and mass balance relationships around the secondary clarifier are,

$$Q_3 = Q_4 + Q_5 \tag{6.30}$$

$$Q_3X_{SS3} = Q_4X_{SS4} + Q_5X_{SS5} \tag{6.31}$$

Stream 4 is the effluent from the system and several components are of concern. Total COD, TSS, and ammonia nitrogen are the most important pollutants monitored in the effluent of wastewater treatment plants. Consequently, effluent constraints are applied on these components and they are discussed in more detail in the following chapter.

6.3.4 Cost Functions

The total cost of the wastewater treatment system is the sum of the costs of all unit processes plus the costs associated with pumping flow between these units. As mentioned previously in Chapter 5, the cost functions utilized in this study are derived from the functions developed by Tang et al. (1984) and the ones introduced by Tyteca (1985). Table 6.7 summarizes these cost functions. As an example, the cost of the

primary clarifier is the summation of its capital cost, operation costs, maintenance costs, and material and supply costs.

The total annual cost in 2003 dollars is used to express the total system cost. Since the capital cost is expressed as a lump sum, a design life and a discount rate are to be assumed to amortize the capital costs. The Engineering News Record construction cost index of 2003 can be used to update the capital costs and the costs for material and supply from the base year they developed in to the year of study. Annual operation and maintenance costs are calculated by multiplying the man-hour requirement by the hourly wage rates. The cost for pumping is the product of the power requirement and the unit power cost.

Table 6.7: Summary of cost functions utilized in the study

Process Unit	Capital (1971$)	Operation (manhours/yr)	Maintenance (manhours/yr)	Material and Supply (1971$/yr)	Power (kW hr/yr)
Primary Clarifier†	$824A^{77}$	$17.1A^{06}$	$9.23A^{06}$	$8.62A^{76}$	–
Primary Sludge Pumping†	$9870Q^{53}$	$257Q^{41}$	$112Q^{43}$	$214Q^{64}$	$23.85QH/\varepsilon_p$
Aeration Tank††	$461V^{71}$	–	–	–	–
Diffused Aeration††	$8533Q_a^{66}$	$187Q_a^{48}$	$74.4Q_a^{55}$	–	–
Secondary Clarifier†	$824A^{77}$	$17.1A^{06}$	$9.23A^{06}$	$8.62A^{76}$	–
Return & Waste Sludge Pumping†	$9870Q^{53}$	$257Q^{41}$	$112Q^{43}$	$214Q^{64}$	$23.85QH/\varepsilon_p$

A is the surface area in m^2, Q is the flow in m^3/hr, V is the volume in m^3, Q_a is the air flow rate in m^3/min, H is the pumping head in meters, and ε_p is the pumping efficiency.

† Tyteca (1985).

†† Tang et al. (1984).

As can be noticed from the above, several constants appear in the models chosen to represent a unit process or another. The reader is reminded that the values shown are just chosen for the purpose of illustration and should he/she use such models, these constants should be determined for the specific situation under study. This is applicable to all parameters shown above which are summarized in Tables 6.3, 6.5 and 6.8. These three tables show all the parameters in the system. In contrast, Table 6.1 along with Table 6.9 summarize the variables in the above description. The following section presents the model in the form of an optimization problem, while

the next introduces an optimization algorithm that is used to solve this optimization problem.

Table 6.8: Summary of parameters in the model other than ASM3 kinetic and stoichiometric parameters

Symbol	Characterization	Value/range	Units
a	Constant in Christoulas model for primary clarification	$1.71-0.03T$	-
b	Constant in Christoulas model for primary clarification	$683.6-21.13T$	mg/L
c	Constant in Christoulas model for primary clarification	0.0035	d/m
k	Settling constant of primary sludge	65-460	m/d
n	Settling constant of primary sludge	1-5	-
kw	Settling constant of wasting sludge	65-460	m/d
nw	Settling constant of wasting sludge	1-5	-
SVI	Sludge Volume Index of sludge	<200	ml/g
H	Side water depth of final clarifier	>3.1	m
ne	Efficiency depends on diffuser and depth at which air pumped	6-15%	
AIR_U	Maximum air input rate	90	$m^3/(min.1000\ m^3)$
AIR_L	Minimum air input rate	20	$m^3/(min.1000\ m^3)$
CRF	Capital Recovery factor	0.0944	
BCI	Base (1971) Cost Index	1581	$
CI	Cost Index for 2003	6581	$
OMW	Operating Maintenance Wages	8.3	$ per hour
EC	Electricity Cost	0.05	$ per kWh
PH	Pumping Head	10.0	meters
PE	Pumping Efficiency	0.6	

Table 6.9: Summary of variables in the model other than state variables

Symbol	Characterization	Units
q	overflow rate of primary settling tank	m/d
A_p	surface area of the primary clarifier	m^2
SRT	sludge retention time	d
HRT	hydraulic retention time	d
V	volume of the aeration tank	m^3
r	sludge recycle ratio	-
w	wastage ratio	-
SR	overflow rate of final clarifier	m/d
A_f	surface area of the final clarifier	m^2
RO_H	Oxygen Requirement for removal of organic matter	kg/d
RO_A	Oxygen Requirement associated with nitrification	kg/d
AFR	Air flow rate	m^3/min

6.4 Optimization Problem

It is well known that any optimization problem is composed of objective function, constraints, and bounds on variables. The following is a mathematical formulation of the optimization problem in this study.

Objective function:

minimize $\quad\quad \sum total\ cost$ $\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad$ (6.32)

subject to

(1) Primary clarifier

$$\frac{X_{SS2}}{X_{SS1}} = 1 - [a \exp(\frac{-b}{X_{SS1}} - cq)] \quad\quad\quad\quad\quad\quad (6.33)$$

$$q = \frac{Q_2}{A_p} \quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad (6.34)$$

$$X_{SS8}(g/L) = [k(n-1)]^{1/n} \left(\frac{n}{n-1}\right) \left(\frac{A_p}{Q_8}\right)^{1/n} \quad\quad (6.35)$$

$Q_1 = Q_2 + Q_8$ $\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad (6.36)$

$Q_1 X_{SS1} = Q_2 X_{SS2} + Q_8 X_{SS8}$ $\quad\quad\quad\quad\quad\quad\quad\quad (6.37)$

(2) Activated Sludge

$SRT = V. X_{H3}/[Q_7. X_{H7} + Q_4. X_{H4}]$ $\quad\quad\quad\quad\quad\quad (6.38)$

$$HRT = \frac{V}{Q_2} \quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad (6.39)$$

$Q_2. x_{i2} - [Q_4. x_{i4} + Q_7. x_{i7}] + r_{xi} V = 0 \quad\quad$ for $x_i \in \{S_I, S_S, S_{NH4}, S_{NOX}, X_I, X_S, X_H,$

$X_{STO}, X_A, X_{SS}\}$ $\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad (6.40)$

$$RO_H = Q_2(S_{S2} + X_{S2} - S_{S3})\left[1 - \frac{(1 + f_{XI} \cdot b_H \cdot \theta_c)Y_H}{1 + b_H \theta_c}\right] \quad (6.41)$$

$$RO_A = Q_2(S_{NH_42} + S_{NOX2} - S_{NH_43})\left[4.57 - \frac{(1 + f_{XI} \cdot b_A \cdot \theta_c)Y_A}{1 + b_A \theta_c}\right] \quad (6.42)$$

$$AFR = \frac{6.0(RO_H + RO_A)}{n_e} \quad\quad\quad\quad\quad\quad\quad\quad (6.43)$$

(3) Secondary clarifier

$$X_{SS4}(mg/L) = \frac{6.21 \cdot \ln(MLSS \cdot SVI)}{0.67 \cdot \ln(H) - \ln(SR)} - 26.43 \qquad (6.44)$$

$$X_{SS5}(g/L) = [k_w(n_w - 1)]^{1/n_w} \left(\frac{n_w}{n_w - 1}\right) \left(\frac{A_f}{Q_s}\right)^{1/n_w} \qquad (6.45)$$

$$Q_3 X_{SS3} = Q_4 X_{SS4} + Q_5 X_{SS5} \qquad (6.46)$$

(4) Design constraints

$$V \leq \frac{1000 AFR}{AIR_L} \qquad (6.47)$$

$$V \geq \frac{1000 AFR}{AIR_U} \qquad (6.48)$$

$$V \geq \frac{(RO_H + RO_A)}{0.10 \ [kg \ O_2 /(m^3 \cdot h)]} \qquad (6.49)$$

(5) Effluent quality constraints

$$COD_{effluent} \geq S_{14} + S_{S4} + X_{14} + X_{S4} + X_{H4} + X_{STO4} + X_{A4} \qquad (6.50)$$

$$TSS_{effluent} \geq X_{SS4} \qquad (6.51)$$

$$NH_{effluent} \geq S_{NH4} + 0.01 S_{14} + 0.03 S_{S4} + 0.02 X_{14} + 0.04 X_{S4} + 0.07(X_{H4} + X_{A4}) \qquad (6.52)$$

(6) Bounds (practical limits)

$x_{lo} \leq x_i \leq x_{up}$ $x_i \in$ all variables, x_{lo} and x_{up} are the lower and upper limits respectively.

6.5 Optimization Using GAMS

The system model developed earlier is highly nonlinear; the objective function and the majority of the constraints are nonlinear. Most constraints are equations with more than one variable; except for the ones specifying effluent water quality and the bioreactor volume sizing limits. Moreover, the problem is poorly scaled, usually with overflow and underflow rates (expressed in the same units) from a separation unit (primary or secondary clarifier) differing in magnitude by several orders of ten.

GAMS (General Algebraic Modeling System) has been utilized by many references as the base to introduce optimization problems. One example is Mays and Tung (1992) who used GAMS for solving optimization problems in the field of hydrosystems engineering and management. Plenty of other references can be noticed in literature. It is a high-level modeling system for mathematical programm-ing problems. It consists of a language compiler and a stable of integrated high-performance solvers. It is specifically designed for modeling linear, nonlinear and mixed integer optimization problems. GAMS is tailored for complex, large scale modeling applications, and allows one to build large maintainable models that can be adapted quickly to new situations. The user can change the formulation quickly and easily, change from one solver to another, and can even convert from linear to nonlinear with little trouble.

A major advantage of GAMS is that it lets the user concentrate on modeling and forget purely technical machine problems such as address calculations, storage assignments, subroutine linkage, and input-output and flow control. Moreover, GAMS language is formally similar to commonly used programming languages. In addition to that GAMS is available for use on personal computers, workstations, mainframes and supercomputers.

The way that GAMS deals with a model is quit efficient. The model is formulated with GAMS language that is generally easy for anyone with programming experience. Then a solver suitable for the type of problem is called. A GAMS program is contained in a disk file, which is normally constructed with a text editor. When GAMS is run, the file containing the program (the input file) is submitted to be processed using a suitable solver. After this processing, the results, which are stored in an output file, can be inspected with a text editor. For more details about the structure of the GAMS input and output files, the reader is encouraged to consult GAMS references like Brooke et al. (1998).

In GAMS there is more than one solver available for different types of problems. Obviously, the problem under consideration is a nonlinear problem. Nonlinear models must be solved with a nonlinear programming (NLP) algorithm. Currently, there are three standard NLP algorithms available for use with GAMS; CONOPT, MINOS and SNOPT. CONOPT is available in two versions, the old

CONOPT and the new CONOPT2. CONOPT2 is the most advanced one so it has been considered as the base solver although other solvers are tested too.

GAMS/CONOPT2 is well suited for models with very nonlinear constraints. If one experiences that MINOS has problems maintaining feasibility during the optimization, he should try CONOPT2. It has a fast method for finding a first feasible solution that is particularly well suited for models with few degrees of freedom.

GAMS/CONOPT is a GRG-based (Generalized Reduced Gradient) algorithm specifically designed for large nonlinear programming problems expressed in the following form:

$$\text{min or max} \quad f(x) \tag{6.53}$$

$$\text{subject to} \quad g(x) = b \tag{6.54}$$

$$lo < x < up \tag{6.55}$$

Where x is the vector of optimization variables, lo and up are vectors of lower and upper bounds, some of which may be minus or plus infinity, b is a vector of right hand sides, and f and g are differentiable nonlinear functions that define the model. The solution process by GRG may terminate to a local optimum, a feasible solution, or to an infeasible solution.

Several factors affecting the solution process, this includes initial values, bounds, and scaling of variables. More elaboration on these factors is given in the coming chapters since they are playing an important role in finding a solution for an optimization problem. Good initial values are important for many reasons. Initial values that satisfy or closely satisfy the constraints reduces the work involved in finding a first feasible solution. Initial values that in addition are close to the optimal ones also reduce the distance to the final point and therefore indirectly the computational effort. The initial values used by CONOPT are all coming from GAMS. The initial values used by GAMS are by default the zero values projected on the bounds. For example, if a variable is free or has a lower bound of zero, then its default initial value is zero. Unfortunately, zero is in many cases a poor initial value for a nonlinear variable. An initial value of zero is especially bad if the variable appears in a product term since the initial derivative becomes zero, and it appears as if the function does not depend on the variable. Therefore, as many sensible initial values as possible should be supplied. An easy possibility is to initialize all variables

to 1. A better alternative is to select reasonable values for some variables that from the context are known to be important, and then use some of the equations of the model to derive values for other variables. Mathematical programs might help in solving the equations and finding values for all the variables.

The second factor is the bounds on the variables. Bounds have two purposes in nonlinear models. Some bounds represent constraints on the reality and practicality that is being modeled, e.g. a variable must be positive. These bounds are called model bounds. Other bounds help the algorithm by preventing it from moving away from an optimal solution. These bounds are called algorithmic bounds. Variables usually can not be left free and practical (model bounds) or algorithmic bounds should be applied on all variables.

The third factor is scaling of variables. It is known that nonlinear as well as linear programming algorithms use the derivatives of the objective function and the constraints to determine good search directions, and they use function values to determine if the constraints are satisfied or not. The scaling of the variables and constraints, i.e. the units of measurement used for the variables and constraints, determine the relative size of the derivatives and of the function values and thereby also the search path taken by the algorithm. If the variables are not properly scaled the algorithm might ignore smaller values considering it as zero tolerances. Variables become well scaled if they are measured in appropriate units.

The above was only a brief description of the basic principles of GAMS and the solver CONOPT. The reader may seek more information about optimization procedure followed by GAMS/CONOPT in GAMS references. The next chapter illustrates utilizing GAMS in solving the aforementioned optimization model to find the optimum size of an activated sludge system.

CHAPTER 7

ILLUSTRATIVE PROBLEM

This chapter is devoted to illustrate the use of the optimization model, introduced in the previous chapter, to obtain an optimal design of a typical complete mix activated sludge system. Normally, the information available at the beginning of the design process is the influent wastewater characteristics as well as the required effluent quality. From such characteristics, stoichiometric and kinetic parameters should be estimated. In addition capital cost and a possible operating cost might be also stated at the beginning where such information usually limits to some extents the design options.

7.1 Influent Characteristics

Influent characteristics comprised of two parts, species concentrations including organics, ammonia/ammonium, and suspended solids which are the subject of treatment and influent flow rate which is the quantity of wastewater to be treated that highly affects the sizes of the treatment units.

In this application, the influent wastewater is assumed to have the characteristics of medium strength wastewater as given by Metcalf and Eddy (1991). The typical composition shown in Table 7.1 is intended to serve as an example and not to indicate the wastewater characteristics at any location. For design purposes, wastewater composition should be determined through specified standard methods and tests. Then design should proceed based on the composition found.

Wastewaters are normally characterized in terms of concentrations of TSS, VSS, BOD_5, Ammonia-N, and Total Kjeldahl Nitrogen (TKN) (as shown in Table 7.1). Total COD is also commonly measured, but its frequency is usually less than that of the other characteristics, although it is increasing.

The composition shown in Table 7.1 can be used directly when considering simple design models like the one presented by Metcalf and Eddy (1991) as the basis for design where BOD_5 or COD will be representing the concentration of the substrate (S) in the model (Equations 3.27 and 3.28). In contrast, when considering extended and more detailed models like the model of Grady et al. (1999), which is based on ASM1, or any other model like ASM3 as the basis for design, the influent composition should be translated into the variables of the model considered. Henze et al. (1987) introduced in detail a procedure to characterize wastewater experimentally and analytically in terms of ASM1 model components. A similar procedure can be followed when characterizing a wastewater in terms of ASM3 model components. On the other hand, Grady et al. (1999) proposed a procedure to translate the influent composition as the one shown in Table 7.1 utilizing a few simplifying assumptions. The following is an explanation of this procedure.

Table 7.1: Typical composition of untreated medium domestic wastewater (Metcalf and Eddy, 1991)

Contaminants	Unit	Concentration
Solids, Total (TS)	mg/L	720
Dissolved, total (TDS)	mg/L	500
Fixed	mg/L	300
Volatile	mg/L	200
Suspended Solids (SS)	mg/L	220
Fixed	mg/L	55
Volatile	mg/L	165
BOD, 5-day, 20°C (BOD_5)	mg/L	220
Total Organic Carbon (TOC)	mg/L	160
Chemical Oxygen Demand (COD)	mg/L	500
Nitrogen (total as N)	mg/L	40
Organic	mg/L	15
Free ammonia	mg/L	25
Nitrites	mg/L	0
Nitrates	mg/L	0

As indicated in Chapter 3, ASM1 and ASM3 are based on COD rather than BOD. In both models, the total COD in an influent wastewater is made up of four components: (1) particulate biodegradable COD (X_S), (2) soluble biodegradable COD (S_S), (3) inert particulate COD (X_I), and (4) inert soluble COD (S_I). In most activated sludge modeling, it is assumed that the concentration of biomass in the influent is negligible compared to the amount formed within the process. That assumption is considered here too, primarily because more research is needed regarding the impact of biomass in the influent (Henze et al., 2000). If no COD data are available, the total

COD of domestic wastewater can be approximated as (Metcalf and Eddy, 1991 and Water Environment Federation, 1992):

$$COD_{total} \approx (2.1)(BOD_5) \tag{7.1}$$

The biodegradable COD can be approximated as follows (Grady et al., 1999)

$$COD_{biodeg} \approx (1.71)(BOD_5) \tag{7.2}$$

Then the inert COD is the difference between the total COD and the biodegradable COD:

$$COD_{inert} = COD_{total} - COD_{biodeg} \tag{7.3}$$

This inert COD must be partitioned into soluble S_I and particulate X_I forms. Experience suggests that 35 to 40 percent of the particulate organic matter in domestic wastewater is non-biodegradable (Henze et al., 2000). Particulate organic matter is represented by the VSS. If one assumes that the composition of the inert particulate organic matter is like that of protein, which has a COD equivalent of 1.5 g COD/g protein, and that protein is totally volatile in a VSS test, then:

$$X_I \approx (0.375)(1.50)(VSS) = 0.56(VSS) \tag{7.4}$$

The soluble inert COD can be calculated by difference:

$$S_I = COD_{inert} - X_I \tag{7.5}$$

Partitioning of the biodegradable COD into slowly and readily (particulate and soluble) biodegradable fractions requires some knowledge of the nature of the wastewater. Additional information suggests that 43% of the biodegradable COD is readily biodegradable (Grady et al., 1999). Consequently,

$$S_S = (0.43)(COD_{biodeg}) \tag{7.6}$$

And

$$X_S = COD_{biodeg} - S_S \tag{7.7}$$

For nitrogen components, the description in ASM1 is much different than that in ASM3. The reader should refer to Chapter 3 to learn about such differences. However, ASM3 description is the recent and the claimed more accurate one. In ASM3, the nitrogen components are of three types: ammonium plus ammonia nitrogen (S_{NH4}), dinitrogen (S_{N2}), and nitrate plus nitrite nitrogen (S_{NOX}). The

dinitrogen contained in the influent can be neglected (Henze et al., 2000). Also, most domestic wastewaters contain no nitrate nitrogen in the influent (Grady et al., 1999). Hence S_{N2} and S_{NOX} are set to zero in the influent. In contrast, because ASM3 assumed that organic compounds (S_I, S_S, X_I, X_S) contain a fixed fraction of organic nitrogen, the influent S_{NH4} cannot be observed directly from analytical measurements but should be computed from wastewater composition: Kjeldahl nitrogen – organic nitrogen, i.e. (Henze et al., 2000):

$$S_{NH4} = TKN - \sum i_N C_i + S_{N2} + S_{NOX} \tag{7.8}$$

Where i_N is the nitrogen content of C_i which is the concentration of S_I, S_S, X_I, and X_S. i_N values are given in Table 6.3.

This procedure is an approximate procedure and there is a high degree of uncertainty associated with it. Rather, in practical situations it is recommended to use an experimental procedure as explained in Henze et al. (2000) which would produce more accurate measures of ASM1 or ASM3 components.

Using the abovementioned procedure the typical composition shown in Table 7.1 can be translated to the form of ASM3 components which is considered here as the base for the optimization model. Table 7.2 shows the data of Table 7.1 translated into ASM3 components. It is obvious that alkalinity is ignored here because its kinetics are ignored in the model as discussed in Chapter 6. The effect of alkalinity on the dynamics of the process is minor as can be noticed from the stoichiometric parameters associated with alkalinity shown in Henze et al. (2000). Moreover, the composition shown in Table 7.2 is for the full ASM3 model while the optimization model is based on an ASM3 reduced model as discussed in Chapter 6.

However, a simpler procedure can be followed and has been utilized by some other researchers. Rousseau et al. (2001) have utilized, as they claimed, a practical fractionation of COD and TKN to the specific components of ASM1. These fractions are indicated in Table 7.3. They have mentioned that a common practice dictates that heterotrophic and autotrophic biomass and DO in the influent are set to zero. It is obvious that this translation is consistent with the model ASM1; however, it agrees noticeably with the method illustrated above. Fractioning of total COD in the two procedures is very similar where X_S forms the largest fraction followed by S_S, X_I, and

S_I respectively. In contrast, description of TKN in ASM1 is different than ASM3 where in ASM3 nitrogen is described only in two variables S_{NH4} and S_{NOX}.

Table 7.2: Typical composition of medium wastewater as translated to ASM3 form

Compounds		Concentration	Units
Dissolved compounds			
S_{O2}	Dissolved oxygen	0	$g\ O_2\ m^{-3}$
S_I	Soluble inert organics	32	$g\ COD\ m^{-3}$
S_S	Readily biodegradable substrates	162	$g\ COD\ m^{-3}$
S_{NH4}	Ammonium	25	$g\ N\ m^{-3}$
S_{N2}	Dinitrogen, released by denitrification	0	$g\ N\ m^{-3}$
S_{NOX}	Nitrite plus nitrate	0	$g\ N\ m^{-3}$
Particulate compounds			
X_I	Inert particulate organics	92	$g\ COD\ m^{-3}$
X_S	Slowly biodegradable substrates	214	$g\ COD\ m^{-3}$
X_H	Heterotrophic biomass	0	$g\ COD\ m^{-3}$
X_{STO}	Organics stored by heterotrophs	0	$g\ COD\ m^{-3}$
X_A	Autotrophic, nitrifying biomass	0	$g\ COD\ m^{-3}$
X_{SS}	Total suspended	230	$g\ SS\ m^{-3}$

Table 7.3: Fractionation of COD and TKN to the specific components of ASM1 (Rousseau et al., 2001)

Symbol	Characterization	% of COD	% of TKN
S_I	Inert soluble organic matter, mg/L as COD	8	-
S_S	Readily biodegradable substrate, mg/L as COD	21	-
X_I	Inert particulate organic matter, mg/L as COD	14	-
X_S	Slowly biodegradable substrate, mg/L as COD	57	-
S_{NH}	Ammonia nitrogen, mg/L as N	-	64
S_{ND}	Soluble biodegradable organic nitrogen, mg/L as N	-	16
X_{ND}	Particulate biodegradable organic nitrogen, mg/L as N	-	20

Beside the above discussed influent characteristics, the influent flow rate is the other characteristic that needs to be defined. According to Metcalf and Eddy (1991) the average flowrate occurring over a 24-hour period based on total annual flowrate data is the flowrate used in evaluating treatment plant capacity and in developing flowrate ratios used in design. Such flowrate varies from one place to another according to the size of the area the treatment plant is going to serve. There exist treatment plants treating a flowrate of 100 m^3/d and there are others treating 500,000 m^3/d. Details of dealing with wastewater flowrates can be found in Metcalf and Eddy (1991). As an average flowrate, the flowrate considered in this application is 36,000 m^3/d (1500 m^3/h). Table 7.4 summarizes the influent characteristics considered in this illustrative problem.

Before proceeding to the design process, one more step is to be considered. That is the characterization of the wastewater in terms of stoichiometric and kinetic parameters as well as the definition of effluent characteristics required.

Table 7.4: Influent characteristics of the illustrative problem

Symbol	Characterization	Value
Q	Flow rate, [m³/hour]	1500
S_I	Inert soluble organic matter, [mg/L as COD]	32
S_S	Readily biodegradable substrate, [mg/L as COD]	162
S_{NH4}	Ammonium plus Ammonia nitrogen, [mg/L as N]	25
S_{NOX}	Nitrate plus nitrite nitrogen, [mg/L as N]	0
X_I	Inert particulate organic matter, [mg/L as COD]	92
X_S	Slowly biodegradable substrate, [mg/L as COD]	214
X_H	Heterotrophic organisms, [mg/L as COD]	0
X_{STO}	A cell internal storage product of heterotrophic organism, [mg/L as COD]	0
X_A	Nitrifying organisms, [mg/L as COD]	0
X_{SS}	Suspended solids, [mg/L as SS]	230

7.2 Stoichiometric and Kinetic Parameters

In order for a model to have utility in the design and operation of wastewater treatment systems, its parameters values which are wastewater specific must be evaluated. In the case of ASM3 model, this is not an easy job since the model contains originally 21 kinetic parameters and 15 stoichiometric parameters. Even when the reduced order model (developed in Chapter 6) is considered the number of kinetic parameters is 12 and stoichiometric parameters are 15. In contrast, the number of parameters for simple models like the one presented in Metcalf and Eddy (1991) is much less which makes the job easier. Only five parameters appear in the simple model discussed in Chapter 3 (see Table 3.1).

Generally, parameters must be evaluated experimentally during treatability studies. Henze et al. (2000) introduced a detailed experimental and analytical procedure to estimate the parameters values of ASM1 model which are applicable to ASM3 kinetic and stoichiometric parameters. However, values of some kinetic and stoichiometric parameters may be obtained from literature or from experience with the wastewater to be treated. In addition some other parameters show little variation from one wastewater to another and can be considered constant. These parameters appear to be about the same for all systems, and fixed values can be assumed. They are listed in Table 7.5 along with their values which are considered satisfactory for most purposes (Henze et al., 1987 and Grady et al., 1999).

Kinetic and stoichiometric parameters are affected by a number of environmental factors. Three are of more importance and deserve mentioning. These

are specific factors (like inhibitors) in the wastewater, pH, and temperature. Typical values are usually given at neutral pH and 20°C. However, such can be adjusted to other temperatures using different approaches, among which is the one proposed by Henze et al. (2000) and discussed before in Chapter 6.

Table 7.5: Parameters which may be assumed

Symbol	Characterization	Value	Units
$b_{A,O2}$	Aerobic endogenous respiration rate of X_A	0.15	d^{-1}
Y_A	Yield of autotrophic biomass per NO_3-N	0.24	g COD_{XA} (g N_{SNOX})$^{-1}$
f_{XI}	Production of X_I in endog. Respiration	0.20	g COD_{XI} (g COD_{XBM})$^{-1}$
$i_{N,XI}$	N content of X_I	0.02	g N (g COD_{XI})$^{-1}$
$i_{N,BM}$	N content of biomass, X_H, X_A	0.07	g N (g COD_{XBM})$^{-1}$

For the purpose of illustration, the typical values of kinetic and stoichiometric parameters given at neutral pH and 20°C and shown in Table 6.3 and Table 6.5 are considered in this study. However, the performance of the model is tested at various temperatures.

7.3 Other Parameters

Other than the stoichiometric and kinetic parameters, other parameters related to primary sedimentation model, secondary sedimentation model, and cost calculations appear in the proposed model.

Values of primary settling constants a and b appearing in the clarification model of primary clarifier are found to be related to temperature. Their values found at 20°C as 1.11 and 261 mg/L, respectively. In contrast, c value is given a constant value in the original model equal to 0.0035 d/m (Christoulas et al., 1998).

For thickening of primary and secondary sludge, the model of Cho et al. (1996) is utilized. Two settling constants characterize this model. For primary sludge, k and n are assumed to be 400 m/d and 2.3, respectively. While for secondary sludge, k_w and n_w are assumed 385 m/d and 1.8, respectively. Such values have been chosen according to recommendations in literature (Cho et al., 1996 and Tang et al., 1984).

For the clarification function of secondary settler, the model of Voutchkov (1992) is utilized. In this model, SVI and H need to be assumed. SVI is assumed to be 150 mL/g while H is assumed the minimum value of 3.7 m as recommended by Metcalf and Eddy (1991).

It should be noted that the above settling constants play a crucial role in controlling the performance of the model as they determine the removal efficiency of suspended solids. Much care should be considered when determining such constants via extensive measurements. Some of the parameters are determined from data collection and regression analysis like primary settling constants. In contrast, *SVI* should be determined through a standard specified experiment.

In calculating the air flow rate, the typical value of field oxygen transfer efficiency (10%) is considered in this illustration.

Finally, the parameters associated with total cost calculations are to be assumed. Such values have a special importance and can be determined from economic studies. The first parameter is the capital recovery factor. Assuming a twenty year design life and a 7% discount rate, using the interest and annuity tables for discrete compounding (Degarmo et al., 1997), the capital recovery factor equals 0.0944. Since the cost functions are developed in the year 1971, they need to be updated to the year of the study. Using Engineering News Record construction cost index, the cost of 1971 (index = 1581) is updated to the cost of 2003 (index = 6581) as follows:

$$Cost(2003) = Cost(1971) \frac{Index(2003)}{Index(1971)} \tag{7.9}$$

Operation and maintenance wages and electricity costs are to be defined according to local costs. In this study they are assumed to be 8.3 and 0.05 dollars, respectively. Finally, the pumping head and the pumping efficiency are needed to calculate the power cost of pumping. They are assumed to be 10 m and 60%, respectively. Table 7.6 summarizes the values of all parameters mentioned above.

7.4 Effluent Characteristics

Three main species are of interest in the effluent. They are organic content, TSS, and ammonia/ammonium nitrogen. Effluent characteristics are to be set according to local regulations. In this study, the effluent characteristics are assumed as recommended in the literature as explained below.

The organic content of effluent from biological treatment processes is usually composed of soluble biodegradable organics, suspended organic material, and non-

biodegradable organics (soluble). According to Metcalf and Eddy (1991) in a well-operating activated sludge plant that is treating domestic wastewater, the soluble carbonaceous BOD_5 in the effluent will usually vary from 2 to 10 mg/L. Suspended organic material will range from 5 to 15 mg/L, and non-biodegradable organics will range from 2 to 5 mg/L. This yields a total organic content range from 9 to 30 mg/L as BOD_5. According to the same reference the BOD_5/COD ratio for typical untreated domestic wastewater varies from 0.4 to 0.8. As a consequence, the effluent total organic content as COD ($CODtot = S_I + S_S + X_I + X_S + X_H + X_A + X_{STO}$) is assumed to be limited by 50 mg/L in this illustration. However, recalling that the inert soluble organics (S_I) in the influent equals 32 mg/L (Table 7.4), these organics are expected to escape all treatment units and appear in the effluent without change. Thus the remaining soluble biodegradable and particulate biodegradable organics should form only 18 mg/L in the effluent. Since the effluent TSS is constrained and since the particulate COD is part of this TSS, there is no need to constrain the particulate COD. Thus a constraint is only required on the soluble part of COD which is formed of inert soluble organics (S_I) and readily biodegradable substrate (S_S). Moreover, it is obvious that S_I is high enough so that efforts must be spent in reducing S_S as much as possible. Hence S_S is only constrained and is set to be less than or equal 2 mg/L.

Table 7.6: Summary of parameter values utilized in the optimization model

Symbol	Characterization	Value	Units
A	Constant in Christoulas model for primary clarification	1.11	-
b	Constant in Christoulas model for primary clarification	261	mg/L
C	Constant in Christoulas model for primary clarification	0.0035	d/m
k	Settling constant of primary sludge	400	m/d
n	Settling constant of primary sludge	2.3	-
kw	Settling constant of wasting sludge	385	m/d
nw	Settling constant of wasting sludge	1.8	-
SVI	Sludge Volume Index of sludge	150	mL/g
H	Side water depth of final clarifier	3.7	m
ne	Efficiency depends on diffuser and depth at which air pumped	10%	
AIR_U	Maximum air input rate	90	$m^3/(min.1000\ m^3)$
AIR_L	Minimum air input rate	20	$m^3/(min.1000\ m^3)$
CRF	Capital Recovery factor	0.0944	
BCI	Base (1971) Cost Index	1581	$
CI	Cost Index for 2003	6581	$
OMW	Operating Maintenance Wages	8.3	$ per h
EC	Electricity Cost	0.05	$ per kWh
PH	Pumping Head	10.0	m
PE	Pumping Efficiency	0.6	

The second characteristic of interest is the TSS. According to the aforementioned reference. the activated sludge process can achieve as low as 10 mg/L of T^{SS} in the effluent. Hence, the TSS in the effluent is constrained to be less than or equal 10 mg/L.

Regarding the ammonia/ammonium nitrogen, the system is assumed to achieve complete nitrification. This means metabolizing all the quantity of ammonia/ammonium nitrogen. From the influent characteristics, the ammonia/ammonium nitrogen in the influent is not high and hence there is no concern of having a high level of nitrate in the effluent. This is one of the reasons that usually encourage the designer to disregard denitrification in the system. Complete nitrification means very low concentration of ammonia/ammonium nitrogen in the effluent. In this illustration, S_{NH4} is set to be less than 1.0 mg/L in the effluent.

7.5   Bounds on Variables

As mentioned in Chapter 3, practical limits on design variables are recommended for more than one reason. They are also sometimes necessary to drive the solution process to terminate at a practical feasible point.

It is recommended according to Metcalf and Eddy (1991) to set the overflow rate of primary clarifier low enough to ensure satisfactory performance at peak rates of flow. Typical design values for primary clarifiers show that the overflow rate could range between 30 - 120 m^3/m^2.d (800 and 3000 gal/ft^2.d). Similarly, the diameter typical range of circular primary clarifiers is $12 - 46$ m $(40 - 150$ ft) which yields a typical range of surface area equals ($115 - 1600$ m^2) (see also Droste, 1997). Only the lower limit on area is considered, the upper is not considered. If the area obtained is above the practical upper limit, the user may consider using more than one reactor according to the area found.

In the activated sludge process, controlled variables include SRT, HRT, and r. The SRT typically ranges between 1 and 15 days for complete mix activated sludge systems. However, when nitrification is considered the typical SRT may reach 20 days. In contrast, the HRT might range between 3 and 15 hours (0.125 and 0.625 d) for complete mix activated sludge systems considering nitrification. For the recycle

ratio, the range is between 0.25 and 1.0 for complete mix activated sludge systems. For single stage nitrification the recycle ratio might reach 1.5.

It has been mentioned previously that MLSS might range between 500 and 6500 mg/L. Values above 6500 mg/L are not recommended because this will increase the cost of secondary sedimentation. This limit is not applicable here since the cost of secondary sedimentation is implemented in the model and the solved value for MLSS considers that cost. However, the lower limit is still considered since values below 500 mg/L are not practical and cannot be accepted in an operating activated sludge system.

Regarding the secondary clarifier, the typical design information given by Metcalf and Eddy (1991) suggests that the overflow rate (SR) could range between 16 – 32 m m^3/m^2.d (400 – 800 gal/ft^2.d). The lower limit is very important because it limits the removal efficiency that can be achieved practically. Otherwise the system will assume impractical perfect clarification. The limit of area of primary clarifier is also applicable for secondary clarifier.

The aforementioned limits on q, SRT, HRT, r, MLSS, and SR are considered in the model. However, the applicability of these bounds and their effect on the solution will be further investigated. New bounds are proposed in this application and they are thereafter considered in the rest of the analysis. As a rule of thumb, if the final optimal solution suggests some variables at their imposed bounds, then the roles of these bounds should be examined in details. Table 7.7 summarizes all bounds applied in the model.

7.6   Optimization Procedure

To optimize a problem using GAMS, a file contains the parameters, variables, and equations forming the model should be built using the GAMS programming language. Such a file has been prepared for the problem under consideration based on the formulation given earlier (Chapter 6). The user is asked to provide four types of information as an input to the model: the influent wastewater characteristics, the model parameters including kinetic and stoichiometric parameters and effluent characteristics, the imposed bounds on variables, and the initial solution that is required to start the optimization.

Table 7.7: Bounds on variables in the illustrative problem

Symbol	Characterization	Units	Lower bound	Upper bound
q	overflow rate of primary settling tank	m/d	30	120
Ap	Surface area of the primary clarifier	m^2	115	-
SRT	sludge retention time	d	1	20
HRT	hydraulic retention time	d	0.125	0.625
r	sludge recycle ratio	-	0.25	1.5
$XSS3$	MLSS	g/m^3	500	-
SR	overflow rate of final clarifier	m/d	16	32

Listing of the prepared GAMS file is given in Appendix A. Parameters, variables, bounds, and initial values are shown in the listing clearly along with their units. The model contains 29 variables, 25 equations (four degrees of freedom), and six inequalities constraints. The cost equation is broken down into individual equations for simplicity which add another 22 variables and equations.

As mentioned in the previous chapter, several factors affect the solution process and much care should be considered at the beginning before a solution is started. Scaling of variables can be noticed obviously in the formulation. Some variables are exaggerated and others are compressed to overcome large difference in values. For example, volume of bioreactor is expected to be of thousands of cubic meters while the wastage ratio w is expected to be a fraction of thousand which means a difference might reach 10^7. As a consequence, the volume is expressed as $1000 \, m^3$ while w is expressed as $100 \, w$ which reduces the difference to tens only.

In terms of bounds, as can be noticed form the listing, all variables are bounded. This is essential to obtain a solution. However, it is recommended as much as possible to widen the upper and lower limits except when a practical limit exists. Hence, variables that have practical upper and lower bounds, as discussed in section 7.5, are assigned such bounds. Other variables are arbitrarily bounded between 10^{-6} and 10^6.

Initial values are the most critical preparation step before processing the problem. The best option is to use mathematical software for generating system designs that can be used as starting solutions in optimization. As mentioned, the problem has four degrees of freedom. This requires defining four decision variables to make the solving process possible. One might choose q, r, SRT, and HRT as decision variables, pick a value for them, solve the model to find other variables, and then use all as an initial point to initialize the optimization. This will assure obtaining a feasible optimal point. Since the model is highly nonlinear model, multiple local optima are

expected to be present. Another initial point might converge to another local optimal. As a consequence several initial points might be needed (see the next section). Examples of mathematical software that can be used include MatLab, Maple, Mathematica, TK solver ... etc.

The author has used Microsoft Excel with the aid of a solver function to obtain a solution for the system of equations and start the optimization with that solution. The solution obtained is not necessarily a feasible solution since it did not consider the simple bounds on variables. The idea was just to find a feasible or close to feasible values for all the variables. The Excel file was designed to work in the following procedure (refer to GAMS listing for variables definitions):

(1) First, primary clarifier is designed (i.e., variables X1, Q2, Q8, XSS8, and Ap are defined) for a given q. This portion of the file gives an exact design of the primary clarifier for a given value of q.

(2) The second portion of the file uses the Solver function in Excel to find values for all state variables in stream 3 (which are the same in the aeration tank) for a given value of SRT and HRT and depending on the design of primary clarifier given in (1).

(3) The third portion is to design the secondary clarifier, given r and the information gained in (1) and (2), using the solver function also w, X2, X3, and Af are calculated.

(4) Other variables are calculated based on the information gained in (1), (2), and (3).

Based on the above, there is no need to mention all the variables when pointing solutions either initial or final. Only decision variables q, SRT, HRT, r and design variables Ap, V, AFR, MLSS, Af, SR, w, total cost, and effluent characteristics are going to be listed. The reader is expected to calculate other variables easily.

Another option is to choose arbitrary initial values (close to what is expected). If the solution is infeasible or nonoptimal, another solution can be initiated from the previous one but with applying slight modification on the variables considering their predictable values. The procedure can be repeated until an optimal point is found. This procedure is time consuming and no guarantee that a feasible solution can be achieved.

Providing the four types of information mentioned above: the influent characteristics, model parameters, bounds, and initial solution we are ready to start the solution process to obtain an optimal design of this illustrative problem.

7.7 Results and Discussion

Solution of the illustrative problem, performance and quality of such solution are discussed in this section. First an initial run is executed to explore the solution. The results of this run are discussed in details. Such results lead to some modifications in the initial run. A final run to obtain the most reasonable optimal design is then presented. It is recommended to read this section in particular concurrently with Chapter 6 "Model Development".

7.7.1 Initial Run

Using Microsoft Excel and considering the decision variables as q, SRT, HRT, and r with values 30 m/d, 10 d, 0.2 d, and 0.4, respectively, other state variables were calculated by solving the nonlinear system of constraint equations. The values obtained were utilized as an initial solution. A sample GAMS output for this run is given in Appendix B. The reader is referred to GAMS User's Guide for interpretation of the output file terms and sections. Table 7.8 shows the final solution obtained along with the used initial solution. The final solution is depicted in a representative way in Figure 7.1.

Table 7.8: Initial and final solution for the initial run

Symbol	Characterization	Units	Initial solution	Solution
q	overflow rate of primary settling tank	m/d	30	120
Ap	Surface area of the primary clarifier	m^2	1199.5	299.8
SRT	sludge retention time	d	10	3.479
HRT	hydraulic retention time	h	4.8	3
V	Volume of the aeration tank	m^3	7197	4497
AFR	Air flow rate in aeration tank	m^3/min	293.6	264.5
$XSS3$	Mixed liquor suspended solids	g/m^3 as SS	5844	4628
r	sludge recycle ratio	-	0.4	0.25
w	100*wastage ratio (Q7/Q2)	-	0.559	0.696
SR	overflow rate of final clarifier	m/d	14.58	18.90
Af	Surface area of the final clarifier	m^2	2454	1890
$SS3$	Effluent readily biodegradable substrates	g/m^3 as COD	0.288	0.568
$SNH3$	Effluent Ammonium/ammonia nitrogen	g/m^3 as N	0.333	0.778
$XSS4$	Effluent suspended solids	g/m^3 as SS	4.15	10
$Cost$	total system cost	$/year	780258	598138

Figure showing a wastewater treatment process flow with Primary clarifier, Aeration Tank, and Secondary clarifier units.

LEGEND:

Symbol	Units
Q	m³/hr
S_I	g/m³
S_S	g/m³
S_{NH4}	g/m³
S_{NOX}	g/m³
X_I	g/m³
X_S	g/m³
X_H	g/m³
X_A	g/m³
X_{STO}	g/m³
X_{SS}	g/m³

Primary: $q=120$ m/d, $A_p=299.8$ m²

Aeration Tank: $V=4497$ m³, $SRT=3.479$ d, $HRT=3$ h

Secondary: $SR=18.9$ m/d, $A_f=1890$ m²

$r = 0.25$

Stream data (in order Q, S_I, S_S, S_{NH4}, S_{NOX}, X_I, X_S, X_H, X_A, X_{STO}, X_{SS}):

Variable	Stream 1	Stream 2	Stream 3	Stream 4	Stream 8	Stream 6	Stream 7
Q	1500	1499	1874	1489	1.014	374.7	10.43
S_I	32	32	32	32	32	32	32
S_S	162	162	0.568	0.568	162	0.568	0.568
S_{NH4}	25	25	0.778	0.778	25	0.778	0.778
S_{NOX}	0	0	28.78	28.78	0	28.78	28.78
X_I	92	70.43	2283	4.934	31980	11089	11089
X_S	214	163.8	474.7	1.026	74387	2305	2305
X_H	0	0	2230	4.819	0	10830	10830
X_A	0	0	126.2	0.273	0	612.8	612.8
X_{STO}	0	0	731.0	1.580	0	3550	3550
X_{SS}	230	176.1	4628	10	79949	22475	22475

Figure 7.1: Illustrative problem final solution after the initial run

The algorithm needed 24 iterations to reach the optimal solution using CONOPT2. The computing time is negligible on a computer with Pentium 4 at 1.8 GHz speed. Number of iterations and computing time generally depends on the starting solution, the model settings in hand, and the algorithm used.

It has been shown in previous studies (e.g., Tang et al., 1984) that the final solution depends on starting solution, bounds, and algorithm used. However, ideally, the optimal solution should not depend heavily on bounds specified for the variables (which are not limiting the solution) or on the algorithm used. If such happens then it is attributed to solution algorithm weakness. This has been explored as follows.

Two optimization runs starting from the same solution (starting point No. 1 in Table 7.9) were made with slightly different bounds on the decision variables. Of course none of the variables for which the bounds were modified is at its bound in the original final solution. The first run was made using the default bound set summarized in Table 7.7. In the second run, the lower and upper bounds of SRT were changed from the default values of 1 and 20 to 0.5 and 40, respectively, and the lower and upper bounds of SR were changed from the default values of 16 and 32 to 5 and 80, respectively. These values have no physical or practical meaning and were used only for this experiment. It has been noticed that the bounds did not affect the final solution and the same design and cost shown in Table 7.9 were obtained. However, the bounds affect the solution process where it needs a different number of iterations to reach the optimal solution. The effect of bounds on number of iterations is illustrated in Table 7.10.

Concerning the solution algorithm, it has been mentioned that there are three standard NLP algorithms available for the use with GAMS, CONOPT, MINOS and SNOPT. In the above analysis CONOPT2 is utilized. MINOS and SNOPT are tested hereafter.

Three solutions were tested using MINOS solver for the three mentioned starting solutions (Table 7.9). The three solutions converged to the same point shown in Table 7.9 with negligible difference in objective function value. The same reached when SNOPT solver was utilized. In terms of number of iterations, the solver type affects highly the solution process. This effect is illustrated in Table 7.10.

This proves obviously the strength of GAMS solvers where neither the nonlimiting bounds nor the algorithm have affected the final solution. In contrast, the effect of starting solution on final solution is expected. Thus it is explored in the coming section.

7.7.2 Effect of Starting Point

It has been mentioned that the model is highly nonlinear and multiple local optima are expected to be present. To examine this issue, different starting solutions were used and found using the same approach utilized in the initial run. Namely Microsoft Excel was used to solve the system equations after specifying the four decision variables (q, SRT, HRT, and r). Three starting points other than the one shown in Table 7.8 were tested. Table 7.9 summarizes the results of the three runs numbered as 2, 3, and 4 as the initial solution shown in Table 7.8 is considered as number 1. Table 7.9 shows that widely different initial solutions converged to essentially the same solution. Although the starting solutions were totally different, the final solution was the same for the three in terms of decision variables, design variables, effluent characteristics, and total cost. A situation like this indicates clearly a robust solution process and the flatness of the objective function. This is compatible with other researchers work. Tang et al. (1984) concluded the same when they optimized the liquid subsystem which is identical to the system considered in this study.

However, this completely does not mean that the starting solution does not affect the solution. In fact, the starting solution affected the solution process but not the final solution. Effect of starting solution on solution process is obvious when number of iterations and computing time are compared. In terms of number of iterations, it differs with different starting solutions. Table 7.10 shows the number of iterations for the different starting solutions presented in Table 7.9. It is obvious from the table that as the initial value of q is closer to the final value, the number of iterations is less which means less effort to reach the final solution.

In summary, theoretically starting solution always affects the quality of final solution and the solution process when GRG2 is used. However, in this particular example the effect on final solution does not exist although it is obvious on the solution process. This indicates that the solution obtained in the initial run

(summarized in Table 7.8) seems to be a robust global optimal solution. This solution is considered the basis for the following analysis

Table 7.9: Summary of system design obtained using different starting points

Variables (Units)		Starting point		
		2	3	4
q, overflow rate of primary settling tank (m/d)	initial	60	90	120
	final	120	120	120
Ap, Surface area of the primary clarifier (m^2)	initial	599.7	399.7	299.8
	final	299.8	299.8	299.8
SRT, sludge retention time (d)	initial	7	2	20
	final	3.479	3.479	3.479
HRT, hydraulic retention time (h)	initial	3.6	3	9.6
	final	3	3	3
V, Volume of the aeration tank (m^3)	initial	5397	4497	14390
	final	4497	4497	4497
AFR, Air flow rate in aeration tank (m^3/min)	initial	285.3	238.6	326.0
	final	264.5	264.5	264.5
$XSS3$, Mixed liquor suspended solids (g/m^3 as SS)	initial	6189	2932	5377
	final	4628	4628	4628
r, sludge recycle ratio	initial	1	0.25	0.5
	final	0.25	0.25	0.25
w, 100*wastage ratio (Q7/Q2)	initial	1.051	0.367	0.619
	final	0.696	0.696	0.696
SR, overflow rate of final clarifier (m/d)	initial	14.29	45.47	17.87
	final	18.9	18.9	18.9
Af, Surface area of the final clarifier (m^2)	initial	2491	788	2000
	final	1890	1890	1890
$SS3$, Effluent readily biodegradable	initial	0.343	1.091	0.229
substrates (g/m^3 as COD)	final	0.568	0.568	0.568
$SNH3$, Effluent Ammonium/ammonia nitrogen (g/m^{3}as N)	initial	0.414	1.857	0.250
	final	0.778	0.778	0.778
$XSS4$, Effluent suspended solids (g/m^3 as SS)	initial	3.96	132.7	9.03
	final	10	10	10
$Cost$, Total system cost ($/year)	initial	875205	528086	799653
	final	598138	598138	598138

Table 7.10: Number of iterations for different solutions

Case	Starting point			
	1	2	3	4
Default bounds	24	30	20	15
Modified bounds	27	25	25	15
Using MINOS solver	37	22	481	17
Using SNOPT solver	39	40	54	38

7.7.3 Further Analysis

Several comments can be withdrawn by exploring the solution illustrated in Table 7.8:

(1) Primary clarifier overflow rate (q) is at its upper limit (120 m/d) and as a result removal efficiency of this unit is low (23.4%) if compared to removal efficiencies of effective units stated in the literature as (40 – 60%). This indicates little significance of the primary clarifier in the system. The importance of this unit in the system can be further examined by relaxing the upper limit of q (Section 7.7.4).

(2) In contrast, the *HRT* and the recycle ratio (r) are at their lower limits. This indicates that the sludge age is overestimated and more savings can be achieved without altering the required effluent quality by reducing the sludge age via releasing the lower limit of *HRT* and r. This is true since the effluent soluble COD (S_S) and the effluent ammonium (S_{NH4}) are far less than their limits (0.568 < 2.0 g/m^3 for S_S and 0.778 < 1.0 g/m^3 for S_{NH4}). Thus an acceptable effluent can be produced with less cost by reducing *HRT* and/or r. This is further explored in Section 7.7.5.

(3) According to Grady et al. (1999), practically, selection of *SRT* for domestic wastewaters is almost always controlled by factors other than soluble substrate removal. This is clear in the solution. Other than the effluent soluble COD (S_S) and the effluent ammonium (S_{NH4}), the third effluent constraint, the suspended solids (X_{SS}), is found to be at its limit 10 g/m^3 which means that this constraint controlled the solution while the other two are relaxed.

(4) The total effluent COD which equals the soluble COD plus particulate COD equals in the effluent 45.2 g/m^3 and the total nitrogen which is the ammonium plus the nitrate/nitrite equals 30.4 g/m^3. Such values fall within the acceptable practical ranges and they were both achieved without being explicitly constrained. This proves the correctness of the assumption made earlier stating that constraining the TSS is adequate and should implicitly constrain the total COD. This is due to the fact that a major portion of total COD is the particulate COD which is constrained by the TSS. Similarly, the amount of total nitrogen is not high which is attributed to low level of ammonia in the influent.

(5) The major portion of the MLSS is heterotrophic biomass (43.37%) while the inert particulate organics comprises 37%. The rest is slowly biodegradable substrate (7.69%), autotrophic biomass (2.54%), and organics stored by heterotrophs

(9.48%). This is compatible with practical expectations, the two main portions constituting the MLSS are inert organics and active biomass.

Based on the comments (1) and (2) mentioned above further analysis is required. The implications associated with a variable being at its specified bound in the final solution may provide useful insights. Relaxing such a bound may imply that the total system cost could be reduced. However, additional research may be needed to justify such relaxation if bounds imposed on the decision variables represent ranges recommended for design or limits within which the process performance model is developed. On the other hand, if the bounds represent the limits outside which process failure will occur, then extrapolation of a process model is inappropriate. For example, it is impractical to have an activated sludge system with very low recycle ratio (less than 25%) although the optimization solution suggests the recycle ratio at its lower bound and relaxing this bound might produce lower cost. In contrast, if an upper bound on a loading rate (like primary clarifier overflow rate) is approached in the optimization solution, then the desirable action is to eliminate that unit not to consider implementing a higher practical upper bound. Such issues are explored hereafter in more details.

7.7.4 Primary Clarifier Overflow Rate

It has been mentioned previously that typical design guidelines for primary settling tank generally call for the overflow rate to be less than or equal to 120 m/day under peak flow conditions. As a result, an upper bound was imposed on the overflow rate in the system model. However, the final design showed that the overflow rate is at this upper bound. This solution suggests that the total system cost may be further reduced by relaxing the upper bound because of a negative marginal cost associated with this variable in the final solution. This is more obvious when the upper bound is set to 240 m/d and a run is executed. The solution yielded a total system cost of 585474 dollars/year which is 2.1% less and again the overflow rate is at its new upper bound which means further reduction can be achieved. All other design variables were also affected.

This raises a major question. Is the primary clarifier a cost-effective unit in the assumed system? To address this question, the primary settling tank was eliminated from the system. The modified system is shown in Figure 7.2. Considering the initial

run starting solution, a run was executed. Table 7.11 summarizes the results. Results obtained for the system with primary clarifier are also listed for the sake of comparison. Figure 7.3 depicts the final design. The effect of the starting solution on the new system design was also studied similar to the base system and the starting solution was found to have no effect on the final solution.
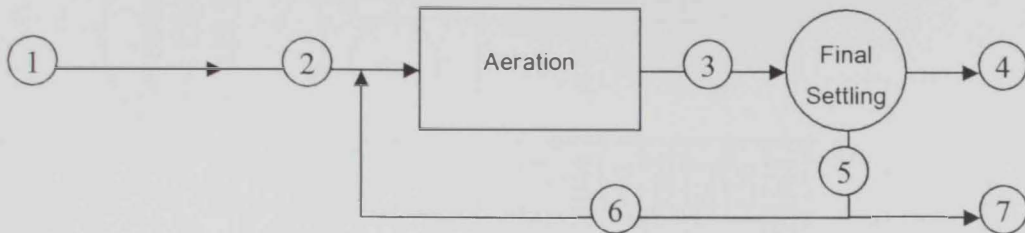
Figure 7.2: System layout without a primary clarifier

Table 7.11: System design optimization without a primary clarifier

Symbol	Characterization	Units	Initial solution	Solution without Primary clarifier	Solution with Primary clarifier
q	overflow rate of primary settling tank	m/d	0	NA	120
Ap	Surface area of the primary clarifier	m^2	0	0	299.8
SRT	sludge retention time	d	10	2.898	3.479
HRT	hydraulic retention time	h	4.8	3.1	3
V	Volume of the aeration tank	m^3	7197	4705	4497
AFR	Air flow rate in aeration tank	m^3/min	293.6	282.3	264.5
$XSS3$	Mixed liquor suspended solids	g/m^3 as SS	5844	4640	4628
r	sludge recycle ratio	-	0.4	0.25	0.25
w	100*wastage ratio (Q7/Q2)	-	0.559	0.892	0.696
SR	overflow rate of final clarifier	m/day	14.58	18.90	18.90
Af	Surface area of the final clarifier	m^2	2454.3	1888	1890
$SS3$	Effluent readily biodegradable substrates	g/m^3 as COD	0.288	0.681	0.568
$SNH3$	Effluent Ammonium/ammonia nitrogen	g/m^3 as N	0.333	0.981	0.778
$XSS4$	Effluent suspended solids	g/m^3 as SS	4.15	10	10
$Cost$	total system cost	$/year	674851	564740	598138

Results indicate a total system cost without the primary clarifier equal to 564740 dollars/year of 5.6% less than the final design cost with the primary clarifier. A comparison between this design and the one with the primary clarifier in the system yielded the following in addition to the reduction in the system cost mentioned above:
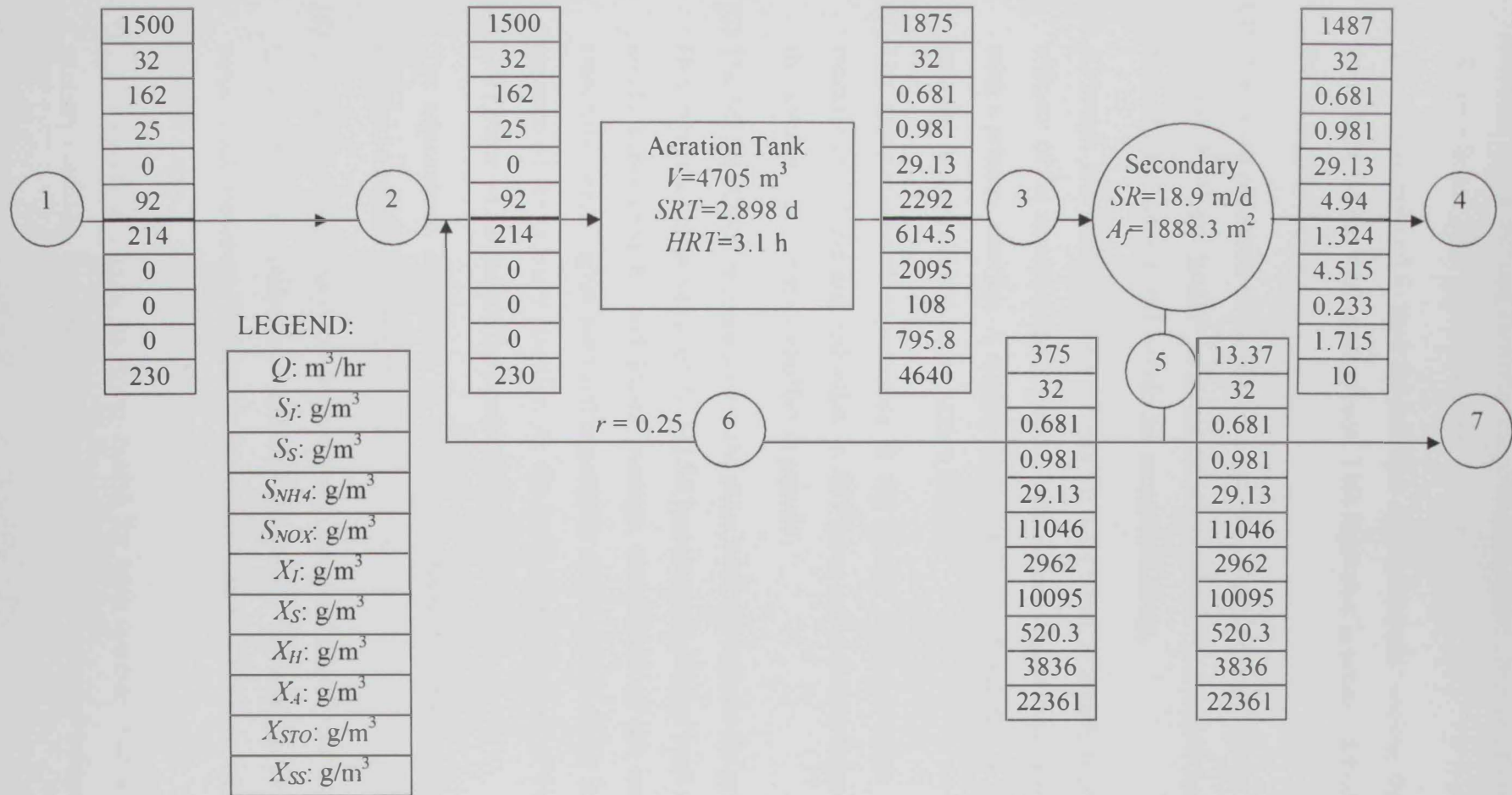
Figure 7.3: Final design for the system without a primary clarifier

(1) Without the primary clarifier, a larger aeration tank is needed to accommodate the increase in the influent flowrate and solids due to the absence of primary clarifier. Hence a larger *HRT* is also required.

(2) The *SRT* is reduced to limit the increase in aeration tank volume that significantly contributes to the total system cost. This reduction is accomplished by increasing the wastage ratio.

(3) Due to the reduction in the *SRT*, the quality of soluble components in the effluent is altered. Worst quality in terms of soluble COD and ammonium/ammonia is noticed although it is still within the acceptable limits.

(4) Although the concentrations of soluble COD and ammonium/ammonia in the effluent of the system without primary clarifier are higher than those in the system with a primary clarifier. A slightly higher *AFR* is noticed, this is attributed to the increase in flowrate and the reduction in *SRT*.

(5) The aforementioned adjustments in the design of the aeration tank, i.e., the increase in volume and reduction in *SRT* have reduced the sludge production to the level as if the primary clarifier is present.

(6) The MLSS (*XSS3*) remains almost the same with and without the primary clarifier. However, the composition of the MLSS has changed. Higher inert and degradable solids concentrations and lower biomass concentrations are noticed. This is reasonable since higher inert and degradable solids present in the influent with the absence of the primary clarifier. At the same time less level of treatment (lower *SRT*) occur which reduce the production of biomass.

(7) The adjustments mentioned in number 5 caused the design of the secondary clarifier to remain unchanged.

(8) The absence of primary clarifier obviously increased slightly the cost of biological treatment in the aeration tank and kept the cost of secondary sedimentation at the same level. However, the total system cost becomes less than when the primary clarifier exists.

(9) The recycle ratio is at its lower bound for both systems with and without the primary clarifier. Thus needs further elaboration as explained below.

7.7.5 Sludge Recycle Ratio

It has been noticed that the sludge recycle ratio is at its lower bound for the systems with and without a primary clarifier. Two runs were done for the system with a primary clarifier and without a primary clarifier after releasing the lower bound on the recycle ratio. The same initial solutions considered in the initial run were used in these tests. Table 7.12 summarizes the results of the two runs.

For the two systems, the recycle ratio has decreased to a very low value compared to the lower practical bound, resulting in a huge reduction in cost (15.5% and 13.6% less cost for the system with and without primary clarifier, respectively). The direct influence of the recycle ratio to the cost is obvious. Low r means low recycle pumping cost which is one of the main contributors to the cost.

Table 7.12: System design optimization after releasing the lower bound on r

Symbol	Characterization	Units	Solution without Primary clarifier	Solution with Primary clarifier
q	overflow rate of primary settling tank	m/d	NA	120
Ap	Surface area of the primary clarifier	m^2	0	299.8
SRT	sludge retention time	d	2.857	2.857
HRT	hydraulic retention time	h	4.9	5.8
V	Volume of the aeration tank	m^3	7378	8730
AFR	Air flow rate in aeration tank	m^3/min	281.7	256.5
$XSS3$	Mixed liquor suspended solids	g/m^3 as SS	2926	2048
r	sludge recycle ratio	-	0.066	0.029
w	100*wastage ratio (Q7/Q2)	-	0.451	0.243
SR	overflow rate of final clarifier	m/d	20.4	21.72
Af	Surface area of the final clarifier	m^2	1753	1652
$SS3$	Effluent readily biodegradable substrates	g/m^3 as COD	0.691	0.691
$SNH3$	Effluent Ammonium/ammonia nitrogen	g/m^3 as N	1	1
$XSS4$	Effluent suspended solids	g/m^3 as SS	10	10
$Cost$	total system cost	\$/year	451715	505201

From Table 7.12, it is clear that the effect of releasing the lower bound of r is almost the same with the presence and absence of the primary clarifier. Even for the system with primary clarifier, design of this unit was not affected at all by the change of r. Hence the performance of the two models can be assumed to be identical. This performance can be explained as follows. Decreasing r is favored because it will reduce the cost associated with recycle pumping and the cost of the secondary clarifier. The two costs are main contributors to total system cost. However this will alter the treatment in the aeration tank since SRT will decrease. To maintain the level of treatment to an acceptable level, SRT should be maintained at its minimum value.

This is accomplished by increasing *HRT* which resulted in increase in the aeration tank volume. This is correct since for the first time the effluent ammonium/ammonia nitrogen is limiting the solution besides the TSS.

This performance looks reasonable. However, this design is considered impractical since the lowest experienced recycle ratio in practice is not less than 25%. As a consequence, engineering intuition always should be considered in such cases. Although the design obtained is less in cost, it is impractical and inapplicable. If the activated sludge process is to be used as the treatment option, the recycle ratio should be kept at its lower bound.

As mentioned above, the low *r* is preferred because the recycle pumping is a main contributor to the total system cost. This can be further investigated by imposing another recycle pumping cost functions. In this investigation, the cost functions coefficients for the return sludge pumping shown in Table 6.7 are reduced by 50%. For example, in the mentioned table the coefficient for return sludge pumping capital cost is 9870. This is reduced to 4935. Other coefficients are reduced similarly. Results (Table 7.13) show higher recycle-ratios for the system with and without a primary clarifier. Further reduction could raise the recycle ratio to the level of the practical limit.

Table 7.13: System design optimization after changing recycle pumping cost function

Symbol	Characterization	Units	Solution without Primary clarifier	Solution with Primary clarifier
q	overflow rate of primary settling tank	m/d	NA	120
Ap	Surface area of the primary clarifier	m^2	0	299.8
SRT	sludge retention time	d	2.857	2.857
HRT	hydraulic retention time	h	7.1	4.4
V	Volume of the aeration tank	m^3	10705	6538.7
AFR	Air flow rate in aeration tank	m^3/min	388.2	256.5
$XSS3$	Mixed liquor suspended solids	g/m^3 as SS	3500	2734
r	sludge recycle ratio	-	0.099	0.056
w	100*wastage ratio (Q7/Q2)	-	1.0	0.340
SR	overflow rate of final clarifier	m/d	19.8	20.7
Af	Surface area of the final clarifier	m^2	1798	1734
$SS3$	Effluent readily biodegradable substrates	g/m^3 as COD	0.691	0.691
$SNH3$	Effluent Ammonium/ammonia nitrogen	g/m^3 as N	1	1
$XSS4$	Effluent suspended solids	g/m^3 as SS	10	10
$Cost$	total system cost	$/year	451715	472222

This indicates a very important issue associated with optimization problems that is the accuracy of cost functions. Cost functions should be chosen to reflect the actual costs in practice and at the same time much care should be devoted to the accuracy of relative costs of various units. In other words, the relation between the capital and operation/maintenance costs for a certain unit and the relation between total cost of a unit and another should be defined clearly. This is a prerequisite to obtain accurate results from optimization studies.

7.7.6 Final Run

From the above analysis, the best economical design would be the system without primary clarifier keeping the recycle ratio at its lower practical bound. However, this is mathematically true while practically it needs further investigation. Much care should be considered when a decision of a treatment plant without a primary clarifier is chosen. Primary clarifier has a crucial role in reducing inert solids coming with the influent which if presented in huge amounts could deteriorate the biological treatment in aeration tanks. Otherwise, the final solution shown in Table 7.11 is the most economical practical solution.

CHAPTER 8

MODEL PERFORMANCE

An optimization model can be used to obtain cost effective designs of an activated sludge system defined by the selected process performance models and parameters. Using an optimization model also enables the designer to analyze process performances systematically and effectively. Detailed design can then be performed following the guidelines or trends suggested from the optimization study.

In this chapter, the role of the optimization model is explored, and it is shown that such a model may be used for more than just identifying a least-cost system design. Expressly, such a model can be used as a tool for the analysis of treatment process performance and of alternative treatment plant configurations. Potentially important research areas and/or design guidelines can also be identified from these insights.

The typical activated sludge system depicted in Figure 6.1 was designed using GAMS/CONOPT2 solver for the design conditions described in Chapter 7. The final design obtained is summarized in Table 7.11. Such a design was analyzed for various conditions to reach a final design claimed to be more representative and cost effective. For the sake of analysis and since it contains primary clarifier, aeration tank, and secondary clarifier, the initial design summarized in Table 7.8 was chosen to provide the basis for the following discussion. Although the cost of the system is overestimated due to the presence of the primary clarifier, it was mentioned that it is still needed for efficient metabolism in the aeration tank and to avoid inhibition effect and bacteria inactivation. Thus, the initial design summarized in Table 7.8 is noted during this chapter as the base design.

It has been mentioned frequently in literature that many factors affect the design and operation of activated sludge systems. These include, *SRT*, MLSS concentration, temperature, DO, oxygen transfer and mixing, and nutrients. Among

them, *SRT* and temperature are of special importance. Thus, their impacts are discussed in this chapter.

One of the main factors to be considered during design is the effect of transient loadings. Their effect is obvious on sizing of various units as well as on the design of the oxygen transfer system. Thus, the effect of different influent characteristics is also investigated in this chapter.

The aim of this chapter is to present the observations and discussions drawn from examinations of the solution obtained under diverse conditions. Such analysis is important for understanding and verifying the system response to various conditions.

8.1   Effect of Solids Retention Time

It has been mentioned that *SRT* is a primary factor determining the performance of activated sludge system. Sufficient *SRT* is required for treatment and for flocculation of suspended solids for proper sedimentation in the final clarifier. In the base design, *SRT* was chosen to be around 3.5 days. Below this value the quality of effluent is not acceptable. It has been noted in literature that the selection of *SRT* is usually controlled by the flocculation requirement not the removal of soluble substrate. Such means that increasing *SRT* above the required value would not affect the effluent quality significantly.

To demonstrate this, several runs have been conducted by fixing *SRT* at gradually increasing value. The first run started with 3.5 days while the last one at 16 days. Figure 8.1 shows the effect of *SRT* on the effluent biodegradable substrate (SS3) and ammonium/ammonia nitrogen (SNH3). The effect on total COD is also shown. Soluble COD decreased rapidly as the SRT was increased from 3.5 to 8 days. After *SRT* of 8 days, the decrease in effluent soluble substrate is very low. The same trend is noticed for the ammonium/ammonia nitrogen. This indicates clearly that after a certain point any addition to the *SRT* will not result in significant enhancements in the effluent quality. This is consistent with practical expectations and literature statements. Grady et al. (1999) have stated clearly that once the *SRT* was enough for effective flocculation and treatment to occur, further increases had only minor effects on soluble substrate removal. Consequently, for domestic wastewaters that contains easily biodegradable substrates, selection of *SRT* is almost always controlled by

factors other than soluble substrate removal. Such was clear in the analysis of the previous chapters (see Section 7.7.3).
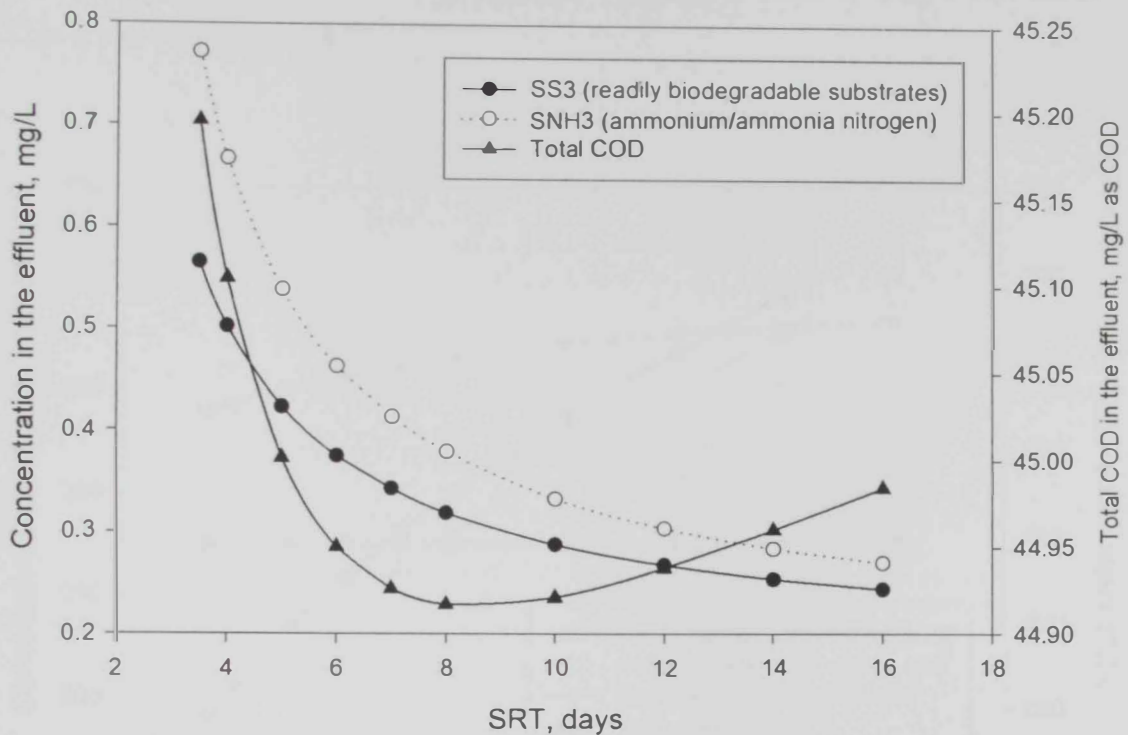
Figure 8.1: Effect of SRT on effluent soluble biodegradable COD, ammonium/ammonia nitrogen, and total COD

In contrary to the above, the total COD in the effluent shows a completely different effect (as shown in Figure 8.1). The COD decreased rapidly as the *SRT* was increased from 3.5 to 8 days. At an *SRT* of 8 days, the COD reached a minimum value. Beyond 8 days the COD increased again. Although such decrease and increase happened only within a range less than 0.5 mg/L, it is still worthy to be noticed. Mathematically, this is attributed to the low decrease in soluble COD after 8 days while the production of biomass and inert particulates which contribute to total COD continue to occur. However, this phenomenon has been noticed in practice and mentioned in literature. Grady et al. (1999) have attributed the increase in effluent COD noticed in systems when the *SRT* increased above a certain value to the production of soluble microbial products.

Effect of increasing *SRT* was obvious on total system cost. Although after 8 days the change in effluent quality was very low, the cost continues to increase with the same rate. Increase in cost is mainly due to the increase in aeration tank volume and oxygen requirement. Both increases are an expected result of higher *SRT* values. This is shown in Figure 8.2.
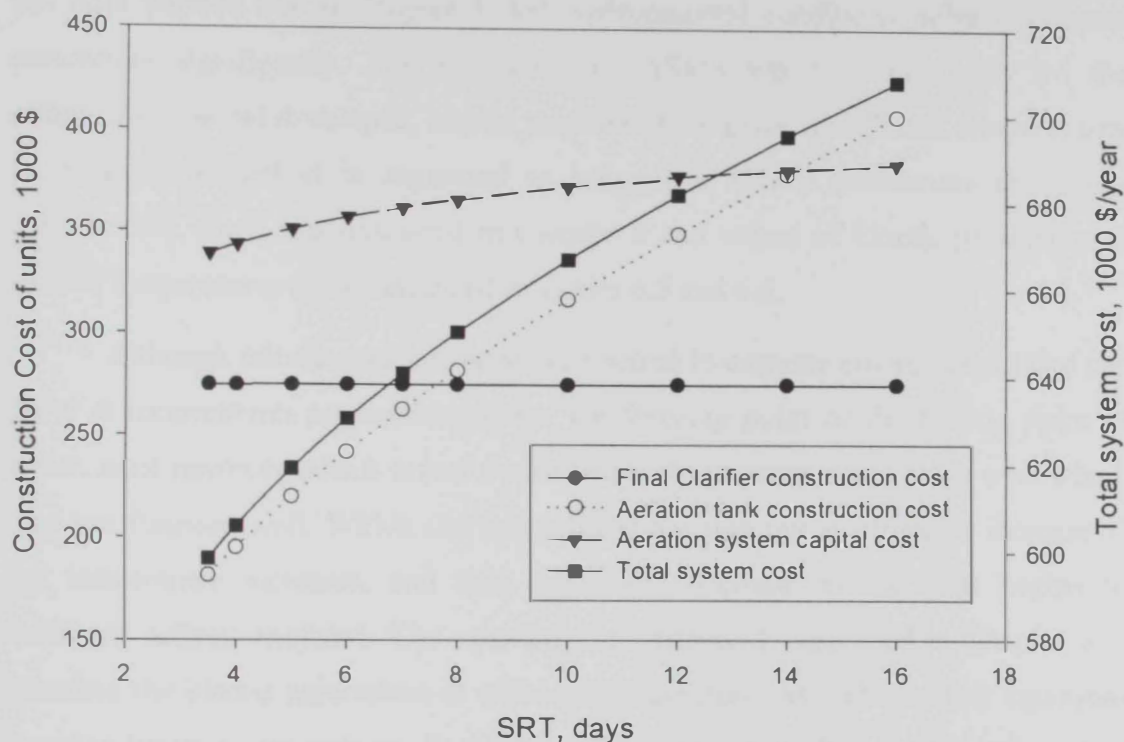
Figure 8.2: Effect of increasing SRT on total system cost

8.2 Effect of Temperature

Wastewater treatment systems are expected to operate in a wide range of ambient temperature. The range varies from less than 10°C to about 50°C. It has been apparently noted in literature that temperature significantly affects different treatment processes. However, this effect varies from one treatment process to another. The effect of temperature on sedimentation processes is still considered a hot spot of research as several researchers have pointed it in one way or another (example is Christoulas et al., 1998). More work is to be done in quantifying the temperature effect on sedimentation processes. In contrast, such effect is obvious on biological treatment. Biological treatment in general and activated sludge in particular is based on biological growth of different species of biomass to remove pollutants from the

wastewater. Temperature affects the performance of activated sludge systems as a result of its impact on the rates of biological reactions.

More technically specking, temperature can exert an effect on biological reactions in two ways: by influencing the rates of reactions and by affecting the rate of diffusion of substrate to the cells. Such an effect is quantified by a change in kinetic and stoichiometric parameters governing the rates of reactions mentioned. It has been pointed out in Chapter 3 that environmental conditions influence kinetic parameters significantly. For a model like ASM3 which is the basis for the optimization model developed, kinetic parameters are given at different temperatures. In addition, a method is suggested to interpolate kinetic parameters at various temperatures. Such was discussed in Chapter 6 and values of kinetic parameters at various temperatures are summarized in Tables 6.5 and 6.6.

Although microorganisms have been found in extreme environments that can grow at temperatures approaching either the freezing point or the boiling point of water, most microorganisms reveal a relatively narrow temperature range over which they can function well. Within that range, most reaction rate coefficients increase as the temperature increases, and then eventually decrease as the heat begins to inactivate cellular enzymes. The equation (Equation 6.13) discussed in Chapter 6 to calculate the kinetic parameters at different temperatures as well as other equations found in literature are only applicable over the range where the coefficient increases with increasing temperature. Microorganisms are grouped into three categories depending on that temperature range. Of great concern in biochemical operations are mesophilic organisms, which grow well over the range of 10-35°C. The two other groups, psychophilic and thermophilic, have ranges on either side and find use only under special conditions.

ASM3 (and ASM1) has been developed based on experience in the temperature range $8 - 23°C$. Outside of this range model application may lead to very significant errors and even model structure may become unsatisfactory.

In this section, the model performance under different temperatures is examined. This is done by optimizing the base model at different temperatures by considering the values of kinetic parameters at these temperatures. In the previous chapter, the base model was designed for a set of kinetic parameters at 20°C. For each

set of kinetic parameters depicted in Tables 6.5 and 6.6 at temperatures 10, 20, 30, 40, and 50°C, a run of the base model was executed. Results are summarized in Table 8.1.

Table 8.1: System design at various temperatures

| Symbol | Units | Temperature, °C | | | | |
		10	20	30	40	50
q	m/d	120	120	120	120	120
Ap	m²	299.797	299.8	299.797	299.797	299.797
SRT	d	8	3.479	5.192	7.083	8.298
HRT	h	6.6	3	3.4	3.7	3.8
V	m³	9924.6	4497	5119.2	5569.5	5771.6
AFR	m³/min	268.4	264.5	307.2	334.2	346.3
$XSS3$	g/m³ as SS	4626.2	4628.0	4617.2	4610.3	4607.1
r	-	0.25	0.25	0.25	0.25	0.25
w	-	0.665	0.696	0.516	0.401	0.349
SR	m/d	18.905	18.90	18.911	18.916	18.918
Af	m²	1890	1890	1893	1894	1895
$SS3$	g/m³ as COD	0.502	0.568	0.284	0.213	0.191
$SNH3$	g/m³ as N	1.0	0.778	0.290	0.224	0.218
$XSS4$	g/m³ as SS	10	10	10	10	10
$Cost$	$/year	653223	598138	621080	635637	642033
Percent change in cost[*]		9.2%	-	3.8%	6.3%	7.3%

[*] Reference to design at 20°C

Obviously, the temperature change did not affect the optimal design of the primary clarifier or the secondary clarifier. Design of both unit operations remains unchanged for the various temperatures examined. Moreover, the primary clarifier overflow rate still at its practical upper bound which indicates that this unit is not effectively participating in the treatment process and economically a reduction in the total cost can be achieved by considering a system without this unit.

Total system cost increased at low temperatures and high temperatures and lowest cost is the cost of a system operating at 20°C. However, the increase in cost at low temperatures differs from the increase at high temperatures due to different reasons.

At low temperatures, the rate of reaction for all the processes is slow. Especially for the autotrophic biomass which is known to have lower reaction rate than heterotrophic biomass. Such low reaction rates of autotrophic biomass affects the treatment process significantly. This type of biomass is responsible for the removal of ammonium/ammonia nitrogen (S_{NH4}). This is at its effluent requirement limit in the design. This indicates that this component has dictated the system to operate at higher

sludge age and higher *HRT* to allow enough time for the autotrophic biomass to remove S_{NH4}. *HRT* is directly related to volume of the aeration tank. And this has increased the system cost.

This becomes clear if we compare the design at 10 and 50°C where the design *SRT* is almost the same while the *HRT* at 10°C is higher and effluent S_{NH4} is at its limit. Hence the volume of aeration tank is higher and so is the cost. At low temperature, the rate is low so the *HRT* increased to the time required to clear the wastewater. In contrast at high temperature, although the *HRT* is much less, the very high growth rate produced better effluent quality of S_S and S_{NH4}.

On the other hand, the condition is more complicated at high temperatures. It is expected that as the temperature increases the rate of reaction increases and shorter sludge age is required. This is not the situation at all as shown in Table 8.1. This is attributed mainly to the bound on the recycle ratio. It is obvious from the table that *r* is the same (at its lower bound) at all temperatures. At low temperatures, as mentioned, reactions rates are low so higher sludge age is required and hence higher recycle ratio too. At high temperatures, reactions rates are faster such less *SRT* is required which means less recirculation. However, since the recirculation is at its lower bound the value remains the same (0.25) which in turns increases *SRT* above the required level. Hence, treatment more than the required level is noticed which in turns increased the volume of aeration tank and the AFR required. Both have caused the increase in cost shown.

Comparing the situation at low and high temperatures, it seems that recirculation controls the biological process and requires longer sludge age at high temperatures. In contrast, at low temperatures soluble substrates rule the situation. This appears clearly if one compares the soluble effluent characteristics at both situations. At low temperatures, soluble components are at their effluent limit while these components at high temperatures are at very low levels.

The effect of the lower bound of *r* on the results obtained can be proved by removing such bound and repeating the same runs shown in Table 8.1. Results are shown Table 8.2. Obviously, completely different designs are obtained at all the temperatures. In contrast, the design at 10°C is very similar to previous design when the lower bound exists except that the recycle ratio is now less and hence cost is less.

This proves the conclusion stated above that at low temperature the effluent requirement of S_{NH4} controlled the design. Also, it proves that at high temperatures overestimation of SRT and hence cost occur because of this bound. It is worthy to mention that at 40 and 50°C the SRT reaches it minimum value (1 day) and hence the cost increases at those two temperatures more than the cost at 30°C. It is important to recall that it is impractical to have activated sludge systems with recycle ratios less than 25%.

Table 8.2: System design at various temperatures after releasing the lower bound on r

Symbol	Units	Temperature, °C				
		10	20	30	40	50
q	m/d	120	120	120	120	120
Ap	m^2	299.797	299.8	299.797	299.797	299.797
SRT	d	8	2.857	1.02	1	1
HRT	h	9	5.8	3.4	3.1	3.3
V	m^3	13422	8730	5055	4628	4980
AFR	m^3/min	268.4	256.5	245.5	272.0	298.8
$XSS3$	g/m^3 as SS	3420.6	2047.7	1384.1	1287.8	1007.9
r	-	0.10	0.029	0.012	0.01	0.006
w	-	0.412	0.243	0.174	0.139	0.089
SR	m/d	19.903	21.722	23.222	23.509	24.512
Af	m^2	1800	1652	1546	1528	1466
$SS3$	g/m^3 as COD	0.502	0.691	1.061	0.504	0.32
$SNH3$	g/m^3 as N	1	1	1	0.404	0.276
$XSS4$	g/m^3 as SS	10	10	10	10	10
$Cost$	$/year	609445	505201	438045	440254	443191
Percent change in cost*		20.6%	-	-13.3%	-12.9%	-12.3%

* Reference to design at 20°C

In summary, temperature affects the activated sludge process design significantly. However, the above discussion is based on the assumption that the kinetic parameters follow in nature the equation mentioned in Chapter 6 for calculating such parameters at different temperatures. For more precise study, such kinetic parameters should be determined at lab using specified experiments at different temperatures. However, this work has a great value in indicating the importance of considering kinetic parameters precisely in the design of activated sludge process.

8.3 Effect of Influent Characteristics

Influent characteristics comprise the wastewater influent flow rate and the concentration of organics contributes to the treatment. Such organics are either soluble or particulate, biodegradable or inert, biomass or substrate. In the base model illustrated in the previous chapter, the influent characteristics were assumed to be for medium flow and strength wastewater. These characteristics are shown in Table 7.4. Obvious from the table the influent characteristics have no biomass at all. In this section, the model performance is examined for various scenarios of influent characteristics.

8.3.1 Influent Flow Rate

In the base solution the influent flow rate was 40,000 m^3/d (1500 m^3/h), which is considered an average for a domestic wastewater treatment. Considering the base solution, performance is examined for other flowrates keeping the concentrations of species the same. Results are summarized in Table 8.3.

Table 8.3: System design at different influent flow rates

Symbol	Units	Influent flow rate, m^3/hour 1000	1500	2000
q	m/d	120	120	120
Ap	m^2	199.9	299.8	399.7
SRT	d	3.479	3.479	3.479
HRT	h	3	3	3
V	m^3	2998	4497	5996
AFR	m^3/min	176.3	264.5	352.7
$XSS3$	g/m^3 as SS	4628.0	4628.0	4628.0
r	-	0.25	0.25	0.25
w	-	0.696	0.696	0.696
SR	m/d	18.90	18.90	18.90
Af	m^2	1260	1890	2520
$SS3$	g/m^3 as COD	0.568	0.568	0.568
$SNH3$	g/m^3 as N	0.778	0.778	0.778
$XSS4$	g/m^3 as SS	10	10	10
$Cost$	$/year	460567	598138	720932
Percent change in cost		-23%	-	20.5%

*Reference to design at 20°C

Clearly, a change in the flow rate affects the system cost but has no effect on the system design. The sizes of various units are changed to accommodate the increase in the amount of wastewater, while biological treatment and their associated variables remain unchanged since the concentrations of influent organics were

unchanged. Such performance is expected and explicable although it is very important. Sizing of units is usually done on the basis of average loads; however, transient conditions should be taken into consideration through a reasonable safety factor to avoid capacity failure.

8.3.2 Strength of Wastewater

It has been mentioned that the base design is assumed a medium strength wastewater in the influent. What will be the performance of the model if the influent was of strong strength wastewater? This is explored in this subsection.

The influent characteristics were varied one at a time to observe the effect of each condition on the system design. As mentioned previously, the influent characteristics comprise soluble or particulate, inert or biodegradable components, and biomass or substrate. Table 8.4 summarizes the system design optimization for various influent conditions along with the influent conditions applied.

In case 1, only readily biodegradable substrate (S_S) is changed to 324 mg/L as COD. Other characteristics are kept unchanged. This resulted in a more expensive system (7.8% increase in cost). The increase in cost is attributed mainly to the increase in aeration tank volume and air flow rate. The volume increase is due to the higher HRT required for the metabolism of the extra S_S concentration. And of course more AFR is needed for this metabolism. However it is noted that the SRT is slightly decreased from the base design (slight increase in w). This is because a higher concentration of heterotrophic biomass is maintained in the aeration tank.

In case 2, only ammonium plus ammonia nitrogen (S_{NH4}) concentration is changed to 50 mg/L as N. Again other characteristics are kept as the base design. This resulted in significant increase in the total system cost (10.6% increase). This is attributed to the significant increase again in the aeration tank volume and the AFR. In contrast to case 1, the AFR increase here is due to the increase in the oxygen requirement of autotrophic biomass while it was there due to the increase in the oxygen requirement of heterotrophic biomass. In this case, SRT suffers significant increase. This is again because of the lower bound imposed on the recycle ratio. However, better quality is noticed in the effluent.

In case 3, both soluble components in case 1 and 2 (S_S and S_{NH4}) are changed together to examine their combined effect. The increase in cost is significant due to

this change (18.1%). Again it is due to the increase in V and AFR which is now at high value due to the increase in the oxygen requirement for both heterotrophic and autotrophic biomass. More increase in HRT is noticed due to the combined increase.

Table 8.4: System design optimization for different influent conditions

Symbol	Units	base	1	2	3	4	5	6	7	8
Influent Conditions										
Q	m³/h	1500	1500	1500	1500	1500	1500	1500	1500	1500
S_I	mg/L as COD	32	32	32	32	32	32	32	64	32
S_S	mg/L as COD	162	324	162	324	162	162	162	324	162
S_{NH4}	mg/L as N	25	25	50	50	25	25	25	50	25
S_{NOX}	mg/L as N	0	0	0	0	0	0	0	0	0
X_I	mg/L as COD	92	92	92	92	184	92	184	184	92
X_S	mg/L as COD	214	214	214	214	214	428	428	428	214
X_H	mg/L as COD	0	0	0	0	0	0	0	0	10
X_{STO}	mg/L as COD	0	0	0	0	0	0	0	0	0
X_A	mg/L as COD	0	0	0	0	0	0	0	0	0
X_{SS}	mg/L as SS	230	230	230	230	299	390	460	460	239
Final Solutions										
q	m/d	120	120	120	120	120	120	113.2	120	120
Ap	m²	299.8	299.8	299.8	299.8	299.5	298.8	315.9	298.1	299.8
SRT	d	3.479	3.444	5.328	5.048	2.857	3.534	2.857	4	3.361
HRT	h	3	3.9	4.2	5.2	3.12	3.6	3.5	5.6	3
V	m³	4497	5799.2	6348.6	7825.2	4672.5	5306.4	5135.5	8322.9	4497
AFR	m³/min	264.5	348.0	380.92	469.51	248.76	318.38	296.07	499.38	263.1
$XSS3$	g/m³ as SS	4628.0	4641.8	4624.4	4637.0	4640.1	4635.5	4646.1	4656.2	4629.6
r	-	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
w	-	0.696	0.928	0.636	0.848	0.9	0.822	1.003	1.176	0.722
SR	m/d	18.90	18.894	18.906	18.897	18.895	18.898	18.891	18.884	18.90
Af	m²	1889.9	1886.4	1890.8	1887.6	1884.9	1881.8	1874.6	1872.0	1889.5
$SS3$	g/m³ as COD	0.568	0.573	0.405	0.42	0.691	0.56	0.691	0.502	0.544
$SNH3$	g/m³ as N	0.778	0.787	0.51	0.534	1	0.764	1	0.667	0.81
$XSS4$	g/m³ as SS	10	10	10	10	10	10	10	10	10
$Cost$	1000$/year	598.1	644.6	661.2	706.2	598.8	638.9	635.4	737.0	597.7
*Percent change in cost**	-	7.7%	10.5%	18%	0.1%	6.8%	6.2%	23.2%	-6.7%	

*Reference to base design

Comparing the above three cases indicates that the S_{NH4} exerts more influence on the system than S_S. This is common since the reaction rate of autotrophic biomass is much less than that for heterotrophic biomass which means more required time for fulfilling the required treatment. Moreover, reasonably the treatment of S_{NH4} is more expensive than the treatment of S_S. All the above three cases have undergone change in the soluble components. The next cases deal with the particulate ones.

In case 4, only the inert particulate organic matter concentration (X_I) has changed to 184 mg/L as COD. This component does not undergo any treatment during the process but it is produced through the aerobic endogenous respiration processes of heterotrophic and autotrophic organisms. So, the amount of X_I increases during the biological treatment and then settles in the final sedimentation tank. The cost of the system after increasing X_I in the influent is not that much different from the base model cost. However, such increase has altered the effluent quality (S_{NH4} at its effluent limit). This is attributed to the following. An increase in X_I in the aeration tank caused a decrease in X_H and X_A to keep the MLSS at its level and hence the volume of the tank at its minimum possible value because it affects the cost significantly. In addition, an extra wastage is required to remove the extra amount of X_I. These actions result in decreasing the SRT significantly and hence altering the effluent quality. Increasing the X_I further in the influent has shown an increase in the system cost due to the increase in cost associated with wastage sludge pumping. The biological treatment remains unaltered. This trend remains valid until the system starts to reach its capacity of removing solids in the primary clarifier and secondary clarifier. Before reaching such limit, the extra amount of X_I added every time is wasted with the wastage sludge out of the system.

The situation is completely different in case 5 when the slowly biodegradable substrate (X_S) is increased to 428 mg/L as COD. X_S is consumed in the hydrolysis process to produce S_S and small amount of S_{NH4}. There is no other reaction that X_S participates in. This is why the influence of increasing X_S is very similar to the influence of increasing S_S. Comparing the system design at the two situations proves this. The only difference comes from the small amount of S_{NH4} produced during hydrolysis. This small amount has required a small increase in the SRT and the HRT. However, it should be mentioned that some amount of X_S has been removed in the primary clarifier and hence not converted to S_S and/or S_{NH4}. Thus, it makes the total amount of S_S and X_S less than the total in case 1. This explains the reduction in AFR required in this case.

In case 6, the influence of combining cases 4 and 5 is explored. The result is a combination of the results of the two cases. The only worthy point to be mentioned is the effective contribution of the primary clarifier. The large increase in influent TSS due to the increase in X_I and X_S has forced the system to rely more on the primary

clarifier to achieve the required treatment. The overflow rate of primary clarifier is not at its upper practical limit which indicates the effectiveness of this unit.

In order to show the economic importance of primary clarifiers in situations like case 6, the same influent characteristics have been applied to a system without a primary clarifier. Results show more load on the secondary clarifier and more total system cost (0.63% increase in cost). In other situations the treatment plant might fail to operate without a primary clarifier.

Case 7 combines all the above mentioned cases. It is analogous to applying a strong strength wastewater instead of the medium strength wastewater considered in the base design. Results show an increase in V, AFR, SRT, HRT, and total system cost. This is expected since the strength of the influent wastewater is higher. However, it is noticed that the design of primary clarifier and secondary clarifier have been unaffected by this change in the strength of influent wastewater. This indicates that the biological treatment alone was able to absorb the increase more economically than the two sedimentation processes.

The last case shown in Table 8.4 (case 8) examines the presence of heterotrophic biomass in influent. The presence of autotrophic biomass in influent is not tested because it is unlikely to happen. Results of case 8 show an identical design to the base one with little difference. The presence of biomass in the influent has helped the system achieve better quality of S_S at lower SRT which at the same time altered the effluent S_{NH4} but it is still at an acceptable limit. The lower SRT has lowered the AFR which in turns cause a decrease in the total system cost. Other than this the system design is similar to the base system design.

In summary, the system optimization has shown reasonable response to various influent conditions. This response differs according to the type and amount of influent characteristics. Soluble pollutants exert different influence than particulate ones. This in conclusion indicates the importance of quantifying influent wastewater characteristics with more care. Uncertainties and shock changes in such characteristics should be taken into consideration when a reliable and robust design is sought.

However, it is worthy of mentioning that incorporated cost functions play a vital role in driving the response of optimization model to different conditions. This is

because an optimization model is always trying to produce the most cost effective design fulfilling the constraints. Therefore, incorporating cost functions more close to reality is a crucial point in developing optimization models. At least relations between different units' costs should reflect the actual relations to assure a reasonable model response.

8.4   Effect of Effluent Characteristics

The effluent requirements applied on the base design are derived from the practical limits recommended in literature. Tightening such limits will indicate the system capacity. Such is discussed briefly in this section.

The system reaches its full capacity when the effluent suspended solids is set to 6 mg/L as SS. Slightly lower than this limit, the primary clarifier reaches its full capacity, so is the secondary clarifier and the biological treatment system. This indicates that for the conditions applied on the base design, the system can not achieve lower concentration than this in the effluent.

CHAPTER 9

CONSIDERING UNCERTAINTY

For the last decades there has been a growing awareness of the existence of random properties "uncertainties" in environmental systems. In general, uncertainty is an inherent property of modeling. It is not realistic to expect that a model of any type performs perfectly. This is especially true when dealing with biological systems which are usually subject to natural variations. Activated sludge process models, as biological systems, are subject to several sources of uncertainties.

Up to now, within the design of wastewater treatment plants, deterministic models were used to evaluate different scenarios on their merits of effluent compliance. One of the remaining important issues when dealing with these deterministic models is the degree of uncertainty linked to their predictions. In other words, to what extent can the predictions of the model be taken for reality? The consideration of uncertainty in the design process could provide the answer for such a question. Design under uncertainty as well as reliability and risk assessment are gaining a great attention from researchers in almost all fields of engineering design and analysis.

The goal of this chapter is to evaluate uncertainties associated with parameters of the optimization model developed in the previous chapters. Formerly, the optimization model has been utilized in an illustrative problem where all parameters were assumed to be deterministic and assigned certain values. However, such parameters, in reality, involve significant variability.

9.1   Sources of Uncertainty

Uncertainties usually arise from several components contributing importantly to the design process. These include mainly the prediction of influent characteristics where the input into the plants is always variable (not only the amount of input but also its

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characteristics), the parameter estimation (including kinetic and stoichiometric), cost information, possible changes in water quality regulations, and the lack of knowledge about the performance of some unit processes. Moreover, design is usually carried out by assuming steady-state conditions, while an operating wastewater treatment plant is more likely to receive influent varying with time in terms of quantity as well as quality.

Three main sources of uncertainty can be identified: uncertainty in the model structure itself, variability in influent wastewater characteristics considered, and uncertainty in the performance models parameters.

By uncertainty in model structure is meant the uncertainty caused by everything that is not modeled, in other words, the uncertainty caused by all processes which are not included in the model. In the optimization model, several performance models as well as cost information equations are implemented. Each involves a degree of uncertainty. For example, the clarification model of primary clarifier was developed to best describe the process of clarification in primary clarifiers. However, verification of this model shows some variability of its results from the real situations even after a proper calibration is done. Another example is the cost information equations. These equations were developed based on observations from different developed wastewater treatment plants and corrected to match the current market value using a cost index. Both the developed equations and the cost index are not deterministic measures and they involve high level of variability.

The other two uncertainties are obvious in the design of activated sludge systems. Fluctuations in flow and characteristics are always expected in the inflow of treatment plants. Influent characteristics were assumed to be deterministic and of medium strength in the illustrative problem in Chapter 7, however in reality this is not the case. Influent flow rate can be controlled and fluctuations can be minimized through the use of pumping or equalization tanks. This is preferred and hence applied in most wastewater treatment plants. In contrast, influent characteristics are greatly fluctuating and difficult to control. As an example, the total COD in the influent to Mafraq wastewater treatment plant has ranged between 265 and 540 mg/L as COD and the TSS has ranged between 124 and 270 mg/L as TSS during the month of

September, 2001[1]. This raises a fundamental question: which concentration should be considered as the base for design? Designers used to design for the worst case which may happen once during the lifetime and hence invest extra unnecessary cost. Design under uncertainty should give a more comprehensive and economical answer to the mentioned question. In such design the fluctuations in influent characteristics are considered based on their probability to occur and influence on design.

Uncertainty in parameters arises from parameter measurements and estimation. Performance models parameters include parameters of clarification model of primary clarifier, settling constants of primary and secondary clarifiers, parameters in secondary clarifier clarification model, and kinetic and stoichiometric parameters of ASM3. In the presented illustrative problem these parameters were assigned values based on literature recommendations. However, all these parameters are subjected to variability. Such variability may be attributed to experimental methods uncertainty or/and temperature dependency. For example, kinetic parameters are characterized through specified experimental methods and at the same time they are temperature dependent.

In more details, the empirical clarification model of primary clarifier (Christoulas et al., 1998) includes three positive parameters a, b (mg/L), and c (d/m). The parameters a and b were found related to temperature and using a linear regression procedure these relations were formulated with r=0.76 significant at 93% probability level and r=0.99 significant at 99% probability level, respectively (see Chapter 2). However, it has been stated that such relations are valid only for a temperature range 15-26°C. Empirically also, c was determined to be 0.0035.

The thickening performance of primary clarifier is modeled according to the solids flux theory as given by Cho et al. (1996) (see Equation 6.5). Two parameters appear in this model (k and n). These parameters came from the empirical model describing the relation between settling velocity and sludge concentration. There are several models used to describe such a relation like the power model as an example. The model is used on the basis of best fitting the relation mentioned. Hence, the parameters mentioned are determined through a regression analysis depending on the empirical model utilized to describe the settling velocity (see Chapter 2). Like the

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[1] Personal communication with Mafraq wastewater treatment plant management.

parameters of the primary clarification model, the uncertainty of these parameters is taken into consideration when their value is to be determined and there is no need again to consider them as uncertain. Cho et al.'s (1996) model is also utilized to describe the thickening performance of the secondary clarifier with model constants k_w and n_w. The discussion above is applicable on these parameters.

The clarification behavior of the secondary clarifier is modeled with the empirical relation of Voutchkov (1992) (Equation 6.26). In this relation the effluent suspended solids is related to MLSS, surface overflow rate, SVI, and sidewater depth in the tank. MLSS and the surface overflow rate are variables in the model while SVI and sidewater depth (H) are considered as parameters. SVI is determined according to standard tests for the wastewater under consideration while H ranges according to practical recommendations between certain values. In the illustrative problem SVI is assigned a value of 150 mL/g while H is assumed as the minimum possible depth which is 3.7 m. SVI of course involves variability while H here assumes the worst case. Obviously, H is a deterministic parameter while SVI, due to errors associated usually with experimental work and site conditions affecting its value, is not a deterministic parameter and should be dealt as uncertain parameter.

The core of the introduced optimization model is the activated sludge model ASM3. This model comprises major number of model parameters. It includes, as all activated sludge models, stoichiometric parameters and kinetic parameters (see Tables 6.3 and 6.5). Both types of parameters are determined practically using experimental procedures. Due to experimental errors, both involve a certain level of uncertainty. In addition to this, as mentioned previously, most kinetic parameters are temperature dependent. Even if the experimental quality control was at its best level and the values obtained for kinetic parameters were of minimum uncertainty, temperature fluctuations in reality add another dimension of variability.

In the introduced model, the stoichiometric and kinetic parameters are assumed values as recommended by the ASM3 developers at 20°C. ASM3 is introduced originally in the form of a stoichiometric and composition matrix, which is reduced to stoichiometric matrix based on suggested values for the stoichiometric and composition parameters appear in the original matrix. In the introduced optimization model, the stoichiometric and composition parameters values suggested by Henze et al. (2000) were used to produce a stoichiometric matrix of ASM3 considered as the

basis for describing activated sludge process in the model (see Chapters 3 and 6). Hence, in this formulation, stoichiometric and composition parameters are imbedded in the model. Thus, kinetic parameters, only, remain as the parameters associated with ASM3 model and they are a major source of uncertainty.

The remaining parameters in the model are divided into two groups. The first is parameters associated with constraints on aeration tank volume. These involve uncertainty, like the parameter describing efficiency of diffuser, which depends on diffuser type, and depth at which air pumped, this uncertainty has a minor effect on model behavior since it does not contribute in the calculations directly. The second group is parameters associated with objective function calculation. Capital recovery factor, cost index, base cost index, operating and maintenance wages, and electricity cost, all cannot be considered as uncertain parameters since designer chose these parameters based on current situation and if changed their rate of change is very low which allows another analysis to be conducted with the new parameters values. In contrast, the other two parameters associated with the calculation of power consumption cost of pumping, i.e., pumping head and pumping efficiency, are uncertain parameters. They are subjected to variability during operation and hence affect the cost of pumping significantly.

Summarizing the above discussion, uncertainty in the introduced optimization model is attributed to several sources. The main source is the influent wastewater characteristics including flowrate, pollutant concentrations, and kinetic parameters. There are other parameters which involve a level of uncertainty but they are of less importance. These are *SVI*, pumping head and pumping efficiency of primary sludge pumps and secondary sludge pumps.

9.2   Sensitivity Analysis

As mentioned previously, sources of uncertainty in the developed model are attributed to influent wastewater characteristics including flowrate, pollutant concentrations, and kinetic parameters. Other minor parameters are ignored in this analysis. Sensitivity of influent flow rate and pollutant concentrations were assessed in Chapter 8 when the model performance was monitored for different conditions. In that analysis it has been concluded that influent flow rate only affects the sizes of different treatment units while biological state variables remain unchanged. Such uncertainty effect can be

conquered by either considering the peak flowrate as the design flow rate or applying a proper safety factor to the influent flowrate. This will account for the variability and will produce a reliable design in terms of influent flowrate.

In contrast to influent flowrate, influent pollutant concentration shows an ·obvious effect on model performance. Different designs and effluent conditions were obtained with different combinations of influent characteristics. Design and effluent were obviously altered even when only one influent characteristic was changed. The best way in dealing with variability of influent characteristics is finding a probability distribution that best describes their viabilities. Such an approach was followed by Rousseau et al. (2001), as mentioned previously. In that study for every component of influent a triangular distribution was imposed between minimum and maximum values calculated according to extensive measurements conducted on several wastewater treatment plants. Then Monte Carlo simulation was used to conduct the uncertainty analysis (Rousseau et al., 2001).

The remaining random parameters are the kinetic parameters. It has been mentioned that these parameters are random due to estimation and due to their temperature dependency. In Chapter 8, the model performance for different sets of kinetic parameters at various temperatures was assessed. In general, temperature variation affects kinetic parameters which in turn affect the model performance significantly. In that analysis, kinetic parameters were all changed according to a method discussed in Chapter 6 to show the values at certain temperatures. However, this is a rare situation which does not happen in real life. Conventionally, a certain parameter might assume a value above or lower than the expected value. In such a situation the model response is questioned. The answer would provide an insight about the value (importance) of kinetic parameters uncertainty. In this section, sensitivity of a model to kinetic parameters variations at low and high temperatures is explored.

At low temperature, 20°C, the kinetic parameters are assigned values suggested by Henze et al. (2000) and shown on Table 6.5. High temperature is considered at 40°C and the corresponding kinetic parameters are given in Table 6.6. For every parameter, at 20 and 40°C, three runs were conducted, one at the suggested parameter value, another at 50% of this suggested value, and the third at 150% of it. At each run other values were kept at their original values. The base model illustrated

in Chapter 7 was considered the base for all conducted runs. Table 9.1 shows the percentage change in objective function (total cost) due to ±50% change in parameter value. A minus sign indicates a reduction in cost while positive one indicates an increase, referred to values obtained (Table 8.1) at zero variability of each relevant temperature.

Table 9.1: Percentage change in optimal total cost due to variations in kinetic parameters

Kinetic Parameters	At 20°C		At 40°C	
	-50%	+50%	-50%	+50%
k_H	-0.1511	0.0469	-0.0615	0.0164
K_X	0.0630	-0.0561	0.0222	-0.0222
k_{STO}	0.0296	-0.0065	-0.0168	0.0048
K_S	0.0083	-0.0083	0.0068	-0.0068
K_{STO}	-0.0006	-0.0002	-0.0139	0.0128
μ_H	-0.0047	-0.0004	0.0318	-0.0099
K_{NH4}	0.0000	0.0000	-0.0006	0.0007
$b_{H,O2}$	**-1.5146**	**1.8623**	**-2.0542**	0.6479
$b_{STO,O2}$	-0.0601	0.0571	-0.0285	0.0263
μ_A	**11.338**	0.0959	-0.1320	0.0323
$K_{A,NH4}$	0.1019	**1.2096**	0.0428	-0.0420
$b_{A,O2}$	0.0042	0.3355	0.0030	-0.0368

It is obvious from the table that variability of kinetic parameters has different effect on the model optimum solution. Even for the same parameter, effect at low temperatures differs from the effect at high temperatures. There is no general trend that can be drawn. Moreover, all the changes are negligible except the ones imposed by the variability of b_H, μ_A, and K_A. The effect of changes imposed by b_H and K_A are still small (less than 2%) and can be neglected. The most apparent effect is due to a reduction in μ_A assumed value by 50% at 20°C. The assumed value at this temperature is 1.0, which means if μ_A becomes for a reason or another 0.5 then a system with 11.3% higher cost is required to achieve the same treatment requirements. This indicates that the system is very sensitive to this parameter and any weak estimation of it would lead to system failure.

Sensitivity of the model to μ_A makes sense. μ_A is the autotrophic maximum growth rate which is responsible for nitrification in the activated sludge process. The developed model has been assumed to perform complete nitrification that lowers the concentration of ammonium/ammonia nitrogen in the effluent to less than or equal to 1.0. It is well known that the growth rate of autotrophic biomass is naturally very

slow. Then any alteration in this growth rate (variability of μ_A) would affect significantly the nitrification process which in many cases limits the solution, especially at low temperatures as has been indicated before. In the case shown, the decrease in the growth rate required the system to increase the SRT to allow more time for nitrification. Hence the system cost increased significantly.

It should be noted that the discussed sensitivity analysis above was conducted by varying one parameter at a time. However, in reality all parameters might show different values at the same time. And hence the combined effect on system performance will be totally different. The aim of the above sensitivity analysis was only to compare the impact of variability of various individual parameters.

In summary, the system is most sensitive to variability of influent characteristics and maximum growth rate of autotrophic biomass (μ_A). Variability of these parameters should be considered in the design of activated sludge plants. Ignoring such variability would imply a serious risk and possibility of failure is expected. Hence such variability is considered in the coming example of uncertainty based design. In contrast, variability of other parameters also exists, however their influence compared to influent characteristics and μ_A influence is minor. In a comprehensive analysis, all random parameters should be considered because a combined effect can be expected.

9.3   Uncertainty Based Optimal Design

It has been shown in the previous section that a group of parameters are random parameters and their variability influence greatly the performance of the model. In general a random parameter is a real valued function that is defined on the sample space which is the total possible outcomes of the process of observations. A random parameter can be discrete or continuous. Statistical properties of a random parameter can be categorized into three types: (1) descriptors showing the central tendency; (2) descriptors showing the dispersion about a central value (variability); and (3) descriptors showing the asymmetry of a distribution. The commonly used statistical properties are the arithmetic mean and median for the first type, variance and standard deviation for the second type, and coefficient of skewness for the third type. Variance and standard deviation which are often used as the measure of the degree of

uncertainty associated with a random parameter are the most common (Mays and Tung, 1992).

In uncertainty analysis, random parameters can be described by probability distributions where several distributions are frequently used. Based on the nature of the random parameter, probability distributions can be classified into discrete distributions and continuous distributions. Continuous distributions can be approximated to discrete distributions. The most common distribution is the normal distribution.

In the design and analysis of systems involving random parameters, the effort is concentrated on approximating the prediction uncertainty. Several techniques can be implemented which all approximate the mean and variance of a model given the variance of a parameter set. The mean is the expected output of the model and the variance is a measure for the uncertainty. One well known and frequently used technique is the Monte Carlo Simulation technique. In this technique, a deterministic model is run repeatedly with every run a different set of parameter values. These parameter values are determined at the beginning of every run form specific probability distributions. Monte Carlo simulation is frequently used since it is conceptually very simple and easy to use given some previously developed random number generators. A common application is the approximation of the prediction uncertainty of the model given known probability distributions for the parameters.

In GAMS, the implemented optimization software in this study, there is an imbedded solver to deal with uncertainty. DECIS is a system for solving large-scale stochastic programs, programs, which include parameters (coefficients and right-hand sides) that are not known with certainty, but assumed to be known by their probability distribution. It employs Benders decomposition and allows using advanced Monte Carlo sampling techniques. DECIS includes a variety of solution strategies (techniques), such as solving the universe problem, the expected value problem, Monte Carlo sampling within Benders decomposition algorithm, and Monte Carlo pre-sampling. For details about the DECIS system consult the DECIS User's Guide, see Infanger (1997). Although DECIS is a powerful tool to assess uncertainties of models, it is quite difficult in model formulation. It deals only with coefficients and right-hand sides which makes it difficult to define random parameters especially when complex models are considered. In the model under study, very complicated rate

equations of pollutants appear. Defining random parameters within these equations is a complex process and further analysis is required to overcome this problem. Because of the aforementioned difficulties associated with applying GAMS/DECIS on the model under study, the uncertain model design is generated using the expected value problem. The expected value problem results from replacing the stochastic parameters by their expectations. Although it can be solved using GAMS/DECIS, it is easier to be solved by employing a solver directly (CONOPT2 or MINOS). Solving the expected value problem may be useful by itself, and it also may yield a good starting solution for solving the stochastic problem.

The random parameters defined in the previous section are assumed a discrete probability distribution shown in Table 9.2. The table shows part of influent characteristics, characteristics not shown are considered as deterministic parameters. From such a distribution, the expected value of each parameter is found, which is in turn implemented in the optimization model.

Table 9.2: Discrete probability distribution of random parameters in the optimization model

Symbol	Outcome (probability)				
Q	1000 (0.04)	1200 (0.06)	1500 (0.8)	1700 (0.06)	2000 (0.04)
S_I	29 (0.05)	30 (0.075)	32 (0.75)	35 (0.075)	36 (0.05)
S_S	100 (0.05)	130 (0.15)	162 (0.6)	170 (0.15)	190 (0.05)
S_{NH4}	19 (0.01)	20 (0.04)	25 (0.9)	40 (0.04)	50 (0.01)
X_I	85 (0.04)	90 (0.06)	92 (0.8)	100 (0.06)	110 (0.04)
X_S	195 (0.04)	200 (0.06)	214 (0.8)	220 (0.06)	230 (0.04)
X_{SS}	210 (0.04)	217 (0.06)	230 (0.8)	240 (0.06)	255 (0.04)
μ_A	0.5 (0.02)	0.7 (0.08)	1 (0.8)	1.1 (0.08)	1.2 (0.02)

The expected value for a discrete distribution is calculated as follows. If X is a discrete random parameter and $f(x)$ is the value of its probability distribution at x, the expected value of X is (Miller and Miller, 1999):

$$E(X) = \sum_x x \cdot f(x)$$

(9.1)

Table 9.3 shows the expected values of the random parameter calculated according to Equation (9.1) and compared to assumed deterministic values. Table 9.4 shows results of uncertain based design model compared to the deterministic model solved as an illustrative problem in Chapter 7.

Table 9.3: Deterministic value versus expected value for random parameters

Symbol	Deterministic value	Expected value
Q	1500	1494
S_I	32	32.125
S_S	162	156.7
S_{NH4}	25	25.6
X_I	92	92.8
X_S	214	213.4
X_{SS}	230	230.02
μ_A	1	0.978

Table 9.4: Uncertainty based design compared to deterministic design of the illustrative problem discussed in Chapter 7

Symbol	Units	Deterministic	Uncertainty based
q	m/d	120	120
Ap	m^2	299.8	298.6
SRT	d	3.479	3.513
HRT	d	0.125	0.125
V	m^3	4497	4479
AFR	m^3/min	264.5	263.1
$XSS3$	g/m^3 as SS	4628.0	4627.5
r	-	0.25	0.25
w	-	0.696	0.688
SR	m/d	18.90	18.9
Af	m^2	1890	1882
$SS3$	g/m^3 as COD	0.568	0.563
$SNH3$	g/m^3 as N	0.778	0.8
$XSS4$	g/m^3 as SS	10	10
$Cost$	$/year	598138	596428

It is noticeable from the new design that more capacity has been added to the system in terms of size to account for uncertainties in influent flowrate. In addition, other design parameters were adjusted to fulfill the treatment requirements of the random influent characteristics. In terms of nitrification, it is obvious that the perfect case of assuming the maximum growth rate (μ_A) as 1.0 is no longer existent and a higher concentration of ammonium/ammonia nitrogen ($SNH3$) appears in the effluent.

CHAPTER 10

CONCLUSIONS AND FUTURE RESEARCH

The activated sludge process is the most widespread biological treatment technique that has attracted for the last forty years the concern of researchers from different fields. Design of the process to meet treatment requirements with minimum costs has always been a challenge. Traditional design methods are based on simple understanding (low accuracy) of the process and hence may ignore critical considerations or involve a large number of sources of uncertainties. Mathematical modeling of the process is a crucial requirement for design and analysis. Models of activated sludge process have ranged from simple to advanced over the last four decades. Several noticeable advancements have occurred in the understanding and modeling of the process. Such models have played a valuable role in introducing the process to the design engineers in more explicable manner. Moreover, they have become the base for new design approaches of higher accuracy that are clearly more reliable than old approaches.

In contrary, even with the very advanced models as the base for design and analysis, there are still remaining issues that need to be considered for a comprehensive design. The main issue is the need for a cost-effective design. Such have triggered the research about developing cost-effective "optimization" models to be used for design and analysis. Such incorporate mathematical performance models with cost information functions. Since 1962, when the first known study in this field was published, several efforts have been devoted to develop models that optimize different wastewater treatment systems including the common activated sludge systems. A literature review of such efforts is given in Chapter 1 with more emphasis on recent studies. Exploration of such literature reveals that various studies differ in many aspects and several improvements and contributions can be done.

This thesis is an attempt to contribute to the formulation of a model that finds the optimum sizing of an activated sludge system taking into consideration the latest progression in activated sludge process understanding and modeling.

In this thesis, an optimization model for use in the design of activated sludge wastewater treatment systems is developed. The model incorporates in addition to activated sludge system, a primary sedimentation unit operation. State-of-the-art process design mathematical models to predict the performance of various unit operations are considered. A typical activated sludge system layout (Figure 6.1) is considered in the system development. Such layout is applicable to most operated activated sludge wastewater treatment plants and with few modifications it can be applied also to various reactor types, flow regimes, and activated sludge process variations.

In the developed model, the objective function is the total cost (capital and operation) of the considered system. The governing equations are the performance models describing the behavior of different unit operations in the system. Constraints involve practical limits on design variables and of course effluent constraints. The overall objective of the model is to size the various unit operations fulfilling the effluent requirements with minimum total system cost. The first unit in the system, the primary clarifier, is modeled considering its two functions, clarification and thickening. Clarification is modeled according to a recent study by Christoulas et al. (1998) while thickening is modeled according to the solids flux theory as given by Cho et al. (1996). Similarly, the last unit, the secondary clarifier, is modeled considering its two functions. Clarification performance is assumed to follow the model of Voutchkov (1992) while thickening is modeled, as in the primary clarifier, according to Cho et al. (1996). Unlike other similar studies in literature, no assumptions are implemented in modeling the behavior of these two units. Many old studies assumed perfect purification in the secondary clarifier, which is impractical. In this study, a model describing this process is implemented with practical limitations on design parameters and variables. Indeed, all behaviors that affect the design are modeled according to recent verified and tested models.

The core part of activated sludge systems is the biological treatment accomplished in aeration tanks. This is modeled according to the recent model ASM3 of the IWA. ASM3 was developed in 1999 in order to create a tool for use in the next

generation of activated sludge models. ASM3 incorporates the latest developments in understanding activated sludge system behavior. It has the ability to simulate organic carbon removal, nitrification, and denitrification. In this study, it has been considered that the system includes only aerobic aeration tanks. Hence the denitrification performance depicted in ASM3 is omitted. This has been done through a systematic approach to derive a new reduced order model.

The reduced order ASM3 based model developed in this study is based on reasonable assumptions such as neglecting the alkalinity that has minor effect on other processes in the model. Such a model can be used for other purposes other than this study and a good idea would be to test it and compare it to other models including the full ASM3 model.

Performance models of the three unit operations were put together to form a system model with mass balance equations. It is worth mentioning that the selected performance models cannot be declared as the best models representing the reality because every model has its particularities and restrictions. This is especially true for empirical models. Such models are developed for certain situations and using them in other situations may be associated with a certain level of error. Using them for a particular treatment plant for a certain type of wastewater requires calibrating their parameters to represent the wastewater/plant under study. The models chosen, although recent and advanced, are only examples of how the models can be incorporated in such type of problems.

To use the optimization framework introduced in this study for certain plant or design, models then should be chosen based on the best representing the wastewater/plant in question. Sometimes developing a specific model would be a better option.

It has been mentioned that optimization problems have three main portions (see Figure 1.1). The aforementioned system model forms the first potion which is mathematical models governing the system. The second portion is the objective function that is the total system cost. Due to the absence of cost functions describing the cost information for wastewater treatment plants in the UAE, it has been decided to use cost information introduced in literature and developed for other regions. The cost functions utilized in this study are derived from the functions developed by Tang

et al. (1984) and the ones introduced by Tyteca (1985) and scaled to 2003 year cost using the Engineering News Record construction cost index of 2003. Therefore, the cost functions considered in this study are only meaningful in the sense that they represent typical relative costs among unit processes. This is justified since the main purpose of the study is not cost estimation but optimization of a typical treatment system where relative costs of different incorporated units is the most important not the cost itself as a figure.

Third portion is the optimization technique. The optimization software GAMS has been used in this study to perform optimization runs. GAMS is a powerful tool that includes various solvers suit a wide variety of problems. The developed model was programmed in GAMS language considering governing equations, objective function, and constraints. Listing of the program is given in Appendix A. GAMS solvers was found to be robust in dealing with various conditions affecting, usually, any optimization problem.

The use of the developed model was demonstrated through an illustrative problem (Chapter 7). In this problem, typical influent wastewater characteristics as well as typical parameters values were considered. Results were discussed considering several aspects. Robustness of solution has been noticed since several starting solutions did not affect the final solution indicating that the obtained solutions are possibly global optimal solutions. Discussion of the results shows the importance of engineering intuition and perception in designing activated sludge systems. Although the results obtained were reasonable and showed good agreement with practical expectations, further analysis was required to refine the obtained design and explore other possibilities. Furthermore, performance of the developed model under various conditions was explored. Effect of *SRT*, temperature, influent characteristics, as well as effluent requirements were investigated via many optimization runs. Results were reasonable and explicable. Valuable information was obtained from such analysis. This indicates obviously that the developed model can be used as an analysis tool in addition to design tool. Moreover, performance analysis could answer several questions raised about the performance of operated activated sludge plants.

The following is a list of specific findings obtained while testing the model performance (Chapters 7 sand 8):

- Objective function which is based on cost information utilized affects significantly the results. Hence, choosing the most representing cost functions especially in terms of relative costs of various units is a crucial requirement to obtain reasonable results.

- One of the main advantages of the model shown in this study is the coupling of three important unit operations in one model accounting for the interactions between the three units. Considering such interaction is very important. A designer could determine whether a primary clarifier is required or a system without it is more economical. In addition, the interaction between the biological treatment in the aeration tank and the sedimentation in the final settling tanks is also modeled. Such interaction remains a big challenge when designing a wastewater treatment system. By using a model like the one proposed in this study, the interactions can be investigated easily and no worries from biological treatment that cause failure in the subsequent settling.

- It has been shown in most of the runs for the system considered that the primary clarifier is not an effective unit and cost savings can be accomplished by considering a system without a primary clarifier. Indeed, this is mathematically true; however, other factors should be taken into consideration. The first is the accuracy of the cost information used. The second is the role of the primary clarifier in reducing the inert solids entering with the influent. Such solids have serious effect on the subsequent biological treatment. It might exert an inhibition effect and alter the activity of the biomass. Such effect is not modeled and hence results should be judged against it. More improvements in results can be achieved by modeling such effects.

- Effect of temperature on model performance is obvious. Huge cost savings can be achieved by controlling the process at different temperatures. It has been shown, at high temperatures, operating at low sludge age by controlling the recycle flow would cause remarkable reduction in cost.

The last chapter in this study explores the issue of considering uncertainty in design of activated sludge systems. Sensitivity analysis was accomplished to find out the most impressive design parameters that affect the design significantly. Finally, an

example of implementing uncertainty in design was given through an expected-value problem that was then compared to deterministic design. As expected, considering the parameters uncertainties in design produced completely different result of higher costs than the deterministic design results.

In conclusion, the developed optimum sizing approach is a good aid for design engineers. An important advantage of such is the combination of simulation and optimization in one formulation that enables users to use it for design and analysis purposes. Future research depending on this work is discussed in the next section.

10.1 Future Research

Several potential research areas in process modeling and design were investigated during the development and utilization the optimization model. However, there are other areas that deserve future investigation. Two main directions can be identified. The first is related to model development and the second is about utilizing the model for various purposes.

Concerning the first direction, several areas need further investigations. These include:

1- Performance models of various unit operations still need more refinements. Models that describe clarification/thickening behavior of settling tanks are very important while more investigations still required. Development of models that account for more design conditions and realistically describe such performance is a crucial requirement. The best option would be to develop specific purpose models when a certain problem is being addressed.

2- Cost functions are great sources of uncertainty in such type of problems. Development of cost functions for UAE would reduce such uncertainty significantly.

3- The developed model can be extended to include other types of reactors, flow regimes, and activated sludge process variations. Reactors with mechanical aeration and plug flow pattern can be imbedded in the model with minimum modifications.

4- Implementing denitrification, which is omitted in this study, is also a possible extension. In this regard, anoxic conditions should be assumed in the aeration tank. Phosphorus removal also can be added. ASM2 model incorporates denitrification and phosphorus removal and can be used for such a purpose.

Concerning the second direction, several studies and research can be conducted utilizing the developed model. These include:

1- Sensitivity analysis where information on the effect of a particular parameter in the model on the overall system design is useful for system design and analysis. Example of such studies are given in this work that can be extended to cover all parameters in the model. Potential research areas can be identified where the system is very sensitive to a particular parameter.

2- Reliability and uncertainty analysis using the system developed can be evaluated. Valuable information can be obtained when uncertainty of parameters is implemented. This information helps to establish guidelines for practical system design. A simple example of implementing parameters uncertainty is given in this study. Such can be extended to couple a more comprehensive uncertainty analysis tool like Monte Carlo simulation to the developed model. Results of such coupling should produce a design that is more reliable.

The developed model can be applied on realistic situations. These include newly designed and operated wastewater treatment systems. A possible option is to apply the model on one of the treatment plants operating in the UAE. In such a case, performance models should be verified and design parameters should be determined through experimental procedures and field measurements. Such a study could reach a valuable design and operation recommendations.

REFERENCES

1. Akça, L., C. Kinaci, and M. Karpuzcu (1993), "A Model for Optimum Design of Activated Sludge Plants", *Water Res.*, 27, 9, pp. 1461-1468.

2. Amano, K., K. Kageyama, S. Watanabe, T. Takemoto (2002), "Calibration of Model Constants in a Biological Reaction Model for Sewage Treatment Plants", *Water Res.*, 36, pp. 1025-1033.

3. Argaman, Y. (1995) "A Steady-State Model for the Single Sludge Activated Sludge System-I. Model Description". *Water Res.*, 29, 1, pp. 137-145.

4. Argaman, Y., and G. Papkov (1995), "A Steady-State Model for the Single Sludge Activated Sludge System-II. Model Application". *Water Res.*, 29, 1, pp. 147-153

5. Asfari, A. (2000), *Fundamentals of Sewage Facilities Design*, 1st ed., Arabic Scientific Organization, Tunis (In Arabic).

6. Ayesa, E., B. Goya, A. Larrea, L. Larrea, and A. Rivas (1998), "Selection of Operational Strategies in Activated Sludge Processes Based on Optimization Algorithms", *Water Sci. Technol.*, 37, 12, pp.327-334.

7. Barker, P.S. and P.L. Dold (1997a), "General Model for Biological Nutrient Removal Activated-Sludge Systems: Model Presentation", *Water Environment Research*, 69, 5, pp. 969-984.

8. Barker, P.S. and P.L. Dold (1997b), "General Model for Biological Nutrient Removal Activated-Sludge Systems: Model Application", *Water Environment Research*, 69, 5, pp. 985-991.

9. Benefield, L. D. and C. W. Randall (1980), *Biological Process Design for Wastewater Treatment*, Prentice-Hall Inc., Englewood Cliffs, NJ.

10. Bischof, F., M. Hofken, and F. Durst (1996), "Design and Construction of Aeration Systems for Optimum Operation of Large Wastewater Treatment Plants", *Water Sci. Technol.*, 33, 12, pp. 189.

11. Brooke, A., D. Kendrick, A. Meeraus, and R. Raman (1998), *GAMS A User's Guide*, GAMS Development Corporation, Washington.

12. Carlsson, B. (1998), *An Introduction to Sedimentation Theory in Wastewater Treatment*, Report, Systems and Control Group, Uppsala University.

13. Chachuat, B., N. Roche, and M.A. Latifi (2001), "Dynamic Optimization of Small Size Wastewater Treatment Plants Including Nitrification and Denitrification Processes" *Computers and Chemical Engineering*, 25, pp. 585-593.

14. Chapman, D. T. (1983), "The Influence of Process Variables on Secondary Clarification," *J. Water Pollution Control Federation*, 55, 12, pp. 1425-1434.

15. Chatellier, P., and J. M. Audic (2000), "A New Model for Wastewater Treatment Plant Clarifier Simulation", *Water Res.*, 34, 2, pp. 690-693.

16. Cho S., F. Colin, M. Sardin, and C. Prost (1993), "Settling Velocity Model of Activated Sludge", *Water Res.*, 27, 7, pp. 1237-1242.

17. Cho, S., H. Chang, and C. Prost (1996), "Steady State Analysis of the Coupling Aerator and Secondary Settling Tank in Activated Sludge Process", *Water Res.*, 30, 11, pp. 2601-2608.

18. Christoulas, D.G., P.H. Yannakopoulos, and A.D. Andreadakis (1998), "An Empirical Model for Primary Sedimentation of Sewage", *Environment International*, 24, 8, pp. 925-934.

19. Columbo, J., and J. Nelson (1994), "Processing Biosolids for Market Value", *Environ. Prot.*, 5, 9, pp. 24.

20. de Melo, J.J., and A.S. Camara (1994), "Models for the Optimization of Regional Wastewater Treatment System" *Euro. J. Oper. Research*, 73, 1.

21. DeGarmo, E., W. Sullivan, J. Bontadelli, and E. Wicks (1997), *Engineering Economy*, 10th ed., Prentice-Hall International, New Jersy.

22. Diehl, S., and U. Jeppsson (1998), "A Model of the Settler Coupled to the Biological Reactor", *Water Res.*, 32, 2, pp. 331-342.

23. Doby, T.A., D.H. Loughlin, J.J. Ducoste, and F.L. de Los Reyes III (2002), "Optimization of Activated Sludge Designs Using Genetic Algorithms", *Water Sci. Technol*, 45, 6, pp. 187–198

24. Droste, R. L. (1997), *Theory and Practice of Water and Wastewater Treatment*, John Wiley & Sons, New York.

25. Dunn, I. J., E. Heinzle, J. Ingham, and J. Prenosil (1992), *Biological Reaction Engineering*, VCH, Weinheim.

26. Dupont, R. and C. Dahl (1995), "A One-Dimensional Model for a Secondary Settling Tank Including Density Current and Short Circuiting", *Water Sci. Technol*, 31, 2, pp. 215-224.

27. Dupont, R. and M. Henze (1992), "Modelling of the Secondary Clarifier Combined with the Activated Sludge Model No. 1", *Water Sci. Technol*, 25, 6, pp. 285-300.

28. Echeverria, R., A. Seco, and J. Ferrer (1993), "Control of Activated Sludge Settleability Using Preaeration and Preprecipitation", *Water Res.*, 27, 2, pp. 293-296.

29. Eckenfelder, W. W. (2000) *Industrial Water Pollution Control*, 3rd ed., McGraw-Hill Inc., Boston.

30. Environmental Protection Agency (1980), *Construction Costs for Municipal Wastewater Treatment Plants: 1973-1978*, EPA report 430/9-80-003, Washington, D.C.

31. Fels, M., J. Pinter, and D. Lycon (1997), "Optimized Design of Wastewater Treatment Systems: Application to the Mechanical Pulp and Paper Industry: I. Design and Cost Relationships", *The Canadian Journal of Chemical Engineering*, 75, pp. 437-451.

32. Fujiwara, O. (1990), "Preliminary Optimal Design Model for Wastewater Treatment Plant", *Journal of Environmental Engineering*, 116, 1, pp. 206-210.

33. Gernaey, K., P.A. Vanrolleghem, and P. Lessard (2001), "Modeling of a Reactive Primary Clarifier", *Water Sci. Technol*, 43, 7, pp. 73-81.

34. Giokas, D., Y. Kim, P. Paraskevas, E. Paleologos, and T. Lekkas (2002), "A Simple Empirical Model for Activated Sludge Thickening in Secondary Clarifiers", *Water Res.*, 36, 13, pp. 3245-3252.

35. Gomez-Quintero, G., I. Queinnec, and J.P. Babary (1999), *A Reduced Nonlinear Model of An Activated Sludge Process*, LAAS-CNRS, France.

36. Grady, C.P.L., G. Daigger, and H. Lim (1999), *Biological Wastewater Treatment*, 2nd ed., Marcel Dekker Inc., New York.

37. Grijspeerdt, K., P. Vanrolleghem, W. Verstraete (1995), "Selection of One-Dimensional Sedimentation: Models for On-Line Use", *Water Science and Technology*, 31, 2, pp. 193-204.

38. Gujer, W., M. Henze, T. Mino, and M. van Loosdrecht (1999), "Activated Sludge Model No. 3", *Water Sci. Technol*, 39, 1, pp. 183-193.

39. Henze, M., C. P. Leslie Grady Jr., W. Gujer, G. Marais, and T. Matsuo (1987), "A General Model for Single-Sludge Wastewater Treatment Systems", *Water Res.*, 21, 5, pp. 505-515.

40. Henze, M., P. Harremoes, J. Jansen, and E Arvin (2002), *Wastewater Treatment: Biological and Chemical Processes*, 3rd ed., Springer, Berlin.

41. Henze, M., W. Gujer, T. Mino, and M. van Loosdrecht (2000), *Activated Sludge Models ASM1, ASM2, ASM2d, and ASM3*. (IWA Scientific and Technical Report No. 3.) London: IWA.

42. Henze, M., W. Gujer, T. Mino, T. Matsuo, M.C. Wentzel, and G. Marais (1995), *Activated Sludge Model No. 2*. (IWA Scientific and Technical Report No. 3.) London: IWA.

43. Hermanowicz, S.W. (1998), "Secondary Clarification of Activated Sludge: Development of Operating Diagrams", *Water Environ. Research*, 70, 10.

44. Infanger, G. (1997), *DECIS User's Guide*, Dr. Gred Infanger, 1590 Escondido Way, Belmont, CA 94002.

45. Jannssen, M., L.N. Hopkins, B. Petersen, and P.A. Vanrolleghem (2000), "Reduction of an Activated Sludge Process Model to Facilitate Controller Tuning", in: R. Van Langeghem (Ed.), *Proceddings of the 14th European Simulation Multi-conference*, Society for Computer Simulation International (SCS), pp. 697–701.

46. Jeppsson, U. (1996), *Modeling Aspects of Wastewater Treatment Processes*, PhD Thesis, Lund Institute of Technology, Lund, Sweden.

47. Kabouris, J.C. (1996), "Modeling, Instrumentation, Automation, and Optimization of Wastewater Treatment Facilities", *Water Environment Research*, 68, 4, pp. 520.

48. Kabouris, J.C. (1997), "Modeling, Instrumentation, Automation, and Optimization of Wastewater Treatment Facilities", *Water Environment Research*, 69, 4, pp. 562.

49. Kabouris, J.C. (1998), "Modeling, Instrumentation, Automation, and Optimization of Wastewater Treatment Facilities", *Water Environment Research*, 70, 4, pp. 590.

50. Kabouris, J.C. (1999), "Modeling, Instrumentation, Automation, and Optimization of Wastewater Treatment Facilities", *Water Environment Research*, 71, 5, pp. 729.

51. Koch, G., M. Kühni, and H. Siegrist (2001), "Calibration and Validation of an ASM3-Based Steady-State Model for Activated Sludge Systems-Part I: Prediction of Nitrogen Removal and Sludge Production" *Water Res.*, 35, 9, pp. 2235-2245.

52. Krebs, P., D. Vischer, and W. Gujer (1995), "Inlet-Structure Design for Final Clarifiers", *J. Environ. Eng.*, 121, 8, pp. 558-564.

53. Kurata, G., K. Tsumura, S. Nakamura, M. Kuwahara, A. Sato, and T. Kanaya (1996), "Retrofit of Biological Nutrient Removal Process Assisted by Numerical

Simulation with Activated Sludge Model No. 2", *Water Sci. Technol.*, 34, 1-2, pp. 221.

54. Lee, T.T., F.Y. Wang, and R.B. Newell (2002) "Robust Model-Order Reduction of Complex Biological Processes", *Journal of Process Control*, 12, 7, pp. 807-821.

55. Lessard, P. and B. Beck (1988), "Dynamic Modeling of Primary Sedimentation", *J. Env. Eng.*, 114, 4, pp. 753-769.

56. Matko, T., N. Fawcett, A. Sharp, and T. Stephenson (1996b), "Recent Progress in the Numerical Modeling of Wastewater Sedimentation Tanks," *Trans IchemE*, 74, pp. 245-258.

57. Mays, L., and Y. Tung (1992), *Hydrosystems Engineering and Management*, 1st ed., McGraw-Hill, New York.

58. Metcalf and Eddy (1991), *Wastewater Engineering: Treatment, Disposal, and Reuse*, 3rd ed., G. Tchobanoglous and F. L. Burton, eds., Tata McGraw-Hill, New Delhi, India.

59. Miller, I., and M. Miller (1999), *John E. Freund's Mathematical Statistics*, 6th ed., Prentice Hall International, New Jersy.

60. Novak, J.T. (1997), "The Effect of Nitrification on Activated Sludge Settling Properties", *Proc. Water Environ. Fed. 70th Annu. Conf. Exposition*, Chicago, III. 1, pp. 377.

61. Padukone, N., and G. Andrews (1989), "A Simple, Conceptual Mathematical Model for the Activated Sludge Process and its Variants", *Water Res.*, 23, 12, pp. 1535-1545.

62. Petrides, D., R. Cruz, and J. Calandranis (1998), "Optimization of Wastewater Treatment Facilities Using Process Simulation", *Computers chem. Engng*, 22, Suppl., S339.

63. Pincince, A.B., B.G. Braley, K.H. Sangrey, and R.D. Reardom (1997), "Minimizing Cost of Activated Sludge Systems", *Water Environ. Research*, 69, 3, pp. 326-330.

64. Pincince, A.B., *et al.* (1995) "Minimizing Costs of Activated Sludge Systems: Aplication to Deer Island Treatment Plant", *Proc. 68th Annu. Conf. Water Environ. Fed.*, Miami Beach, FL, 1, 693.

65. Pipes, W., and Y. Kim (1996), "Discussion of: A Simple Empirical Model for Hindered Settling in Activated Sludge Clarifier, N. Roche, J. Vaxelaire, C. Prost, 67, 5, pp. 775 (1995)", *Water Environment Research*, 68, 1, pp. 123.

66. Potter, T.G., C.C. Tseng, and B. Koopman (1998), "Nitrogen Removal in a Partial Nitrification/Complete Denitrification Process", *Water Environ. Research*, 70, pp. 334.

67. Queinnec, I., and D. Dochain (2001), "Modelling and Simulation of the Steady-State of Secondary Settlers in Wastewater Treatment Plants", *Water Science and Technology*, 43, 7, pp. 39-46.

68. Rensink, J.H., and W. H. Rulkens (1997), "Using Metazoa to Reduce Sludge Production", *Water Sci. Technol.*, 36, 11, pp. 171.

69. Reynolds, T. D. and P.A. Richards (1996), *Unit Operations and Processes in Environmental Engineering*, PWS Publishing, Boston.

70. Rivas, A., E. Ayesa, A. Galarza, and A. Salterain (2001), "Application of Mathematical Tools to Improve the Design and Operation of Activated Sludge Plants. Case Study: The New WWTP of Galindo-Bilbao Part I: Optimum Design", *Water Sci. Technol.*, 43, 7, pp. 157.

71. Roche, N., J. Vaxelaire, and C. Prost (1995), "A Simple Empirical Model for hindered Settling in Activated Sludge Clarifier", *Water Environment Research*, 67, 5, pp. 775-780.

72. Rousseau, D., F. Verdonck, O. Moerman, R. Carrette, C. Thoeye, J. Meirlaen, and P.A. Vanrolleghem (2001), "Development of a Risk Assessment Based Technique for Design/Retrofitting of WWTPs". *Water Science and Technology*, 43, 7, pp. 287-294.

73. Samuelsson, P., M. Ekman, and B. Carlsson (2001) "A Java Based Simulator of Activated Sludge Processes", *Mathematics and Computer in Simulation*, 56, pp. 333-346.

74. Scuras, S.E., A. Jobbagy, and C.P. Leslie Grady (2001), "Optimization of Activated Sludge Reactor Configuration: Kinetic Considerations" *Water Res.*, 35, 18, pp. 4277-4284.

75. Steffens, M. A., P. A. Lant, and R. B. Newell (1997), "A systematic approach for reducing complex biological wastewater treatment models" *Water Res.*, 31, 3, pp. 590-606.

76. Suidan, M.T., F. Saunders, C. Godfrey, and H. Stewart (1983), "Wastewater Treatment: Sensitivity Analysis", *Journal of Environmental Engineering*, 109, 1, pp. 120-138.

77. Takacs, I., G. Patry, and D. Nolasco (1991), "A Dynamic Model of the Clarification-Thickening Process", *Water Res.*, 25, 10, pp. 1263-1271.

78. Tang, C., E. D. Brill Jr., and J. Pfeffer (1984), *Mathematical Models and Optimization Techniques for Use in Analysis and Design of Wastewater Treatment Systems*, Research Report No. 194, Water Resources Center, University of Illinois.

79. Tang, C., E. D. Brill Jr., and J. Pfeffer (1987a), "Comprehensive Model of Activated Sludge Wastewater Treatment System", *Journal of Environmental Engineering*, 113, 5, pp. 952-969.

80. Tang, C., E. D. Brill Jr., and J. Pfeffer (1987b), "Optimization Techniques for Secondary Wastewater Treatment System", *Journal of Environmental Engineering*, 113, 5, pp. 935-951.

81. Tench, H.B. (1994), "A Theory of the Operation of Full Scale Activated Sludge Plants", *Water Res.*, 28, pp. 1019.

82. Truax, D.D. (1994), "Wastewater Treatment Plant Optimization and Automation", *Water Environment Research*, 66, 4, pp. 383.

83. Truax, D.D. (1995), "Optimization, Modeling, and Automation of Wastewater Treatment Facilities", *Water Environment Research*, 67, 4, pp. 499.

84. Tyteca, D. (1985), *Mathematical Models for Cost-Effective Biological Wastewater Treatment*, in *Mathematical Models in Biological Wastewater Treatment*, S.E. Jorgensen and M.J. Gromiec, eds., Elsevier Science Publishers, Amsterdam.

85. U.S. Department of the Army Corps of Engineers (1987), *Evaluation Criteria Guide for Water Pollution Prevention, Control, and Abatement Programs*, Technical Manual TM 5-814-8, Washington, D.C.

86. Ueberl, J., and W.H. Hager (1997), "Improved Design of Final Settling Tanks" *J. Environ. Eng.*, 123, pp. 259.

87. Vanderhasselt, A., and P. Vanrolleghem (1999), "Estimation of Sludge Sedimentation Parameters from Single Batch Settling Curves", *Water Res.*, 34, 2, pp. 395-406.

88. Vanrolleghem, P. A., U. Jeppsson, J. Carstensen, B. Caslsson, and G. Olsson (1996), "Integration of Wastewater Treatment Plant Design and Operation – A

Systematic Approach Using Cost Functions", *Wat. Sci. Tech.*, 34, 3-4, pp. 159-171.

89. Voutchkov, N. (1992), "Relationship for Clarification Efficiency of Circular Secondary Clarifiers", *Wat. Sci. Tech.*, 26, 9, pp. 2539-2542.

90. Voutchkov, N., and P.F. Boulos (1993), "Heuristic Screening Methodology for Regional Wastewater-treatment Planning", *J. Environ. Eng.*, 119, pp. 603.

91. Water Environment Federation and ASCE (1992), *Design of Municipal Wastewater Treatment Plants*, vol. 1 and 2, WEF, Alexandria, VA.

92. Watts, R. W., S. A. Svoronos, and B. Koopman (1996), "One-Dimensional Modeling of Secondary Clarifiers Using a Concentration and Feed Velocity-Dependent Dispersion Coefficient", *Water Res.*, 30, 9, pp. 2112-2124.

93. Wentzel, M.C., and G.A. Ekama (1997), "Principles in the Design of Single-Sludge Activated-Sludge Systems for Biologecal Removal of Carbon, Nitrogen, and Phosphorus", *Water Environment Research*, 69, 7, pp. 1222-1231.

94. Wett, B. (2002), "A Straight Interpretation of the Solids Flux Theory for a Three-Layer Sedimentation Model", *Water Res.*, 36, pp. 2949-2958.

95. Wright, D. G., and D. R. Woods (1993), "Evaluation of Capital Cost Data. Part 7: Liquid Waste Disposal with Emphasis on Physical Treatment", *The Canadian Journal of Chemical Engineering*, 71, pp. 575-590.

96. Wright, D. G., and D. R. Woods (1994), "Evaluation of Capital Cost Data. Part 8: Liquid Waste Disposal with Emphasis on Biological Treatment", *The Canadian Journal of Chemical Engineering*, 72, pp. 342-351.

97. Yuan, Z., H. Bogaert, G. Vansteenkiste, and W. Verstraete, (1998), "Sludge Storage for Countering Nitrogen Shock Loads and Toxicity Incidents" *Water Sci. Technol.*, 37, 12, pp. 173.

98. Yuan, Z., P.A. Vanrolleghem, and G.C. Vansteenkiste (1997), "Modeling Error Identification of Activated Sludge Models", *Water Science and Technology*, 36, 5, pp. 81-88.

99. Zhao, H., S.H. Isaacs, H. Soeberg, and M. Kummel (1994), "A Novel Control Strategy for Improved Nitrogen Removal in an Alternating Activated Sludge Process – Part I", *Water Res.*, 28, 3, pp. 521-534.

List of GAMS Input File

```
$Title Optimization of Activated Sludge Model   (ASM,SEQ=68)

$Ontext

24/4/2003
    Optimizing a system includes primary clarifier and activated
    sludge process. The activated sludge is modeled based on the
    a model derived from the recent ASM3 model which was proposed
    by IWA in 1999

$Offtext
$Offlisting
*$onsymxref
*$onsymlist


Parameter        U1, U2, U3;

Scalar

* Influent characteristics

        Q1      Flow rate (m3 per hour)         /1500/
        SI1     Inert soluble organic matter (mg per L as COD)    /32/
        SS1     Readily biodegradable substrate (mg per L as COD)       /161/
        SNH1    Ammonium plus Ammonia nitrogen (mg per L as N)  /25/
        SNO1    Nitrate plus nitrite nitrogen (mg per L as N)   /0/
        XI1     Inert particulate organic matter (mg per L as COD)       /92/
        XS1     Slowly biodegradable substrate (mg per L as COD)  /214/
        XH1     Heterotrophic organisms (mg per L as COD)        /0/
        XSTO1   A cell internal storage product of hectographs (mg per L as COD) /0/
        XA1     Nitrifying organisms (mg per L as COD)   /0/
        XSS1    Suspended solids (mg per L as SS)        /230/

* Parameters for sedimentation models

        a       constant in Christoulas Model     /1.11/
        b       constant in Christoulas Model (mg per L)  /261/
        c       constant in Christoulas Model (day per m)      /0.0035/

        k       settling constant of primary sludge (m per day) /400/
        n       settling constant of primary sludge /2.3/
        kw      settling constant of final clarifier (m per day)/385/
        nw      settling constant of final clarifier /1.8/

        SVI     Sludge volume index (ml per g) /150/
        H       Side water depth of final settler (m) /3.7/;

* Kinetic parameters in ASM3

scalar
        kH      Hydrolysis rate constant          /3/
        Kx      Hydrolysis saturation constant    /1/
        kSTO    Storage rate constant    /5/
        Ks      Saturation constant for substrate        /2/
        kkSTO   Saturation constant for XSTO     /1/
        mH      Heterotrophic max. growth rate of XH     /2/
        KNH     Saturation constant for ammonium        /0.01/
        bH      Aerobic endogenous respiration rate of XH       /0.2/
        bSTO    Aerobic respiration rate for XSTO       /0.2/
        mA      Autotrophic max. growth rate of XA       /1/
        KA      Ammonium substrate saturation for XA     /1/
```

```
        bA       Aerobic endogenous respiration rate of XA        /0.15/;

Scalar
* Stoichiometric parameters

        fD       Fraction of biomass contributing to biomass debris (mg debris COD per
                 mg biomass COD)   /0.2/
        YH       Aerobic yield of heterotrophic biomass (mg COD per mg COD) /0.63/
        YA       Yield of autotrophic biomss (mg COD per mg COD) /0.24/

* Other parameters

        ne       Efficiency depends on diffuser and depth at which air pumped /10/
        AIRU     Minimum air input rate m3 per min times 1000m3 /90/
        AIRL     Minimum air input rate m3 per min times 1000m3 /20/

        SCOD     Soluble degradable COD restriction in the effluent (g per m**3) /2/
        TSS      Total suspended solids restriction in the effluent(g per m**3) /10/
        SNH      Ammonia nitrogen in the effluent(g per m**3) /1.0/

        CRF      Capital Recovery factor /0.0944/
        BCI      Base (1971) Cost Index   /1581/
        CI       Cost Index for 2003   /6581/
        OMW      Operating_Maintenance Wages ($per hr) /8.3/
        EC       Electiricity Cost ($ per kWhr) /0.05/
        PH       Pumping Head (meters) /10.0/
        PE       Pumping Efficiency /0.6/;

        U1 = CRF * CI/BCI;
        U2 = CI/BCI;
        U3 = EC*23.85*PH/PE;

Free Variables

* State variables

        Q2       m3 per hour
        Q5       m3 per hour
        Q8       m3 per hour
        SS3      g per m3
        SNH3     g per m3
        SNO3     g per m3
        XI3      kg per m3
        XS3      kg per m3
        XH3      kg per m3
        XA3      kg per m3
        XSS3     kg per m3
        XSS8     kg per m3
        XSTO3    g per m3

* Other variables

        q        overflow rate of primary settling tank (m per day)
        Ap       surface area of the primary clarifier (100m**2)
        X1       XSS2 divided by XSS1

        SRT      sludge retention time (days)
        HRT      hydraulic retention time (days)

        V        volume of the aeration tank (1000m**3)
        r        sludge recycle ratio which is the ratio of Q6 to Q2
        w        100*wastage ratio which is the ratio of Q7 to Q2

        Af       surface area of the final clarifier (100m**2)
        SR       overflow rate of final clarifier (m per day)
        ROH      Oxygen Requirement for removal of organic matter (kg per day)
        ROA      Oxygen Requirement associated with nitrification (kg per day)
        AFR      Air flow rate  (m**3 air per min)

        X2       100*XSS4 divided by XSS3
        X3       XSS5 divided by XSS3

* Cost function variables

        CCPST, COPST, CMPST, CSPST
        CCPSP, COPSP, CMPSP, CSPSP, CPPSP
        CCAT, CCDAA, CODAA, CMDAA
```

```
         CCFST, COFST, CMFST, CSFST
         CCRSP, CORSP, CMRSP, CSRSP, CPRSP

         cost      cost function (dollars);

Equations
         C1, C2, C3, C4, C5, C6, C7, C8, C9, C10, C11, C12, C13, C14, C15
         C16, C17, C18, C19, C20, C21, C22, C23, C24, C25, C26, C27, C28, C29
         C30, C31, C32, C33, C34, C35, C36, C37, C38, C39, C40, C41, C42
         C43, C44, C45, C46, C47, C48, C49, C50, C51, C52

         objfun     objective function ;

* Primary Settling Tank Design
C1..
X1 =e= 1-(a*exp(-b/XSS1-c*q));
C2..
q =e= 24*Q2/(1e2*Ap);
C3..
Q1 =e= Q2 + Q8;
C4..
XSS8 =e= (k*(n-1))**(1/n)*(n/(n-1))*((1e2*Ap)/(Q8*24))**(1/n);
C5..
Q1*XSS1 =e= Q2*XSS1*X1 + Q8*1e3*XSS8;

* Activated Sludge Design
C6..
HRT =e= (1e3*V)/(24*Q2);
C7..
SRT =e= HRT/((1-(1e-2*w))*(1e-2*X2)+(1e-2*w)*X3);
C8..
0 =e= SS1 - SS3 + HRT*(1e3*XH3)*(kH*(XS3/XH3)/(KX+XS3/XH3)-kSTO*SS3/(KS+SS3));
C9..
0 =e= SNH1 - SNH3 +
HRT*(1e3*XH3*(0.01*kH*(XS3/XH3)/(KX+XS3/XH3)+0.03*kSTO*SS3/(KS+SS3)
         - 0.07*mH*SNH3/(KNH+SNH3)*(XSTO3/(1e3*XH3))/(kkSTO+XSTO3/(1e3*XH3))+0.066*bH)
         - 4.24*mA*SNH3/(KA+SNH3)*1e3*XA3 + 0.066*bA*1e3*XA3);
C10..
0 =e= SNO1 - SNO3 + HRT*4.17*mA*SNH3/(KA+SNH3)*1e3*XA3;
C11..
0 =e= XI1*X1 - 1e3*XI3*(HRT/SRT) + HRT*(0.2*bH*1e3*XH3+0.2*bA*1e3*XA3);
C12..
0 =e= XS1*X1 - 1e3*XS3*(HRT/SRT) + HRT*1e3*XH3*(-kH*(XS3/XH3)/(KX+XS3/XH3));
C13..
0 =e= XH1*X1 - 1e3*XH3*(HRT/SRT) + HRT*1e3*XH3*(mH*SNH3/(KNH+SNH3)
         *(XSTO3/(1e3*XH3))/(kkSTO+XSTO3/(1e3*XH3))-bH);
C14..
0 =e= XSTO1*X1 - XSTO3*(HRT/SRT) + HRT*(0.85*kSTO*SS3/(KS+SS3)*1e3*XH3-
         1.60*mH*SNH3/(KNH+SNH3)*(XSTO3/(1e3*XH3))/(kkSTO+XSTO3/(1e3*XH3))
         *1e3*XH3-bSTO*XSTO3);
C15..
0 =e= XA1*X1 - 1e3*XA3*(HRT/SRT) + HRT*(mA*SNH3/(KA+SNH3)*1e3*XA3-bA*1e3*XA3);
C16..
XSS3 =e= 0.75*XI3 + 0.75*XS3 + 0.9*(XH3+XA3) + 0.6*(XSTO3/1000);
C17..
1+r =e= (1-(1e-2*w))*(1e-2*X2) + ((1e-2*w)+r)*X3;
C18..
ROH =e= (24/1000)*Q2*(SS1+XS1*X1-SS3)*(1-((1+fD*bH*SRT)*YH)/(1+bH*SRT));
C19..
ROA =e= (24/1000)*Q2*(SNH1+SNO1-SNH3)*(4.57-((1+fD*bA*SRT)*YA)/(1+bA*SRT));
C20..
AFR =e= 6*((ROH + ROA)/24)/ne;
C21..
SR =e= 24*((1-(1e-2*w))*Q2)/(100*Af);
C22..
(1e-2*X2)*1e3*XSS3 =e= 6.21*log(XSS3*SVI)/(0.67*log(H)-log((SR/24)))-26.43;
C23..
X3*XSS3 =e= (kw*(nw-1))**(1/nw)*(nw/(nw-1))*((1e2*Af)/((r+1e-2*w)*24*Q2))**(1/nw);

* Effluent Water Quality Standards
C24..
SCOD =g= SS3;
C25..
TSS =g= 1e3*XSS3*(1e-2*X2);
C26..
SNH =g= SNH3;
```

```
* Mixing and oxygen transfer Requirements in Aeration Tank
C27..
1000*V =g= ((ROH + ROA)/24)/0.10;
C28..
1000*V =g= 1000*AFR/AIRU;
C29..
1000*V =l= 1000*AFR/AIRL;

* Cost Functions
* Primary clarifier
C30..
CCPST =e= 824*(1e2*Ap)**.77;
C31..
COPST =e= 17.1*(1e2*Ap)**.6;
C32..
CMPST =e= 9.23*(1e2*Ap)**.6;
C33..
CSPST =e= 8.62*(1e2*Ap)**.76;

* Primary Sludge Pumping
C34..
CCPSP =e= 9870*Q8**.53;
C35..
COPSP =e= 257*Q8**.41;
C36..
CMPSP =e= 112*Q8**.43;
C37..
CSPSP =e= 214*Q8**.64;
C38..
CPPSP =e= Q8;

* Aeration tank
C39..
CCAT  =e= 461*(1e3*V)**.71;
C40..
CCDAA =e= 8533*AFR**.66;
C41..
CODAA =e= 187*AFR**.48;
C42..
CMDAA =e= 74.4*AFR**.55;

* Final Clarifier
C43..
CCFST =e= 824*(1e2*Af)**.77;
C44..
COFST =e= 17.1*(1e2*Af)**.6;
C45..
CMFST =e= 9.23*(1e2*Af)**.6;
C46..
CSFST =e= 8.62*(1e2*Af)**.76;

* Return and Waste Sludge
C47..
Q5 =e= (r+(1e-2*w))*Q2;
C48..
CCRSP =e= 9870*Q5**.53;
C49..
CORSP =e= 257*Q5**.41;
C50..
CMRSP =e= 112*Q5**.43;
C51..
CSRSP =e= 214*Q5**.64;
C52..
CPRSP =e= Q5;

objfun..
cost =e= U1*(CCPST+CCPSP+CCAT+CCDAA+CCFST+CCRSP)
        + OMW*(COPST+CMPST+COPSP+CMPSP+CODAA+CMDAA+COFST+CMFST+CORSP+CMRSP)
        + U2*(CSPST+CSPSP+CSFST+CSRSP)
        + U3*(CPPSP+CPRSP);
*
*   bounds
*
q.lo = 30;        q.up=120;
Ap.lo = 1.15;    Ap.up=1e6;
SRT.lo = 1;      SRT.up = 20;
HRT.lo = .125;   HRT.up = .625;
```

```
r.lo = .25;        r.up = 1.5;
XSS3.lo= 0.5;      XSS3.up=1e6;
SR.lo = 16;        SR.up = 32;

Q2.lo=1e-6;        Q2.up=1e6;
Q8.lo=1e-6;        Q8.up=1e6;
X1.lo = 1e-6;      X1.up=1e6;
SS3.lo=1e-6;       SS3.up=1e6;
SNH3.lo=1e-6;      SNH3.up=1e6;
SNO3.lo=1e-6;      SNO3.up=1e6;
XI3.lo=1e-6;       XI3.up=1e6;
XS3.lo=1e-6;       XS3.up=1e6;
XH3.lo=1e-6;       XH3.up=1e6;
XA3.lo=1e-6;       XA3.up=1e6;
XSS8.lo=1e-6;      XSS8.up=1e6;
XSTO3.lo=1e-6;     XSTO3.up=1e6;
V.lo = 1e-6;       V.up=1e6;
ROH.lo = 1e-6;     ROH.up = 1e6;
ROA.lo = 1e-6;     ROA.up = 1e6;
AFR.lo = 1e-6;     AFR.up = 1e6;
Q5.lo= 1e-6;       Q5.up=1e6;
w.lo = 1e-6;       w.up= 1e6;
Af.lo = 1e-6;      Af.up=1e6;
X2.lo = 1e-6;      X2.up=1e6;
X3.lo = 1e-6;      X3.up=1e6;

*  initial values
q.l=     30.000  ;
Ap.l=    11.995  ;
X1.l=    0.679   ;
Q2.l =   1499.392            ;
Q8.l=    0.608   ;
XSS8.l=          182.493  ;

SRT.l=  10.000  ;
HRT.l=  0.200   ;
V.l=    7.197   ;
r.l=    0.400   ;

w.l=    0.559   ;
X2.l=   0.071   ;
X3.l=   3.450   ;
Af.l=   24.543  ;

SS3.l=  0.288   ;
SNH3.l=         0.333    ;
SNO3.l=         31.153   ;
XI3.l=  4.126   ;
XS3.l=  0.259   ;
XH3.l=  2.398   ;
XA3.l=  0.149   ;
XSTO3.l= 438.221           ;

XSS3.l=         5.844    ;

ROH.l=  7798.453          ;
ROA.l=  3945.738          ;
AFR.l=  293.605         ;

Q5.l= 608.140   ;
SR.l=   14.580  ;

Model ASM3_MODEL /all/;
Option NLP = CONOPT2    ;
Solve ASM3_MODEL using nlp minimizing cost;
```

APPENDIX B

List of GAMS Output File

        24/4/2003
             Optimizing a system includes primary clarifier and activated
             sludge process. The activated sludge is modeled based on the
             a model derived from the recent ASM3 model which was proposed
             by IWA in 1999

COMPILATION TIME   =   0.010 SECONDS   0.7 Mb   WIN197-119

---- C1   =E=

C1..   - (0.0011)*q + X1 =E= 1 ; (LHS = 1.0003, INFES = 0.0003 ***)

---- C2   =E=

C2..   - (0.02)*Q2 + q + (2.5011)*Ap =E= 0 ; (LHS = -0.0003, INFES = 0.0003 ***)

---- C3   =E=

C3..   - Q2 - Q8 =E= -1500 ; (LHS = -1500)

---- C4   =E=

C4..   (130.4917)*Q8 + XSS8 - (6.6143)*Ap =E= 0 ;
       (LHS = 0.0134, INFES = 0.0134 ***)

---- C5   =E=

C5..   - (156.17)*Q2 - (182493)*Q8 - (608)*XSS8 - (344860.16)*X1 =E= -345000 ;
       (LHS = -345115.7926, INFES = 115.7926 ***)

---- C6   =E=

C6..   (0.0001)*Q2 + HRT - (0.0278)*V =E= 0 ;
       (LHS = 2.2675858E-6, INFES = 2.2675858E-6 ***)

---- C7   =E=

C7..   SRT - (50.0212)*HRT + (17.2611)*w + (4.9763)*X2 + (2.7974)*X3 =E= 0 ;
       (LHS = -0.0042, INFES = 0.0042 ***)

---- C8   =E=

C8..   (917.1512)*SS3 - (488.7272)*XS3 + (120.1729)*XH3 + (807.9715)*HRT =E=
       161 ; (LHS = 161.8823, INFES = 0.8823 ***)

---- C9   =E=

C9..   - (27.4845)*SS3 + (72.9904)*SNH3 - (4.8873)*XS3 - (5.8243)*XH3
       + (209.861)*XA3 + (0.0194)*XSTO3 + (122.7627)*HRT =E= 25 ;
       (LHS = 24.8855, INFES = 0.1145 ***)

---- C10 =E=

C10.. - (69.9346)*SNH3 + SNO3 - (208.3436)*XA3 - (155.216)*HRT =E= 0 ;
      (LHS = 0.1098, INFES = 0.1098 ***)

---- C11 =E=

C11.. (20)*XI3 - (8)*XH3 - (6)*XA3 - 92*X1 - (8.252)*SRT + (312.21)*HRT =E= 0
      ; (LHS = -0.026, INFES = 0.026 ***)

---- C12 =E=

C12.. (508.7272)*XS3 + (5.7012)*XH3 - 214*X1 - (0.518)*SRT + (727.1593)*HRT
      =E= 0 ; (LHS = 0.1259, INFES = 0.1259 ***)

---- C13 =E=

C13.. - (12.5972)*SNH3 + (50.7292)*XH3 - (0.2776)*XSTO3 - (4.796)*SRT
      - (0.0198)*HRT =E= 0 ; (LHS = -0.004, INFES = 0.004 ***)

GAMS Rev 119 Windows NT/95/98   06/04/03 00:51:27 PAGE   4
Optimization of Activated Sludge Model (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- C14 =E=

C14.. - (778.7285)*SS3 + (20.1555)*SNH3 - (92.1598)*XH3 + (0.5042)*XSTO3
      - (0.8764)*SRT - (0.3081)*HRT =E= 0 ; (LHS = -0.0616, INFES = 0.0616 ***)

---- C15 =E=

C15.. - (16.7709)*SNH3 + (0.0375)*XA3 - (0.298)*SRT + (0.0279)*HRT =E= 0 ;
      (LHS = 0.0056, INFES = 0.0056 ***)

---- C16 =E=

C16.. - 0.75*XI3 - 0.75*XS3 - 0.9*XH3 - 0.9*XA3 + XSS3 - 0.0006*XSTO3 =E= 0 ;
      (LHS = 1.7400000E-5, INFES = 1.7400000E-5 ***)

---- C17 =E=

C17.. - (2.45)*r - (0.0345)*w - (0.0099)*X2 - (0.4056)*X3 =E= -1 ;
      (LHS = -1, INFES = 8.4689000E-6 ***)

---- C18 =E=

C18.. - (5.1852)*Q2 + (25.4057)*SS3 - (5436.8194)*X1 - (123.3364)*SRT + ROH
      =E= 0 ; (LHS = 23.8521, INFES = 23.8521 ***)

---- C19 =E=

C19.. - (2.6316)*Q2 + (159.9623)*SNH3 - (4.0903)*SRT + ROA =E= 0 ;
      (LHS = -0.0529, INFES = 0.0529 ***)

---- C20 =E=

C20.. - 0.025*ROH - 0.025*ROA + AFR =E= 0 ; (LHS = 0.0002, INFES = 0.0002 ***)

GAMS Rev 119 Windows NT/95/98   06/04/03 00:51:27 PAGE   5
Optimization of Activated Sludge Model (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- C21 =E=

C21.. - (0.0097)*Q2 + (0.1466)*w + (0.5941)*Af + SR =E= 0 ;
      (LHS = 1.4198, INFES = 1.4198 ***)

---- C22 =E=

C22.. - (0.1189)*XSS3 - (1.6001)*SR + (58.44)*X2 =E= -26.43 ;
      (LHS = -28.6727, INFES = 2.2427 ***)

---- C23 =E=

C23.. (0.0075)*Q2 + (3.45)*XSS3 + (27.618)*r + (0.2762)*w - (0.4564)*Af
      + (5.844)*X3 =E= 0 ; (LHS = -0.001, INFES = 0.001 ***)

```
---- C24   =G=

C24..  - SS3 =G= -2 ; (LHS = -0.288)

---- C25   =G=

C25..  - (0.71)*XSS3 - (58.44)*X2 =G= -10 ; (LHS = -4.1492)

---- C26   =G=

C26..  - SNH3 =G= -1 ; (LHS = -0.333)

---- C27   =G=

C27..  1000*V - 0.4167*ROH - 0.4167*ROA =G= 0 ; (LHS = 2303.5871)

---- C28   =G=

C28..  1000*V - 11.1111*AFR =G= 0 ; (LHS = 3934.7222)
```

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Optimization of Activated Sludge Model (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

```
---- C29   =L=

C29..  1000*V - 50*AFR =L= 0 ; (LHS = -7483.25)

---- C30   =E=

C30..  - (12423.612)*Ap + CCPST =E= 0 ;

       (LHS = -193534.0593, INFES = 193534.0593 ***)

---- C31   =E=

C31..  - (60.1932)*Ap + COPST =E= 0 ; (LHS = -1203.3619, INFES = 1203.3619 ***)

---- C32   =E=

C32..  - (32.4902)*Ap + CMPST =E= 0 ; (LHS = -649.5339, INFES = 649.5339 ***)

---- C33   =E=

C33..  - (119.498)*Ap + CSPST =E= 0 ; (LHS = -1886.0251, INFES = 1886.0251 ***)

---- C34   =E=

C34..  - (6609.3438)*Q8 + CCPSP =E= 0 ; (LHS = -7582.0396, INFES = 7582.0396 ***)

---- C35   =E=

C35..  - (141.3233)*Q8 + COPSP =E= 0 ; (LHS = -209.5722, INFES = 209.5722 ***)

---- C36   =E=


C36..  - (63.9531)*Q8 + CMPSP =E= 0 ; (LHS = -90.4267, INFES = 90.4267 ***)
```

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Optimization of Activated Sludge Model (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

```
---- C37   =E=

C37..  - (163.8281)*Q8 + CSPSP =E= 0 ; (LHS = -155.6367, INFES = 155.6367 ***)

---- C38   =E=

C38..  - Q8 + CPPSP =E= 0 ; (LHS = -0.608, INFES = 0.608 ***)

---- C39   =E=

C39..  - (24910.666)*V + CCAT =E= 0 ;
       (LHS = -252509.9479, INFES = 252509.9479 ***)

---- C40   =E=
```

```
C40..  - (815.8421)*AFR + CCDAA =E= 0 ;
       (LHS = -362932.2999, INFES = 362932.2999 ***)

---- C41  =E=

C41..  - (4.6757)*AFR + CODAA =E= 0 ; (LHS = -2860.0134, INFES = 2860.0134 ***)

---- C42  =E=

C42..  - (3.1728)*AFR + CMDAA =E= 0 ; (LHS = -1693.7247, INFES = 1693.7247 ***)

---- C43  =E=

C43..  - (10537.427)*Af + CCFST =E= 0 ;
       (LHS = -335870.2211, INFES = 335870.2211 ***)

---- C44  =E=

C44..  - (45.2039)*Af + COFST =E= 0 ; (LHS = -1849.0672, INFES = 1849.0672 ***)
```

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

```
---- C45  =E=

C45..  - (24.3996)*Af + CMFST =E= 0 ; (LHS = -998.0638, INFES = 998.0638 ***)

---- C46  =E=

C46..  - (100.6325)*Af + CSFST =E= 0 ; (LHS = -3249.7674, INFES = 3249.7674 ***)

---- C47  =E=

C47..  - (0.4056)*Q2 + Q5 - (1499.392)*r - (14.9939)*w =E= 0 ;
       (LHS = 0.0016, INFES = 0.0016 ***)

---- C48  =E=

C48..  - (257.1055)*Q5 + CCRSP =E= 0 ;
       (LHS = -295011.5672, INFES = 295011.5672 ***)

---- C49  =E=

C49..  - (2.3997)*Q5 + CORSP =E= 0 ; (LHS = -3559.3819, INFES = 3559.3819 ***)

---- C50  =E=

C50..  - (1.2468)*Q5 + CMRSP =E= 0 ; (LHS = -1763.3542, INFES = 1763.3542 ***)

---- C51  =E=

C51..  - (13.6255)*Q5 + CSRSP =E= 0 ; (LHS = -12947.206, INFES = 12947.206 ***)

---- C52  =E=

C52..  - Q5 + CPRSP =E= 0 ; (LHS = -608.14, INFES = 608.14 ***)
```

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Equation Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

```
---- objfun  =E=  objective function

objfun..  - 0.3929*CCPST - 8.3*COPST - 8.3*CMPST - 4.1626*CSPST - 0.3929*CCPSP
      - 8.3*COPSP - 8.3*CMPSP - 4.1626*CSPSP - 19.875*CPPSP - 0.3929*CCAT
      - 0.3929*CCDAA - 8.3*CODAA - 8.3*CMDAA - 0.3929*CCFST - 8.3*COFST
      - 8.3*CMFST - 4.1626*CSFST - 0.3929*CCRSP - 8.3*CORSP - 8.3*CMRSP
      - 4.1626*CSRSP - 19.875*CPRSP + cost =E= 0 ; (LHS = 0)
```

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

```
---- Q2  m3 per hour
```

Q2
```
                    (.LO, .L, .UP = 1.0000000E-6, 1499.392, 1000000)
        (-0.02)    C2
          -1       C3
    (-156.17)      C5
       (0.0001)    C6
      (-5.1852)    C18
      (-2.6316)    C19
      (-0.0097)    C21
       (0.0075)    C23
      (-0.4056)    C47
```

---- Q5   m3 per hour

Q5
```
                    (.LO, .L, .UP = 1.0000000E-6, 608.14, 1000000)
          1        C47
    (-257.1055)    C48
      (-2.3997)    C49
      (-1.2468)    C50
     (-13.6255)    C51
          -1       C52
```

---- Q8   m3 per hour

Q8
```
                    (.LO, .L, .UP = 1.0000000E-6, 0.608, 1000000)
          -1       C3
     (130.4917)    C4
    (-182493)      C5
    (-6609.3438)   C34
     (-141.3233)   C35
      (-63.9531)   C36
     (-163.8281)   C37
          -1       C38
```

---- SS3   g per m3

SS3
```
                    (.LO, .L, .UP = 1.0000000E-6, 0.288, 1000000)
     (917.1512)    C8
      (-27.4845)   C9
     (-778.7285)   C14
       (25.4057)   C18
          -1       C24
```

---- SNH3   g per m3

SNH3
```
                    (.LO, .L, .UP = 1.0000000E-6, 0.333, 1000000)
      (72.9904)    C9
     (-69.9346)    C10
     (-12.5972)    C13
      (20.1555)    C14
     (-16.7709)    C15
     (159.9623)    C19
          -1       C26
```

---- SNO3   g per m3

SNO3
```
                    (.LO, .L, .UP = 1.0000000E-6, 31.153, 1000000)
          1        C10
```

---- XI3   kg per m3

XI3
```
                    (.LO, .L, .UP = 1.0000000E-6, 4.126, 1000000)
        (20)       C11
        -0.75      C16
```

---- XS3   kg per m3

XS3
                (.LO, .L, .UP = 1.0000000E-6, 0.259, 1000000)
    (-488.7272) C8
      (-4.8873) C9
    (508.7272) C12
       -0.75   C16

---- XH3 kg per m3

XH3
                (.LO, .L, .UP = 1.0000000E-6, 2.398, 1000000)
    (120.1729) C8
     (-5.8243) C9
     (-8)   C11
      (5.7012) C12
     (50.7292) C13
    (-92.1598) C14
       -0.9   C16

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- XA3 kg per m3

XA3
                (.LO, .L, .UP = 1.0000000E-6, 0.149, 1000000)
     (209.861) C9
   (-208.3436) C10
      (-6)   C11
      (0.0375) C15
       -0.9   C16

---- XSS3 kg per m3

XSS3
                (.LO, .L, .UP = 0.5, 5.844, 1000000)
       1   C16
     (-0.1189) C22
      (3.45)   C23
     (-0.71)   C25

---- XSS8 kg per m3

XSS8
                (.LO, .L, .UP = 1.0000000E-6, 182.493, 1000000)
       1   C4
   (-608)   C5

---- XSTO3 g per m3

XSTO3
                (.LO, .L, .UP = 1.0000000E-6, 438.221, 1000000)
      (0.0194) C9
     (-0.2776) C13
      (0.5042) C14
     -0.0006   C16

---- q overflow rate of primary settling tank (m per day)

q
                (.LO, .L, .UP = 30, 30, 120)
     (-0.0011) C1
       1   C2

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- Ap surface area of the primary clarifier (100m**2)

Ap
                (.LO, .L, .UP = 1.15, 11.995, 1000000)
      (2.5011) C2
     (-6.6143) C4
  (-12423.612) C30
     (-60.1932) C31

```
     (-32.4902) C32
     (-119.498)  C33

---- X1   XSS2 divided by XSS1

X1
                   (.LO, .L, .UP = 1.0000000E-6, 0.679, 1000000)
         1         C1
  (-344860.16)     C5
        92         C11
       -214        C12
    (-5436.8194)   C18

---- SRT   sludge retention time (days)

SRT
                   (.LO, .L, .UP = 1, 10, 20)
         1         C7
      (-8.252)     C11
      (-0.518)     C12
      (-4.796)     C13
      (-0.8764)    C14
      (-0.298)     C15
    (-123.3364)    C18
      (-4.0903)    C19

---- HRT   hydraulic retention time (days)

HRT
                   (.LO, .L, .UP = 0.125, 0.2, 0.625)
         1         C6
     (-50.0212)    C7
     (807.9715)    C8
     (122.7627)    C9
    (-155.216)     C10
     (312.21)      C11
     (727.1593)    C12
      (-0.0198)    C13
      (-0.3081)    C14
       (0.0279)    C15
```

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- V   volume of the aeration tank (1000m**3)

```
V
                   (.LO, .L, .UP = 1.0000000E-6, 7.197, 1000000)
     (-0.0278)     C6
    1000           C27
    1000           C28
    1000           C29
  (-24910.666)     C39
```

---- r   sludge recycle ratio which is the ratio of Q6 to Q2

```
r
                   (.LO, .L, .UP = 0.25, 0.4, 1.5)
     (-2.45)       C17
     (27.618)      C23
   (-1499.392)     C47
```

---- w   100*wastage ratio which is the ratio of Q7 to Q2

```
w
                   (.LO, .L, .UP = 1.0000000E-6, 0.559, 1000000)
     (17.2611)     C7
     (-0.0345)     C17
      (0.1466)     C21
      (0.2762)     C23
     (-14.9939)    C47
```

---- Af   surface area of the final clarifier (100m**2)

```
Af
                   (.LO, .L, .UP = 1.0000000E-6, 24.543, 1000000)
```

```
        (0.5941)  C21
        (-0.4564)  C23
   (-10537.427)  C43
        (-45.2039)  C44
        (-24.3996)  C45
       (-100.6325)  C46
```

---- SR   overflow rate of final clarifier (m per day)

```
SR
                 (.LO,  .L,  .UP = 16, 16, 32)
        1        C21
     (-1.6001)  C22
```

---- ROH Oxygen Requirement for removal of organic matter (kg per day)

```
ROH
                 (.LO,  .L,  .UP = 1.0000000E-6, 7798.453, 1000000)
        1        C18
     -0.025   C20
     -0.4167  C27
```

---- ROA Oxygen Requirement associated with nitrification (kg per day)

```
ROA
                 (.LO,  .L,  .UP = 1.0000000E-6, 3945.738, 1000000)
        1        C19
      -0.025   C20
      -0.4167  C27
```

---- AFR Air flow rate (m**3 air per min)

```
AFR
                 (.LO,  .L,  .UP = 1.0000000E-6, 293.605, 1000000)
        1        C20
     -11.1111  C28
     -50       C29
   (-815.8421) C40
      (-4.6757) C41
      (-3.1728) C42
```

---- X2   100*XSS4 divided by XSS3

```
X2
                 (.LO,  .L,  .UP = 1.0000000E-6, 0.071, 1000000)
      (4.9763)  C7
     (-0.0099)  C17
     (58.44)    C22
     (-58.44)   C25
```

---- X3   XSS5 divided by XSS3

```
X3
                 (.LO,  .L,  .UP = 1.0000000E-6, 3.45, 1000000)
      (2.7974)  C7
     (-0.4056)  C17
      (5.844)   C23
```

---- CCPST

```
CCPST
                 (.LO,  .L,  .UP = -INF, 0, +INF)
        1        C30
     -0.3929  objfun
```

---- COPST

```
COPST
                 (.LO,  .L,  .UP = -INF, 0, +INF)
```

```
       1     C31
      -8.3   objfun
```

---- CMPST

CMPST
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C32
      -8.3   objfun
```

---- CSPST

CSPST
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C33
      -4.1626  objfun
```

---- CCPSP

CCPSP
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C34
      -0.3929  objfun
```

---- COPSP

COPSP
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C35
      -8.3   objfun
```

---- CMPSP

CMPSP
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C36
      -8.3   objfun
```

---- CSPSP

CSPSP
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C37
      -4.1626  objfun
```

---- CPPSP

CPPSP
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C38
     -19.875  objfun
```

---- CCAT

CCAT
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C39
      -0.3929  objfun
```

---- CCDAA

CCDAA
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C40
      -0.3929  objfun
```

---- CODAA

CODAA
```
             (.LO,  .L,  .UP = -INF,  0,  +INF)
       1     C41
      -8.3   objfun
```

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- CMDAA

CMDAA
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C42
      -8.3   objfun

---- CCFST

CCFST
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C43
      -0.3929   objfun

---- COFST

COFST
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C44
      -8.3   objfun

---- CMFST

CMFST
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C45
      -8.3   objfun

---- CSFST

CSFST
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C46
      -4.1626   objfun

---- CCRSP

CCRSP
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C48
      -0.3929   objfun

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Column Listing   SOLVE ASM3_MODEL USING NLP FROM LINE 357

---- CORSP

CORSP
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C49
      -8.3   objfun

---- CMRSP

CMRSP
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C50
      -8.3   objfun

---- CSRSP

CSRSP
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C51
      -4.1626   objfun

---- CPRSP

CPRSP
                (.LO, .L, .UP = -INF, 0, +INF)
        1   C52
      -19.875   objfun

---- cost   cost function (dollars)

cost
              (.LO, .L, .UP = -INF, 0, +INF)
       1   objfun

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Optimization of Activated Sludge Model   (ASM,SEQ=68)
Model Statistics   SOLVE ASM3_MODEL USING NLP FROM LINE 357

MODEL STATISTICS

BLOCKS OF EQUATIONS   53   SINGLE EQUATIONS   53
BLOCKS OF VARIABLES   51   SINGLE VARIABLES   51
NON ZERO ELEMENTS   180   NON LINEAR N-Z   101
DERIVATIVE POOL   13   CONSTANT POOL   77
CODE LENGTH   2281

GENERATION TIME   =   0.050 SECONDS   1.9 Mb   WIN197-119

EXECUTION TIME   =   0.050 SECONDS   1.9 Mb   WIN197-119

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Optimization of Activated Sludge Model   (ASM,SEQ=68)

                  S O L V E   S U M M A R Y

     MODEL   ASM3_MODEL   OBJECTIVE cost
     TYPE   NLP   DIRECTION MINIMIZE
     SOLVER CONOPT2   FROM LINE 357

**** SOLVER STATUS   1 NORMAL COMPLETION
**** MODEL STATUS   2 LOCALLY OPTIMAL
**** OBJECTIVE VALUE   597963.4482

 RESOURCE USAGE, LIMIT   0.110   1000.000
 ITERATION COUNT, LIMIT   24   10000
 EVALUATION ERRORS   0   0

     C O N O P T 2   Windows NT/95/98   version 2.071F-007-042
     Copyright (C)   ARKI Consulting and Development A/S
                     Bagsvaerdvej 246 A
                     DK-2880 Bagsvaerd, Denmark

 Using default control program.

** Optimal solution. There are no superbasic variables.

 CONOPT time Total   0.080 seconds
   of which: Function evaluations   0.000 = 0.0%
             Derivative evaluations   0.000 = 0.0%

 Work length =   0.13 Mbytes
   Estimate =   0.13 Mbytes
   Max used =   0.07 Mbytes

                        LOWER   LEVEL   UPPER   MARGINAL

---- EQU C1   1.000   1.000   1.000   729.053
---- EQU C2   .   .   .   194.823
---- EQU C3   -1500.000   -1500.000   -1500.000   -239.412
---- EQU C4   .   .   .   -81.991
---- EQU C5   -3.450E+5   -3.450E+5   -3.450E+5   -0.081
---- EQU C6   .   .   .   -4.036E+5
---- EQU C7   .   .   .   693.505
---- EQU C8   161.000   161.000   161.000   -33.487
---- EQU C9   25.000   25.000   25.000   -16.498
---- EQU C10   .   .   .   EPS
---- EQU C11   .   .   .   -81.230
---- EQU C12   .   .   .   -39.521
---- EQU C13   .   .   .   -62.499
---- EQU C14   .   .   .   -38.328
---- EQU C15   .   .   .   243.120
---- EQU C16   .   .   .   -3882.459
---- EQU C17   -1.000   -1.000   -1.000   33432.343
---- EQU C18   .   .   .   10.024

---- EQU C19   .   .   .   10.024

                          LOWER   LEVEL   UPPER   MARGINAL

---- EQU C20   .   .   .   400.976
---- EQU C21   .   .   .   6642.583
---- EQU C22   -26.430   -26.430   -26.430   3844.212
---- EQU C23   .   .   .   1755.007
---- EQU C24   -2.000   -0.567   +INF   .
---- EQU C25   -10.000   -10.000   +INF   3851.514
---- EQU C26   -1.000   -0.775   +INF   .
---- EQU C27   .   95.775   +INF   .
---- EQU C28   .   1562.836   +INF   .
---- EQU C29   -INF   -8706.590   .   .
---- EQU C30   .   .   .   0.393
---- EQU C31   .   .   .   8.300
---- EQU C32   .   .   .   8.300
---- EQU C33   .   .   .   4.163
---- EQU C34   .   .   .   0.393
---- EQU C35   .   .   .   8.300
---- EQU C36   .   .   .   8.300
---- EQU C37   .   .   .   4.163
---- EQU C38   .   .   .   19.875
---- EQU C39   .   .   .   0.393
---- EQU C40   .   .   .   0.393
---- EQU C41   .   .   .   8.300
---- EQU C42   .   .   .   8.300
---- EQU C43   .   .   .   0.393
---- EQU C44   .   .   .   8.300
---- EQU C45   .   .   .   8.300
---- EQU C46   .   .   .   4.163
---- EQU C47   .   .   .   251.455
---- EQU C48   .   .   .   0.393
---- EQU C49   .   .   .   8.300
---- EQU C50   .   .   .   8.300
---- EQU C51   .   .   .   4.163
---- EQU C52   .   .   .   19.875
---- EQU objfun   .   .   .   1.000

   C1
   C2
   C3
   C4
   C5
   C6
   C7
   C8
   C9
   C10
   C11
   C12
   C13
   C14
   C15
   C16
   C17
   C18

   C19
   C20
   C21
   C22
   C23
   C24
   C25
   C26
   C27
   C28
   C29
   C30
   C31

                          LOWER   LEVEL   UPPER   MARGINAL

```
C32
C33
C34
C35
C36
C37
C38
C39
C40
C41
C42
C43
C44
C45
C46
C47
C48
C49
C50
C51
C52
objfun  objective function
```

	LOWER	LEVEL	UPPER	MARGINAL
---- VAR Q2	1.0000E-6	1498.986	1.0000E+6	.
---- VAR Q5	1.0000E-6	385.148	1.0000E+6	.
---- VAR Q8	1.0000E-6	1.014	1.0000E+6	.
---- VAR SS3	1.0000E-6	0.567	1.0000E+6	.
---- VAR SNH3	1.0000E-6	0.775	1.0000E+6	.
---- VAR SNO3	1.0000E-6	28.772	1.0000E+6	.
---- VAR XI3	1.0000E-6	2.289	1.0000E+6	.
---- VAR XS3	1.0000E-6	0.475	1.0000E+6	.
---- VAR XH3	1.0000E-6	2.227	1.0000E+6	.
---- VAR XA3	1.0000E-6	0.126	1.0000E+6	.
---- VAR XSS3	0.500	4.628	1.0000E+6	.
---- VAR XSS8	1.0000E-6	79.949	1.0000E+6	.
---- VAR XSTO3	1.0000E-6	728.647	1.0000E+6	.
---- VAR q	30.000	120.000	120.000	-194.225
---- VAR Ap	1.150	2.998	1.0000E+6	.
---- VAR X1	1.0000E-6	0.766	1.0000E+6	.
---- VAR SRT	1.000	3.487	20.000	.

```
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Optimization of Activated Sludge Model  (ASM,SEQ=68)
```

	LOWER	LEVEL	UPPER	MARGINAL
---- VAR HRT	0.125	0.125	0.625	5.6669E+5
---- VAR V	1.0000E-6	4.497	1.0000E+6	.
---- VAR r	0.250	0.250	1.500	4.2057E+5
---- VAR w	1.0000E-6	0.694	1.0000E+6	.
---- VAR Af	1.0000E-6	18.899	1.0000E+6	.
---- VAR SR	16.000	18.903	32.000	.
---- VAR ROH	1.0000E-6	6731.810	1.0000E+6	.
---- VAR ROA	1.0000E-6	3831.029	1.0000E+6	.
---- VAR AFR	1.0000E-6	264.071	1.0000E+6	.
---- VAR X2	1.0000E-6	0.216	1.0000E+6	.
---- VAR X3	1.0000E-6	4.857	1.0000E+6	.
---- VAR CCPST	-INF	66540.251	+INF	.
---- VAR COPST	-INF	523.712	+INF	.
---- VAR CMPST	-INF	282.682	+INF	.
---- VAR CSPST	-INF	657.501	+INF	.
---- VAR CCPSP	-INF	9943.170	+INF	.
---- VAR COPSP	-INF	258.473	+INF	.
---- VAR CMPSP	-INF	112.673	+INF	.
---- VAR CSPSP	-INF	215.917	+INF	.
---- VAR CPPSP	-INF	1.014	+INF	.
---- VAR CCAT	-INF	1.8083E+5	+INF	.
---- VAR CCDAA	-INF	3.3841E+5	+INF	.
---- VAR CODAA	-INF	2718.113	+INF	.
---- VAR CMDAA	-INF	1597.789	+INF	.
---- VAR CCFST	-INF	2.7466E+5	+INF	.
---- VAR COFST	-INF	1580.754	+INF	.
---- VAR CMFST	-INF	853.237	+INF	.
---- VAR CSFST	-INF	2664.445	+INF	.
---- VAR CCRSP	-INF	2.3158E+5	+INF	.

```
---- VAR CORSP         -INF   2951.485   +INF      .
---- VAR CMRSP         -INF   1448.899   +INF      .
---- VAR CSRSP         -INF   9665.307   +INF      .
---- VAR CPRSP         -INF    385.148   +INF      .
---- VAR cost          -INF   5.9796E+5  +INF      .
```

```
  Q2   m3 per hour
  Q5   m3 per hour
  Q8   m3 per hour
  SS3   g per m3
  SNH3  g per m3
  SNO3  g per m3
  XI3   kg per m3
  XS3   kg per m3
  XH3   kg per m3
  XA3   kg per m3
  XSS3  kg per m3
  XSS8  kg per m3
  XSTO3 g per m3
  q   overflow rate of primary settling tank (m per day)
  Ap  surface area of the primary clarifier (100m**2)
  Xl  XSS2 divided by XSSl
  SRT  sludge retention time (days)
  HRT  hydraulic retention time (days)
```

```
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```

```
  V   volume of the aeration tank (1000m**3)
  r   sludge recycle ratio which is the ratio of Q6 to Q2
  w   100*wastage ratio which is the ratio of Q7 to Q2
  Af  surface area of the final clarifier (100m**2)
  SR  overflow rate of final clarifier (m per day)
  ROH Oxygen Requirement for removal of organic matter (kg per day)
  ROA Oxygen Requirement associated with nitrification (kg per day)
  AFR Air flow rate  (m**3 air per min)
  X2  100*XSS4 divided by XSS3
  X3  XSS5 divided by XSS3
  CCPST
  COPST
  CMPST
  CSPST
  CCPSP
  COPSP
  CMPSP
  CSPSP
  CPPSP
  CCAT
  CCDAA
  CODAA
  CMDAA
  CCFST
  COFST
  CMFST
  CSFST
  CCRSP
  CORSP
  CMRSP
  CSRSP
  CPRSP
  cost  cost function (dollars)
```

```
**** REPORT SUMMARY :        0     NONOPT
                             0  INFEASIBLE
                             0  UNBOUNDED
                             0     ERRORS
```

```
EXECUTION TIME        =       0.000 SECONDS    0.7 Mb     WIN197-119
```

```
USER: Contract & Purchasing Dept.                    G010122:1131AE-WIN
      United Arab Emirates University                        DC3190
```

```
**** FILE SUMMARY
```

```
INPUT     C:\DOCUMENTS AND SETTINGS\ABDULHAMEED\MY DOCUMENTS\MY DOCS\GAMS FILES
          \APPENDIXA.GMS
OUTPUT    C:\WINDOWS\GAMSDIR\APPENDIXA.LST
```

النموذج المطور والذي يتميز بقدرته الفعالة على حل النماذج الرياضية غير الخطية.

تقدم الأطروحة أيضاً مثالاً لتطبيق النموذج المطور بشكل عملي، حيث تم التوصل إلى نتائج هامة وعملية تبين أن استخدامات النموذج تتعدى الوصول إلى التصميم الأمثل الأكثر اقتصادية، حيث يمكن استخدامه كأداة تحليل لدراسة تأثير مختلف المتغيرات على التصميم. وبشكل عام أظهرت الدراسة العديد من النتائج المفيدة فيما يتعلق بتأثير درجات الحرارة وتأثير تغير مواصفات مياه الصرف الصحي على التصميم. كما أظهر التطبيق أن التحكم المناسب في عملية المعالجة يؤدي بشكل واضح إلى وفورات هائلة في التكلفة سواء كانت تكلفة الإنشاء أو تكلفة التشغيل.

ومن المتوقع أن يمثل النموذج المقدم في هذه الأطروحة أداة فعالة تساعد مصممي عمليات المعالجة في الوصول إلى تصاميم أكثر اقتصادية تأخذ بالاعتبار مختلف المتغيرات التي تؤثر على عملية التصميم وتفاعل مختلف عمليات المعالجة مع بعضها البعض وهو ما تفتقر إليه معظم طرق التصميم التقليدية. كما يمكن استخدام النموذج في تقييم عمليات المعالجة تحت التشغيل واقتراح بعض الحلول لتحسين أداء هذه العمليات. وكاقتراح للأبحاث القادمة، تقترح الأطروحة تطبيق النموذج المطور على محطات المعالجة في دولة الإمارات سواء كانت تحت التشغيل أو مازالت في مرحلة التصميم.

ملخص الأطروحة

يعتبر الماء بالنسبة للإنسان عصب الحياة، ونظرا لاستمرار تناقص موارد المياه فقد شكل تلوث المياه ومعالجتها هاجساً لكثير من العلماء والباحثين طوال القرن الماضي. وتعتبر مياه الصرف الصحي بعد معالجتها أحد مصادر المياه البديلة التي تستخدم بشكل أساسي لأغراض الزراعة التجميلية في الدولة.

تعالج مياه الصرف الصحي عادة في محطات معالجة تتضمن عدة عمليات متعاقبة تؤدي في النهاية إلى إنتاج مياه صالحة للاستخدام. أحد هذه العمليات هي المعالجة باستخدام الحمأة المنشطة وهي من أكثر عمليات المعالجة البيولوجية شيوعا نظرا لما تتميز به من تكلفة منخفضة وقدرة عالية على إنتاج مياه نقية ومرونة تناسب معظم أنواع المياه المستخدمة. وعلى هذا فقد استحوذت هذه العملية البيولوجية على كثير من الاهتمام من قبل العلماء الذين وضعوا الكثير من النظريات والنماذج الرياضية التي تصف هذه العملية.

تصميم عملية الحمأة المنشطة بشكل تقليدي يعتمد بشكل أساسي على التجربة والخطأ وعلى خبرة المصمم السابقة، وفي كثير من الأحيان فإن التصميم المقترح قد لا يمثل التصميم الأكثر اقتصادية وقد لا يأخذ بالاعتبار تفاعل مختلف وحدات المعالجة مع بعضها البعض، كما أن هناك الكثير من عوامل الشك (العوامل غير المحددة) التي تدخل في تصميم هذه العملية. من هنا تظهر أهمية تطوير نموذج رياضي متكامل يجمع بين أحدث النظريات التي تصف عملية الحمأة المنشطة ومعلومات تكلفة البناء والتشغيل للوصول إلى التصميم الأمثل الذي يأخذ بالاعتبار عوامل الشك (العوامل غير المحددة)، وهو ما يمثل الهدف الرئيسي للأطروحة.

تقدم الأطروحة نموذجاً رياضياً يمكن استخدامه في تصميم نظام معالجة نموذجي يتضمن حوض ترسيب أولي بالإضافة إلى عملية الحمأة المنشطة والتي تتضمن حوض تهوية للمعالجة البيولوجية وحوض ترسيب ثانوي. يجمع النموذج بين أحدث النظريات التي تصف أداء عمليات المعالجة وبين المعادلات التي تصف تكلفة بناء وتشغيل هذه العمليات. وقد تم صياغة النموذج في إطار تصميم أمثل لمسألة تقليدية، الهدف فيها هو الوصول إلى التصميم الأكثر اقتصادية مع الوفاء بمتطلبات النقاوة المطلوبة. وقد تم استخدام برنامج GAMS في حل

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جامعة الإمارات العربية المتحدة
عمادة الدراسات العليا

تصميم أمثل باعتبار عوامل الشك لعمليات الحمأة المنشطة المستخدمة في معالجة مياه الصرف الصحي

أطروحة مقدمة من الطالب
عبدالحميد عبدالله عرواني

بكالوريوس هندسة مدنية
جامعة الإمارات العربية المتحدة (2000)

استكمالا لمتطلبات الحصول على درجة الماجستير في
علوم موارد المياه

يونيو 2003