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MONITORING, MODELING, AND CONTROL OF NUTRIENT REMOVAL IN THE ACTIVATED SLUDGE PROCESS (Under the Direction of M. BRUCE BECK)

The activated sludge process has been one of the most widely used biological processes for treating wastewater containing inorganic and organic pollutants. However, increased process complexity in the activated sludge system makes it more vulnerable to external disturbances, such as large variations in flow. Under these circumstances, analysis in mathematical form stands out with the potential benefits of improving understanding of process performance under dynamic conditions, and optimizing operation to treat greater volumes of wastewater, to deal with higher variability in influent load, and meet even more stringent discharge standards.

The objectives of this research are accordingly (i) to develop an understanding of the dynamic behavior of the biological nutrient removal processes in the activated sludge process and (ii) to determine efficient control strategies to achieve more reliable plant operation.

The dissertation begins with an extensive review of (i) dynamic behavior of the process; (ii) dynamic models of the activated sludge processes; and (iii) control schemes applied to the system. Collection of high-quality data at Athens Wastewater Treatment Facility No. 2 for an extended period of time is then described. The University of Georgia's Environmental Process Control Laboratory (EPCL) demonstrates enormous potential in retrieving high-frequency, high-quality field data, which is a prerequisite for success in the subsequent development of process models and process control strategies for these systems. A high-order model was then built up to simulate the nutrient removal processes, including especially a modified means of characterizing transport and mixing of both solute and particulate matter in the activated sludge system. Subsequently, the calibrated model was validated to test its robustness of under conditions different from those of calibration. The model is considered to be acceptable in its performance and thus

adequate for its subsequent application in studies of process control. Using the model thus identified, a detailed assessment of control strategies for the activated sludge process has been conducted. This is focused on storm event control. The manipulated control variables are confined to those routinely used, i.e., recycle rate, wastage rate, step-feed, and step-sludge. Even so, the operational flexibility of the activated sludge process can thus be fully exploited.

INDEX WORDS: Activated sludge system, Real time monitoring, Dynamic modeling, Signal processing, Model calibration, Model validation, Control strategies

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by

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Over the past decades eutrophication of surface water as a result of nutrient discharges has become manifest. Occurrences of elevated levels of nitrogen and phosphorus in various chemical forms give rise to algae blooms, which cause oxygen depletion, odor problems, aesthetic problems and serious disturbance of aquatic ecosystems. Consequently, legislation has imposed stricter demands on effluent nutrients. The nutrients present in the wastewater can be removed either physically or converted by means of chemical or biological processes. Due to its consistent efficiency in removing inorganic and organic pollutants from wastewater, activated sludge is currently one of the most widely used biological processes for treating domestic wastewater. However, increased process complexity in the activated sludge system makes it more vulnerable to disturbances such as large variations in flow, load and temperature. Sometimes the process loses its stability suddenly for no obvious reason, which underlines the importance of taking operational elements into consideration during the design process (Andrews, 1974). Thus there is a continuous need to better understand this biological wastewater treatment process. One strategy to fulfill this need is to model this process in mathematical form, which offers a number of potential benefits:

- □ Improve understanding of process performance under dynamic conditions;
- Optimize design of wastewater treatment plants;
- Optimize operation (prevent process failure and enhance treatment efficiency) to treat larger volumes of water, to deal with higher variability in influent load, and meet even more stringent discharge standards (Lijklema, 1973).

Reliable design and operation of biological wastewater treatment systems demands effective modeling of the various nutrient removal processes involved. Steadystate models are normally used for process design, and they can also be used for prediction of process performance. Due to the dynamic nature of biological processes and the unsteady disturbances to which they are subject, dynamic models are essential for describing the operation of biological wastewater treatment processes (for example, activated sludge process), and establishing the most effective real-time control strategy. Therefore, mathematical analysis is widely used to optimize or pinpoint the most costeffective operational solution. By imposing feasible control strategies on the activated sludge system, wastewater treatment plants can save money from more reliable and costeffective operation, and can, in principle, save even more money from deferring plant capacity expansion. Carucci et al. (1999) optimized the operating of a large wastewater treatment plant by shutting down one of the three treatment lines and overloading the other two. The potential cost savings of the vacuum-exhaust control (VEC) strategy for the city of Houston, Texas, 69th Street Treatment Complex was examined by Clifft et al. (1988). At 80% of design loading VEC was found to provide an oxygen-utilization efficiency of 94.9% as compared to 77.0% for the conventional control method. Based on the expected turn-down capability of Houston's oxygen production facilities, their simulations indicated that the VEC strategy would more than double the possible cost savings of the conventional control method. Additional savings at 80% and 100% of design loading were estimated to be \$113,000 and \$46,500 per year for the VEC strategy. As a rule of thumb, constructing a good mathematical model requires high-quality real world data of sufficient quantity. Reliable, on-line, 'intelligent' instruments can certainly satisfy this demand. However, lack of either first-class high-frequency real-time field data, reflecting the process responses to dynamic perturbations, or reasonably good process models, has long prevented environmental scientists from achieving better reconciliation of mathematical models with real data (Beck, 1986).

The University of Georgia's recently commissioned Environmental Process Control Laboratory (EPCL) opens up an opportunity to correct the above-mentioned data limitation. It has proven to be an effective tool in revealing actual process performance on-line. As an integrated facility equipped with multiple automatic instruments, it can be deployed in the wastewater treatment plant or other features of the aquatic environment, such as an aquaculture pond, for continuous sampling campaigns over extended periods, typically of the order of months. The data thus retrieved provide a solid foundation for subsequent model development, calibration, and application (Beck and Liu, 1998).

In 1986, the International Association of Water Quality (IAWQ), now International Water Association (IWA), published the Activated Sludge Model No. 1 (ASM1) (Henze et al., 1986), which describes biological nitrification and denitrification processes in the activated sludge system. In 1999, its most recent successor, the ASM3 (Gujer et. al., 1999) was published. ASM3 relates to the ASM1 and corrects for some defects of ASM1. As an extension to ASM1, the IWA published the ASM2 (Gujer et. al., 1995) in 1995. ASM2's successor, the ASM2d (Henze et. al., 1999), was later published in 1999. In addition to what is already described in ASM1/ASM3, enhanced biological phosphorus removal (EBPR) and two other chemical processes are incorporated in ASM2/ASM2d. The completeness of ASM2/ASM2d is certainly a significant contribution. They represent the state of art in modeling the activated sludge process. However, they have not been successfully calibrated against extended experimental observations collected from actual wastewater treatment processes. They are definitely not universally best for every single practical application, and will certainly suffer from problems of a lack of identifiability in model calibration. In other words, it is not possible to find a uniquely best set of parameter values for the model. Rather than a final answer, ASM2/ASM2d establishes a platform for further model development under different circumstances (Gujer et. al., 1995).

1.2 OBJECTIVES OF RESEARCH

The primary objectives of this research are to:

- Develop an understanding of the dynamic behavior of the biological nutrient removal processes in the activated sludge system with data acquired with the EPCL;
- Determine efficient control strategies to achieve more reliable plant operation.

These goals are achieved via the following steps:

- Retrieval of first-class high-frequency real-time field data over an extended period of time through a case-study experiment at the Athens Wastewater Treatment Facility #2 in Athens, GA;
- □ Signal pre-processing directed at the goal of model development;
- Development of a new dynamic model to simulate various nutrient removal processes in the activated sludge system. The detailed steps include:
 - Characterization of solute and particulate transport (through the aeration tank and secondary clarifier);
 - Development of nitrogen removal model (nitrification and denitrification);
 - Development of model for carbonaceous compound removal;
 - Development of phosphorus removal model.
- Development and assessment of feasible control strategies for operational management of the activated sludge process based on the resulting model.

1.3 CONTENTS AND CONTRIBUTIONS

There are seven chapters in the dissertation. This chapter has set out the background and objectives of the research.

In Chapter 2, the nature of the dynamic behavior of the activated sludge process is reviewed and presented. Subsequently, aspects of dynamic models and process control strategies for the activated sludge system are collected from the literature. Following this review, three aspects are dealt with in Chapter 3. One aspect is the sampling campaign arrangements at the Athens Wastewater Treatment Facility #2, including case study site description, sampling regime and retrieved data streams. The second aspect is the Environmental Process Control Laboratory (EPCL), including its hardware, software, and quality assurance and quality control (QA/QC) aspects. The sampling campaign succeeded in acquiring first-class extensive experimental results collected from an actual wastewater treatment process. Such comprehensive data, to the best of our knowledge, exist nowhere else in the published literature. The third aspect deals with data pre-processing, which concerns how raw data are processed without bias before being presented for model development. This work also provides an opportunity for assessing transport and mixing properties of both solute and particulate materials as they pass through the bioreactor and secondary clarifier of the activated sludge process, although these features will not be covered in this dissertation.

Chapter 4 deals with the subject of model development, with emphasis laid on characterization of solute and particulate transport through the aeration tank and secondary clarifier, together with nitrification and denitrification model development. This is where the proposed model is significantly different from what has usually been reported in the literature. This new model substantially enhances our capability to match the field data.

Following Chapter 4, Chapter 5 deals with model calibration. Complete calibration was carried out for the nitrification and denitrification processes. The overall strategy was to achieve successful nitrification fitting, because it is the best defined process and is most sensitive to external upsets of the various biological processes. Within the restricted span of the dissertation, efforts were then made to obtain acceptable carbon and phosphorus calibrations.

Based on the model thus identified, process control strategies are simulated in Chapter 6 to assess their effectiveness in improving plant operations during storm events. Advanced algorithms are tested to assess whether the process can be effectively controlled using the routinely manipulated control variables.

The principle conclusions of this research and recommendations for future work are given in Chapter 7.

CHAPTER 2

DYNAMICS, MODELING, AND CONTROL OF THE ACTIVATED SLUDGE PROCESS—A REVIEW

2.1 INTRODUCTION

The purpose of this chapter is to provide a general view of what has been captured in the literature on dynamics, modeling, and control of the activated sludge process.

The chapter is organized as follows. In section 2.2, the dynamic behavior of the activated sludge process is summarized. Based on a comprehensive understanding of process behavior thus provided, dynamic modeling of the activated sludge process is reviewed in section 2.3, which includes the reactor model (section 2.3.1) and secondary clarifier model (section 2.3.2). Section 2.4 deals with model calibration. Sensitivity analysis and control of the activated sludge process are then reviewed in sections 2.5 and 2.6 respectively.

2.2 DYNAMIC BEHAVIOR

2.2.1 Activated Sludge Process

The activated sludge process is a suspended growth system consisting of two stages, aeration and sludge/liquid separation. In theory, in the aeration stage extremely high rates of microbial growth and respiration can be achieved with unlimited food and oxygen, resulting in the utilization of available inorganic and organic matter to produce oxidized end products or the biosynthesis of new microorganisms. Air is added to the system either by surface agitators or via diffusers. Aeration has dual functions: supply of oxygen to the aerobic microorganisms for respiration, and maintenance of the microbial flocs in a continuous state of agitated suspension, ensuring maximum contact between the surfaces of the flocs and the substrates in wastewater. The mixed liquor (mixture of wastewater and microbial biomass) is subsequently displaced into secondary clarifiers. This is the final stage in the activated sludge process, where the flocculated biomass settles rapidly out of suspension to the bottom of the secondary clarifiers forming sludge under quiescent conditions, with the clarified effluent, which should be free of solids, discharged as the final effluent. It is a characteristic of the activated sludge process that a fraction of the settled sludge is recycled back to the aeration tank to achieve efficient nutrient removal. The surplus solids are wasted.

2.2.2 Microbial Biomass

Mixed liquor suspended solids (MLSS) concentration is a crude measurement of the amount of biomass within the aeration tank. In biological wastewater treatment a wide variety of microorganisms are found, including viruses, bacteria, fungi and protozoa. But the most widely occurring and abundant group of microorganisms is bacteria, and it is this group of biomass that is most important in terms of utilizing the inorganic and organic substrate in the wastewater.

The inorganic and organic matter in wastewater is utilized in a series of enzymatic reactions. Enzymes are pure proteins or proteins combined with either an inorganic or low molecular weight organic molecule. Enzymes act as catalysts to form complexes with the organic substrate, which they subsequently convert to a specific product. The enzyme is then released to catalyze the same reaction over and over again. Enzymes have such a high degree of substrate specificity that bacterial cells must produce a different enzyme for each substrate utilized. There are basically two types of enzymes. *Extracellular* enzymes convert substrates extracellularly into forms that can be taken into the cell for further breakdown by the *intracellular* enzymes, which are involved in synthesis and energy reactions within the cell. Normally the product of one enzymatic reaction

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immediately combines with another enzyme until the final end product required by the cell is reached after a sequence of enzyme-substrate interactions (Gray, 1990).

The most commonly used model, relating microbial growth to substrate utilization, is the Monod relationship. It is assumed in this model that the growth rate of biomass is not only a function of microorganism concentration, but also of limiting substrate concentrations. The relationship between the specific growth rate of biomass (μ) and the residual concentration of growth-limiting substrate (S) is described as follows:

$$\boldsymbol{m} = \boldsymbol{m}_m * \frac{S}{K_s + S}$$

where,

- μ_m -Maximum specific growth rate (d⁻¹) of microorganisms at saturation concentration of growth limiting substrate;
- S–Growth limiting substrate concentration (mg/L);
- K_s–Saturation coefficient (mg/L) which is the concentration of growth-limiting substrate at which the specific growth rate of microorganisms equals one-half of the maximum specific growth rate ($\mu = \mu_m / 2$).

2.2.3 Inorganic and Organic Substrate

Organic Substrate

Wastewater normally contains different kinds of organic matter, which all have in common at least one carbon atom (and thus are also known as carbonaceous compounds). They can be oxidized either chemically or biologically to yield carbon dioxide and water. Through these reactions the microorganisms obtain the energy necessary for their growth. A measurement of each individual organic matter is impossible. Therefore different collective analyses, such as Total Organic Carbon (TOC), Chemical Oxygen Demand (COD), and Biochemical Oxygen Demand (BOD), are adopted.

Inorganic Substrate (Nitrogen and Phosphorus)

Nitrogen and phosphorus together are known as nutrients in the wastewater. Nitrogen in sewage arises primarily from metabolic conversions of excreta-derived compounds, whereas 50% or more of the phosphorus arises from synthetic detergents (Gray, 1990). The principal forms of nutrients occurring in municipal wastewater are: NH_4^+ (ammonium), NO_2^- (nitrite), NO_3^- (nitrate), and PO_4^{3-} (orthophosphate). Perhaps the most widespread example of pollution through nitrogen and phosphorus discharges occurs through its ability to promote growth of algae. Significant seasonal and annual trends in effluent NH_4^+ variability were found in six out of seven nitrification plant studies in Ohio (Rossman, 1984).

2.2.4 Biochemical and Chemical Reactions

2.2.4.1 Nitrification

The microbial oxidation of ammonium occurs in two distinct stages, each of which involves different species of nitrifying bacteria. The first stage is the oxidation of ammonium to nitrite by *Nitrosomonas*. In the second stage nitrite is oxidized to nitrate by *Nitrobacter*. In practice it is the oxidation of ammonium that is generally believed to be the rate-limiting step in the overall process. The whole nitrification process requires a high input of oxygen.

In the context of sequential *Nitrosomonas-Nitrobacter* activity and a low-level of nitrite presence, the conversion of ammonium to nitrite by *Nitrosomonas* has long been deemed as the rate-limiting step for complete conversion of ammonium to nitrate (Benefield and Randall, 1980). Several studies, however, have observed the presence of a considerable amount of nitrite, a so-called 'nitrite buildup' (Silverstein and Schroeder 1983; Randall and Buth 1984; Abeling and Seyfried 1992; Mauret 1996). The possible conditions under which an elevated nitrite concentration might be realized in a nitrification system are reduced temperatures, limiting oxygen presence, elevated pH,

free ammonia presence, elevated solids wastage, acute process loading, and unusual nitrate reduction (Alleman, 1984; Balmelle et al., 1992; Mauret et al., 1996; Surmacz-Górska et al., 1997). In a similar study, Yang and Alleman (1992) discovered that nitrite buildup did not correlate well with either dissolved oxygen concentration or free ammonia. The concentration of free hydroxylamine and other intermediates in nitrification were the principal factor behind nitrite accumulation in batch nitrifying systems. Some researchers took inhibition of nitrite oxidation to nitrate as a more economical method of nitrogen removal in comparison with the traditional method of nitrogen removal (Turk and Mavinic 1989; Akunna et al., 1993).

Oxidation of nitrite by *Nitrobacter* in a chemostat system was reported to depend strongly on the presence of the ammonium oxidizer *Nitrosomonas* (Gee et al., 1990). Nitrite oxidation in the absence of ammonium required a 10-day retention time, whereas complete oxidation of ammonium was achieved at a retention time of 2.7 days. Reduction of nitrite and nitrate was reported to occur simultaneously and was dependent on the oxidation of ferrous to ferric iron (Nielsen and Nielsen, 1998).

Organic matter is known to "poison' the nitrifiers, so that increases in organic loading results in rapid decreases in the rate of nitrification. This is probably due to the increased activity of heterotrophs, which, because of their more rapid growth rates, successfully out-compete the nitrifying bacteria for access to dissolved oxygen and nutrients. This direct competition from heterotrophs is a major cause of nitrification failure (Gray, 1990; Henze et al., 1995b).

2.2.4.2 Denitrification

This is a biological process which occurs under low dissolved oxygen (anoxic) conditions and when a suitable carbon source is available. Oxidized forms of nitrogen, nitrite or nitrate, are reduced to molecular nitrogen form. The rate of denitrification occurring in secondary clarifiers, characterized by flocs buoyed up to the surface by

nitrogen gas, is extremely low due to a number of factors, the most important being insufficient carbon substrate for metabolism (Gray, 1990).

Two studies emphasized the role of denitrification in oxidation ditches (Huang and Drew, 1985; Rittmann and Langeland, 1985). In both investigations, significant denitrification was observed in the anoxic zone. Mateju et al. (1992) reviewed biological denitrification including its microbiology, reaction stoichiometry, bioreactors, and unit processes. Theoretically, denitrification can only proceed under anoxic conditions where a suitable carbon source, such as methanol, is available. Nitrate is progressively reduced to nitrogen gas by heterotrophic bacteria via the following route:



The enzymes associated with denitrification are synthesized when conditions become advantageous for denitrification (Knowles, 1982). However, it has been shown that denitrification can also occur with certain species in the presence of oxygen (Meiberg et al., 1980). Casey et al. (1994) found that aerobic denitrification only proceeded with nitrite, but not with nitrate and was enhanced under conditions of low readily biodegradable substrate in the aerobic phase. The influence of oxygen concentration (Krul and Veeningen, 1977; Simpkin and Boyle, 1988; Downes, 1988; Kokufuta et al., 1988), pH (Allison and MacFarlane, 1989), temperature (Lewandowski, 1982), nitrate, and intermediate product concentrations (Skrinde and Bhagat, 1982; Snyder et al., 1987) on denitrification performance have been investigated in detail. Denitrification rates by activated sludge increased from 0.001 to 0.02 mg N/mg MLSS/h in a solution containing 250 mg/L NO₂ when the pH was increased from 6 to 8 (Glass et al., 1997). Thomsen et al. (1994) demonstrated that pH significantly affected intermediate accumulation in denitrification. At pH 5.5 nitrate, significant quantities of nitrite, and

 N_2O were formed, whereas at pH 8.5 lower concentrations of intermediate products were formed. Batch tests determined that denitrification could occur at a DO concentration of up to 0.11 mg/L (Eliosov et al. 1997). Two models of denitrification intermediates were presented by Wild et al. (1995). Model I included the reduction of nitrate, nitrite, and nitrous oxide with noncompetitive inhibition by O_2 and NO_2^- . To improve the fitting of data, Model II incorporated the process of synthesis and decay of denitrification enzymes and the component "enzyme saturation coefficient." It was shown that increasing the anoxic zone DO concentration from 0 to 0.2 mg/L reduced denitrification activity by approximately 50%. The production and consumption of nitric oxide during denitrification and its implications were reviewed by Ye et al. (1994).

2.2.4.3 Aerobic Heterotrophic Conversion of Organic Matter

Organic matter in the wastewater can be separated into the following categories:

- Dissolved easily biodegradable organic matter, which can enter the biodegradation process directly;
- Dissolved biologically inert organic matter, which can not be biodegraded;
- Suspended slowly biodegradable organic matter, which must undergo cell external hydrolysis before being available for biodegradation;
- □ Suspended biologically inert organic matter, which cannot be biodegraded.

The most important factors for aerobic conversions of organic compounds are temperature, oxygen, pH, toxic substances, nitrogen, and phosphorus (Henze et al., 1995b). Ammonium assimilation by heterotrophs can take place in preference to nitrification in the aeration tank, with only the surplus ammonium after assimilation being subject to nitrification. Ammonium assimilation by heterotrophs was demonstrated to be proportional to COD consumption, and consequently significantly reduced the availability of ammonium for nitrification. Besides this effect, organic loading produced an inhibitory effect on ammonium oxidation. Production of toxic compounds by heterotrophs is a possible reason (Hanaki et al., 1990a). Another likely reason for the inhibitory effect is the localized competition between the heterotrophs and the nitrifiers for common substrates (Painter, 1977). Research on nitrification at low levels of dissolved oxygen, with and without organic loading in a suspended-growth reactor, reveals that a low DO concentration enhances the inhibitory effect of organic loading through heterotrophic activity on ammonium oxidation.

2.2.4.4 Phosphorus Removal

Phosphorus in the wastewater can be categorized into the following fractions:

- Dissolved inorganic orthophosphate, which is available for immediate biological metabolism;
- Dissolved inorganic poly-phosphate, which requires further decomposition to the more assimilable orthophosphate form;
- Dissolved organic phosphate, which requires further decomposition to the more assimilable orthophosphate form;
- □ Suspended organic phosphate.

Chemical Phosphorus Removal

In a biological wastewater treatment system precipitation of phosphorus via addition of metals, such as iron or aluminum salts, is a traditional strategy for phosphorus removal. But chemicals are very expensive, and the inorganic residues left from the chemicals can occasionally cause pollution problems. Recent advances in our understanding have led to the development of biological processes for phosphorus removal, which is a natural mechanism for removal of phosphorus by the phosphorusaccumulating bacteria in the mixed liquor. But biological phosphate removal is heavily dependent on the formation of these polyphosphate-accumulating bacteria. In cases where the ratio of COD to phosphate in the influent is too low to produce enough biomass for poly-phosphate storage, it is sensible to supplement the biological phosphorus removal process with chemical phosphorus precipitation.

Biological Phosphorus Removal

The first indication of biological phosphate removal occurring in a wastewater treatment plant was described by Srinath et al. (1959). They observed that sludge from a certain treatment plant when aerated exhibited excess phosphorus uptake, more than needed for cell growth. Milbury et al. (1971) defined some basic requirements for phosphorus removal by stating that the reactor must operate as a plug-flow scheme, and the first part of the reactor should not be well aerated. They also found that the sludge had a maximum capacity for accumulating phosphate. In the second half of the 1970s research in microbiology expanded. Fuhs and Chen (1975) concluded that bacteria of the genus Acinetobacter were responsible for biological phosphate removal. They postulated that an anaerobic phase was needed to produce volatile acids, which are the substrates for phosphate-removing organisms. Acinetobacter-type organisms could use these substrates under aerobic conditions for growth and excessive phosphate uptake. Rensink (1981) was the first to report that substrate might be sequestered as poly-hydroxybutyrate (PHB) by strictly aerobic organisms under anaerobic conditions at the expense of energy stored as poly-phosphate. He was therefore the first to make a direct mechanistic link between phosphate release and uptake in the biological phosphorus removal (BPR) process. Rather than providing a stress factor, the anaerobic phase not only supplies polyphosphate-accumulating bacteria with volatile fatty acids, but also offers a competitive advantage for their substrate uptake over other heterotrophic bacteria. This basic hypothesis was further developed and put in a more biochemical framework by subsequent researchers (Comeau et al., 1986; Wentzel et al., 1986; Arun et al., 1987; Smolders et al., 1996; Maurer et al., 1997). It was realized that phosphorus-accumulating bacteria take up easily degradable organic matter in the anaerobic zone as poly-

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hydroxybutyrate (PHB) or poly-hydroxyvalerate (PHV) (Comeau et al. 1987). The energy required for storage of PHB/PHV is produced via decomposition of the intracellular polyphosphate. As a result, the phosphorus accumulating bacteria will release phosphate in connection with the storage of organic matter. Under aerobic conditions, the phosphorus accumulating bacteria consume PHB/PHV. The energy produced is used by the phosphorus accumulating bacteria for growth and storage of phosphate in polyphosphate form. The phosphorus accumulating bacteria can also take up phosphate under anoxic conditions with nitrate serving as the oxidant, which was demonstrated by Comeau et al. (1987) and Kerrn-Jespersen et al. (1993). They found that the phosphorus uptake was more rapid under aerobic conditions than under anoxic conditions. Faster uptake under aerobic conditions was explained by the presence of two groups of bacteria. One group of bacteria could use both nitrate and oxygen as oxidants, while the other could only use oxygen as oxidant. Deterioration of enhanced biological phosphorus removal by the domination of microorganisms without polyphosphate accumulation was also observed (Satoh et al., 1994). Biological phosphorus removal has been observed in an aerated bioreactor in which no formal anaerobic zone is available (Brewer et al., 1995; Cinar et al., 1998). Storage of poly-phosphate might stop if phosphorus content within phosphorus-accumulating bacteria is too high (Gujer et al., 1995). Jones and Stephenson (1996) observed anaerobic phosphate release and aerobic phosphate uptake over a wide range of temperatures (5-45°C).

2.2.4.5 Hydrolysis Process

Many high molecular weight organic substrates cannot be utilized directly by the microorganisms. But they can be transformed into readily biodegradable substrates through enzymatic reactions external to the cell, which are usually called hydrolysis processes. Typically, these processes are considered to be surface reactions, which occur between the organisms, which provide the enzymes, and the slowly biodegradable

substrates. Hydrolysis can possibly take place under three different conditions: aerobic, anoxic, and anaerobic conditions. It is assumed that only heterotrophic organisms may catalyze these processes. Rates of hydrolysis processes under anoxic and anaerobic conditions are slower than that under aerobic conditions (Henze et al., 1985).

2.2.5 Factors Affecting the Activated Sludge Process

Pitman et al. (1988) reviewed the various factors responsible for obtaining low nitrogen and phosphorus effluent concentrations at biological nutrient removal plants. Effectiveness of the activated sludge process is generally affected by the nature of the crude sewage (the wastewater entering the plant prior to treatment) as well as environmental, climatic, and hydrological factors.

Biological activity of sludge flocs and their settling characteristics are affected by wastewater composition. In a conventional activated sludge process a BOD: N: P ratio of 100:6:1 is required to maintain the optimal nutrient balance for heterotrophic activity. The presence of toxic or inhibitory substances affects the metabolic activity of aerobic heterotrophs, although activated sludge does become acclimated to low concentrations of toxic substances with time (Gray, 1990; Henze et al., 1995).

Randall et al. (1982) examined the impact of temperature changes on various facets of activated sludge metabolism using experimental results and literature data. They argued that an increase in temperature of the mixed liquor enhances the inorganic and organic substrate removal of the activated sludge system. However, it might lead the system to the state of being oxygen-limited due to enhanced microbial activities at higher temperature. In the activated sludge system all the biochemical reaction rates, such as organic substrate stabilization, production of cellular material, maintenance energy requirements, oxygen utilization, auto-oxidation of cellular mass, and nitrification, follow the Arrhenius relationship over the 5-20 $^{\circ}$ C range (Gray, 1990).

$$K_T = K_{20} * \boldsymbol{q}^{T-20}$$

where,

T-temperature (°C);

 K_T -reaction rate constant (d⁻¹) at temperature T;

 θ -temperature coefficient.

The growth rate of nitrifying bacteria increases considerably with temperature over the range of 8-30°C, with Nitrosomonas having a 9.5 per cent increase per 1°C rise. Below 10°C the nitrification rate drops sharply, while above 10°C the rate is almost directly proportional to the temperature (Gray, 1990). Nitrosomonas isolated from activated sludge has an optimum growth rate at 30°C (Loveless and Painter, 1968) although a slightly higher range has also been reported at 30-35°C (Buswell et al., 1954). There is little growth of *Nitrobacter* below 5°C and no growth at all below 4°C. Nitrobacter has a slightly higher optimum temperature for growth at 35°C, although maximum growth has been reported up to 42°C (Gray, 1990). Nitrifying bacteria are especially sensitive to sudden variation of temperature. Lower nitrification performance at colder temperatures resulted from numerous factors, including lower reactant diffusivity in the bulk liquid and in the biofilm, low dissolved oxygen concentrations compared with saturation, and lower metabolic rate of the microorganisms. Pöpel and Fischer (1998a and 1998b) found that the traditional temperature coefficient tended to exaggerate the influence of temperature on effluent concentrations, particularly for plants with low loading.

The effect of oxygen absence on the activated sludge dynamics was studied by feeding an activated sludge pilot plant with a synthetic substrate (Maurines-Carboneill et al., 1998). Most of the bacteria revived after 8 days of anaerobiosis, indicating that some bacteria possess the capability of enduring oxygen limitation up to a limited amount of time. An analysis of operational data from 23 activated sludge plants led to the

conclusion that surface aeration systems did not nitrify as well as diffused systems due to the DO shortage (Maier and Krauth, 1988). It was also concluded that it is necessary for surface aerators to maintain an oxygen concentration of about 2.5 mg/L at the surface to facilitate complete nitrification, whereas for diffused air plants 1.5mg/L oxygen concentration over the whole tank surface was adequate.

The nitrification process itself reduces the alkalinity in water. It is favored by moderate pH of 7.2-9.0, with an optimum at 8.0-8.4. Below 8.0 the nitrification rate decreases until it is completely inhibited at pH below 5 (Wild et al., 1971). A decrease in pH inhibits the nitrifying organisms in such a way that their growth rates become lower than their wastage rates in the surplus sludge, which results in subsequent loss of nitrification. This can be controlled by addition of chemicals or provision of an anoxic zone to reduce the concentration of nitric and nitrous acids in the recycled sludge. Even though nitrifying bacteria can be acclimatized to slightly acidic conditions, it may take several weeks to do so. For example, a pH shift from 7.0 to 6.0 required 10 days of acclimatization before nitrification eventually returned to its former rate (Haug and McCarty, 1972). Antoniou et al. (1990) studied the effect of temperature and pH on the nitrifying bacteria. A functional relationship for the dependence of the maximum specific growth rate on both temperature and pH was verified via batch experiments with sludge from a local wastewater treatment plant. An optimum pH of approximately 7.8 was determined and the maximum specific growth rate was found to be a monotonically increasing function of temperature in the range of 15-25°C.

The nitrification process is inhibited by various kinds of substances, such as metals, sulphur, phenol and cyanide (Henze et al., 1995b). Even a limited inhibition could cause the nitrification to cease completely. However, this will not take place instantaneously, but only after a washout period over several weeks. The toxicity of copper and nickel on the activity of *Nitrosomonas* sp. was equal to or greater than *Nitrobacter* sp. (Lee et al., 1997).

A sudden increase in the hydraulic loading to the aeration tank due to storm events or recirculation of wastewater within the plant will increase discharge of mixed liquor to the secondary clarifiers. This will result in a reduction of MLSS in the aeration tank with more sludge stored within the secondary clarifiers. Sludge being stored for longer periods within the secondary clarifiers before being recycled to the aeration tank may adversely affect the viability of the microorganisms. Increased flow also reduces the effectiveness of the secondary clarifiers by increasing the upward flow rate, which extends the sludge blanket toward the surface with the possibility that some of the sludge is discharged with the final effluent resulting in pollution in the receiving water body. The effects of transient loads on full-scale nitrification were modeled using performance data from an activated sludge plant (Bliss and Barnes, 1986). Computer simulations indicated that hydraulic transients caused by on/off pump control increased effluent suspended solids and that dynamic models were useful in sizing pumps and selecting pumping strategies which minimized transient effects (Nakamura et al., 1986).

The normal operation of the sludge handling system leads to recycling of supernatant with a high content of ammonium to the inlet end of the reactor. The load from the supernatant made about 15% of the total nitrogen load on the plant on an average basis but could reach up to 65% during working hours. The digester supernatant and its impact on daily variation in ammonium and nitrate have been examined in detail and the examination has formed the basis for a mathematical modeling of the system in order to test strategies for optimized handling (Jansen et al., 1993). Studies in the past have neglected the effects of recycled supernatant when using optimization techniques for choosing the best design from numerous alternatives. This can lead to serious errors in optimization and faulty selection of the "best" design. Arun et al. (1988) described an approach, which would enable incorporation of the recycle supernatant to any optimization techniques. Separate nitrification of supernatant from dewatering processes was investigated in a pilot SBR (Mossakowska et al., 1997). During the nitrification

process nitrite was accumulated until ammonium nitrogen was depleted in the reactor. It was shown that nitrite accumulation was primarily caused by the initial concentration of ammonium and secondly by the oxygen concentration.

2.3 MODELS

2.3.1 Dynamic Modeling of the Nutrient Removal Processes in Aeration Tank

Recent developments in modeling the activated sludge process were reviewed by Lessard and Beck (1991). Orhon et al. (1999) provided a comprehensive coverage of the experimental information required for the activated sludge treatment of industrial wastewater, such as choice of parameters for organic carbon removal, the value of basic relationships between major parameters, the merit of size distribution for the evaluation of pretreatment, COD fractionation and its implication in system design, major kinetic and stoichiometric coefficients for process modeling. Henze (1992, 1995c) provided a guideline on how to characterize the wastewater for activated sludge process modeling.

The improvement in modeling the dynamics of activated sludge wastewater treatment process using a distributed parameter approach has been discussed (Lee et al., 1997). The hydraulic model employed in that study considers backmixing or intermixing, which can represent the actual process more accurately than the idealised flow schemes commonly employed for modeling and/or design of the activated sludge bioreactor. A computational algorithm, based on the global orthogonal collocation procedure, for the activated sludge process was developed in this work. Dochain et al. (1997) suggested how to use asymptotic observers to model and validate reaction schemes independent of the reaction kinetics. The model represents the hydrodynamics, transport dynamics, and biochemical conversions in an activated sludge system. Yuan et al. (1997) used an observer-based approach to identify modeling error in a denitrifying bioreactor. The method was used to pinpoint errors in model structure and parameters and identify additional laboratory experiments to improve calibration. Gao et al. (1997) applied a

cybernetic approach to the modeling of mixed substrate biodegradation and found reasonably good agreement with experimental data. Additional mechanisms, such as the interaction of enzyme synthesis with multiple substrates and biomass adsorption of substrate, need to be included to improve model accuracy. A procedure has been developed to improve the accuracy of an existing mechanistic model of the activated sludge process, previous described by Lessard and Beck (1991) using neural network (Cote et al., 1995). The coupling of the mechanistic model with neural network models resulted in a hybrid model yielding accurate simulations of five key variables of the activated sludge process.

Watson et al. (1994) showed that there is no general rule or global "optimal level" of modeling, and the required modeling detail is a function of influent flow and loading levels, processes to be simulated, and the purpose of the model. The degree of detail is often constrained by data availability and reliability. Steffens et al. (1997) proposed a systematic approach for reducing complex models of biological wastewater treatment processes. They provided a means of quantifying the interaction between state variables, the "speed" of a state and whether it is a candidate for reduction.

2.3.1.1 Nitrification

Mathematical modeling of the nitrification process in wastewater treatment systems was reviewed (Prosser, 1990). One of the early comprehensive dynamic models for the nitrification process was developed by Poduska and Andrews (1975), which successfully employed Monod kinetics to depict the growth rate of the nitrifying bacteria. In the model there were five state variables: ammonium, nitrite, nitrate, *Nitrosomonas* and *Nitrobacter*. This model was later evaluated with data collected at Norwich Sewage Works in eastern England, where it was recognized that the model was limited in its ability to characterize nitrifier growth principally because of the assumption that growth of nitrifying bacteria was independent of dissolved oxygen concentration (Beck, 1981).

Beck (1984) later reported a model for the activated sludge process accounting for CBOD removal and 2-stage nitrification, with crude mechanisms for simulating the behavior of a toxic substance, a bulking sludge, and a rising sludge. This model has in fact been developed as a surrogate of the real system for developing and testing the fuzzy controller of Tong et. al. (1980). In ASM1 Henze et al. (1986) considered for simplicity that the autotrophic conversion of ammonium to nitrate is a single-step process. In ASM2 (Henze et al., 1995a) the nitrification process is modeled similar to that in ASM1, the difference being that phosphorus is taken up as nutrient by the single nitrifying species. ASM3 (Gujer et. al., 1999) includes the possibility to differentiate decay rates of nitrifiers under aerobic and anoxic conditions. Nowak et al. (1995) and Huang et al. (1996) extended ASM1 by using a 2-step nitrification with nitrite as intermediate product. Lessard (1989) and Chen (1993) calibrated their models based on ASM1 with data, especially for nitrification. Andreottola et al. (1997) introduced nitrite-buildup into a dynamic simulation model based on ASM1. The new model was successfully reconciled with actual data. Nowak et al. (1995) developed an extended nitrification model on the basis of ASM1 in order to control the process under inhibiting conditions. Model elements for competitive and non-competitive inhibition as well as for biodegradation of the inhibitor were added. Operational as well as simulation results showed that nitrifying activated sludge plants may become acclimatized to inhibitory compounds, but had to be protected from peak loads of both nitrogen (ammonium, nitrite and nitrate) and inhibitory compounds.

Spies et al. (1988) discussed the importance of DO in obtaining full nitrification. Operating procedures that increase oxygen transfer were found to be most effective in increasing the rate of nitrification (Gullicks and Cleasby, 1990). Strenstrom and Song (1991) introduced a model accounting for oxygen transport limitation and competition for oxygen between heterotrophs and nitrifiers. The simulation results indicated that DO could limit nitrification rates at concentrations ranging from 0.5 to 4.0 mg/L. The nitrifying bacteria are more sensitive to low oxygen concentrations than the heterotrophic bacteria. It is generally accepted that nitrification does not occur below 0.2-0.5 mg/L. However, no inhibition is found at oxygen concentrations greater than 1.0 mg/L (Wild et al. 1971; Sharma 1977). Under DO concentrations that limit nitrification, the Nitrosomonas and Nitrobacter compete with each other for the available oxygen (Stenstrom and Song, 1991). The outcome of the competition will be determined by their specific affinities for oxygen as well as their population sizes. Result of a mixed continuous culture experiment showed that the specific affinity for oxygen of Nitrosomonas was in general higher than that of Nitrobacter. When aerobic conditions were switched to anoxic, *Nitrobacter* was washed out and nitrite accumulated. However, when nitrifying bacteria grew at low oxygen concentrations, the specific oxygen affinity of Nitrobacter increased and became as great as that of Nitrosomonas. Due to its larger population size, *Nitrobacter*, the nitrite-oxidizing bacteria, became the better competitor for oxygen, and ammonium was then accumulated in the fermenter (Laanbroek et al., 1994). Ossenbruggen et al. (1991) used oxygen uptake rate and dynamic models to describe interactions between the organisms responsible for nitrification.

2.3.1.2 Denitrification

The most important factor influencing nitrogen gas bubble evolution in the secondary clarifier tanks is the rate of biological denitrification (Henze et al., 1993). Rising sludge due to nitrogen bubbles will rarely appear, even at higher ammonium influent concentrations if return sludge is adjusted appropriately (Siegrist et al., 1994). In order to control denitrification in the secondary clarifier, a model was developed which included a variable sludge volume in the clarifier related to scraper interval under the assumption of constant sludge mass in the entire system. Hamilton et al. (1992) presented two models in their paper. One considered bioreactions in the secondary clarifier, while the other neglected them but incorporated a more detailed model of the clarification and

thickening functions. In ASM2 (Henze et al., 1995a) denitrification is assumed to be inhibited by oxygen, and the maximum growth rate of heterotrophic biomass due to denitrification is reduced relative to its value under aerobic conditions. This accounts for the fact that not all the heterotrophic organisms may be capable of denitrification or that denitrification may only proceed at a reduced rate.

2.3.1.3 Simultaneous Nitrification and Denitrification

Nitrification and denitrification are usually regarded as separate processes, occurring in different layers of water and requiring individual reactors for separate wastewater treatment. A separate anoxic zone for denitrification is generally chosen close to the point where the settled sewage and return sludge are fed to the reactor. The process can also be configured to remove the nutrients by providing mixed and nonaerated zones and internal process recycle streams to create the anoxic and anaerobic environments needed for biological nitrogen and phosphorus removal. The design and operation of biological nutrient removal activated sludge systems using these well-defined anoxic and anaerobic zones has evolved during the past 20 years to the point where it is a widely applied technology. However, researches in the 1980s and 1990s established that nitrifiers and denitrifiers were not as metabolically fastidious as previously thought, and strict segregation was not necessary. Nutrient removal has been observed numerous times in the activated sludge facilities that do not possess explicitly defined anoxic and anaerobic zones. Nitrogen losses from aerated facilities have been observed frequently (Applegate et al., 1980; Drews and Greef, 1973; Rittmann and Langeland, 1985; van Huyssteen et al., 1990; van Munch et al., 1996; Bertanza, 1997). This phenomenon has been referred to as simultaneous nitrification and denitrification because it was assumed that these two biological processes were occurring simultaneously in the same aerated biological aerator. It is well known that full-scale bioreactors do not provide an entirely uniform environment throughout. Examples of such bioreactors include oxidation ditches

and plants with oxygen transfer devices, such as mechanical surface aerators that cause large scale recirculation of the mixed liquor (Applegate et al., 1980; Cinar et al., 1998; Drews and Greef, 1973; Grady et al., 1999; Randall et al., 1992; Rittmann and Langeland, 1985; and van Huyssteen et al., 1990). In such facilities intense oxygen transfer occurs in one portion of the bioreactor, limited oxygen transfer occurs throughout the rest of the bioreactor, and mixed liquor is recycled between the aerated and nonaerated zones. Simultaneous nitrification and denitrification occurred at nitrogen concentration up to 11 mg/L nitrogen in a combined system with a low DO (Hanaki et al., 1990b). The Orbal process for the treatment of wastewater was claimed to have simultaneous nitrification and denitrification in the outer lane in the presence of a dissolved oxygen concentration of 1.5 mg/L. Denitrification continued for some time before oxygen replaced nitrate as the terminal electron acceptor and the length of the lagged response was a function of the duration of the anoxic conditions (O'Neill et al.,1995). Dynamic simulations of nitrification and denitrification were performed using ASM1 calibrated to field conditions at full-scale industrial and municipal wastewater treatment plants (Coen et al., 1997). Standard analyses and respirometry were used to determine heterotrophic and autotrophic growth parameters, and the simulations showed how various anoxic volumes, recycle flow rates, and feed schemes impacted system performance.

2.3.1.4 Phosphorus Removal

Barnard (1983) summarized the early work on phosphorus removal in activated sludge plants in the USA. Discovery of simultaneous nitrogen and phosphorus removal, as well as full-scale experiments, was discussed. Several researchers reviewed the microbiology and biochemistry of enhanced biological phosphorus removal (van Loosdrecht et al., 1997a and 1997b; Smolders et al., 1996; Mino et al., 1998). Wentzel et al. (1991) and Toerien et al. (1990) described the kinetics of biological phosphorus

removal in nitrification and denitrification activated sludge systems. Limitations in current models were stated to be due to inadequate understanding of denitrification rates in biological nutrient removal systems. Barker and Dold (1996) collaborated on a literature review of denitrification behavior in biological excess phosphorus removal (BEPR) activated sludge systems. It was found that a significant fraction of poly-P organisms could use nitrate as an electron acceptor in the absence of oxygen for oxidation of stored PHB and simultaneous uptake of phosphorus. Murnleitner et al. (1997) presented an integrated metabolic model for the aerobic and denitrifying biological phosphorus removal

Temmink et al. (1996) reported that the high effluent phosphate level that occurred during and after short periods of low organic loading resulted from the slow rate at which depleted poly-ß-hydroxybutyrate (PHB) storage was replenished. The authors recommend using decreased aeration times during and after periods of low organic loading to obtain faster rates of biological phosphorus removal recovery. Phosphorus removal was found to be affected by nitrogen in activated sludge plants (Barnard, 1982). Below a COD/N ratio of 10:1, it is very difficult to control phosphate removal. A double-Monod kinetic expression was used to model the biodegradation capability of Pseudomonas denitrificans at varying concentrations of nitrate or nitrite (Kornaros and Lyberatos, 1997). The presence of high concentrations of nitrate or nitrite (up to 400 mg/L as nitrogen) caused a severe decrease in the specific growth rate and cell yield of phosphorus-accumulating microorganism. Gerber et al. (1987) studied the interactions between phosphate, nitrate and organic compounds in biological nutrient removal processes. The phosphate release was controlled primarily by the nature of the substrate rather than the creation of an anaerobic state. And it was concluded that phosphate uptake and release occur simultaneously in the presence of fatty acids, which also render the best overall phosphate removal during aeration. Stephens and Stensel (1998) reported that longer aeration times and low acetate concentrations significantly decreased phosphorus
removal efficiency in sequential batch reactors (SBR).

The effect of temperature on the biological phosphorus removal (BPR) process was investigated by Brdjanovic et al. (1997) and Tasli et al. (1997). Kumar et al.(1998) compared EBPR at 25 and 10°C, and found that the effluent phosphorus concentrations were the same although phosphorus release was lower at 10°C. Anaerobic phosphorus release reached the maximum rate at 20°C, and the rate of aerobic conversion increased continuously as the temperature increased from 5-30°C. The temperature coefficients for the anaerobic and aerobic processes were determined to be 1.078 for temperature between 5 and 20°C, and 1.057 for temperatures between 5 and 30°C, respectively

Ante et al. (1994) developed a mathematical model based on ASM1, including the kinetics of enhanced biological phosphorus elimination in particular. Model assumptions and interaction between the phosphorus accumulating and nitrogen eliminating organisms were considered in the light of recent literature. Smolders et al. (1994 and 1995) proposed a metabolic model for the biological phosphorus removal process. The model was based on the bioenergetics and stoichiometry of the metabolism. It was able to describe the dynamic behavior of all the components in a sequenced batch reactor during the anaerobic and aerobic phases very well over a wide range of sludge retention time (SRT) values. ASM2 was used to explain a rise in phosphate levels due to an accumulation of polyhydroxyalkanoates in an alternating type activated sludge pilot plant (Isaacs et al., 1995).

2.3.1.5 Combined Nitrogen and Phosphorus Removal

Developments in modeling the kinetics of three types of activated sludge system were reviewed by Wentzel et al. (1992). Examples of research areas that require attention to complete the development of the general kinetic model were found to be denitrification by poly-P organisms, and calibration and verification of the model for cyclic flow and load. A river water quality model was presented for the prediction of the concentration fields of the water quality variables in an oxidation ditch performing carbon oxidation, nitrification and denitrification (Stamou, 1997). This model involved the one-dimensional convection-dispersion equations for all the variables, which are described in ASM1. Hydrodynamic effects were represented in a model developed by Stamou (1994).

The effect of temperature on the overall nitrogen/phosphorus removal process in a biological nutrient removal (BNR) plant was studied (Choi et al., 1998a, 1998b). Ninety percent of nitrification occurred at temperatures of 8°C, but with decreased denitrification resulting in lower phosphorus removal due to elevated nitrate concentrations in the return sludge. The phosphorus removal was not affected at low temperature when nitrification did not occur. The operations of a biological phosphorus removal and a biological nitrogen removal plant at low temperatures were compared (Ydstebo and Bilstad, 1997). The authors reported that biological phosphorus removal was accomplished at 5° C with 0.6mg/L total phosphorus in the effluent, and biological nitrogen removal was accomplished at 6-8°C with an average of 0.25 mg/L total phosphorus and 5.3-9.6 mg/L total nitrogen in the effluent. To aid in design and optimization of temperature-sensitive biological treatment processes, the effect of dynamic temperature changes was modeled by Scherfig et al. (1996). The model predicts the hourly temperature in biological treatment tanks within 0.5°C during a 1-month period when the hydraulic retention time ranges between 12 and 36 hours. The authors showed that the daily temperature fluctuations are strongly dependent on the local wind conditions, and they recommended the use of windbreaks or tank covers to reduce temperature variability in treatment systems. Based on IAWQ activated sludge models, the cold temperature operation of a full-scale wastewater treatment plant was successfully simulated at temperatures down to 12°C (Funamizu and Takakuwa, 1994). Aeration basin temperature was modeled by Sedory and Stenstrom (1995) to account for changing weather conditions and wastewater characteristics. Evaporation and relative humidity were found to have a major impact on wastewater cooling.

Dold et al. (1986) reviewed the antecedents to the general activated sludge model proposed by the IAWPRC Task Group on modeling of the activated sludge system. Modifications to the Group model were proposed and sets of experimental data from a wide range of single sludge systems were presented to validate the model. A modification to the IAWPRC model was proposed which permitted predetermination of denitrification rates (Griffiths, 1994). Extension of this modification may well provide insight into the effect of "selectors" on activated sludge systems and the varying specific growth rates measured for both heterotrophs and autotrophs. ASM2, with the recommended default and calibration parameters, was used to successfully predict the effluent nutrient concentrations with the exception of ammonia and nitrate (Mino et al., 1995). Modeling difficulties were attributed to uncertainty in the anaerobic hydrolysis rate, anaerobic substrate uptake rate, and poly-\(\beta\)-hydroxy-alcanoic acids (PHA) yield. This case study showed how relevant calibration procedures could be developed with limited static data. Sen and Schwinn (1997) reviewed several case histories for biological nutrient removal systems. Cinar et al. (1998) applied ASM2 to characterize the nutrient removal of four ASPs. It was concluded that the model was successful in modeling the plants having surface aeration and diffused aeration but could not characterize the performance of the two oxidation ditch plants. The authors suggested that the incompleteness of fundamental understanding of the specific hydraulic flow pattern and bioreactor configuration of these two oxidation ditch plants might have contributed to the failure. The complexities of activated sludge process modeling were investigated in a modification of the ASM1 to include biological phosphorus removal (Barker and Dold, 1997a). Model simulations showed reasonable predictions; however, some uncertainties, particularly with respect to denitrification, suggest that some kinetic and stoichiometric parameters must be calibrated for different reactor configurations, flow regime, and microbial communities (Barker and Dold, 1997b). Maurer et al. (1998) presented a dynamic model based on ASM2 for the description of enhanced biological phosphorus removal. With the aid of 18

batch-experiments and measurements from a wastewater treatment pilot plant, a set of kinetic parameters was estimated, which was able to reproduce satisfactorily the nutrient removal behavior of the investigated sludge. Cinar et al. (1996) concluded that the application of ASM2 to biological nutrient removal processes in the oxidation ditch using available plant operating and design data was a difficult task. For bioreactor processes, ASM2 was robust in modeling heterotrophs, requiring calibration of only one parameter, whereas for extension to the modeling of phosphorus accumulating organisms and nitrifiers, calibration of three and two additional parameters was required, respectively. Dynamic models for NDBEPR (Nitrification Denitrification Biological Excess Phosphorus Removal) systems that achieved combined biological removal of nitrogen and phosphorus were presented (Ducato et al., 1995; Takacs et al., 1995; Bogdan et al., 1998). The model included the biological reactors and the secondary settler.

The Orbal process bioreactor consists of three closed-loop reactors in series. Oxygen input to each stage can be varied to allow the creation of different environments (Smith, 1996). As a result, spatially varying environments can be created throughout the bioreactor. The performance of seven full-scale, staged, closed-loop bioreactor activated sludge plants were studied to characterize their overall nutrient removal performance and the effect of operating parameters on nutrient removal. This evaluation was conducted in the context of an overall evaluation of simultaneous biological nutrient removal, which hypothesizes that three mechanisms may be responsible for simultaneous biological nutrient removal: (1) mixing pattern, that is, the bioreactor macroenvironment, (2) the floc microenvieonment, and (3) novel microorganisms (Daigger et al., 2000). In comparing full-scale performance of a Pasveer oxidation ditch with a Carrousel facility, Mulready et al. (1982) found that the latter system consistently produced a higher quality effluent. The results of experimental and mathematical modeling of simultaneous processes of organic and nitrogen removal in an industrial wastewater treatment plant were presented by Derco et al. (1994).

2.3.1.6 Tracer Study

Wastewater of three treatment plants was characterized using ASM1 (Siegrist et al., 1992). The mixing characteristics of the aeration tanks were determined with sodium bromide as a tracer. Newell et al. (1998) fitted model predictions for a conservative tracer passing through a series of completely mixed reactors of a nutrient removal plant to Rhodamine dye data profile and concluded that the number of model compartments could be reduced to 66% of the number of plant tanks. The performance of large-scale aeration tanks from the viewpoints of substrate behavior and characteristics of fluid flow was evaluated (Iida, 1988). The results of tracer studies applied to mathematical models indicated that flow was subject to high longitudinal and latitudinal dispersion in the aeration tanks. However, the flow patterns were too complicated to describe satisfactorily with the mathematical models; a key problem lay in trying to join a transient system to an equilibrium model. Horan et al. (1991) used four tracing agents to test mixing efficiency in an activated sludge reactor. There are many different methods available for analyzing the results of tracer studies on the aeration lanes of activated sludge plants. If the results are to be used for modeling, it is necessary to calculate the number of tanks in series to allow the dispersion within the tanks to be accurately modeled (Burrows et al., 1999). The effects of aeration tank mixing characteristics on activated sludge process performance were evaluated both experimentally and theoretically (Shimizu et al., 1993). A multistage tower aeration tank outperformed a completely mixed aeration tank with respect to removal efficiency and sludge settlability in experiments, and modeling results indicated plug flow with partial mixing to be the optimal design. Simultaneous differential equations of plug-flow reactors resulting from mass balances on substrate and biomass around an infinitesimal volume element were solved analytically taking the longitudinal biomass gradient into account under steady-state conditions (San, 1989). It was found that kinetic coefficients have pronounced effects upon mean solids residence times, suggesting that more realistic studies reflecting field conditions are required for

determination of kinetic coefficients and for evaluation of treatability experiment. A 3D hydraulic model was found to be successful when applied to plant layouts with surface aerators (Alex et al., 1998). Interpretation of reactor mixing characteristics in the light of dissolved oxygen data was considered when the performance of a wastewater treatment works was simulated using a dynamic modeling package from WRc, STOAT (Stokes, 1997).

2.3.2 Dynamic Modeling of the Secondary Clarifier

Krebs (1995) summarized the successes and shortcomings of clarifier modeling. A dynamic model essential for implementation of control strategies for the activated sludge process was developed for the thickening function of the secondary clarifier (Tracy et al., 1974). Dick (1972, 1985) and Vitasovic (1989) predicted the solids concentration of the recycled sludge and the resulting mixing liquor solids for varying conditions of operation based on the basic sedimentation characteristics of the sludge. This model incorporated all requisite operating parameters of the unit. Simulations were presented which illustrated the utility of the model as a tool in the design and operation of activated sludge wastewater treatment plants. Solids gravity flux variability was found to be appreciable at three operating municipal wastewater treatment facilities investigated during a 14-month study (Morris et al., 1989). Choi et al. (1993) carried out research to determine an appropriate settling velocity model to calculate secondary settlers of an activated sludge system by using the solid flux theory. Takacs et al. (1991) presented a model, which was an alternate form of the settling velocity model, allowing for the development of a unified approach to dynamic modeling of the clarification and thickening functions of settlers. A paper written by Jeppson et al. (1996a, 1996b) supported and illustrated recent theoretical results on the mathematical modeling of the secondary clarifier. A new settler model was compared with a traditional layer model by means of numerical simulations. An important conclusion was that a layer model dividing the settler into only ten layers (normally used in setter models) is too crude an approximation to capture the detailed dynamic behavior of the settler. Hasselblad and Xu (1998) described a process to determine settling parameters from dynamic, full-scale rectangular clarifiers that can be used to model the clarifier performance. One and twodimensional hydrodynamic models of secondary clarifiers were discussed by Krebs (1995). The lack of current knowledge and the potential for significant improvements in modeling were addressed. Two one-dimensional algorithms were evaluated by means of numerical simulations (Jeppsson et al., 1996). The second algorithm, which is based on a percentage vector that describes the different particulate biological components as fractions of the total suspended solids concentration, was computationally efficient and did not exhibit any oscillatory behavior. In Chen (1993)'s model it was assumed that when the mixed liquors from the reactor flow into the secondary clarifier, suspended solids matter pass entirely into the thickening zone, and any suspended solids subsequently found in the clarification zone are as a result of resuspension from the thickening zone. This conceptualization of resuspension is considered an important step towards improving the clarification model.

Dupont et al. (1992) took their biokinetic expressions from ASM1, whereas their sludge-thickening component was based on a parabolic partial differential equation derived from a material balance. The model predicted the sludge blanket height and the suspended solids concentration profile in the secondary clarifier under both steady state and dynamic conditions. A dynamic model of the activated sludge process was presented, in which emphasis was put on the hydrodynamic aspects of the system (Zhong et al., 1996). Hasselblad et al. (1998) applied a simple linear dynamic model to secondary clarifier performance. The dynamic modeling could accurately predict movement of the sludge blanket height in secondary clarifiers and also indicated the level of the current limiting solids flux. When coupling a model of the biological reactor to a model of the settler, difficulties appear because of the different representations used for the particulate

material. A combined model of the entire activated sludge process needs to include an algorithm for the dynamic propagation of the biological components of the particulate material through the secondary clarifier. In particular, this is of importance for an accurate description of the sludge that is recycled to the biological reactor. Diehl and Jeppsson (1996) presented an ASP model based on the coupling of ASM1 model with a one-dimensional partial differential equation (PDE) settling model based on the Kynch sedimentation theory. Halttunen (1996) presented a simple thickening model, based on the use of a combination of plug-flow reactor (PFR) and continuous stirred-tank reactor (CSTR) modules to simulate short-circuiting, with the time variation of solids concentration being inversely proportional to a thickening factor and the solids concentration. Applied to a full-scale plant, the model simulated observed sludge blanket volume in the clarifier. Predicted RAS concentration deviations from the measurements were approximately 5 to 15%. Research into denitrification in the secondary clarifier has been conducted (Henze et al., 1993; Siegrist et al., 1994).

Operation of clarifiers in series can produce better settling sludge than operating clarifiers in parallel, since better settling microorganisms were selected in the series arrangement (Kim et al., 1998). According to Siegrist et al., (1994), denitrification in the secondary clarifiers is primarily due to hydrolysis of particulate degradable COD and decay of biomass. The reduction factor of these processes under anoxic as compared to aerobic conditions was investigated in the treatment plants of the City of Zurich.

2.4 CALIBRATION

Very few case studies in calibration of dynamic process models with field data, as opposed to individual kinetic models tested on laboratory batch data, have been reported in the literature. Nitrification performance data from an activated sludge plant was simulated using a simple model based on growth requirements of nitrifiers only and a complete-mix flow pattern (Bliss et al., 1986). With the experimental data of investigations at the treatment plants Zurich-Glatt, Wattwil, and Zurich-Werdholzli the ASM1 was calibrated for COD removal and nitrification for a typical Swiss municipal wastewater (Siegrist et al., 1992). The calibrated model was verified with experimental data of the Zurich-Werdholzli treatment plant during an ammonia shock load. Lessard (1989) and Chen (1993) calibrated their activated sludge process models with the field data collected from the Norwich Sewage Works. The actual data were sampled on a 3hour basis over a 10-day period from May 7 to 17, 1986. Ducato et al. (1995) presented a dynamic model for a NDBEPR (Nitrification Denitrification Biological Excess Phosphorus Removal system) that achieved combined biological removal of nitrogen and phosphorus. The model included the biological reactors and the secondary settler. The model was verified with reference to an existing urban wastewater treatment plant with a three-stage configuration (anaerobic, anoxic, aerobic reactors). Carlsson et al. (1996) conducted studies on enhanced biological phosphorus removal operation in activated sludge processes on a pilot plant at the Sjoelunda wastewater treatment plant in Malmoe, Sweden since 1986. Results derived from daily, composite samples taken over 3-year formed the basis of a general description of the water quality and the performance of the process. In addition, intensive field studies and laboratory studies have been used as ways of investigating certain phenomena in more detail. Rouleau et al. (1997) conducted an evaluation exercise using data collected through intensive sampling during rain events. The results show that it is possible to simulate the behavior of a plant during a wet weather period with the usual models found in the literature. An activated sludge model for biological N- and P-removal was developed, which describes anoxic and aerobic Puptake based on bacteria metabolism (van Veldhuizen et al., 1999). This model was calibrated over 2-day measurements in two treatment plants. The model appeared to be able to give an adequate description of the performance of these treatment plants under different conditions.

2.5 SENSITIVITY ANALYSIS

A sensitivity analysis was performed on a kinetic-based model for wastewater stabilization ponds (Prats and Llavador, 1994). The analysis revealed that the stability of the simulation was dependent on mean residence time of the process and the kinetic parameter being adjusted. The authors indicated that their method of analysis should be applicable to any kinetic model of biochemical reactors. van Veldhuizen et al. (1999) analyzed the sensitivity of the effluent concentrations and the sludge production as a function of the stoichiometric and kinetic coefficients, the influent composition, the setup of the hydraulic model and the flows of air, mixed liquor and return sludge. This analysis showed that besides a limited number of model parameters (12%) the influent characterization influenced the model output significantly. von Sperling (1993) described an adaptation of the regionalized sensitivity analysis on Monte Carlo simulations for the parameter estimation and sensitivity analysis of an activated sludge model. A dynamic model of an activated sludge waste treatment plant was described, examples of typical simulation results were offered, and a sensitivity analysis of an example plant was given in Tanthapanichakoon et al. (1981). Sensitivity studies for the most important parameters and operating conditions were done, using plant data (Scheer et al., 1996). Recommendations were given, based on these studies, for the optimization of the EBPR process. These recommendations illustrate the most effective means of improving the operating conditions for EBPR (e.g. increase of amount of readily available organic substrate, decrease of sludge age) with regard to an increase of the biologically removed phosphorus content. A structured model for sequencing batch reactors, which was developed earlier and tested successfully against a number of experimental data sets, was used in a study to investigate the sensitivity of model predictions to some of the system kinetic parameters for a wide range of parameter values (Abasaeed, 1997).

2.6 PROCESS CONTROL

Hartley (1985) provided an overall presentation on the subject of activated sludge process control. Chen (1993) developed a systematic procedure by which models of the activated sludge process can be identified and a more complete framework within which suitable control strategies for the activated sludge process can be derived. In the activated sludge process the principal objective of process control are to damp out input variation, minimize effluent variation, prevent and recover from process upsets (Lumbers, 1989; Leeuw et al., 1996; Otterpohl et al., 1994; Tench, 1994; Spanjers et al., 1998; Solly et al., 1997; Rosen and Morling, 1998; van Veldhuizen et al., 1999; Nolasco et al., 1994; Bradstreet and Johnson, 1994; Carucci et al., 1999). Control of these factors is directly related to process reliability in terms of meeting discharge requirements and minimizing energy expenditure. Control strategies have been developed for all these objectives, but results have been mixed for a variety of reasons.

Publications aimed at improving the performance of treatment plants, that were not meeting standards, were provided (Stover and Cowan, 1985; the U.S. EPA, 1985). In both publications improvement of operational procedures played the key role. The use of contract operation rather than in-house public service employees was addressed by several authors (Carter,1985; Peterson,1985). Other procedures for improving treatment plant performance include the use of effective management techniques (Andrews, 1974), record systems (Trout, 1985), and video taping (Nicolai, 1984). Video tapes were used to train personnel to install and maintain equipment properly. However, automation and real-time control of wastewater treatment plants have increasingly become important topics of research and development in recent years. Real-time control is considered a desirable goal for moderate and large facilities to attain better treatment efficiencies, improved compliance, and reduced operating costs. For an efficient implementation, realtime control must be supported by adequate modeling methodologies that take into full account the dynamic properties of the treatment system. Capodaglio (1994) evaluated the requirements of such applications, the properties of available modeling approaches, and some application examples.

2.6.1 Manipulated Variables

Variables, which can truly be manipulated in the activated sludge process, are summarized below.

□ Sludge Recycle Rate

Sludge is returned to the aeration tank in order to maintain sufficient microbial biomass for oxidation of nutrients in the wastewater. The traditional strategy to control return sludge is to use a fixed percentage of the plant inflow. Brune (1985) developed an optimal control strategy for the recycle flow rate. Simulation of the controlled system showed that for this specific plant the biomass concentration in the aeration tank could be lowered significantly. However, great care must be exercised when recycle rate is increased significantly because the hydraulic and solids loadings on the settler are also eventually increased, which could destroy the positive impact brought by such an action (Lessard, 1989).

□ Sludge Wastage Rate

The normal objective of sludge wasting is to stabilize the concentration of mixed liquor suspended solids (MLSS) in the aeration tank. Holmberg et al. (1984) found out that the activated sludge process was not sensitive to the recycle flow rate control, while waste flow control with the objective to keep the sludge concentration constant was promising. In a case study done by Georgakakos et al. (1990), a new control approach was compared with the commonly proposed recycle ratio control strategy, and a constant recycle and wastage rate strategy. Wastage regulation was a much more effective control input than recycle for reducing effluent organic variability. The control framework proposed could be extended to include other regulation schemes and account for process and input uncertainties. Vacari et al. (1989) compared several control algorithms for

activated sludge waste rate by computer simulation. There are two ways to implement sludge wastage: from the clarifier or from the mixed liquor. Considering the considerable hydraulic effect on the secondary clarifier of a change in recycle rate, the adjustment of wastage is preferred in practice over the manipulation of recycle rate, provided they can achieve the same objective. The slow responses of the activated sludge system to wastage rate determine that it may be more suitable for long-term control.

□ Step Feed

Step-feed control is the manipulation of the spatial distribution of influent flow along the aeration tank. It can equalize the oxygen supply and demand in the plug-flow configuration. It is used when hydraulic shocks act on the system, such as a rain event. Detailed studies of step-feed control can be found in the dissertations by Lessard (1989) and Chen (1993). Research conducted by Sorour et al. (1993) provided a better understanding of applying step feed control considering the role of biomass activity in the settler. They developed a procedure for varying the feed point to ensure optimum operation under varying conditions. Strategies for operation and control of the secondary clarifiers were presented by Keinath (1985). Step-feed control was considered effective for clarifiers overloaded in regard to both clarification and thickening. The model that included the ASM2 was developed for simulating the performance of the plant in Sapporo City (Funamizu et al., 1997). Simulated results showed that the choice of the sewage step-feed ratio did not affect the nitrogen removal and that the denitrification rate in the anoxic zone was controlled by the hydrolysis rate of the slowly biodegradable organic matters. The results of the lab-scale experiment and simulation results showed that the addition of the readily biodegradable organic matters like fermentation products of the primary settler sludge was effective to improve the performance of nitrogen removal. A step-feed activated sludge configuration with pre-anoxic zones at or following each point of influent addition was found to achieve reliable nitrification whereas plug flow regimes with pre-anoxic zones did not (Stephenson and Luker, 1994).

Since step-feed control shortens the retention time of influent in the aeration tank and a poor nutrient removal may result. This is especially true in the case of nitrification, as a result of the short hydraulic retention time (Lessard, 1989).

□ Step Sludge

Step-sludge control is manipulation of spatial distribution of recycle sludge along the aeration tank. Zickefoose (1981) proved that step-sludge control could handle highly variable wastewater with a weak organic loading.

Step-sludge control transfers sludge from the aeration tank to the secondary clarifier in such a faster way that suspended solids might be carried over in the effluent when the sludge blanket is high. Also in order to prevent the sludge blanket from overflowing as a result of implementing step-sludge control, the sludge wastage rate has to be increased which inevitably causes the reduction in sludge age. From these perspectives step-sludge control has to be exercised with caution.

□ Aeration Control

The ability to control the rate of aeration should be a primary consideration in plant design. This is especially true when it is recognized that the energy requirements of aeration equal to 60 to 80% of all energy costs to operate the plant, and that with control, savings of from 25 to 50% are possible (Robert, 1983). Buhr et al. (1984) adopted the step-aeration approach, the most versatile of the activated sludge modifications. A simulation study (Healey, 1989) permitted the selection of an effective DO control system that would adjust aeration rates during diurnal and seasonal changes in inlet water quality. The advanced simulation model, STREAM, was used to simulate wastewater treatment plants (van der Kuij et al., 1994). The model was successfully employed to optimize aeration control and evaluate a number of technological process adjustments, without the need for costly research, at the Kralingseveer wastewater treatment plant. The result of this approach was to improve the treatment process. Czeczot (1998) discussed

the minimal-cost control of the modified activated sludge process with varying level of wastewater in the aeration tank.

2.6.2 Control Algorithms

In spite of the fact that there are now many examples of conventional PID controllers being used in reality (Beck, 1986), most of the more advanced algorithms are discussed in theoretical terms. Von Jeszenszky et al. (1976) found out that sludge recycle control provided substantial improvement for all models. On-off feedback control was best for linear models, and feedforward control was more effective for Monod kinetics. Chen (1993) found that combined feedback control of returned sludge flow and wastage flow is not advantageous in controlling storm events and improving effluent performance. However, combined control provides a good biomass control and can thus be important for long-term operation.

Other control algorithms have attracted attention as well (Tong et al., 1980). Fuzzy inferential control was applied to activated sludge by detecting the presence and quantity of inhibitory or dangerous compounds in the influent and taking steps to prevent process upset (Müller et al., 1997). The behavior of the system was tested with and without process control, and improved reliability was observed with fuzzy control. Serra et al. (1994) showed an application of artificial intelligence to help wastewater treatment plant operators with process control. Other researchers have proposed "expert system" type models (Patry et al., 1989; Lai and Berthouex, 1990). Expert systems consist of a set of rules, defined by a domain expert, that are linked with the historic (past and present) database of the treatment system.

Although various algorithms have been developed for control of the activated sludge process, few of them have ever been applied in practice, except for DO control (Olsson and Andrew, 1978; Holmberg, 1984). The reasons are numerous and an assessment of some of these controllers is therefore conducted in Chapter 6.

2.6.3 Operational Indicators

The effects of control actions taken are in turn reflected in operational indicators, among which the following are frequently used:

□ Effluent water quality

The most obvious indicator of system performance is the effluent water quality, such as effluent ammonium, nitrite, nitrate, and COD concentrations. And they are the values subject to legislative regulations.

 \square MLSS

MLSS is a crude measurement of biomass in the system. The minimization of system cost based on the balance between aeration tank volume and clarifier surface area was addressed by Pincince et al. (1995). The authors argue that the ideal MLSS value for treatment of a given wastewater can be determined using cost curves for aeration tanks and clarifiers and present a case study based on their methods.

DO DO

DO profile in the aeration tank is an overall reflection of the activated sludge process including its hydraulic and organic loading, reaction rate, and the degrees of completion of the various reactions. Olsson and Andrews (1978) found that DO profile is a good qualitative indicator of the system's overall performance.

□ Sludge blanket height

This is an empirical indicator for the performance of the secondary clarifier.

There are also many other operational indicators. Fujimoto et al. (1981) and Spanjers et al. (1998) used respiration rate as an operational indicator on treatment facilities. Oxidation and reduction potential (ORP) has been in use for a long time as a measurement of the factors that contribute to electron activity, such as pH, chemical constituents of the system, the variety of biological activity and temperature (Peddie et al., 1990). Certainly none of the above indicators is on its own a good enough measure of the state of the process. Alternatively, the combination of some of the indicators should be able to represent the process better.

2.7 CONCLUSIONS

From this extensive literature review of dynamic modeling of the activated sludge process, we can draw the following conclusions. Most published biological nutrient removal models are based on the ASM2, either simplified or extended according to actual application. However, their calibration is greatly limited by the availability of real field conditions at full-scale wastewater treatment over extended periods. Also there have hardly been any reports on successful calibration of such models to high quality field data at full-scale wastewater treatment plants. The purpose of the next Chapter is to summarize the sampling campaign carried out in the Athens Wastewater Treatment Facility No. 2 and data processing for model development later in Chapter 4.

CHAPTER 3

DATA COLLECTION AND DATA PRE-PROCESSING

3.1 INTRODUCTION

It can generally be concluded from the preceding literature review chapter that scarcity of on-line experimental data is one of the reasons that have long obstructed better reconciliation of the model with observed behavior (Beck, 1986), and implementation of efficient control actions (Lessard and Beck, 1991). The University of Georgia's Environmental Process Control Laboratory (EPCL), however, turns out to be outstanding in achieving the state of being "data rich". This chapter describes the sampling campaign covering February, March, and April, 1998, hardware and software aspects of the EPCL, some interpretations of the resulting data, and aspects of data pre-processing for the subsequent development of the model in Chapter 4.

3.2 SAMPLING CAMPAIGN

3.2.1 Case Study Site Description

The Athens Wastewater Treatment Facility No. 2, one of the three operated by Athens-Clarke County, was constructed in 1963. It was designed to provide primary and secondary treatment of sewage from the western residential area of Athens-Clarke County. The preliminary treatment stage involves the removal of solids in the influent as it passes through screens. During the biological treatment stage the concentration of carbon and nitrogen-based pollutants are reduced. Sludge produced in the aeration tanks settles to the bottom of the secondary clarifiers, where it is collected and then either returned to the aeration tanks to aid in the treatment process or thickened and digested in the anaerobic digesters. After digestion the sludge is pumped to the dewatering building, where it is mixed with polymers and run through centrifuges to achieve the separation of solids from excess water (supernatant). Usually, when the solid content in the sludge is higher than 20%, it is hauled and discharged into a sanitary landfill. The excess water (supernatant), however, is returned to the aeration tanks. Effluent of the secondary clarifiers is disinfected by chlorine, which therefore completes the whole treatment process before the effluent is discharged to the Middle Oconee River. Figure 3.1 shows the process configuration at the Athens Wastewater Treatment Facility No. 2, Athens, GA. The annual average wastewater flow to the plant is about 5MGD, and the corresponding hydraulic detention time in the aeration tank is about 24 hours if two tanks operate together.



Figure 3.1 Process Configurations at the Athens Wastewater Treatment Facility No. 2 during Spring ,1998.

There are two aeration tanks in the plant, each of which has three channels (Orbal facility). The sewage flows in sequence from the outer channel through middle channel into the inner one. Effluent from the two tanks is combined and then split equally into three secondary clarifiers for water-sludge separation and sludge settlement. Under normal operation the returned activated sludge is pumped from the bottom of the secondary clarifiers back to the outer channels of the aeration tanks. Figure 3.2 shows the overview of the plant. The outer channel, middle channel and inner channel typically contain 50 to 55%, 30 to 35%, and 15 to 20% of the total bioreactor volume, respectively. A typical design and operating strategy for the Orbal process is to provide less oxygen to the outer channel than required to meet the full process oxygen demand (Smith, 1996). Typical calculations indicate that approximately 70% of the total process oxygen demand will be exerted in the outer channel. However, the oxygen supply provided in the outer channel is only 50 to 70% of the calculated demand. This is done to allow nitrification and denitrification to occur in the outer channel. Grady et al. (1999) and Randall et al. (1992) discussed manipulation of oxygen supply to control nitrification and denitrification. Mixed liquor can also be recycled from the inner channel to the outer channel to transport nitrate-nitrogen formed in the middle and inner channels back to the outer channel, where it can be denitrified. Nitrogen removal efficiency greater than 90% can be achieved using these strategies (Applegate et al., 1980; Drews and Greef, 1973; and Smith, 1996). The Orbal process can also be operated in the step-feed mode by feeding all or a portion of the process influent to the middle and inner channels. This operating mode would typically be used during high wet-weather flow conditions to avoid overloading the secondary clarifer and causing thickening failure in the secondary clarifier. But this operating mode might not support simultaneous biological nutrient removal (Daigger et al., 2000).



Figure 3.2 Overview of Athens Wastewater Treatment Plant No. 2

In the activated sludge system as shown in Figure 3.2, both the influent wastewater and the returned sludge are added at the front of the aeration tank. The tank is equipped with surface mechanical aerators to provide oxygen to the mixed liquor along the length of the tank. As the mixed liquor proceeds along the tank the organic matter is utilized with the desired level of removal being controlled by the time it takes to reach the outlet at the far end. In theory the sludge growth rate should be discernible as the mixed liquor moves along the tank with an initially rapid rate of removal of organic substrate (waste) becoming progressively slower as the mixed liquor makes its way along the tank. There should also be discernible nutrient concentration gradients along the tank. The rate of oxygen utilization also changes along the length of the aeration tank, so the oxygen supply may be deficient at the inlet where demand is greatest and be in excess at the outlet where the demand is lowest. But this can be overcome by a modification known as step aeration. The term "step aeration" suggests a gradual reduction in the aeration along the tank. It is vital to ensure that the aeration at the outlet end of the tank is also sufficient to maintain the sludge in suspension as well as supply the required oxygen. It is easier to introduce step aeration where surface aerators are used, and the different

aeration rates are achieved by altering their speed of rotation. The same function can also be performed by introducing the influent wastewater incrementally at several points along the length of the tank with all the recycled sludge still introduced into the aeration tank at the inlet end. A useful advantage of this "step feed" strategy is that the proportion of influent wastewater entering the tank at each stage can be varied according to changes in the organic or hydraulic loadings. This gives the process a considerable degree of flexibility in operation. A third strategy similar to "step aeration" or "step feed" is to feed the returned sludge incrementally at several points along the length of the tank with the influent wastewater entering the system at a single point at the inlet and the aeration being uniform along the length of the tank at the same time.

3.2.2 Sampling Regime

The sampling campaign started on February 1, 1998, and was completed on April 28, 1998. Considering that operating just one aeration tank, as opposed to both, should achieve equally good treatment efficiency and, more importantly, save power costs, the plant stopped feeding the influent to aeration tank No.1 on 11 February 1998, using it subsequently as a backup. The sampling campaign was divided into two sections. The first sampling section (02/01/98~04/06/98) was a comprehensive monitoring of the whole activated sludge system. Figure 3.3 shows the plant schematic with sampling locations for the first section. The second sampling campaign (04/20/98~04/28/98) was focused around one of the secondary clarifiers, with measurements for the aeration tank effluent, returned activated sludge just before its mixing with the sewage in the aeration tank, and the secondary clarifier outflow as seen in Figure 3.4. The results of the second sampling campaign will not be discussed in this dissertation.



Figure 3.3 Plant Schematic with Sampling Locations in the First Sampling Section



Figure 3.4 Plant Schematic with Sampling Locations in the Second Sampling Section

The first section of the campaign was relatively more successful. During the second section frequent blockage of the returned activated sludge (RAS) pumps were encountered due to the thick, viscous nature of the RAS.

Table 3.1 summarizes the observations recorded at various sampling locations in the first part of the campaign. By the end of that period more than 2000 time points of data with 45-minute time interval for each of these observations were obtained. Table 3.2 provides a summary for the observations made in the second part of the campaign.

Sampling Locations	Observations
Crude Sewage	Flow, Ammonium, TOC,
	NO _x , Ortho-phosphate-P
Aeration Tank 2	Respirometry,
Outer Channel	Ammonium, TOC
Aeration Tank 2	Respirometry, NO _x ,
Middle Channel	Ortho-phosphate-P, DO
Aeration Tank 2	Respirometry, Ammonium,
Inner Channel	TOC, DO, MLSS
Secondary Clarifier	Sludge Blanket Level
Secondary Clarifier Effluent	NO _x , Ortho-phosphate-P
	Turbidity

 Table 3.1 Observations at Different Sampling Locations

 in the First Sampling Section

Table 3.2 Observations at Different Sampling Locations in the Second Sampling Section

Sampling Locations	Observations
Aeration Tank 2	Respirometry
Inner Channel Effluent	Ammonium, TOC
	NO _x , Ortho-phosphate-P
	DO, MLSS
Returned Activated Sludge	Respirometry
At The Aeration Tank Inlet	Ammonium, TOC
	NO _x , Ortho-phosphate-P
Secondary Clarifier	Respirometry
	Ammonium, TOC
	NO _x , Ortho-phosphate-P
	DO, Turbidity
	Sludge Blanket Level

3.3 ENVIRONMENTAL PROCESS CONTROL LABORATORY (EPCL)

3.3.1 Introduction

The Environmental Process Control Laboratory (EPCL) was manufactured by Capital Controls (Minworth Systems Limited) in Birmingham, England. It can be used in many contexts, especially in the study of municipal and industrial wastewater treatment, protection of surface water quality, and groundwater contamination. It is also referred to as being "mobile" in the sense that it can be moved around and deployed at different sites for real-time water quality monitoring as opposed to an ordinary "stationary" laboratory in a building. In this research the EPCL was fully adopted to support the development of a dynamic process model for the activated sludge system at the Athens Wastewater Treatment Facility No. 2 in Athens, GA.

3.3.2 Hardware Aspects

As seen in Figure 3.5, the two white trailers essentially comprise the so-called Environmental Process Control Laboratory (EPCL). Each of the trailers is equipped with on-line automatic monitors. One trailer houses a respirometer, ammonium monitor, TOC monitor, and turbidity sensor (specifically for determining samples with low suspended solid concentrations, such as secondary clarifier effluent) coupled with a homogenizer and a debubbler. The other trailer has a respirometer, NO_x (nitrite and nitrate) monitor and orthophosphate monitor. Figure 3.6 presents the interior view of the trailer. Other than the on-board monitors, such out-board monitors as two DO probes, a mixed liquor suspended solid (MLSS) probe, and a sludge blanket level sensor are installed outside the trailers in the process tanks. For instance, the sludge blanket level sensor was fixed to the bridge over the secondary clarifier to monitor the movement of sludge blanket level. Most of these monitors have the capability of self-cleaning and self-calibration operations. The calibration frequency is preset by the commands issued at the "Trailer

Terminal" interface, which is explained below. Some monitors, such as DO probes, can even perform the function of self-correction, which is to correct its operation against driftings or malfunctions. The following is brief definitions of the monitors in the EPCL, their corresponding analytical methods and the calibration frequency against the standards (where appropriate).



Figure 3.5 External View of the EPCL



Figure 3.6 Internal View of the EPCL

Each on-board and out-board monitor will be defined in summary note form as follows.

On-board Monitors

□ Respirometer

In tank form designed by Capital Controls (MSL), housing the following monitors with their respective working ranges: temperature (0-50°C); pH (0-14); DO (0-10gm⁻³); MLSS (0-10,000gm⁻³); and conductivity (0-20,000µS).

□ Ammonium Monitor

Designed by Capital Controls (MSL) with heat-insulated, temperature-controlled housing, ion-selective probe (a Cole Palmer 27502-00 gas sensing electrode), and a pH probe for monitoring the pH of a caustic buffer together with a temperature sensor. Operating range is 0-100gm⁻³, with daily calibration against 1gm⁻³ and 10gm⁻³ standard solutions.

D TOC Monitor

Pollution and Process Monitoring Protoc 100 TOC analyzer using and infra-red CO_2 detector (0-3,000ppm) to provide an operating range of 0-500gm⁻³ TOC. Calibration is on a daily basis using clean water and a standard solution of 200gm⁻³ TOC.

Orthophosphate Monitor

Designed by Capital Controls (MSL) using colorimetric detection and having an operating range of 0-20gm⁻³; calibration is on a daily basis using a combination of an obscured light source, an empty cell, and a standard solution of 10gm⁻³ orthophosphate.

□ Nitrate/TON (NO_x) Monitor

Prototype Capital Controls (MSL) design using colorimetric detection of NO_2 and 0-30gm⁻³ TON. Calibration is on a daily basis using a combination of an obscured light source, an empty cell, and a standard solution of 10gm⁻³ TON made up of 90% nitrate and 10% nitrite.

□ Turbidity Sensor

Infra-red reflection (Nephelometric) detection operating over 0-500ftu ranges and designed by Capital Controls (MSL). Calibration is done manually.

Out-board Monitors

Sludge Blanket Level Sensor

Zellweger Analytics SLM-3000 sludge level monitor and profiler with 4-way detector.

DO Probe

Clarke-type DO cell designed by Capital Controls (MSL), with Danfoss Evita 085G0022 DO cartridge. The probe is pneumatically operated, self-cleaning and self-calibrating against air in the respirometer and against air blown into the housing on a daily basis.

□ MLSS Probe

Optical detection system designed by Capital Controls (MSL), pneumatically operated and self-cleaning. Requires manual calibration against either a formazin standard, clean water, or gravimetric analysis of grab samples of sludge on a daily basis.

Each of the trailers can continuously receive samples from three locations on site. All the samples are pretreated through the ultrafilters to ensure being solid-free before being presented to the instrumentations on-board. Filtrates from these ultrafilters enter each individual monitor in turn through pneumatical valves. For instance, the ammonium monitor is presented in turn with a sample from each of the three streams, producing a measurement of the ammonium concentration in each of the three sample lines once every 45 minutes.

Day-to-day records of all measurements taken in 15-minute sampling interval are stored in the on-board data loggers, together with data recorded with a much shorter time interval down to one second. The data with much shorter time intervals are essentially electrical signals gathered during the calibration cycles of each instrument for diagnostic purposes. Since the data logger can only hold a data volume of up to several days, the on-line data amassed are accordingly transferred from the local system to a remote UNIX based workstation through a phone line on a regular basis.

In order to check the operational status or set up the operational mode of the monitors, the instruments can be accessed in three ways:

- □ Front panel access, which is for access to the operational information of the instruments with the keypads on the faces of the instruments;
- Local PC access, which is to hook up a personal PC to the communication port onboard such that the operational information of the instruments can be assessed and/or changed with one of the three-letter commands;
- Remote PC access, which is to remotely check or reset the operational information of the instruments through a phone line without even being in the EPCL in person.

3.3.3 Software Aspects

The software package of the EPCL has the following components:

□ FT for windows (Capital Controls, 1994)

This enables stored data to be retrieved directly from the monitors, and is usually used for the purpose of instrument testing.

GFX (Capital Controls, 1996)

This presents the data in graphical form. It can also export data in a format compatible with the "Translate for Windows" program, which makes it possible to add or edit "GFX for Windows" data.

- Translation for Windows (Capital Controls, 1996)
 This allows a file with time-stamped data to be converted into a form where it is manipulated by GFX. It is designed to facilitate viewing and limited editing of the data.
- **□** Trailer Terminal (Capital Controls, 1996)

This provides a platform for accessing and controlling the operations of the instrumentations.

3.3.4 Quality Assurance and Quality Control (QA/QC)

The detailed data files logged at a resolution of down to 1-second intervals are collected principally for QA/QC purposes. Whenever a monitor is engaged in a self-calibration, a detailed data file is triggered with diagnostic information being logged. By checking the diagnostic information of an individual monitor, a judgement can be made as to whether the monitor is functioning properly, or otherwise identify possible problems. For example, the measurements taken by the orthophosphate monitor involve a colorimetric mechanism. Figure 3.7 shows the PO₄ Head Output curve when the orthophosphate monitor is functioning properly, while the curve as seen in Figure 3.8 denotes problems with the monitor. The PO₄ Head Output is the voltage generated by the amplifier and thus is an easy indicator of the proper functioning of the monitor. In Figure 3.8 there is none of the color development phase as seen in Figure 3.7 (the flat platform near the tail end of the curve), which is caused by reagent failure, either because of

reagent exhaustion or difficulty in reagent delivery due to blockage somewhere along the transportation line. QA/QC procedures for all the other monitors are listed in Appendix I.

Note that the figures shown in this section (Section 3.3.4) are all in GFX (introduced in Section 3.3.3) format. In GFX, units are indicated at the upper right corners of the curves.



Figure 3.7 Normal PO₄ Head Output Curve



Figure 3.8 Problematic PO₄ Head Output Curve

Regular intra- and inter-laboratory checking of the standard solutions for the monitors is also essential for ensuring quality of the data quality. Using again the example of the orthophosphate monitor, it gave a reading of 10.00mg/L to a 10mg/L standard solution. The standard solution was also sent to the on-site laboratory at the Athens Wastewater Treatment Facility No.2 for inter-laboratory checking, where the result was 10.13mg/L.

3.3.5 Practical Experience With The On-line Monitors

The on-line monitors require attentive service and maintenance if they are to function properly. A complete daily routine was consistently exercised during the sampling campaign to ensure smooth operation of the EPCL. This daily routine basically included checking instrument status through the built-in software and visual inspection. Checking logged diagnostic data for each monitor with the EPCL built-in software allows one to determine the possibility of existing problems, such as those indicated in Figure 3.8 for the orthophosphate monitor. A summary of the daily maintenance checklist is included in Appendix II.

Typical of the raw time-series is that for ammonium concentration in the crude sewage shown in Figure 3.9. The abrupt decrease of ammonium concentration labeled as 1 was caused by the misplacement of the sodium hydroxide suction tube. Sodium hydroxide is used for the ammonium monitor to maintain an ideal alkalinity. After the normal NaOH transport into the ammonium monitor was recovered, the diagnostic curve returned to a "normal" value. On some occasions the ammonium monitor failed due to a faulty ammonia probe, in which case the voltage output of the probe was extremely low compared to the desired, optimum value. In order to overcome this kind of problem, the ammonia probe was replaced, and the monitor was recalibrated and left to stabilize for at least 24 hours before the normal measurement was resumed. Generally, after replacement and recalibration, the voltage output would increase to the optimum level. The period labeled as 2 was when the ammonium monitor was suspended for maintenance. Again, this figure is in the GFX (Section 3.3.3) format and the unit is at the upper right corner.



Figure 3.9 Ammonium Concentrations in the Crude Sewage -An Example of Raw Time Series

3.3.6 Use Of On-line Monitors For Process Monitoring And Control

So far no constraint has been imposed on the effluent orthophosphate concentration at the Athens facility. Thus the whole activated sludge process can in principle be monitored and manually controlled by measuring the ammonium concentration, nitrate concentration and DO concentration in the secondary clarifier effluent. Integration of on-line monitors into plant operation would introduce ways of improving overall plant performance, since the on-line data can be used directly by the plant operators for detecting process faults and for advanced process control. This process is essentially a preliminary reflection on the data and its potential use before formal modeling and control studies are carried out, as now illustrated.

Use of On-line Monitors for Process Fault Detection

The activated sludge process requires a thorough control of the dissolved oxygen concentrations in the aeration tanks. It is important to sustain a given dissolved oxygen level for efficient nitrification and reasonable energy consumption. However, during the period of Figure 3.10, dissolved oxygen concentration in the middle channel (lower plot) was consistently deficient compared to dissolved oxygen concentration in the inner channel (upper plot).



Figure 3.10 Dissolved Oxygen Concentration in (a) the Inner channel and (b) Middle Channel (bottom) of Aeration Tank #2.

In order to obtain complete nitrification, ammonium in the crude sewage has to be oxidized to nitrite and eventually to the end product of nitrate. However, during the time period from 5 February, 1998 to 5 April, 1998, there was an obvious ammonium buildup in both the outer channel and inner channel of aeration tank #2, and correspondingly loss of nitrification in terms of nitrite and nitrate decreases in the middle channel and clarifier effluent (See Figure 3.11). In Figure 3.11 the plots in the left column are in the sequence of ammonium concentration in the outer channel, and nitrite and TON concentrations in the middle channel of aeration tank #2, while those in the right column are in the sequence of ammonium concentration in the inner channel, and nitrite and TON concentrations in the secondary clarifier effluent. Ammonia probe failure was pointed out and the biggest drop in ammonium concentration was due to the dilution effect of a major rain event.



Figure 3.11 Ammonium Concentration in the Outer (a) and Inner (b) channel, Nitrite Concentrations in the Middle (c) and Secondary clarifier Effluent (d), TON in the Middle (e) and Secondary Clarifier Effluent (f) from 02/05/98 to 04/05/98.

Use of On-line Monitors for Process Control

During heavy rain events, the high flow rate to the plant leads to high concentrations of suspended solids in the secondary clarifier effluent.



Figure 3.12 (a) Influent Flow, (b) MLSS In Inner Channel of Aeration Tank #2, (c) Sludge Blanket Level, and (d) Secondary Effluent Suspended Solids Concentration During One of the Rain Events.

Figure 3.12 shows the wastewater flow, the MLSS concentration in the inner channel of aeration tank No.2, the sludge blanket level in one of the secondary clarifiers, and the suspended solids concentration in the effluent of one of the secondary clarifiers during a major rain event. The figure shows that the influent flow jumped to over 1300m³/h from an average 800m³/h, thus diluting the mixed liquor concentration in the aeration tank. The sludge blanket level increased on this occasion with a time delay due to transport reason, causing heavy sludge loss with the effluent. In cases of this kind the plant adopted alternative strategies. The process control action taken then was that the
influent was directed to the inner channel of the aeration tank, bypassing the first two. The returned activated sludge, however, still went through regular route, that is, from outer channel through middle channel to inner channel. But its rate was much slower than under "normal weather" condition. Influent flow was switched back to the outer channel when it fell to a normal level.

Aspects of process control, including sludge age control, feedback control of return sludge rate and sludge wastage rate, step feed, step sludge, and DO set-point control, will be experimented fully in Chapter 6.

3.4 INTRODUCTION TO SMOOTHING ALGORITHM OF TVP ANALYSIS TOOLBOX

Great efforts were made to make sure that the sampling process was carried out as smooth as possible, as described in the previous sections in this Chapter. However, things still went wrong in spite of these efforts. The ammonium probe failed; the sample line became blocked; a switch was left in the "off" position after manual maintenance of an instrument; a lawn-mower destroyed one of the sample lines by driving over it; and so on. These factors result in gaps, drift, and potential outliers in the retrieved data sets. Their corrupting effects have to be accounted for before the raw data are presented for development and evaluation of the mathematical model in Chapters 4 and 5. The goal is to generate a set of refined, clean data, yet without tainting them in the process. Among the many approaches available for signal pre-processing, our work has employed a package of software constructed around the algorithms of recursive estimation and timeseries analysis (Young, 1984; Young and Benner, 1991), in particular, a smoothing algorithm based on filtering theory, as illustrated through applications in processing hydrological data in Young and Beven (1994) and Young et. al., (1997). This software is based on a Kalman filter, and processes the time-series data first forwards and then backwards in time. Smoothed estimates of the components of the observed signal, typically its long-term trend and seasonal components, are therefore generated at each point in time with the benefit of having processed the entire record of the data. The net effect of this is to give the software the ability to deal successfully with outliers and to interpolate across gaps in the data (Beck and Liu, 1999). Interpolation across the gaps in the model input data stream is especially important for implementing a model, since these are the forces driving the model and must be given for each period of the solution scheme.

3.5 EXAMPLE OF SMOOTHED MODEL INPUT DATA

Using ammonium concentration in the crude sewage as an example, the procedures taken to obtain the trend, seasonal component, and smoothed traces of the raw data with a so-called Dynamic Harmonic Regression (DHR) algorithm in the Time Variable Parameter (TVP) toolbox are summarized as follows.

The basic DHR model contains trend, seasonal, cyclical, and white noise components: i.e.,

$$y_t = T_t + S_t + C_t + e_t$$
 $t = 1, 2, ..., N$ (3.1)

where,

 S_t -Seasonal component; C_t -Cyclical component; T_t -Trend; e_t -White noise. The seasonal component is described as:

$$S_{t} = \sum_{i=1}^{N} \{a_{i,t} \cos(\mathbf{w}_{i}t) + b_{i,t} \sin(\mathbf{w}_{i}t)\}$$
(3.2)

where,

a_{i,t} and b_{i,t}-Stochastic time variable parameters (TVP's);

 ω_i (i=1,2,...,R_s)–Fundamental and harmonic frequencies associated with the seasonality.

The cyclical component is described as:

$$C_{t} = \sum_{i=1} \{ \boldsymbol{a}_{i,t} \cos(f_{i}t) + \boldsymbol{b}_{i,t} \sin(f_{i}t) \}$$
(3.3)

where,

 $\alpha_{i,t}$ and $\beta_{i,t}$ -Stochastic time variable parameters (TVP's);

 f_i (i=1,2,...,R_c)–Fundamental and harmonic frequencies associated with the cyclical component.

In the TVP analysis toolbox, however, the cyclical component is not taken into account. Therefore, the DHR model is simplified into the following format:

In order to specify an appropriate form of equation (3.4) for a specific data stream, it is necessary to identify the number and values of the fundamental and harmonic frequencies associated with the seasonality in equation (3.2). This is accomplished by referring to the empirical spectral properties of y in equation (3.4) using standard methods of spectral analysis (Priestley, 1981). There are many ways to compute the empirical spectrum, but one of the most useful is the Auto Regression (AR) spectrum of the data, with the AR order identified from the *Akaike Information Criterion (AIC)* (Priestley, 1981). The AR spectrum has many advantages, since it is particularly smooth and resolves the model peaks very well.

Shown in Figure 3.13 are the raw data of ammonium concentrations in the crude sewage with gaps.



Figure 3.13 Raw Data of Ammonium Concentration in the Crude Sewage

After processing with the DHR algorithm, the AR spectrum is computed, whereby an autoregression model (of order p, which is to be determined) is used to model the time series plotted in Figure 3.13 in the time domain, i.e., where time t is the basic independent variable. In this fitting process, different AR models are tried, i.e., different values of p are tried and the AIC is then used to identify that value of p giving the best fit. Having found the appropriate AR (p) model, the AR spectrum is computed by changing the independent variable from time (t) to the frequency domain (See Figure 3.14). The curve with smooth peaks is then plotted out in Figure 3.15.



Figure 3.14 AR Spectrum of Order 16 for Ammonium Concentration in the Crude Sewage



Figure 3.15 Raw and Smoothed Data of Ammonium Concentration in the Crude Sewage

From the AR spectrum curve in the frequency domain, the number of fundamental and harmonic components can be specified. Thus the overall structure of the DHR model can be determined, i.e., the choice of R_s in equation (3.2).



Figure 3.16 Raw Data of Ammonium Concentration in the Crude Sewage and its Trend



Figure 3.17 Seasonal Component of Ammonium Concentration in the Crude Sewage

Therefore, the variations of the DHR model's component parameters over time can be estimated, which then generates the trend and diurnal oscillation curves as shown in Figure 3.16 and Figure 3.17.

Either of the two different traces for ammonium concentration in the crude sewage with the gaps filled can possibly be used as model input data stream. One trace is the pure smoothed data produced from the DHR algorithm (the curve plotted as a continuous line in Figure 3.15. The other option is the raw data with its gaps filled with data produced from the DHR algorithm. Table 3.3 illustrates the difference between these two different model input traces using some numbers from the observed sequence. The effect of these two different input traces for ammonium concentration in the crude sewage on the simulation results of ammonium concentration in the inner channel is plotted out in Figure 3.18.

Raw Data with Gaps	Smoothed Data from DHR Analysis	Raw Data with Gaps Filled with Data from DHR Analysis
1.5	1.3	1.5
2	1.8	2
3.1	3.2	3.1
NaN	3.4	3.4
NaN	3.6	3.6
4.3	4.3	4.3
3.7	3.4	3.7
5	4.8	5
4.4	4.1	4.4
4	3.9	4
3.9	3.6	3.9
3.8	3.5	3.8
NaN	3.6	3.6
3.5	3	3.5
4	3.7	4

 Table 3.3 Difference of Two Model Input Traces

In Table 3.3, "NaN" denotes gap in the raw data stream.



Figure 3.18 Comparison of Model Result of Ammonium Concentration in the Inner Channel with Different Model Input Streams

In Figure 3.18 the sequence plotted in circle is the raw data of ammonium concentration in the inner channel corrupted with gaps, and the sequence plotted in diamonds is the smoothed data of ammonium concentration in the inner channel without gaps. The curve plotted in red line is the model simulation result of ammonium concentration in the inner channel with smoothed ammonium concentration in the crude sewage as model input, while the curve plotted in blue line is the model simulation result of ammonium concentration in the inner channel with data produced from DHR analysis as model input. There is little difference between the two model simulation results, but the trace in circles, i.e., raw ammonium concentration in the inner channel data with gaps, provides a better model fitting result. So what can be concluded from this exercise is that using raw

data with gaps filled with data produced from DHR algorithm as model input is a reasonable way to pre-process the raw data streams and it does a better job in allowing us to reconcile the model with the data.

Other than smoothing the raw data set, worthy of mention in passing is the potential of the TVP toolbox in assessing transport and mixing properties of both solute and particulate materials as they pass through the bioreactor and secondary clarifier of the activated sludge process. This can be achieved through identifying Active Mixing Volume (AMV) models (Young and Lees, 1993) using the seasonal components extracted from the same variable, such as ammonium-N, at different sampling locations.

3.6 CONCLUSIONS

This Chapter describes how high quality data was collected in the case study at Athens Wastewater Treatment Facility No.2, and how the resulting data streams are preprocessed without bias by the so-called "TVP analysis toolbox" before being presented for model development in the next Chapter.

The scope and quality of the data retrieved from Athens Wastewater Treatment Facility No.2 with EPCL are found to be better than what have been presented in the key published papers mentioned in Chapter 2 (Bliss et al., 1986; Siegrist et al., 1992; Ducato et al., 1995; Carlsson et al., 1996; Rouleau et al., 1997).

CHAPTER 4

MODEL DEVELOPMENT

4.1 INTRODUCTION

This chapter is organized as follows. The purposes for which the model is to be developed are stated first in section 4.2. Following a brief statement of the overall structure of the model (section 4.3), section 4.4 sets out the key modification of the aerator sub-model with respect to characterization of solute and particulate transport. The clarification and thickening functions of the secondary clarifier model are discussed in section 4.5. A key assumption is that all particulate matter entering the settler from the aerator passes into the thickening zone of the model. Any suspended solids subsequently found in the clarification zone are represented by an upward flux at the interface between the thickening and clarification zones. Discussion of the features of the model is given in section 4.6.

4.2 PURPOSE OF THE MODEL

The model should be able to simulate general behavior of the wastewater treatment plant, i.e., carbonaceous oxidation, nitrification, denitrification, and orthophosphate-P removal. To satisfy the above objectives, it is unavoidable that a large dynamic model will have to be developed. Such a model would apparently be undesirable because of its lack of identifiability (Beck, 1987). However, the model is to be used as a test bed for the development of control strategies. As with any model, the model is inevitably a simplification of highly complex kinetic processes. Despite the uncertainty surrounding many of the mechanisms in the model (Chen, 1993), such a model will be acceptable as long as the sensitivity of the control strategies to such uncertainty can be

established. In other words, if we can find a control strategy that works well irrespective of the gross uncertainty attaching to the model, using such a large model is legitimate.

In the following sections, a dynamic multiple-species model is thus developed in order to meet the above objective. In Chapter 5 the model's parameters are established with reference to field data. "Trial-and-error" calibration is adopted for this case study. In Chapter 6, the calibrated model is subsequently used for the assessment of control strategies to explore the extent to which such a capability could improve process performance.

4.3 OVERALL ARRANGEMENT OF THE MODEL

Generally speaking, the model contains three independent sub-models according to their functions in the whole system. The aerator is where the various biological reactions take place. The thickener is where the biomass is concentrated and then recycled back to the reactor/aerator. The clarification zone is where solid/liquid separation takes place.

The thickening and clarification functions occur within the secondary clarifier. The arrangement of the sub-models is shown in Figure 4.1. The state variables for each sub-model, as well as their inputs, are listed in Table 4.1. Input variables except return sludge flow and wastage sludge flow rates were monitored by the EPCL in a continuous manner at the Athens Wastewater Treatment Facility No.2. Return sludge flow and wastage flow rates were monitored on a daily basis by the plant. All the sub-models are predominantly "mechanistic" in nature. The parameters associated with these mechanisms are summarized in Table 4.2. In Figure 4.1, "XA" represents the aeration tank; "XB1" represents the thickening zone; "XB2" represents the compaction zone; "XC" represents the clarification zone.



Figure 4.1. Schematic Structure of the Activated Sludge Model

Table 4.1 Defin	itions of State	Variables	and Input	Variables
	in the Activat	ed Sludge	Model	

Variable	Definition
XA _{i,j}	State variables in the ith CSTR of the aerator sub-model
	j=1 Ammonium (S _{NH4})
	2 Nitrosomonas (X _{SO})
	3 Nitrite (S_{NO2})
	4 Nitrate (S_{NO3})
	5 Nitrobacter (X _{ba})
	6 Easily biodegradable substance (S _{easy})
	7 Fermentation products (S _{ferment})
	8 Heterotroph (X _H)
	9 Slowly biodegradable substance (X _{slow})
	10 Ortho-phosphate-P (S_{PO4})
	11 Phosphorus-accumulating bacteria (X _{PAO})
	12 Cell internal organic storage substance (X _{PHA})
	13 Poly-P (X_{PP})
	14 Dissolved oxygen (S_{O2})
	15 Mixed Liquor Suspended Solids (X _{MLSS})
$XB1_{i,s}$ and	State variables in the ith CSTR of the thickening zone sub-
$XB2_{i,s}$	model and in the compaction zone sub-model respectively
	s=1 SS (X _{SS})
	2 Ammonium (S_{NH4})
	3 Nitrite (S_{NO2})
	4 Nitrate (S _{NO3})

	5 Easily biodegradable substance (S _{easy})	
	6 Fermentation products (S _{ferment})	
	7. Slowly biodegradable substance (X_{slow})	
	8. Ortho-phosphate-P (S_{PO4})	
XC _{i,s}	State variables in the ith CSTR of the clarification zone sub-	
	model	
	s=1 SS (X _{SS})	
	2 Ammonium (S _{NH4})	
	3. Nitrite (S _{NO2})	
	4. Nitrate (S _{NO3})	
	5. Easily biodegradable substance (S _{easy})	
	6. Fermentation products $(S_{ferment})$	
	7. Slowly biodegradable substance (X_{slow})	
	8. Orthophosphate-P (S_{PO4})	
U_{m}	Measured input variables	
	m=1 Influent flow (Q _I)	
	2 Returned activated sludge flow (Q_R)	
	3 Wasted sludge flow (Q_W)	
	4 Influent ammonium (S_{NH4})	
	5 Influent nitrite (S_{NO2})	
	6 Influent nitrate (S_{NO3})	
	7 Influent total organic carbon (S_{TOC})	
	8 Influent ortho-phosphate-P (S _{PO4})	
	Note: no measurement of input DO concentration is	
	available.	
UC_k	Input variables to the secondary clarifier, k has the same	
	meaning as in XC _{i,s}	
\mathbf{V}_1	Volume of water in the outer channel of the aeration tank	
V_2	Volume of water in the middle channel of the aeration tank	
V_3	Volume of water in the inner channel of the aeration tank	
A _{SC}	Cross sectional area of the secondary clarifier	
Aa	Cross sectional area of the aeration tank	
H _{Ci}	Height of the ith CSTR of the clarification zone	
H_{SBi}	Height of the ith CSTR of the thickening zone other than the	
	top one	
H _{CZ}	Height of the compaction zone	
H _{SBM}	Height of the top CSTR in the thickening zone	
	Note: H_{Ci} , H_{SBi} , and H_{SBM} are time-varying. While Hcz is	
	tixed.	

In this table, CSTR represents Continuously Stirred Tank Reactor.

Symbols	Definition
max_vel	Settling velocity of suspended solids in the
	thickening zone (feet/h)
para	Settling parameter (l/mg)
vel	Settling velocity of suspended solids from top CSTR
	to bottom CSTR in the aeration tank (feet/h)
alpha1	Ratio of $Q_{above}/(Q_I+Q_R)$ in the aeration tank, where
1	O_{above} is the flow through the upper layer CSTRs in
	the aeration tank (%)
alpha2 & c	Parameters in $Q_{\text{Resuspension}} = alpha2^*(Q_1 + Q_R)^{\wedge}c$
ratio1	Fraction of active volume in the outer channel of the
	aeration tank (%)
ratio2	Fraction of active volume in the middle channel of
	the aeration tank (%)
ratio3	Fraction of active volume in the inner channel of the
	aeration tank (%)
beta	Fraction of RAS flow dropping into the 1 st bottom
	CSTR upon entering the aeration tank due to the
	density difference between water and sludge (%)
alpha45	Minimum flux from the thickening zone to the
L	clarification zone (mg/l)
alpha46	Resuspension factor(d/MG)
alpha54	Settling velocity of solids in the clarification zone of
-	the secondary clarifier (feet/h)
K _{O2_SO} , K _{O2_ba}	Saturation coefficients for oxygen (mg O ₂ /l)
$K_{O2}H, K_{O2}PAO$	
a2	Biomass growth factor in the bottom CSTRs (d ⁻¹)
b2	Biomass decay factor in the bottom CSTRs (d^{-1})
b1	Biomass decay factor in the upper CSTRs (d ⁻¹)
Y _{SO}	Yield coefficient for Nitrosomonas (g Nitrosomonas
	formed/g NH ₄ oxidized)
Y _{ba}	Yield coefficient for Nitrobacter (g Nitrobacter
	formed/g NO ₂ oxidized)
Y _h	Yield coefficient for heterotroph (g/g)
Y _{PAO}	Yield coefficient for phosphorus-accumulating
	bacteria (g PAO formed/g PHA)
b _{so}	Decay coefficient for Nitrosomonas (d ⁻¹)
b _{ba}	Decay coefficient for Nitrobacter (d^{-1})
b _h	Decay coefficient for heterotroph (d^{-1})
b _{PAO}	Decay coefficient for phosphorus-accumulating
	bacteria (d ⁻¹)
b _{PHA}	Rate constant for lysis of cell internal organic storage
	substance (d ⁻¹)
b _{PP}	Rate constant for lysis of poly-P (d ⁻¹)

 Table 4.2 Parameters in the Activated Sludge Model

	Rate constant for storage of poly- $P(d^{-1})$
q pp Q _{aba}	Rate constant for storage of cell internal organic
Чрна	storage substance (d^{-1})
:	Phosphorus content of biomass
lpbm V	Saturation constant of ammonium for Nitrosomonas
K _{NH4_SO}	(mg NH /l)
TZ	Saturation constant of nitrite for Nitrobacter (mg
K _{NO2_ba}	NO (1)
17	NO ₂ /1)
K _{PO4_SO}	Saturation constant of phosphorus for Nitrosomonas $(m \in \mathbb{R}^{d})$
	(iiig 1/1) Seturation constant of phoenhoms for Nitrohester
K _{PO4_ba}	Saturation constant of phosphorus for Nitrobacter $(m \sim D^{1})$
K _{easy_h}	Saturation constant of easily biodegradable substance
V	for heterotroph (mg/l)
K _{NO3_h}	Saturation constant of nitrate for neterotroph (mg/l)
K _{NH4_h}	Saturation constant of ammonium for heterotroph
17	
K _{po4_h}	Saturation constant of phosphorus for heterotroph
TZ	$(\operatorname{mg} P/I)$
K _{NO2_h}	Saturation constant of nitrite for heterotroph (mg/l)
f _{xi}	Fraction of inert substance in biomass lysis
K _h	Rate constant of hydrolysis (d ⁻)
K _{x_h}	Saturation constant for slowly biodegradable
	substance
q _{fe}	Maximum fermentation rate (d ⁻¹)
eda_NO ₃	Anoxic hydrolysis reduction factor
K _{fe}	Saturation coefficient for fermentation of easily
	biodegradable substance (mg/l)
K _{NH4_PAO}	Saturation constant of ammonium for phosphorus-
	accumulating bacteria (mg/l)
K _{easy_PAO}	Saturation constant of easily biodegradable substance
	for phosphorus-accumulating bacteria (mg/l)
K _{PO4_PAO}	Saturation constant of phosphorus for phosphorus-
	accumulating bacteria (mg P/l)
K _{NO3_PAO}	Saturation constant of nitrate for phosphorus-
	accumulating bacteria (mg N/l)
K _{A_PAO}	Saturation constant of fermentation product for
	phosphorus-accumulating bacteria (mg/l)
K _{PP}	Saturation constant of poly-P for phosphorus-
	accumulating bacteria (mg PP/mg PAO)
K _{PHA}	Saturation constant of cell internal storage organic
	substance for phosphorus-accumulating bacteria (mg
	PHA/mg PAO)
K _{max}	Maximum ratio of Poly-P/phosphorus-accumulating
	bacteria (mg PP/mg PAO)
K _{ipp}	Inhibition coefficient for poly-P storage (mg PP/mg

	PAO)
const	Oxygen transfer coefficient (d^{-1})
gamma_PO ₄	Poly-P requirement per cell internal storage organic
-	substance stored (g P/g)
gamma_pha	Cell internal storage organic substance requirement
	for poly-P storage (g/g)
ratio_so	Ratio of Nitrosomonas in the returned activated
	sludge (%)
ratio_ba	Ratio of Nitrobacter in the returned activated sludge
	(%)
ratio_h	Ratio of heterotroph in the returned activated sludge
	(%)
ratio_pao	Ratio of phosphorus-accumulating bacteria in the
-	returned activated sludge (%)
ratio_pha	Ratio of cell internal storage organic substance in the
	returned activated sludge (%)
ratio_pp	Ratio of poly-P in the returned activated sludge (%)
ratio_slow	Ratio of slowly biodegradable substance (%)
deni1	Denitrification rate of nitrite in the upper CSTRs of
	all three channels (d^{-1})
deni2	Denitrification rate of nitrate in the upper CSTRs of
	all three channels (d^{-1})
deni3	Denitrification rate of nitrite in the bottom CSTRs of
	all three channels (d^{-1})
deni4	Denitrification rate of nitrate in the bottom CSTRs of
	all three channels (d^{-1})
percentage_easy	Ratio of easily biodegradable substance in TOC (%)
percentage_ferment	Ratio of fermentation products in TOC (%)
percentage_inert	Ratio of inert substance in TOC (%)
miu_so	Maximum growth rate of <i>Nitrosomonas</i> (d ⁻¹)
miu_ba	Maximum growth rate of <i>Nitrobacter</i> (d^{-1})
miu_h	Maximum growth rate of heterotroph (d^{-1})
miu_pao	Maximum growth rate of phosphorus-accumulating
	bacteria (d ⁻¹)

The reactor sub-model is essentially a modified version of the ASM2 model (Henze et al., 1986 and 1995a; Lessard, 1989) especially in terms of using a new way to characterize solute and particulate transport in the activated sludge process. Because of the interactions between the reactor and the secondary clarifiers, particulate substrates and various biomass in the reactor outlet have to be aggregated into a single quantity, mixed liquor suspended solids (MLSS), before they are directed to the secondary

clarifier, and then disaggregated into individual parts, i.e., various biomass fractions, based on their ratios to the total solids (MLSS) before they are transferred back to the aeration tanks. Detailed discussion of each sub-model now follows.

4.4 AERATOR MODEL STRUCTURE

4.4.1 Conceptual Model Structure

The conceptual structure of the biochemical and microbial interactions in the aerator model is shown in Figure 4.2.



Figure 4.2 Conceptual Structure of the Biochemical and Microbial Interactions in the Aerator Model

Here PAO represents phosphorus-accumulating organisms; Poly-P represents polyphosphate, and PHA represents cell internal storage organic substances. Biomass decay refers to decay of *Nitrosomonas*, *Nitrobacter*, heterotroph and phosphorus-accumulating organisms.

The biochemical processes described in the aerator model as shown in Figure 4.2 are:

- □ Aerobic growth of autotrophs (*Nitrosomonas* and *Nitrobacter*)
- Oxic/anoxic growth of heterotrophs
- Oxic/anoxic growth of phosphorus-accumulating bacteria
- Decay of autotrophs, heterotrophs, and phosphorus-accumulating bacteria
- Oxic/anoxic hydrolysis of slowly biodegradable substances
- **G** Fermentation
- **Denitrification from nitrite and nitrate**
- □ Oxic/anoxic storage of poly-P
- □ Storage of cell internal storage organic substances
- □ Lysis of cell internal storage organic substances
- □ Lysis of poly-P.

4.4.2 Solute and Particulate Transport Characterization in the Activated Sludge Process

The mixing of biomass and nutrients in the aeration tank is normally supposed to be perfect in the vertical dimension according to conventional activated sludge theory. However, for an activated sludge system with surface aeration, mechanical mixing may not be ideal in reality, i.e., partial mixing is achieved at the upper part of the tank with dead zones created at the lower part of the tank (Nielsen et al., 1999). In other words, the surface aerators can only mix well a certain amount of mixed liquor (biomass and nutrients), referred to as the effective mixing volume, such that the rest is left largely unmixed at the bottom of the tank, to which significant amounts of particulate matter may also settle. From this perspective the contents in the tank are not uniform in concentration in the vertical direction, so that a two-layer model might better characterize this vertical variation, which is subsequently confirmed by model simulation results in Chapter 5. The first step towards the development of this two-layer model is to decompose the threedimensional tank into a series of CSTRs in the horizontal based on the channel configuration. Due to the vertical variation, the physical tank is then divided into layers, a top and a bottom layer. The volume of each top CSTR is a fraction of the total volume of the channel to which it belongs, which results in parameters ratio1, ratio2, and ratio3 in Table 4.2. These three parameters are respectively fractions of active volumes (%) in the outer, middle, and inner channel of the aeration tank. The flow velocity in the top layer is significantly higher than that in the bottom layer (Hunze et al., 2000), with alpha1 (as seen Table 4.2) being the ratio of flow running through the top CSTRs to the combination of crude sewage and return sludge flows. Crude sewage and returned activated sludge are separately fed to the top layer of the outer channel of the aeration tank, and the effluent from the inner channel of the aeration tank leaves the system from the same top layer. Upon entry into the aeration tank, part of the returned activated sludge, denoted as a ratio (beta in Table 4.2) of the total amount of return sludge flow, is assumed to drop to the bottom layer of the outer channel very quickly, as a function of its high density about 10,000 mg/L. Because of hydraulic effects it is assumed that there exist exchanges between the top layers and the bottom layers immediately below, i.e., upward and downward fluxes. The upward flow is conceptualized as a linear function ('alpha2' in Table 4.2) of the power ('c' in Table 4.2) of the combination of crude sewage and return sludge (Simons et al., 1992). Conceptually, suspended solids in the top layers undergo free gravitational settling into the bottom layers immediately below as well, and the

settling velocity of suspended solids is described in the same manner as that of Dick and Young (1972).

The need for separate solute and particulate transport characterization was not recognized in the first place. In the next Chapter (Chapter 5) the full "story" of how the changes were made will be explained. The parameters mentioned immediately above in this section become crucial in making the model fit the data, although we shall have no hard empirical evidence to assess the "correctness" of the values chosen for them.

The resulting model structure is presented in Figure 4.3.



Figure 4.3 Structure of the 2-Layer Model in the Aeration Tank

With the aeration tank being divided into top and bottom layers, it is essentially assumed that the transport of solute and particulates in the aeration tank now involves more complicated mechanisms, most notably that the mixed liquor suspended solids (biomass) cannot be treated, in effect, as a solute. The mathematical counterpart of the conceptual structure depicted in Figure 4.3 is shown in Figure 4.4. In this figure each block represents a conceptual CSTR unit. For instance, upper(1) refers to the first CSTR element in the top layer.



 $V_{settling} = vel^*exp(-para^*X), \ where \ X \ is the suspended solids \ concentration \\ Q_{d1} = Q_{resus} + Q_{bottom} - beta^*Q_R \\ Q_{d2} = Q_{resus} \\ Q_{d3} = Q_{resus} - Q_{bottom}$

Figure 4.4 Characterization of Solids Transportation in the Aeration Tank

4.4.3 Representative Model Equations

Differential equations for the component mass balances of the top first CSTR element in the aerator are given in Table 4.3, while those equations for the component mass balances of the bottom first CSTR element are given in Table 4.4. For example, $S_{NH4_upper}(1)$ refers to ammonium concentration in the top first CSTR element.

$\Box \quad Ammonium (S_{NH4})$

$$\frac{dS_{NH4_upper}(1)}{dt} = \frac{Q_{I} * S_{NH4}(crudeSewage) + (1 - beta) * Q_{R} * S_{NH4}(thicken3)}{V_{upper1}} + \frac{-(Q_{above} + Q_{d1}) * S_{NH4_upper}(1) + Q_{resus} * S_{NH4_bottom}(1)}{V_{upper1}} \\ - \frac{\mathbf{m}_{SO} * \frac{S_{NH4_upper}(1)}{K_{NH4_SO} + S_{NH4_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)}}{Y_{SO}} \\ - \frac{\frac{S_{PO4_upper}(1)}{K_{po4_SO} + S_{PO4_upper}(1)}}{Y_{SO}} \\ - 0.022 * \mathbf{m}_{H} * \frac{S_{easy_upper}(1)}{K_{easy_H} + S_{easy_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_H} + S_{NH4_upper}(1)} \\ * \frac{\frac{S_{PO4_upper}(1)}{K_{PO4_H} + S_{PO4_upper}(1)}}{K_{H_upper}(1)} * X_{H_upper}(1)$$

$$-0.07 * \mathbf{m}_{PAO} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_pao} + S_{NH4_upper}(1)} \\ * \frac{S_{PO4_upper}(1)}{K_{PO4_pao} + S_{PO4_upper}(1)} * \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1)$$

$$+0.01*K_{H}*\frac{S_{O2_upper}(1)}{K_{O2_H}+S_{O2_upper}(1)}*\frac{S_{slow_upper}(1)/X_{H_upper}(1)}{K_{X_H}+S_{slow_upper}(1)/X_{H_upper}(1)}*X_{H_upper}(1)$$

+0.031*[b_{SO}*X_{SO_upper}(1)+b_{ba}*X_{ba_upper}(1)+b_{pao}*X_{pao_upper}(1)+b_{h}*X_{h_upper}(1)]

$\Box \quad \text{Nitrite} (S_{\text{NO2}})$

$$\begin{aligned} \frac{dS_{NO2_upper}(1)}{dt} &= \frac{Q_I * S_{NO2}(crudeSewage) + (1 - beta) * Q_R * S_{NO2}(thicken3)}{V_{upper1}} + \\ &= \frac{-(Q_{above} + Q_{d1}) * S_{NO2_upper}(1) + Q_{resus} * S_{NO2_bottom}(1)}{V_{upper1}} \\ &+ \frac{\mathbf{m}_{SO} * \frac{S_{NH4_upper}(1)}{K_{NH4_SO} + S_{NH4_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)}}{Y_{SO}} \\ &= \frac{\frac{S_{PO4_upper}(1)}{K_{SO}} * X_{SO_upper}(1)}{Y_{SO}} \\ &- \frac{\mathbf{m}_{ba} * \frac{S_{NO2_upper}(1)}{K_{NO2_ba} + S_{NO2_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_ba} + S_{O2_upper}(1)}}{Y_{ba}} \\ &= \frac{\frac{S_{PO4_upper}(1)}{K_{PO4_ba} + S_{PO4_upper}(1)} * X_{ba_upper}(1)}{Y_{ba}} - deni1 * \frac{S_{NO2_upper}(1)}{S_{NO2_upper}(1) + K_{NO2_b}} \end{aligned}$$

 $\Box \quad \text{Nitrate } (S_{\text{NO3}})$

$$\begin{aligned} \frac{dS_{NO3_upper}(1)}{dt} &= \frac{Q_I * S_{NO3}(crudeSewage) + (1 - beta) * Q_R * S_{NO3}(thicken3)}{V_{upper1}} + \\ \frac{-(Q_{above} + Q_{d1}) * S_{NO3_upper}(1) + Q_{resus} * S_{NO3_bottom}(1)}{V_{upper1}} \\ &+ \frac{\mathbf{m}_{ba} * \frac{S_{O2_upper}(1)}{K_{O2_ba} + S_{O2_upper}(1)} * \frac{S_{NO2_upper}(1)}{K_{NO2_ba} + S_{NO3_upper}(1)}}{Y_{ba}} \\ + \frac{\frac{S_{PO4_upper}(1)}{K_{PO4_ba} + S_{PO4_upper}(1)} * X_{ba_upper}(1)}{Y_{ba}} - deni2 * S_{NO3_upper}(1)} \end{aligned}$$

$\Box \quad Nitrosomonas (X_{SO})$

$$\frac{dX_{SO_upper}(1)}{dt} = \frac{(1-beta) * Q_R * (ratio_so * X_{RAS}) + Q_{resus} * X_{SO_bottom}(1)}{V_{upper1}} + \frac{-(Q_{d1} + Q_{above}) * X_{SO_upper}(1) - A_a * vel * \exp[-para * X_{SO_upper}(1)] * X_{SO_upper}(1)}{V_{upper1}} + \mathbf{m}_{SO} * \frac{S_{NH4_upper}(1)}{K_{NH4_SO} + S_{NH4_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_SO} + S_{PO4_upper}(1)} + \frac{S_{SO_upper}(1)}{K_{SO_upper}(1) - b_{SO} * X_{SO_upper}(1)} + \frac{S_{SO_upper}(1)}{K_{SO_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)} + \frac{S_{PO4_upper}(1)}{K_{PO4_SO} + S_{PO4_upper}(1)} + \frac{S_{SO_upper}(1)}{K_{SO_upper}(1) - b_{SO} * X_{SO_upper}(1)} + \frac{S_{SO_upper}(1)}{K_{SO_upper}(1) - b_{SO} * X_{SO_upper}(1)} + \frac{S_{SO_upper}(1)}{K_{SO_upper}(1)} + \frac{S_{SO_upper}(1)}{$$

 $\Box \quad \text{Nitrobacter} (X_{ba})$

$$\begin{aligned} \frac{dX_{ba_upper}(1)}{dt} &= \frac{(1-beta) * Q_R * (ratio_ba * X_{RAS}) + Q_{resus} * X_{ba_bottom}(1)}{V_{upper1}} + \\ \frac{-(Q_{d1} + Q_{above}) * X_{ba_upper}(1) - A_a * vel * \exp[-para * X_{ba_upper}(1)] * X_{ba_upper}(1)}{V_{upper1}} \\ &+ \mathbf{m}_{ba} * \frac{S_{O2_upper}(1)}{K_{O2_ba} + S_{O2_upper}(1)} * \frac{S_{NO2_upper}(1)}{K_{NO2_ba} + S_{NO2_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_ba} + S_{PO4_upper}(1)} \\ &* X_{ba_upper}(1) - b_{ba} * X_{ba_upper}(1) \end{aligned}$$

□ Easily Biodegradable Substance (S_f)

$$\frac{dS_{easy_upper}(1)}{dt} = \frac{Q_I * S_{easy}(crudeSewage) + (1 - beta) * Q_R * S_{easy}(thicken3)}{V_{upper1}} + \frac{-(Q_{above} + Q_{d1}) * S_{easy_upper}(1) + Q_{resus} * S_{easy_bottom}(1)}{V_{upper1}}$$

$$-\frac{\mathbf{m}_{H} * \frac{S_{easy_upper}(1)}{K_{easy_H} + S_{easy_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)}}{Y_{H}} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} * \frac{S_{O2_upper}(1)}{Y_{H}} * \frac{S_{O2_upper}(1)}{Y_$$

$$\frac{S_{NH4_upper}(1)}{K_{NH4_H} + S_{NH4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_H} + S_{PO4_upper}(1)} * X_{H_upper}(1) + K_{H} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} * \frac{S_{slow_upper}(1) / X_{H_upper}(1)}{K_{X_H} + S_{slow_upper}(1) / X_{H_upper}(1)} * X_{H_upper}(1)$$

□ Fermentation Products (S_A)

$$\begin{aligned} \frac{dS_{A_upper}(1)}{dt} &= \frac{Q_I * S_A (crudeSewage) + (1 - beta) * Q_R * S_A (thicken3)}{V_{upper1}} + \\ \frac{-(Q_{above} + Q_{d1}) * S_{A_upper}(1) + Q_{resus} * S_{A_bottom}(1)}{V_{upper1}} \\ -q_{PHA} * \frac{S_{A_upper}(1)}{K_{A_PAO} + S_{A_upper}(1)} * \frac{X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PP} + X_{PP_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1) \\ + b_{PHA} * X_{PHA_upper}(1) \end{aligned}$$

 $\Box \quad Heterotroph (X_H)$

$$\frac{dX_{H_upper}(1)}{dt} = \frac{(1-beta) * Q_R * (ratio_h * X_{RAS}) + Q_{resus} * X_{H_bottom}(1)}{V_{upper1}} + \frac{-(Q_{d1} + Q_{above}) * X_{H_upper}(1) - A_a * vel * \exp[-para * X_{H_upper}(1)] * X_{H_upper}(1)}{V_{upper1}} + \mathbf{m}_H * \frac{S_{easy_upper}(1)}{K_{easy_H} + S_{easy_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_H} + S_{NH4_upper}(1)}$$

$$*\frac{S_{PO4_upper}(1)}{K_{PO4_H} + S_{PO4_upper}(1)} *X_{H_upper}(1) \\ -b_{H} *X_{H_upper}(1)$$

\Box Slowly Biodegradable Substance (X_S)

$$\frac{dX_{slow_upper}(1)}{dt} = \frac{Q_I * X_{slow}(crudeSewage) + (1 - beta) * Q_R * X_{slow}(thicken3)}{V_{upper1}} + \frac{-(Q_{above} + Q_{d1}) * X_{slow_upper}(1) + Q_{resus} * X_{slow_bottom}(1)}{V_{upper1}} + \frac{-A_a * vel * \exp[-para * X_{slow_upper}(1)] * X_{slow_upper}(1)}{V_{upper1}} + \frac{-K_H * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} * \frac{X_{slow_upper}(1) / X_{H_upper}(1)}{K_{X_H} + X_{slow_upper}(1) / X_{H_upper}(1)} * X_{H_upper}(1) + (1 - f_{X_I}) * [b_{so} * X_{so_upper}(1) + b_{ba} * X_{ba_upper}(1) + b_h * X_{H_upper}(1) + b_{pAO} * X_{PAO_upper}(1)]$$

 $\ \ \, \square \quad Ortho-phosphate-P\left(S_{PO4}\right)$

$$\frac{dS_{PO4_upper}(1)}{dt} = \frac{Q_I * S_{PO4}(crudeSewage) + (1 - beta) * Q_R * S_{PO4}(thicken3)}{V_{upper1}} + \frac{-(Q_{above} + Q_{d1}) * S_{PO4_upper}(1) + Q_{resus} * S_{PO4_bottom}(1)}{V_{upper1}}$$

$$-q_{PP} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_PAO} + S_{PO4_upper}(1)} * X_{PAO_upper}(1)$$

$$* \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{IPP} + K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}$$

$$= i_{PBM} * \mathbf{m}_{PAO} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_pao} + S_{NH4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4} + S_{PO4_upper}(1)}$$

$$* \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PAA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1)$$

$$= i_{PBM} * \mathbf{m}_{SO} * \frac{S_{NH4}(1)}{K_{NH4_SO} + S_{NH4}(1)} * \frac{S_{O2}(1)}{K_{O2_SO} + S_{O2}(1)} * \frac{S_{PO4}(1)}{K_{PO4_SO} + S_{PO4}(1)} * X_{SO_upper}(1)$$

$$= i_{PBM} * \mathbf{m}_{SO} * \frac{S_{NO2}(1)}{K_{NO2_ba} + S_{NO2}(1)} * \frac{S_{O2}(1)}{K_{O2_ba} + S_{O2}(1)} * \frac{S_{PO4}(1)}{K_{PO4_ba} + S_{PO4}(1)} * X_{ba_upper}(1)$$

$$= 0.004 * \mathbf{m}_{H} * \frac{S_{easy_upper}(1)}{K_{easy_uH} + S_{easy_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_uH} + S_{O2_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_uH} + S_{NH4_upper}(1)}$$

$$* \frac{S_{PO4_upper}(1)}{K_{PO4_uPP} + S_{PO4_upper}(1)} * X_{H_upper}(1) + \frac{S_{O2_upper}(1)}{K_{O2_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{NH4_uH} + S_{NH4_upper}(1)}$$

$$+ g_{PO4} * q_{PHA} * \frac{S_{A_upper}(1)}{K_{A_PAO} + S_{A_upper}(1)} * \frac{X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PP} + X_{PP_upper}(1) / X_{PAO_upper}(1)} + b_h * X_{h_upper}(1)$$

$$+ b_{PP} * X_{PP_upper}(1)$$

□ Phosphorus-Accumulating Organisms (X_{PAO})

$$\begin{aligned} \frac{dX_{PAO_upper}(1)}{dt} &= \frac{(1-beta) *Q_{R} * (ratio_pao *X_{RAS}) + Q_{resus} *X_{PAO_bottom}(1)}{V_{upper1}} + \\ &= \frac{-(Q_{d1} + Q_{above}) *X_{PAO_upper}(1) - A_{a} * vel * \exp[-para *X_{PAO_upper}(1)] *X_{PAO_upper}(1)}{K_{O2_PAO} * S_{O2_upper}(1)} * \frac{S_{Vupper1}}{K_{NH4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4} + S_{PO4_upper}(1)} \\ &= \frac{X_{PAA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1) \\ &= \frac{X_{PAA_upper}(1) / X_{PAA_upper}(1)}{K_{PHA_upper}(1) / X_{PAA_upper}(1)} * X_{PAA_upper}(1) \end{aligned}$$

□ Cell Internal Storage Substance (X_{PHA})

$$\frac{dX_{PHA_upper}(1)}{dt} = \frac{(1-beta) *Q_{R} * (ratio_pha * X_{RAS})}{V_{upper1}} + \frac{-(Q_{above} + Q_{d1}) * X_{PHA_upper}(1) + Q_{resus} * X_{PHA_bottom}(1)}{V_{upper1}} + \frac{-A_{a} * vel * \exp[-para * X_{PHA_upper}(1)] * X_{PHA_upper}(1)}{V_{upper1}}$$

$$\begin{split} &- \boldsymbol{g}_{PHA} * q_{PP} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_PAO} + S_{PO4_upper}(1)} \\ &* \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{IPP} + K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)} \\ &* X_{PAO_upper}(1) \end{split}$$

$$-\frac{m_{PAO} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_pao} + S_{NH4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4} + S_{PO4_upper}(1)} *}{Y_{PAO}} * \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{Y_{PAO}} * X_{PAO_upper}(1)} * X_{PAO_upper}(1) + q_{PHA} * \frac{S_{A_upper}(1) / X_{PAO_upper}(1)}{K_{A_PAO} + S_{A_upper}(1)} * \frac{X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PP} + X_{PP_upper}(1) / X_{PAO_upper}(1)} * (1) +$$

$\Box \quad \text{Poly-P}\left(X_{\text{PP}}\right)$

$$\frac{dX_{PP_upper}(1)}{dt} = \frac{(1 - beta) * Q_R * X_R * ratio_pp - (Q_{above} + Q_{d1}) * X_{PP_upper}(1)}{V_{upper1}} + \frac{Q_{resus} * X_{PP_bottom}(1) - A_a * vel * exp[-para * X_{PP_upper}(1)] * X_{PP_upper}(1)}{V_{upper1}} + q_{PP} * \frac{S_{O2_upper}(1)}{S_{O2_upper}(1) + K_{O2_PAO}} * \frac{S_{PO4_upper}(1)}{S_{PO4_upper}(1) + K_{PO4_PAO}} \\ * \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{X_{PHA_upper}(1) / X_{PAO_upper}(1)} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{IPP} + K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)} \\ * X_{PAO_upper}(1) - g_{PO4} * q_{PHA} * \frac{S_{A_upper}(1)}{K_{A_PAO} + S_{A_upper}(1)} * \frac{X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PP} + X_{PP_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1) \\ - b_{PP} * X_{PP_upper}(1) \end{cases}$$

$\square Dissolved Oxygen (S_{O2})$

$$\begin{split} \frac{dS_{O2_upper}(1)}{dt} &= n1*const*[C_{S} - S_{O2_upper}(1)] + \\ \frac{-(Q_{above} + Q_{d1})*S_{O2_upper}(1) + Q_{resus}*S_{O2_bottom}(1)}{V_{upper1}} \\ &+ (1 - \frac{3.43}{Y_{SO}})*\mathbf{m}_{SO}*\frac{S_{NH4_upper}(1)}{K_{NH4_SO} + S_{NH4_upper}(1)}*\frac{S_{O2_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)} \\ &* \frac{S_{PO4_upper}(1)}{K_{PO4_SO} + S_{PO4_upper}(1)}*X_{SO_upper}(1) \\ &+ (1 - \frac{1.14}{Y_{ba}})*\mathbf{m}_{ba}*\frac{S_{O2_upper}(1)}{K_{O2_ba} + S_{O2_upper}(1)}*\frac{S_{NO2_upper}(1)}{K_{NO2_ba} + S_{NO2_upper}(1)} \\ &* \frac{S_{PO4_upper}(1)}{K_{PO4_ba} + S_{PO4_upper}(1)}*X_{ba_upper}(1) \\ &+ (1 - \frac{1}{Y_{H}})*\mathbf{m}_{H}*\frac{S_{easy_upper}(1)}{K_{easy_H} + S_{easy_upper}(1)}*\frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} \\ &* \frac{S_{NH4_upper}(1)}{K_{NH4_H} + S_{NH4_upper}(1)}*\frac{S_{PO4_upper}(1)}{K_{PO4_H} + S_{PO4_upper}(1)}*X_{H_upper}(1) \end{split}$$

$$+ (1 - \frac{1}{Y_{PAO}}) * [\mathbf{m}_{PAO} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH 4_upper}(1)}{K_{NH 4_pao} + S_{NH 4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4} + S_{PO4_upper}(1)} * \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1)]$$

$$+ \frac{M_{PHA} + M_{PHA_upper}(1) / X_{PAO_upper}(1)}{S_{O2_upper}(1) + K_{O2_PAO}} * \frac{S_{PO4_upper}(1)}{S_{PO4_upper}(1) + K_{PO4_upper}(1)} * \frac{X_{PAO_upper}(1)}{K_{PO4_upper}(1) + K_{PO4_upper}(1)}$$

$$+ \frac{M_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA_upper}(1) / X_{PAO_upper}(1)} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PAO_upper}(1) + K_{PHA}} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{PAO_upper}(1) / X_{PAO_upper}(1) + K_{MAX}}$$

Table 4.4 Differential Equations for the Bottom First CSTR Element in the Aerator(all notations are given in Table 4.1 and 4.2)

 $\Box \quad Ammonium (S_{NH4})$

$$\frac{dS_{NH4_bottom}(1)}{dt} = \frac{beta^*Q_R^*S_{NH4}(thicken3) + Q_{d1}^*S_{NH4_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom})^*S_{NH4_bottom}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom})^*S_{NH4_bottom}(1)}{V_{bottom1}} + \frac{0.03^*q_{fe}^*\frac{K_{NO3_H}}{K_{NO3_H} + S_{NO3_bottom}(1)}^*\frac{S_{easy_bottom}(1)}{K_{fe} + S_{easy_bottom}(1)}^*X_{H_bottom}(1)} + \frac{0.01^*K_H^*h_{NO3}^*\frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)}^*\frac{S_{slow_bottom}(1)/X_{H_bottom}(1)}{K_{X_H} + S_{slow_bottom}(1)/X_{H_bottom}(1)} + \frac{0.022^*m_H^*h_{NO3}^*\frac{S_{easy_bottom}(1)}{K_{easy_H} + S_{easy_bottom}(1)}^*\frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)}^*\frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} + \frac{S_{NO3_bottom}(1)}{K_{X_H} + S_{slow_bottom}(1)/X_{H_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{NO3_H} + S_{easy_bottom}(1)}^*\frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} + \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)}^*\frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} + \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{NO3_H} + S_{NO4_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{NH4_H} + S_{NH4_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{H_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{H_bottom}(1)} + \frac{S_{NH4_bottom}(1)}{K_{N} + S_{NH} + S_{N} + S_{$$

$$-0.07 * \mathbf{m}_{PAO} * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{NH4_bottom}(1)}{K_{NH4_pao} + S_{NH4_bottom}(1)} \\ * \frac{S_{PO4_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} * \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PHA} + X_{PHA_bottom}(1) / X_{PAO_bottom}(1)} * X_{pao_bottom}(1) \\ + 0.031 * b_{2} * [X_{SO_bottom}(1) + X_{ba_bottom}(1) + X_{pao_bottom}(1) + X_{h_bottom}(1)]$$

$\Box \quad \text{Nitrite} (S_{\text{NO2}})$

$$\frac{dS_{NO2_bottom}(1)}{dt} = \frac{beta * Q_R * S_{NO2}(thicken3) + Q_{d1} * S_{NO2_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom}) * S_{NO2_bottom}(1)}{V_{bottom1}} - deni3 * S_{NO2_bottom}(1)$$

 $\Box \quad \text{Nitrate } (S_{\text{NO3}})$

$$\begin{aligned} \frac{dS_{NO3_bottom}(1)}{dt} &= \frac{beta * Q_R * S_{NO3}(thicken3) + Q_{d1} * S_{NO3_upper}(1)}{V_{bottomi}} + \\ \frac{-(Q_{resus} + Q_{bottom}) * S_{NO3_bottom}(1)}{V_{bottomi}} \\ &= \frac{1 - Y_H}{2.86 * Y_H} * \mathbf{m}_H * \mathbf{h}_{NO3} * \frac{S_{easy_bottom}(1)}{K_{easy_H} + S_{easy_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &* \frac{S_{NH4_bottom}(1)}{K_{NH4_H} + S_{NH4_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} * X_{H_bottom}(1) \\ &= 0.21 * \mathbf{m}_{PAO} * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{NH4_bottom}(1)}{K_{NH4_pao} + S_{NH4_bottom}(1)} \\ &* \frac{S_{PO4_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} * \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_PAO} + S_{PO4_bottom}(1)} \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_PAO} + S_{PO4_bottom}(1)} \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_PAO} + S_{PO4_bottom}(1)} \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1) / X_{PAO_bottom}(1)}{K_{PO4_PAO} + S_{PO4_bottom}(1)} \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1) / X_{PAO_bottom}(1)}{K_{PAA_bottom}(1) / X_{PAO_bottom}(1)} \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PAA_bottom}(1) \times K_{PAA_bottom}(1)} \\ &* \frac{X_{PAA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PAA_bottom}(1) - deni4 * S_{NO3_bottom}(1)}$$

$\Box \quad Nitrosomonas (X_{SO})$

$$\frac{dX_{SO_bottom}(1)}{dt} = \frac{beta * Q_R * (X_R * ratio_so) + Q_{d1} * X_{so_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom}) * X_{so_bottom}(1) + A_a * vel * \exp[-para * X_{SO_upper}(1)] * X_{SO_upper}(1)}{V_{bottom1}}$$

 $\Box \quad \text{Nitrobacter} (X_{ba})$

$$\frac{dX_{ba_bottom}(1)}{dt} = \frac{beta * Q_R * (X_R * ratio_ba) + Q_{d1} * X_{ba_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom}) * X_{ba_bottom}(1) + A_a * vel * \exp[-para * X_{ba_upper}(1)] * X_{ba_upper}(1)}{V_{bottom1}} - b_2 * X_{ba_bottom}(1)$$

□ Easily Biodegradable Substance (S_f)

$$\frac{dS_{easy_bottom}(1)}{dt} = \frac{beta * Q_R * S_{easy}(thicken3) + Q_{d1} * S_{easy_upper}(1)}{V_{bottom}} + \frac{-(Q_{resus} + Q_{bottom}) * S_{easy_bottom}(1)}{V_{bottom}} + \frac{-(Q_{resus} + Q_{bottom}) * S_{easy_bottom}(1)}{V_{bottom}} + \frac{-(q_{fe} * \frac{K_{NO3_H}}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{S_{easy_bottom}(1)}{K_{fe} + S_{easy_bottom}(1)} * X_{H_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{easy_H} + S_{easy_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{S_{NH4_bottom}(1)}{K_{H_H} + S_{NH4_bottom}(1)} + \frac{S_{PO4_bottom}(1)}{K_{H_bottom}(1)} * X_{H_bottom}(1)}{K_{H_bottom}(1)} + K_{H} * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{S_{slow_bottom}(1) / X_{H_bottom}(1)}{K_{X_H} + S_{slow_bottom}(1) / X_{H_bottom}(1)} * X_{H_bottom}(1)$$

$\Box \quad Fermentation Product (S_A)$

$$\frac{dS_{A_bottom}(1)}{dt} = \frac{beta * Q_R * S_A(thicken3) + Q_{d1} * S_{A_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom}) * S_{A_bottom}(1)}{V_{bottom1}}$$

$$-q_{PHA} * \frac{S_{A_bottom}(1)}{K} + \frac{X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K} + \frac{X_{PAO_bottom}(1) / X_{PAO_bottom}(1)}{K} + \frac{X_{PAO_bottom}(1)}{K} + \frac{X_{PAO_bottom}(1) / X_{PAO_bottom}(1)}{K} + \frac{X_{PAO_bottom}(1) / X_{PAO_bo$$

$$K_{A_pao} + S_{A_bottom}(1) + K_{PP} + K_{PP_bottom}(1) / K_{PAO_bottom}(1) + q_{fe} * \frac{K_{NO3_H}}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{S_{easy_bottom}(1)}{K_{fe} + S_{easy_bottom}(1)} * X_{H_bottom}(1) + b_{PHA} * X_{PHA_bottom}(1)$$

$\Box \quad Heterotroph (X_H)$

$$\frac{dX_{H_bottom}(1)}{dt} = \frac{beta * Q_R * (X_R * ratio_h) + Q_{d1} * X_{H_upper}(1)}{V_{bottom}} + \frac{-(Q_{resus} + Q_{bottom}) * X_{H_bottom}(1) + A_a * vel * \exp[-para * X_{H_upper}(1)] * X_{H_upper}(1)}{V_{bottom}} + \mathbf{m}_H * \mathbf{h}_{NO3} * \frac{S_{easy_bottom}(1)}{K_{easy_H} + S_{easy_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ * \frac{S_{NH4_bottom}(1)}{K_{NH4_H} + S_{NH4_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} * X_{H_bottom}(1) \\ - b_2 * X_{H_bottom}(1)$$

$$\frac{dX_{slow_bottom}(1)}{dt} = \frac{beta * Q_R * X_{slow}(thicken3) + Q_{d1} * X_{slow_upper}(1)}{V_{bottom1}} + \frac{-(Q_{resus} + Q_{bottom}) * S_{slow_bottom}(1) + A_a * vel * \exp[-para * X_{slow_upper}(1)] * X_{slow_upper}(1)}{V_{bottom1}} + \frac{-K_H * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} * \frac{X_{slow_bottom}(1) / X_{H_bottom}(1)}{K_{X_H} + X_{slow_bottom}(1) / X_{H_bottom}(1)} * X_{H_bottom}(1)} + (1 - f_{X_1}) * b_2 * [X_{so_bottom}(1) + X_{ba_bottom}(1) + X_{H_bottom}(1) + X_{H_bottom}(1$$

$\Box \quad Ortho-phosphate-P(S_{PO4})$

$$\begin{aligned} \frac{dS_{PO4_bottom}(1)}{dt} &= \frac{beta^* Q_R^* S_{PO4}(thicken3) + Q_{d1}^* S_{PO4_upper}(1)}{V_{bottoml}} + \\ &= \frac{-(Q_{resus} + Q_{bottom})^* S_{PO4_bottom}(1)}{V_{bottoml}} \\ &+ g_{PO4}^* q_{PHA}^* \frac{S_{A_bottom}(1)}{K_{A_PAO} + S_{A_bottom}(1)}^* \frac{X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K_{PP} + X_{PP_bottom}(1) / X_{PAO_bottom}(1)} * X_{PAO_bottom}(1) \\ &+ 0.01^* q_{fe}^* \frac{K_{NO3_H}}{K_{NO3_H} + S_{NO3_bottom}(1)}^* \frac{S_{easy_bottom}(1)}{K_{fe} + S_{easy_bottom}(1)} * X_{H_bottom}(1) \\ &- q_{PP}^* \mathbf{h}_{NO3}^* \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} * X_{PAO_bottom}(1) \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PHA} + X_{PHA_bottom}(1) / X_{PAO_bottom}(1)} * \frac{K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K_{IPP} + K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)} \\ &- 0.004^* \mathbf{m}_H^* \mathbf{h}_{NO3}^* \frac{S_{easy_bottom}(1)}{K_{easy_H} + S_{easy_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &* \frac{S_{NH4_bottom}(1)}{K_{NH4_H} + S_{NH4_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} \\ &* \frac{S_{NH4_bottom}(1)}{K_{NH4_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} \\ &* \frac{S_{NH4_bottom}(1)}{K_{NH4_bottom}(1)} * \frac{S_{NO3_bottom}(1)}{K_{PO4_H} + S_{PO4_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bottom}(1)} \\ &= i_{PBM}^* \mathbf{m}_{PAO}^* \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_H} + S_{NO3_bott$$

$$= t_{PBM} + \mathbf{m}_{PAO} + \mathbf{n}_{NO3} + \frac{1}{K_{NO3_{PAO}} + S_{NO3_{bottom}}(1)} + \frac{1}{K_{NH4_{pao}} + S_{NH4_{bottom}}(1)} \\ * \frac{S_{PO4_{bottom}}(1)}{K_{PO4_{pao}} + S_{PO4_{bottom}}(1)} * \frac{X_{PHA_{bottom}}(1) / X_{PAO_{bottom}}(1)}{K_{PHA} + X_{PHA_{bottom}}(1) / X_{PAO_{bottom}}(1)} * X_{PAO_{bottom}}(1) \\ + 0.01 * b_{2} * [X_{SO_{bottom}}(1) + X_{ba_{bottom}}(1) + X_{pao_{bottom}}(1) + X_{h_{bottom}}(1)] \\ + b_{PP} * X_{PP_{bottom}}(1)$$

$$\frac{dX_{PAO_bottom}(1)}{dt} = \frac{beta * Q_R * (X_R * ratio_pao) + Q_{d1} * X_{PAO_upper}(1)}{V_{bottoml}} + \frac{-(Q_{resus} + Q_{bottom}) * X_{PAO_bottom}(1) + A_a * vel * \exp[-para * X_{PAO_upper}(1)] * X_{PAO_upper}(1)}{V_{bottoml}} + \mathbf{m}_{PAO} * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{NH4_bottom}(1)}{K_{NH4_pao} + S_{NH4_bottom}(1)} \\ * \frac{S_{PO4_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} * \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PHA} + X_{PHA_bottom}(1) / X_{PAO_bottom}(1)} * X_{pao_bottom}(1) \\ - b_2 * X_{PAO_bottom}(1)$$

□ Cell Internal Storage Substance (X_{PHA})

$$\begin{aligned} \frac{dX_{PHA_bottom}(1)}{dt} &= \frac{beta * Q_R * X_R * ratio_pha + Q_{d1} * X_{PHA_upper}(1)}{V_{bottoml}} + \\ \frac{-(Q_{resus} + Q_{bottom}) * X_{PHA_bottom}(1) + A_a * vel * \exp[-para * X_{PHA_upper}(1)] * X_{PHA_upper}(1)}{V_{bottoml}} \\ &+ q_{PHA} * \frac{S_{A_bottom}(1)}{K_{A_PAO} + S_{A_bottom}(1)} * \frac{X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K_{PP} + X_{PP_bottom}(1) / X_{PAO_bottom}(1)} * X_{PAO_bottom}(1) \\ &- g_{PHA} * q_{PP} \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_PAO} + S_{PO4_bottom}(1)} * X_{PAO_bottom}(1) \\ &* \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{K_{PHA} + X_{PHA_bottom}(1) / X_{PAO_bottom}(1)} * \frac{K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K_{IPP} + K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)} \end{aligned}$$

$\Box \quad Poly-P(X_{PP})$

$$\begin{aligned} \frac{dX_{PP_bottom}(1)}{dt} &= \frac{beta * Q_R * ratio_pp * X_{RAS} + Q_{d1} * X_{PP_upper}(1)}{V_{bottoml}} + \\ \frac{-(Q_{resus} + Q_{bottom}) * X_{PP_bottom}(1) + A_a * vel * \exp[-para * X_{PP_upper}(1)] * X_{PP_upper}(1)}{V_{bottoml}} \\ -g_{PO4} * q_{PHA} * \frac{S_{A_bottom}(1)}{K_{A_PAO} + S_{A_bottom}(1)} * \frac{X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{X_{PP_bottom}(1) / X_{PAO_bottom}(1) + K_{PP}} \\ * X_{PAO_bottom}(1) + q_{PP} * \mathbf{h}_{NO3} * \frac{S_{NO3_bottom}(1)}{K_{NO3_PAO} + S_{NO3_bottom}(1)} * \frac{S_{PO4_bottom}(1)}{K_{PO4_pao} + S_{PO4_bottom}(1)} \\ * \frac{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)}{X_{PHA_bottom}(1) / X_{PAO_bottom}(1)} * \frac{K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)}{K_{IPP} + K_{MAX} - X_{PP_bottom}(1) / X_{PAO_bottom}(1)} \end{aligned}$$

 $\Box \quad \text{Dissolved Oxygen} (S_{O2})$

$$\frac{dS_{O2_bottom}(1)}{dt} = \frac{Q_{d1} * S_{O2_upper}(1) - Q_{resus} * S_{O2_bottom}(1) - Q_{bottom} * S_{O2_bottom}(1)}{V_{bottoml}}$$

Those equations in Table 6.3 and 6.4 include not only features of characterization of solids and nutrient transport in the aeration tank, but also the two-step nitrification process from ammonium to nitrite and then nitrate, and simultaneous nitrification and denitrification in the upper CSTRs. Nitrogen losses from aerated facilities have been observed frequently (Applegate et al., 1980; Drews and Greef, 1973; Rittmann and Langeland, 1985; van Huyssteen et al., 1990; van Munch et al., 1996; Bertanza,1997). As quoted in Chapter 2, O'Neill et al. (1995) reported that simultaneous nitrification and denitrification in the outer lane of the Orbal process took place in the presence of a DO concentration of 1.5 mg/L. The biochemistry of ortho-phosphate-P removal in the current model is based on that is proposed in ASM2.
4.5 SECONDARY CLARIFIER MODEL STRUCTURE

In the Athens Wastewater Treatment Facility case study, only one secondary clarifier was monitored for its dynamic behavior. Therefore, it is assumed that the aeration tank effluent is equally distributed among the three secondary clarifiers, and that equal amounts of wastage sludge are drawn from each individual clarifier. In the secondary clarifier, particulate transport is characterized based on the classical flux theory (Dick & Young, 1972), with the solutes simply being routed through each individual CSTR in the thickening zone, compaction zone, and clarification zone, assuming that no biochemical reactions take place there. As drawn in Figure 4.1, the secondary clarifier is represented by three conceptual zones, i.e., clarification, thickening, and compaction zones. The descriptions for each of these sub-models are given below.

4.5.1 Clarification Sub-Model

The volume of the clarification zone varies due to the time-varying nature of the sludge blanket level in the thickening zone, and is equal to the volume of the secondary clarifier above the sludge blanket. The mixing characteristics of the clarification zone are approximated by 4 identical CSTR elements in the vertical direction of variable volumes by calibration.

4.5.1.1 Characterization of Solute Matter in the Clarification Zone

Seven soluble state variables are quantified in the clarification sub-model. Effluent from the aeration tank is the input to three secondary clarifiers. Their compositions in the influent to the clarification zone are assumed to be equal to their compositions in the effluent from the aerator. They are routed in a simple manner through the CSTR elements as follows (Table 4.5).

Table 4.5 Differential Equations for Solutes Routed through CSTR Elements In the Clarification Sub-Model (all notations are given in Table 4.1 and 4.2)

□ Mth layer, i.e., bottom-most layer which is at the interface with the thickening zone

$$\frac{dXC_{M,j}}{dt} = \frac{(Q_I - Q_W)/3 * (XA_{3,j} - XC_{M,j})/A_s}{H_{CM}}$$

□ 1st layer, i.e., top-most layer

$$\frac{dXC_{1,j}}{dt} = \frac{(Q_I - Q_W)/3 * (XC_{2,j} - XC_{1,j})/A_s}{H_{C1}}$$

In Table 4.5, $XC_{1,j}$ is the concentration of solute j in the clarifier effluent. For suspended solid matter, the mass balances are somewhat different, as now discussed.

4.5.1.2 Characterization of Particulate Matter in the Clarification Zone

Suspended solids in the clarification zone are involved in two kinds of movement. On the one hand, they are subject to settling, which takes place throughout the clarification zone. On the other hand, they are also subject to resuspension (Chen, 1993). Both characteristics are described below.

Sludge Settling Velocity in the Clarification Zone

To be simple, the settling velocity of suspended solids is assumed in this model to be constant in the clarification zone and denoted by parameter α_{54} . This simplification is justified by the fact that sludge concentration in the clarification zone is very low such that sludge particulates can be assumed to settle discretely and thus follow Stoke's Law. Furthermore, since only a certain range of floc sizes can be carried up into the clarification zone and then settle (Mueller et al., 1967; Magara et al., 1976; Otterpohl and Freund, 1992), it is reasonable to assume that the distributions of floc density and diameter for settleable suspended solids in the clarification zone do not show significant variations. As a result, it is acceptable to assume that their average settling velocity is largely invariant with time.

Sludge Resuspension in the Clarification Zone

The feed point for the secondary clarifier is below the surface of the unit. Upon entering the secondary clarifier, the solids stream is assumed to behave as a submerged waterfall passing entirely into the thickening zone. Therefore, any suspended solids subsequently found in the clarification zone are there as a result of resuspension from the thickening zone. Resuspension occurring at the interface layer between the clarification zone and the thickening zone is characterized by a mass flux in the model below denoted by Q_{t} . The resuspension flow can be related to a number of factors, among which hydraulic shocks are the most important (Zhou and McCorquodale, 1992; Chen, 1993). The resuspension flow is quantified as follows (Chen, 1993):

 $Q_{tt} = a_{45} + a_{46} * Q_{in} * X_{MLSS}$

where,

 α_{45} -Minimum suspended solids flux. In other words, there is always some upward movement of particles into the clarification zone;

 α_{46} -Hydraulic effect factor;

X_{MLSS}–MLSS from inner channel of the aeration tank.

These two components constitute a form of Pflanz empirical expression for relating the transfer flux into the clarifier to both the flow rate and suspended solids concentration of the influent to the secondary clarifier (Pflanz, 1969; Takacs, 1986).

Given the flux Q_{tt} , the mass balance equation of suspended solids in the bottommost layer of the clarification zone is then expressed as follows:

$$\frac{dXC_{M,1}}{dt} = \frac{Q_{tt} + a_{54} * (XC_{M-1,1} - XC_{M,1}) - (Q_I - Q_W) / 3 * XC_{M,1} / A_s}{H_{CM}}$$

where the last term on the numerator describes the upflow of particulate matter from the top CSTR of the thickening zone to the bottom CSTR of the clarification zone (Takacs et al., 1991).

4.5.2 Thickening Sub-Model

4.5.2.1 Flux Theory

The thickening model is based on the conventional flux theory (Dick & Young, 1972). According to this theory, there exists a limiting flux of solids that the thickening zone can take for each specific operating condition, including return sludge flow, wastage sludge flow, and MLSS concentration in the aeration tank effluent. The total flux of solids, G_t , in a continuous thickening zone is a function of the solids flux due to gravitational settling of particulate matter (G_g), and of the solids flux due to the downward movement of solids (G_u) caused by withdrawal of sludge at a flow rate V_u (the underflow), as described in the following equation:

 $G_t = G_g + G_u = C_i * (V_s + V_u)$

where,

 C_i -Concentration of sludge in the thickening zone; V_s-Gravity settling velocity of sludge in the thickening zone; V_u=(Q_R+Q_W)/(3*A_S) – Downward bulk velocity of the sludge. Any applied flux exceeding the limiting flux will result in a continual increase of the sludge blanket as long as the applied flux is higher than the limiting flux. The sludge blanket concentration will then be equal to the limiting concentration corresponding to the limiting flux associated with the given operating condition (Dick & Young, 1972).

4.5.2.2 Representative Equations

The thickening zone is represented by 2 CSTR elements with variable volume by calibration, where variation is due to the increase or decrease of the sludge blanket height. When there is no blanket, the compression layer is fed directly by the aeration tank effluent. The concentration of suspended solids in the upper-most layer of the thickening zone is never allowed to exceed the limiting concentration defined by the flux theory for the given operating conditions. But once at its limiting concentration, the blanket volume is allowed to expand with overloading or to contract with underloading. The compaction zone, on the other hand, comprises just one CSTR element with fixed volume by calibration as shown in Figure 5.1.

Eight state variables are simulated in the thickening zone model: seven for the solute state variables and one for the particulate state variables. The seven solute state variables are routed through the thickening zone and the compaction zone as conservative, non-reacting substances. The feed point for the secondary clarifier is located near the surface of the unit, and the solids stream is assumed to behave as a submerged waterfall entering the sludge blanket at height H_{SBM} directly.

For the uppermost layer of the thickening zone, the suspended solids concentration is described as:

$$\frac{dXB1_{i,j}}{dt} = \frac{\left[(Q_I + Q_R)/3\right]^* X_{in}/A_s + a_{54}^* XC_{1,j} - Q_{tt} - XB1_{i,j}^* (V_{s,i} + V_u)}{H_{SBM}}$$

where,

 X_{in} -Suspended solids concentration from the aerator to the secondary clarifier; XC_{1,i}-Suspended solids concentration at the bottom-most CSTR of the clarification zone.

After each iteration of the calculation, the resulting concentration of the blanket will be checked against the flux theory. If a violation has occurred, such as exceeding the limiting concentration, the concentration of the blanket will be reset to the limiting value with the flux imbalance being provided for by an increase in the blanket height. For the layer beneath the uppermost layer of the thickening zone:

$$\frac{dXB1_{i-1,j}}{dt} = \frac{XB1_{i,j} * (V_{s,i} + V_u) - XB1_{i-1,j} * (V_{s,i-1} + V_u)}{H_{SBi}}$$

For the compaction zone:

$$\frac{dXB2_{RAS}}{dt} = \frac{XBl_{1,j} * (V_{s,1} + V_u) - XB2_{RAS} * V_u}{H_{CZ}}$$

The volume of the compaction zone is invariant. The settling velocity of the suspended solids in the thickening zone is described by an exponential function as follows (Dick and Young, 1972):

where,

max_vel and para –Constants;

X–Suspended solids concentration in each individual CSTR.

4.6 DISCUSSIONS

4.6.1 Assumptions and Restrictions Associated With the Model

As with every model, the proposed model is certainly a compromise. Inevitably it has some assumptions and restrictions, which are summarized as follows:

- During the simulation, it is assumed that there is no evidence of any abnormal behavior, such as sludge bulking taking place;
- □ The system operated at a constant pH level;
- The concentration of suspended solids is completely uniform within any vertical layer within the thickening zone;
- □ The mass flux into the top CSTR of the thickening zone cannot exceed the mass flux that element is capable of taking, nor can it exceed the mass flux which the element immediately below it is capable of taking;
- □ The gravitational settling velocity of solids in the thickening zone and the compaction zone is a function of the suspended solids concentration;
- □ No significant biological reactions take place in the secondary clarifier.

4.6.2 Implementation of the Model

A computer program written in Matlab was developed for use on a PC. The simulation results from the program were then analyzed within the environment of Matlab in terms of checking the model results against the real data.

4.7 CONCLUSIONS

A dynamic multiple species model has been developed and presented in this Chapter for simulating nutrient removal processes in the activated sludge process. Its major innovations compared to ASM2 are:

□ Characterization of solute and particulate transport in the activated sludge process;

- Differentiation of functions of *Nitrosomonas* and *Nitrobacter* in the nitrification process;
- □ Characterization of simultaneous nitrification and denitrification in the aerated environment.

The proposed model will be used to improve understanding of activated sludge process performance under dynamic conditions. A detailed examination of the model, i.e., calibration by trial and error, validation and sensitivity analysis, follows in Chapter 5.

CHAPTER 5

MODEL CALIBRATION, VALIDATION, AND SENSITIVITY ANALYSIS

5.1 INTRODUCTION

A comprehensive dynamic model has been developed in the preceding chapter. Now it is time to assess the validity of the model's structure, in particular in respect of its novel features. The calibration method adopted hereby is trial-and-error. However, in identifying such a complicated model with so many parameters the harsh reality is that it is hard to retrieve a uniquely best set of parameters when checked against real world data. In section 5.2 calibration results are presented, and possible reasons for mismatches between model results and data are discussed. Thereafter, another set of data is used for model validation in section 5.3. Sensitivity analysis is finally carried out at the end of this Chapter in section 5.4.

5.2 MODEL CALIBRATION RESULTS AND DISCUSSIONS

The field data used for this calibration exercise were part of the data retrieved from the comprehensive sampling campaign from the Athens Wastewater Treatment Facility No.2 case study illustrated in Chapter 3. It is some 19 days long, from March 12, 1998 to March 31, 1998.

Shown below in Figure 5.1 is the influent flow data for the calibration period, averaging about 4 MGD. On the 6^{th} day there was a moderate rain event. Unfortunately, control actions taken by the plant during the event, such as step-feed or step-sludge, were not recorded. Figure 5.2 plots out the return sludge rate during this 19-day period. These data were recorded roughly on a daily basis (missing for some days), and it is therefore assumed in the model that return sludge rate remains constant over a 24h period (straight

line in Figure 5.2).



Figure 5.1 Influent Flow for Model Calibration



Figure 5.2 Return Sludge Flow for Model Calibration

The same applies to sludge wastage rate. Sludge processing personnel in the treatment plant worked only 4 days a week; sludge wastage was stopped during their off days.

Sludge wastage rate was never above 0.22 MGD due to the wastage capacity. DO concentration in the outer channel of the aeration tank was measured by treatment plant personnel on a daily basis, while those in the middle and inner channels were continuously measured by the out-board DO probes of EPCL (Table 3.1). It is apparent that DO concentration in the middle channel was consistently lower relative to those in the other two channels. The average temperature of the sewage for the 19 days was around 16°C. However, temperature for the last four days or so (day 14 onwards) was about 5°C higher. PH value of the sewage stayed relatively stable at about 6.5. As cited in Chapter 2, Wild et al. (1971) found that nitrification rate decreases below pH of 8.0 until it is completely inhibited at pH below 5.

5.2.1 Nitrification and Denitrification

The calibration exercise started with nitrification, the reason being that this is generally believed to be the most well defined process, and thus should be a relatively "simple" process to identify. In order to illustrate how model results were improved by progressively changing model structures, ammonium concentration in the outer channel of the aeration tank is therefore selected as the focus of discussion in this section. The outer channel is closer to the feeding points of crude sewage and return sludge compared to middle and inner channels, and better simulation results are expected there. It is also believed that description of how fitting for ammonium concentration in the outer channel was achieved, step by step, should be able to give a good account of how the 2-layer model come to be proposed.

At the beginning of the simulation exercise, nitrification was simulated as a onestep process, from ammonium directly to nitrate, based on the configuration in ASM2 (Henze et al., 1995a). Under this configuration, nitrifiers are treated as one entity



Figure 5.3 Fitting of Ammonium Concentration (mg/L) in the Outer Channel with Nitrifiers Simulated as One Entity

The line in dots is the raw data, and the continuous line is the simulation result. The mismatch is quite obvious, especially from the beginning to about day 8 and then from day 14 onwards. The curve consistently underestimates ammonium concentration until about day 8 and then overestimates afterwards. The mismatch from day 14 onwards can be corrected somewhat by taking higher temperature into account, because growth rate of the nitrifying bacteria increases considerably with temperature over the range of 8-30°C (Gray, 1990). But what was frustrating is that if it was managed to increase ammonium concentrations before day 8 to the level of the raw data by changing the values of some parameters, say, maximum growth rate of nitrifiers, the overestimation after day 8 became even more obvious. It seems that the nitrifiers perform differently after the rain event. After the rain event they did not consume as much ammonium as before the rain event, probably due to the hydraulic shock. It seemed that the model structure had to be

adjusted somehow to relieve this predicament. It was then realized that the mismatch up to day 8 might be corrected by associating only *Nitrosomonas* with ammonium removal rather than the combination of *Nitrosomonas* and *Nitrobacter*. The reason is that even though both nitrifiers play important roles in the nitrification process, they in fact oxidize different substrates, i.e., *Nitrosomonas* oxidizes ammonium to nitrite, while *Nitrobacter* subsequently oxidizes the intermediate products to nitrate, and therefore they cannot be simply combined as one entity.

Efforts were then made to simulate nitrification as a two-step process instead of a simple one-step process. Essentially, nitrifiers, previously assumed as an entity, were split up into two entities, i.e., *Nitrosomonas* and *Nitrobacter*. Therefore one more state variable, nitrite, is introduced into the model as an intermediate product. The corresponding model result is shown in Figure 5.4.



Figure 5.4 Fitting of Ammonium Concentration (mg/L) in the Outer Channel with Nitrifiers Simulated as Two Entities

Clearly, the new model structure does a better job in matching the data before day 8 compared to Figure 5.3, even though not equally well after day 8. Undeniably, this has been a significant improvement. Looking closer at the mismatch after day 8 in Figure 5.4,

one would guess that probably after the rain event *Nitrosomonas* could not grow fast enough to assimilate the ammonium such that the model overestimates the residual ammonium. Unobserved *Nitrosomonas* concentration at the same location does not confirm this speculation (Figure 5.5). Even though the concentration of *Nitrosomonas* dropped during the rain event, it managed to recover afterwards. If *Nitrosomonas* is grown even faster than it is in Figure 5.5, the mismatch after day 8 might be corrected, but the fit before day 8 will definitely be corrupted. Another guess for the mismatch after day 8 is that even though *Nitrosomonas* were still retained in the aeration tank after the rain event, they simply resided somewhere there without converting much ammonium. In other words, solutes and particulates might travel through different routes in the aeration tank. During the storm event, solutes pass through the top of the tank very quickly, while biomass resides elsewhere within the tank such that little contact is made between solutes and biomass.



Figure 5.5 *Nitrosomonas* Concentration (mg/L) in the Outer Channel During the 19-day Period

It appears that a 1-layer tanks-in-series model might be an oversimplification of the reality, and the model structure needs to be further adjusted in order to improve the model results. Attention was then directed to characterization of mixed liquor suspended solids (MLSS) transport in the aeration tank. At this point a two-layer model was built up with 3 CSTRs in series in both the top and bottom layers. In other words, each channel in the aeration tank is represented by a CSTR on the top of another one at the bottom. Volume of CSTR on the top layer represents the effective mixing volume. As a result of this new 2-layer configuration, exchange flows (upward and downward flows) between CSTRs on the top and the ones immediately below, and free settling of suspended solids from top CSTRs to bottom CSTRs, need to be accounted for. The upward flow is considered a linear function of $(Q_R+Q_W)^{-1}$ based on previous investigations on suspended solids (Simons et al., 1992), where QR and QW are return sludge rate and wastage flow rate, and c is a constant. Downward movement flows can therefore be calculated based on mass balances around each individual CSTR on the bottom layer. Instinctively, flow through the top CSTRs (effective mixing volume) should be much higher than that through the bottom CSTRs (ineffective mixing volume), and the combination of which should amount to the flow leaving the aeration tank, i.e., crude sewage flow plus return sludge flow. Another assumption made was that due to the high density nature of return sludge, it may well be that a small part of the return sludge settles immediately to the bottom CSTR upon entering the aeration tank, which was backed up by the simulation results (Section 5.5.2.1). After exhaustive "trial-and-error" model calibration, it turns out that the free settling velocity of suspended solids from top CSTRs to bottom CSTRs, denoted as 'vel' in Table 4.2, is zero, which implies that mixing in the effective mixing zone is so good that no free settling of suspended solids occurs (Section 5.5.2.4). Figure 5.6 below shows fitting of MLSS in the inner channel of the aeration tank

against real data based on the 2-layer configuration. Generally speaking, the result is very good except obvious mismatches between day 6 and 7 and around day 11.



Figure 5.6 Fitting of MLSS in the Inner Channel During the 19-day Period



Figure 5.7 MLSS in the Upper and Bottom Middle Channel

Intuitively, MLSS should be denser in the bottom CSTRs than in the relevant top CSTRs, and this is confirmed by the model results as well. For example, MLSS concentration in the upper middle CSTR is higher than that in bottom middle CSTR, as seen in Figure 5.7.

Based on this 2-layer structure, solute transport was characterized. Figure 5.8 shows the new model results in comparison with those shown in Figure 5.3 and Figure 5.4. Improvements are fairly evident. The model tracks the data reasonably well until shortly after day 7. From day 10 to day 14 the model underestimates observed conditions. After that the model overestimates them. One explanation is that some parameters considered constant are in reality variable, such as effective mixing volumes due to shifting and on and off controls of the aerators. Clearly the 2-layer model further improves the model results after day 8 without degrading the fitting before day 8. The step-by-step process described so far embodies a significant success in reaching a good model structure to properly describe the nitrification dynamics in the aeration tank.



Figure 5.8 Comparison of Ammonium Concentration in the Outer Channel with Different Model Structures

Comparison of model results for ammonium concentration in the inner channel with different model structures is shown in Figure 5.9. These results are quite encouraging as well.



Figure 5.9 Comparison of Ammonium Concentration in the Inner Channel with Different Model Structures

Fittings of nitrite and nitrate concentrations in the middle channel and secondary clarifier effluent are shown in Figure 5.10.



Figure 5.10 Fittings of Nitrite and Nitrate Concentrations in the Middle channel of the Aeration Tank and in the Secondary Clarifier Effluent

As seen in the upper two figures in Figure 5.10, nitrite data stays at a fairly low level and the model results largely catch the level. However, in the bottom two figures the model overestimates nitrate in the middle channel after about day 8. In order to correct this mismatch, different denitrification mechanisms of nitrate were tried including introducing the actions of heterotrophic bacteria capable of denitrification in the upper CSTRs other than in the bottom CSTRs, because according to Meiberg (1980) denitrification can occur in the presence of oxygen. But unfortunately the mismatch became more evident with this effort. So the results in Figure 5.10 are about the best achievable at present. Literature show that denitrification is not only affected by oxygen and nitrate concentrations, intermediate product concentrations, such as NO and N₂O,

have impacts on denitrification performance as well (Skrinde and Bhagat, 1982; Snyder et al., 1987), which are not incorporated in the model.



(a) Nitrosomonas Concentration in the upper and lower CSTRs of the middle channel



CSTRs of the middle channel

Figure 5.11 Nitrifiers Profiles in the Upper and Lower CSTRs of the Middle Channel

The two plots in Figure 5.11 show *Nitrosomonas* and *Nitrobacter* concentrations in the upper and lower CSTRs of the middle channel. Nitrifier concentrations in the bottom CSTRs are higher than those in the upper CSTRs. The benefits of introducing the 2-layer model will be described again later in section 5.2.4 in terms of improvement in matching the behavior of the sludge blanket height.

5.2.2 Carbonaceous Substrate Oxidation

Shown in Figure 5.12 are the total organic carbon (TOC) concentrations in the upper CSTR of the outer and inner channels.



Figure 5.12 Fittings of TOC Concentrations in the Upper CSTRs of the Outer and Inner Channels

The mismatches of TOC from day 14 to about day 18 in the outer channel and mismatch of ammonium during the same period of time (Figure 5.8) can be correlated with a slight underestimation of DO in the outer channel during the same period of time (Figure 5.15). The reason is that due to the underestimation of DO smaller amounts of easily biodegradable substrates and ammonium are taken up by the biomass. Underestimation of TOC in the inner channel between day 6 and day 7 could be

attributed to overestimation of DO during the same period of time. Partitioning of the influent TOC into various constituents, easily biodegradable substances, fermentation products, and inert substances, could be another source of error, especially if these relative ratios are considered constants in the model (Chen, 1993) as is the case here. Despite these limitation, the model by and large catches the ups and downs of the TOC data.

Figure 5.13 shows heterotroph concentrations in the upper and bottom CSTRs of the outer and inner channels. Figure 5.14 shows the fermentation product concentration in the upper and bottom CSTRs. Figure 5.15 shows the simulation results of DO concentrations in the outer and inner channels.



Figure 5.13 Heterotroph Concentrations in the Upper and Bottom CSTRs of the Outer and Inner Channels



Figure 5.14 Fermentation Product Concentration in the Upper and Bottom CSTRs of the Outer and Inner Channels



Figure 5.15 Fittings of DO Concentrations in the Outer and Inner Channel

As seen in Figure 5.15, the model consistently overestimates DO in the outer channel from day 4 to about day 14, while it underestimates DO in the inner channel from about day 9 until about day 15. The possible reasons for the mismatches are as follows. Firstly, an incoming dissolved oxygen concentration of zero is assumed in the model, which is not the case in reality (Bocken et al., 1989), especially during higher crude sewage flows. Secondly, the oxygen transfer is not only a function of the number of active aerators, but also several other variables including temperature, wastewater characteristics (Casey and Karmo, 1974), and turbulence (Horan, 1989). Since there were no DO concentration measurements for the last four days, it is assumed that DO concentrations stay constant for the last four days, which might have contributed to the mismatch from day 14 onwards for DO in the outer channel.

5.2.3 Phosphorus

Biological phosphorus removal is a new but quite complex process. Introduction of a very detailed mechanistic model for the processes responsible for biological phosphorus removal is, however, premature. Despite extensive research efforts, the behavior and physiology of PAOs in the biological phosphate removal processes have not yet been fully understood, as discussed in Chapter 2. The IAWQ Task Group acknowledged the simplicity in the current model, and realized that it is the base for further development (Henze et al., 1999).

Shown in Figure 5.16 are the simulation results of phosphorus concentrations in the middle channel and secondary clarifier effluent, which is admittedly poor. There is an obvious surge of phosphorus concentration in data from day 4 to day 8, which is missed by the model. It correlates with the small surge from the rain event and very low DO in the inner channel as seen in Figure 5.15. During the calibration process, if the surge is better simulated, results for the rest part became much worse than what is shown in

Figure 5.16. Figure 5.17 shows the phosphorus-accumulating bacteria concentration in the upper and bottom CSTR of the middle channel.



Figure 5.16 Fittings of Ortho-phosphate-P Concentration in the Middle Channel and the Secondary Clarifier



Figure 5.17 Phosphorus-Accumulating Bacteria in the Upper and Bottom CSTRs of the Middle Channel

According to plant operation records, no step sludge/feeding control actions were taken around rain event on day 6, which is suspicious. But the surge in data suggests that large amount of phosphorus might have been released due to low DO concentration caused by under-aerated sludge. In order to interpret the data, various control alternatives were experimented, albeit speculative. One alternative, feeding all the influent into the inner channel while leaving the sludge in the outer channel, achieves better result in simulating the surge (Figure 5.18). Unfortunately, by doing so simulation results for other monitored variables, such as TOC, are seriously deteriorated.



Figure 5.18 Ortho-Phosphate-P Concentration (mg/L) in the Middle Channel and Secondary Clarifier Effluent

Both poly-phosphorus and PHA occur only associated with phosphorusaccumulating bacteria, and are so-called "cell internal stored materials". In ASM2 (Henze et al., 1995a) poly-phosphorus and PHA are not included in the mass of phosphorusaccumulating bacteria. Calibration results show that concentrations of these two cell internal stored materials stay at zero during the whole simulation window. Any further calibration would cause their concentrations in the bottom CSTRs lower that those in the upper CSTRs, which is impossible. Model calibration results essentially suggest that biological phosphorus removal in terms of ortho-phosphate-P uptake and release did not actually take place in the aeration tank with the presence of the phosphorus-accumulating bacteria, even though carbon removal still takes place in the system as identified in Section 5.2.2. The phenomenon of anaerobic substrate removal without phosphorus release was first reported by Fukase et al. (1985). Cech and Hartman (1990, 1993) reported that when glucose was fed to an anaerobic-aerobic system, some organisms grew significantly which were able to induce breakdown of the phosphorus removal process. They termed the organism "G-bacterium"; however, the microbiological aspects of the organism are not well understood (Matsuo, 1994).

The phosphorus results obtained are certainly far from satisfactory. However, they pinpoint the need for more research and thus result in the development of enhanced models based upon sounder principles. Several limitations associated with current orthophosphate-P removal models might have contributed to the mismatches. For example, it is assumed in ASM2 that the PAOs can grow aerobically on stored PHA only, not on fermentation products directly; and PHAs represent all the carbon storage materials in PAO cells in the present models, although glycogen or carbohydrate has been proposed as another carbon storage material (Mino et al., 1987; Satoh et al., 1992).

5.2.4 MLSS and Sludge Blanket Height

Shown in Figure 5.19 is the result for the sludge blanket level (above tank base) in one of the secondary clarifiers based on the 1-layer model configuration. Obviously the model consistently overestimates the sludge blanket level, indicating that more MLSS are

transferred to the secondary clarifier than actually was the case. However, by implementing the 2-layer model, better results were obtained, as seen in Figure 5.20.



Figure 5.19 Fittings of Sludge Blanket Level with 1-layer model



Figure 5.20 Fittings of Sludge Blanket Level with 2-layer model

Correspondingly, MLSS simulation results (Figure 5.21) show that by implementing 2-layer model, the model detects more MLSS retained in the aeration tank, which therefore decreases the level of sludge blanket in the secondary clarifier (Figure 5.20).



Figure 5.21 Comparison of MLSS Concentrations in the Inner Channel with Different Model Structures



Figure 5.22 Fitting of MLSS in the Aeration Tank Outlet

The figure above (Figure 5.22) shows MLSS simulation results in the inner channel of the aeration tank, with the solids out of the aeration tank being routed through the secondary clarifier. In other words, in the model concentration of return sludge (RAS) into the aeration tank is the simulation result of that from the compaction zone of the secondary clarifier, rather than real data. The 2-layer model does a good job getting the phasing right and also the amplitude correct except for the tail end of the record. Since the treatment plant did not keep regular detailed records of the sludge wastage, including wastage sludge rate, sludge wastage rates for some days, including the last four days or so in this simulation, have to be assumed zero, which might have been higher than that in actual operation.

5.2.5 Returned Activated Sludge

Shown in Figure 5.23 is return sludge concentration from bottom of one clarifier. The model result largely matches the level. The overestimation at the tail end might be related to the unavailability of wastage flow rate data as illustrated in Section 5.2.4 as well. Returned activated sludge (RAS) concentration data were provided by the plant. No data are available for several days. Under this situation, it is assumed that RAS data for the missing days are the same as those days immediately before. RAS samples are taken before the sludge being fed into the aeration tank. So this is essentially a mixture of return sludge pumped from the bottom of all three secondary clarifiers. Since operations of all three secondary clarifiers are not exactly the same, RAS measurement by the plant might be a biased representation of the actual RAS concentration pumped from the bottom of the secondary clarifier being monitored and simulated. Also for simplicity, the flux theory was used to simulate the secondary clarifier behavior. It is a very classic theory, which has been widely used, but has some limitations (Morris et al., 1989).



Figure 5.23 Fitting of RAS Concentration in One Secondary Clarifier

The corresponding suspended solid concentration in the secondary clarifier effluent is shown in 5.24. Since measurements of effluent suspended solid concentrations are in a different unit (FTU) other than mg/L generated by the model, unfortunately no comparison can be made between data and model result.



Figure 5.24 SS Concentration in One Secondary Clarifier Effluent

5.3 MODEL VALIDATION

A second set of data, other than the one used for model calibration, will now be utilized for model validation. These data cover the time span from February 13, 1998 to March 2, 1998, some 18 days in length. There are major rain events involved in this data set, ones that are much heavier than that seen in the calibration data. The purpose is to test how robust and successful the model is when subjected to substantial external disturbances.



Figure 5.25 Influent Flow for Model Validation

Shown above in Figure 5.25 is the influent flow data for model validation use, averaging 6-7 MGD with the maximum at about 9 MGD. On the 4th, 5th, 10th, 11th and 15th day there were rain events. Sometime on the 4th day, influent was fed into the inner channel of the aeration tank, and return sludge was still directed into the outer channel of the aeration tank being simulated. No control actions were recorded for other rain events in actual operation.



Figure 5.26 Return Sludge Flow for Model Validation



Figure 5.27 Wastage Sludge Flow for Model Validation

Figure 5.26 and Figure 5.27 are the return sludge flow rate and wastage sludge flow rate respectively. Again some of these flow rates data were missing in the treatment plant records. So, if the return sludge flow data is missing, it is assumed to be the same as

the day immediately before. If it is sludge wastage flow data, it is assumed to be zero. And it is assumed that return sludge rate and sludge wastage sludge rate remain constant over a 24h period.

Relatively speaking, DO concentrations were lower in the middle and inner channels compared to that in the outer channel. The average sewage temperature was about 15.2°C, and pH of the sewage stayed at about 6.3.

5.3.1 MLSS

Figure 5.28 shows the model validation result for MLSS concentration in the inner channel of the aerator. Figure 5.29 shows the MLSS concentration in the upper and bottom CSTRs of the inner channel. Clearly the 2-layer model simulates the MLSS concentration very well, even when the rain events took place with full routing of MLSS through the clarifier.



Figure 5.28 Model Validation Result of MLSS Concentration in the Upper Inner Channel



Figure 5.29 MLSS Concentration in the Upper and Bottom CSTRs of the Inner Channel

The underestimation at the front end is probably due to the time-varying feature of parameter 'beta' (Table 4.2), which is nevertheless assumed constant in the model. This specific parameter is the fraction of RAS flow dropping into the bottom 1st CSTR upon entering the aeration tank due to its high density. Its value should be changing with the number of active aerators in the outer channel, i.e., the more aerators on, the smaller its value. According to the plant operation records, there were 3 aerators on at the starting point of the calibration window, while there were 5 operating aerators at the starting point of the validation window. An alternative smaller value of 'beta' was tried, and the mismatch was minimized.

According to plant records, during certain time period between day 4 and day 5 half of the influent flow was fed directly into the inner channel of the aeration tank being simulated, with the other half directed to the unused aeration tank. In actual operation the flow fed into the inner channel might have been less than half, which could have caused the sharp drop just before day 4 in the simulation result (Figure 5.28). An alternative value was tested and the speculation was confirmed.

The mismatch at the tail end is again attributed to insufficient wastage sludge flow data. On the very last day sludge was being wasted, but no actual flow data was recorded. So it is assumed to follow the data of the day immediately before, which might have not been the case in reality.

All in all, the results above further confirm that 2-layer model is a good approximation of the real system studied.

5.3.2 Returned Activated Sludge

Figure 5.30 shows the model validation result of RAS concentration in one of the secondary clarifiers. Obviously the model result misses a peak in data between day 4 and day 6. The data, however, are quite suspicious, because at that time the sludge was wasted at a relatively high rate.



Figure 5.30 Model Validation Result of RAS Concentration in One of the Secondary Clarifiers
5.3.3 Sludge Blanket Height



Figure 5.31 Model Validation Result of Sludge Blanket Height in One of the Secondary Clarifiers

Figure 5.31 shows the model validation result of sludge blanket level. Due to high flow rate during the rain event, mixed liquor suspended solids are washed out of the aeration tank into the secondary clarifier (Figure 5.28) such that the sludge blanket level gradually builds up (as seen in Figure 5.31) if the recycle and waste sludge flow rates are relatively small, and therefore more suspended solids are seen in the secondary clarifier effluent (Figure 5.32). Since the measurement of suspended solids in the secondary effluent is in the unit of FTU, check against model result is impossible.



Figure 5.32 Simulation Result of Suspended Solids in the Secondary Clarifier Effluent

In Figure 5.31 the model overestimates the data after day 6, but manages to correct itself to a certain extent towards the end. The time-invariant parameters for the thickening model cannot lead to accurate simulation performance on an hourly basis (Chen, 1993). Under extreme conditions, flux theory model may lead to unrealistic sludge profiles (Krebs, 1995). Dupont and Dahl (1995) attempted to include the known effects of density current and short-circuiting from inlet to sludge recirculation into a 1D model, and present some promising results. Siegrist et al. (1995) included a 2D hydrodynamic model in their study and found improved agreement with the experimental observations. In order to keep the model as simple as possible, additional attempts have not been made in the current model.

5.3.4 Ammonium-N

Figure 5.33 shows the model validation result of ammonium concentration in the outer and inner channels of the aeration tank. Figure 5.34 shows *Nitrosomonas* concentration at the corresponding locations.



Figure 5.33 Model Validation Result of Ammonium Concentration in the Outer and Inner Channel



Figure 5.34 Nitrosomonas Concentration in the Upper CSTRs of the Outer and Inner Channels

During the rain event half of the influent was fed directly into the inner channel according to the plant record, which might be an inaccurate data as discussed in Section

5.3.1. This might be the reason the model detects more ammonium downstream than upstream between day 4 and day 6, as seen in Figure 5.33.

Towards the end of the period, the model underestimates ammonium concentration in both the outer and inner channels. One possible reason for the mismatches towards the end of the simulation window is the overestimation of DO in both the outer and inner channels as seen in Figure 5.35. Data around day 15 are suspicious because the ammonia probe failed on day 15. Calibration of the probe was carried out once a day in the early morning. Once a new probe is installed, it needs 24 hours to stabilize (Capital Controls, 1997).



Figure 5.35 DO Concentration in the Upper CSTRs of the Outer and Inner Channels

The consistent overestimation of DO concentration in the outer channel might be due to the fact that the measurement was taken at the tail end of the aerator, where DO levels can be expected to be lower than that at the head end of the aerator. Another explanation for the overestimation is that during the rain event when influent flow is fed directly into the inner channel and the return sludge is still fed into the outer channel, the number of aerators operating in the outer channel is usually decreased to 2 to keep the solids in suspension. But this decrease in aerator number might not have been recorded.

5.3.5 Nitrite-N

The following figure shows the simulation results of nitrite in the middle channel and secondary clarifier effluent. The model largely matches the fairly low nitrite level.



Figure 5.36 Model Validation Result of Nitrite Concentration in the Middle Channel and the Secondary Clarifier Effluent

5.3.6 Nitrate-N

Figure 5.37 shows the model validation results of nitrate concentrations in the middle channel and the secondary clarifier effluent using the parameter values obtained from model calibration exercise. Obviously, denitrification rates for the calibration data do not quite apply to the validation data, especially in reference to the top plot in Figure 5.37. This is due to the fact that more aerators were on during the validation data period

than for the calibration data period, and also there were more major rain events involved in the validation data period such that less denitrification takes place.



Figure 5.37 Initial Model Validation Results of Nitrate Concentrations in the Middle Channel and the Secondary Clarifier Effluent



Figure 5.38 Model Validation Result of Nitrate Concentration in the Upper Middle Channel and the Secondary Clarifier Effluent

Therefore, smaller denitrification rate values were chosen. Figure 5.38 shows the model validation result of nitrate concentration in the upper middle channel and secondary clarifier effluent. The model successfully simulated the surge for nitrate in the middle channel, however, less so for that in the secondary clarifier effluent. The underestimation of nitrate concentration in the secondary clarifier (bottom plot in Figure 5.38) can be somehow correlated with overestimation of ammonium concentration in the inner channel in Figure 5.33. The source of error can also be associated with inaccurate operational data on the percentage of influent feeding into the inner channel during the rain events as discussed in Section 5.3.1.

5.3.7 TOC

Figure 5.39 shows the model validation result of TOC concentration in the outer and inner Channel respectively. The model does a relatively good job in matching the data. The mismatches in the inner channel from day 3 to about day 5 correlate with underestimations of DO (as shown in Figure 5.35). Partitioning of the aerator influent TOC into various constituents could be another source of error, as discussed in Section 5.2.2.



Figure 5.39 Model Validation Result of TOC Concentration in the Outer and Inner Channel

5.3.8 Ortho-Phosphate-P

Figure 5.40 shows the model validation results of phosphorus concentration in the middle channel and the secondary clarifier effluent. The possible explanation for the overestimation of ortho-phosphate-P between day 6 to about day 9 is lower organic loading during the same period of time. The mismatch might have resulted from the slow rate at which depleted poly- β -hydroxybutyrate (PHB) storage is replenished (Temmink et al., 1996), as reported in Chapter 2.



Figure 5.40 Model Validation Result of Phosphorus Concentration in the Upper Middle Channel and Secondary Clarifier Effluent

5.4 PARAMETER ESTIMATION

The parameters in the proposed model were changed on a "trial and error" basis to achieve the best fit between the results generated by the model and the data obtained from the sample campaign. No more complicated calibration techniques were adopted due to large number of parameters in the current model. The final parameter values are listed in Table 5.1 below, for which the simulation results have already been illustrated in Sections 5.2 (calibration) and 5.3 (validation).

No.	Symbols	Proposed Values	Final Estimated		
		for ASM2	Values		
Second	Secondary Clarifier				
1	max_vel		200		
2	para		4.6e-4		
3	alpha45		8.0		
4	alpha46		1e-6		
5	alpha54		43		
Aeration Tank					
6	vel		0		
7	alpha1		1.00		
8	alpha2		0.16		
9	c		1		
10	ratio1		0.32		
11	ratio2		0.20		
12	ratio3		0.68		
13	beta		0.18		
14	a2		0.008		
15	b2		0.05		
16	b1		0.095		
Autotro	Autotrophic Biomass				
17	K ₀₂ so		0.5		
18	miu so		0.45		
19	miu ba		0.45		
20	$K_{O2 ba}$		0.5		
21	Y _{SO}		0.14		
22	Y _{ba}		0.24		
23	b _{so}		0.15		
24	b _{ba}		0.10		
25	K _{NH4} SO		1.7		
26	K _{NO2 ba}		0.35		
27	K _{PO4} so		0.01		
28	K _{PO4 ba}		0.01		
29	deni1		380		
30	deni2		380		
31	deni3		20		
32	deni4		30		
Hetero	Heterotrophic Biomass				
33	miu h	3.0~6.0	3.8		
34	K _{O2 h}	0.20	0.20		
35	Y _h	0.63	0.625		
36	b _h	0.20~0.40	0.20		
37	K _{easy h}	4.00	4.00		

Table 5.1 Parameter Values for the Current Model and ASM2 ($10^{\circ}C$ and $20^{\circ}C$)

38	K _{NO3_h}	0.50	0.50	
39	K _{NH4 h}	0.05	0.05	
40	Kpo _{4 h}	0.01	0.01	
41	K _{NO2 h}		0.32	
42	q_{fe}^{-}	1.5~3.0	1.50	
43	K _{fe}	20.00	20.00	
Phosph	orus-accumulating bac	teria		
44	miu_pao	0.67~1.00	0.70	
45	K _{O2 PAO}	0.20	0.20	
46	Y _{PAO}	0.63	0.625	
47	b _{PAO}	0.10~0.20	0.10	
48	bpp	0.10~0.20	0.10	
49	b _{pha}	0.10~0.20	0.10	
50	q _{pp}	1.00~1.50	1.50	
51	q _{pha}	2.00~3.00	3.00	
52	K _{NH4 PAO}	0.05	0.05	
53	K _{PO4} PAO	0.01	0.01	
54	K _{NO3 PAO}	0.50	0.50	
55	K _{A PAO}	4.00	4.00	
56	- K _{PP}	0.01	0.01	
57	K _{PHA}	0.01	0.01	
58	K _{max}	0.34	0.34	
59	K _{ipp}	0.02	0.02	
Hydrolysis				
60	K _h	2.00~3.00	2.50	
61	K _{x_h}	0.10~0.30	0.10	
62	eda_NO ₃	0.60	0.60	
Others				
63	i _{pbm}	0.02	0.02	
64	f _{xi}	0.10	0.10	
65	gamma_PO ₄	0.40	0.40	
66	gamma_pha	0.20	0.20	
67	const		8	
68	ratio_so		0.011	
69	ratio_ba		0.020	
70	ratio_h		0.018	
71	ratio_pao		0.05	
72	ratio_pha		0	
73	ratio_pp		0	
74	ratio_slow		0	
75	percentage_easy		1	
76	percentage_ferment		0	
77	percentage_inert		0	

5.5 SENSITIVITY ANALYSIS

5.5.1 Sensitivity of Biomass to Initial Conditions

In this section alternative initial concentrations for different kinds of biomasses are tested to obtain a sense of the characteristic time constants of their dynamic behavior.

Now sensitivity of various categories of biomass to initial conditions will be tested. Location for each kind of biomass is picked purely at random. For each kind of biomass their initial concentrations in each channel are taken the same values.

5.5.1.1 Nitrosomonas



Figure 5.41 Sensitivity of Nitrosomonas to Different Initial Concentrations

Figure 5.41 shows sensitivity of Nitrosomonas to different initial concentrations. It reaches stable state in less than 1.5 days.

5.5.1.2 Nitrobacter



Figure 5.42 Sensitivity of Nitrobacter to Different Initial Concentrations

Figure 5.42 shows sensitivity of Nitrobacter to different initial concentrations. It reaches stable state in a bit less than 2 days.

5.5.1.3 Heterotroph



Figure 5.43 Sensitivity of Heterotroph to Different Initial Concentrations

Figure 5.43 shows sensitivity of heterotroph to different initial concentrations. It reaches stable state in a bit over 2 days.

5.5.1.4 Phosphorus-Accumulating Organisms (PAO)



Figure 5.44 Sensitivity of PAO to Different Initial Concentrations

Figure 5.44 shows sensitivity of PAO to different initial concentrations. It reaches stable state in a bit over 2 days as well.

5.5.2 Hydraulic/Transport Structure

The following part will show why the particular values for the parameters associated with hydraulic/transport structure of the model have been chosen.

5.5.2.1 'beta'



Figure 5.45 MLSS Concentration in the Inner Channel

'beta' is the fraction of return sludge immediately dropping into the bottom CSTR of the outer channel upon entering the aeration tank. The chosen value for 'beta' in the model is 0.18.





Figure 5.46 MLSS Concentration in the Inner Channel

'alpha1' is the ratio of flow running through top CSTRs to combined flow (crude sewage + return sludge) entering the aeration tank. The chosen value for 'alpha1' in the model is 1.0, which implies that all flow goes through the upper CSTRs and none through the bottom CSTRs.

5.5.2.3 'alpha2'



Figure 5.47 MLSS Concentration in the Inner Channel

'alpha2' is resuspension coefficient. The chosen value for 'alpha2' in the model is 0.16, which gives the best fit.

5.5.2.4 'vel'

'vel' is free settling velocity of solids from top CSTRs to bottom CSTRs immediately below. The chosen value for 'vel' in the model is 0. The difference of assigning different values to this parameter may not be obvious graphically, but this value is in fact confirmed by Hunze et al. (2000).

Some further sensitivity analysis is done in Section 6.5.

5.6 CONCLUSIONS

The model has been evaluated against two sets of field data retrieved from the same sampling campaign described in Chapter 3. The first set of data is used for model calibration. The second study deals with model validation. Evaluation of the model against both sets of data demonstrates the model's capability to simulate the nutrient removal processes in an activated sludge system. However, it is also found that there are flaws in the model structure, in particular, with respect to biological phosphorus removal. Yet, since they are not found to be critical, the model can reasonably be used for the assessment of operational control strategies in the next Chapter.

CHAPTER 6

DEVELOPMENT AND EVALUATION OF PROCESS CONTROL STRATEGIES

6.1 INTRODUCTION

Various control strategies for the activated sludge process have been reviewed in Chapter 2. A dynamic model was then developed in Chapter 4 and the results of its evaluation against field data were presented in Chapter 5. Therefore, we are now in a position to explore the application of this model in the development and evaluation of process control strategies for the activated sludge system. The strategies to be studied in this Chapter are for short-term operational control under transient variations in the influent, especially for the case of storm run-off events.

This Chapter is organized as follows. Overall design of the assessment is specified in section 6.2, including control objectives and selection of the criteria for evaluation of the various strategies. A comprehensive set of control algorithms is then set out in section 6.3. The results of testing these algorithms are presented in section 6.4.

6.2 SPECIFICATION OF THE ASSESSMENT

6.2.1 Configuration of The Activated Sludge Process

A typical conventional activated sludge plant configuration to be studied in this Chapter is basically the same as that at the Athens Wastewater Treatment Facility No. 2. All the influent flow passes through the entire plant without any diversions of excess flow. By doing so, the effects of storm events and their control can be fully revealed. The aeration tank has three circular channels. Both crude sewage and returned activated sludge can be distributed to any of the three channels at will. In other words, this configuration allows manipulation of the following variables:

- Distribution of crude sewage to various channels of the aeration tank (i.e., step-feed control); and
- Distribution of recycled activated sludge to various channels of the aeration tank (i.e., step-sludge control).

Moreover, in the outer channel of the aeration tank six pairs of surface mechanical aerators are installed in northwest, north, northeast, southeast, south, and southwest directions respectively. Each pair of aerators always works together. There are also two surface aerator units each in the middle channel and the inner channel. The aerators can be turned on and off or their depth of immersion can be changed (as stated in Chapter 3) to make dissolved oxygen concentration in each channel reach the given desired level.

Finally, the constraints for the other control variables (recycle and wastage flow rates) are:

- □ recycle flow (Q_R) : $0 \le Q_R \le 2.5$ million gallon/day
- □ sludge wastage flow (Q_w): $0 \le Q_w \le 0.22$ million gallon/day

6.2.2 Influent Flow Water Quality

The influent flow water quality data used for the control study are extracted from the model validation data streams as described in Section 5.3. The simulation period is from February 13, 1998 to February 22, 1998, with the total number of days amounting to 10. The reason for choosing this period is that there are major rain events (indicated in Figure 6.1) involved on day 4 (February 16, 1998) and day 5 (February 17, 1998), which provide a good testing bed for subsequent experimentation with different control strategies. Figure 6.1 to Figure 6.9 show influent flow (Figure 6.1), corresponding water quality characteristics of the crude sewage as it passed forward to the activated sludge unit (Figure 6.2 to Figure 6.6), DO concentration measurements in three channels (Figure 6.7), MLSS concentration measurement in the inner channel (Figure 6.8), and sludge blanket level measurement in one of the secondary clarifies (Figure 6.9). Figure 6.7 to Figure 6.9 complete the picture of actual operating conditions.



Figure 6.1 Influent Flow



Figure 6.2 Influent Ammonium Concentrations



Figure 6.3 Influent Nitrite Concentrations



Figure 6.4 Influent Nitrate Concentrations



Figure 6.5 Influent TOC Concentrations



Figure 6.6 Influent Ortho-phosphate-P Concentrations

Obviously, influent concentrations of ammonium, TOC and orthophosphate-P decrease during the rain events due to the dilution effects. However, influent nitrite and

nitrate concentrations increase during the storm events, something which can be attributed to many factors, including acute high loadings (Alleman, 1984; Henze, 1995).



Figure 6.7 DO Concentration Measurements in All Three Channels



Figure 6.8 MLSS Concentration Measurement in Inner Channel



Figure 6.9 Sludge Blanket Level Measurement in One Secondary Clarifier

During the storm events, DO concentrations increase in all three channels (Figure 6.7) due to elevated turbulency. Mixed liquor suspended solids, however, are flushed out of the aeration tank (Figure 6.8) because of the hydraulic surge, such that sludge begins to build up in the secondary clarifier subsequently (Figure 6.9).

6.2.3 Objectives and Constraint Functions For Process Control Strategies

The following two control objectives will be used in the study:

- Ensure that the plant satisfies the regulatory effluent standards for suspended solids (SS), and ammonium, and specified standards for nitrite, nitrate, TOC and orthophosphate-P;
- □ Improve the effluent water quality as much as possible under the conditions provided.

The objectives above can be expressed in the following mathematical forms such that the settings of the manipulated control variables (returned sludge flow, wastage flow, etc.) during the simulation can be chosen on the basis of their satisfaction. □ Effluent in compliance

$$P_1 = \frac{1}{T} \int_0^T Y_{eff} dt \le Y_{std}$$

where,

Y_{eff}–Simulated effluent water quality, and

Y_{std}–Desired effluent standard over the period of T.

For SS and ammonium, the current regulatory effluent standards are 30 and 17 mg/L respectively. While the effluent standards for nitrite, nitrate, TOC and ortho-phosphate-P to be used in this control study are assumed to take the values of 0.2 (Effler et al., 1996), 2.5 (Orth et al., 1988), 30 (Orth et al., 1988), and 2 mg/L (Werumeus Buning et al., 1991) respectively;

- T-Sampling period over which the actual behavior of the process is checked against its desired behavior.
- □ Maximum water quality

$$P_{2} = Min\{\int_{0}^{T} (w1*\frac{Y_{SS}}{Y_{std,SS}} + w2*\frac{Y_{NH3}}{Y_{std,NH3}} + w3*\frac{Y_{TOC}}{Y_{std,TOC}} + w4*\frac{Y_{P}}{Y_{std,P}} + w5*\frac{Y_{NO2}}{Y_{std,NO2}} + w6*\frac{Y_{NO3}}{Y_{std,NO3}})dt\}$$

where,

Y_{ss},Y_{NH3},Y_{TOC},Y_P,Y_{NO2},Y_{NO3}–Simulated effluent water quality for the suspended solids, ammonium, TOC, ortho-phosphate-P, nitrite, and nitrate respectively;

w1, w2, w3, w4, w5, w6–Weighting coefficients. In this study, SS and ammonium are given priority over TOC, ortho-phosphate-P, nitrite and nitrate, so that w1, w2, w3, w4, w5, and w6 are chosen as 0.3, 0.3, 0.1, 0.1, 0.1, and 0.1 respectively.

The formulation immediately above is based on Chen's work (1993), but including more water quality indicators, specifically TOC, ortho-phosphate-P, nitrite and nitrate.

For these two objectives, the control variables are changed on an hourly basis, except the step-feed and step-sludge controls as discussed below, because effluent quality is more easily influenced by relatively fast transient disturbances. More detailed discussion of the design of the control algorithms will be continued in section 6.3.

6.2.4 Criteria for Evaluation of Controlled Plant Performance

It is hard to differentiate between the performances of the various control strategies in the context of the entire sewage plant management without simulating the behavior of the whole treatment plant and the receiving water quality. Such a more complete assessment is beyond the scope of this dissertation, although extensive work has been done on evaluating the impacts of effluent discharges on receiving water quality (Reda,1993; Duchesne et al., 2000; Schutze et al., 2000). In this control study, evaluation of the controlled plant performance will be based primarily on the performance of the activated sludge process itself. For each strategy tested, the following summaries will be used for comparative assessment:

- □ Maximum concentrations of SS, ammonium, nitrite, nitrate, TOC, and orthophosphate-P in the secondary clarifier effluent over the storm period;
- □ Average effluent loadings of SS, ammonium, nitrite, nitrate, TOC, and orthophosphate-P in the secondary clarifier effluent over the storm period;
- Graphs of the effluent and activated sludge process responses, including the effluent SS, effluent ammonium, mixed liquor suspended solid concentration, and sludge blanket height.

The threshold values mentioned above are set at 30, 17, 0.2, 2.5, 30, and 2 mg/L for SS, ammonium, nitrite, nitrate, TOC and ortho-phosphate-P respectively. They are also approximately the maximum effluent concentrations found during normal operation at Athens wastewater treatment facility No.2.

6.3 DETAILED SPECIFICATION OF THE CONTROL ALGORITHMS

Many combinations of the control strategies presented in Chapter 2 might be used for exploring the operational flexibility of the activated sludge process. Such an exhaustive analysis, however, would be beyond the scope of this dissertation. Only the practical strategies and those clearly demonstrating both the potential of the model and the importance of simple control actions will be examined herein.

More specifically, the following four groups of control strategies are to be investigated for the control of storm disturbances.

- Conventional Control
 - Returned sludge flow rate (Q_R) is manipulated in proportion to the influent flow rate (Q_{in});
 - Sludge wastage flow rate (Q_w) is chosen as a function of the calculated sludge age. Sludge age is calculated as the total amount of sludge in the system divided by the rate of sludge loss from the system. The equation is defined as follows (Grady et al., 1999).

$$t_{s} = \frac{VX}{Q_{w}X_{w} + Q_{e}X_{e}}$$

where,

t_s–Sludge age;

V–Volume of liquid in the aeration tank;

X–MLSS concentration in the aeration tank;

Q_w–Sludge wastage rate;

X_w–Sludge concentration in the sludge wastage flow;

Q_e-Secondary effluent flow;

Xe-Suspended solid concentration in the secondary effluent;

- □ Feedback Control
 - Q_R is changed as a function of the effluent quality;
 - Q_w is similarly manipulated as Q_R .
- □ Advanced Control
 - Step Feed
 - Step Sludge
- DO Set-Point Control

6.3.1 Base Case Control

One simulation run is essential to provide a reference point for comparisons among the various control strategies. Return sludge flow and wastage sludge flow data used in this simulation run are the same as those in the actual operation of the Athens Wastewater Treatment Facility No. 2. Figure 6.10 shows return sludge flow from one of the three secondary clarifiers for the entire simulation window. Due to the limited number of facilities in EPCL, only this specific clarifier was monitored during the sampling campaign described in Chapter 3. During actual plant operation return sludge flow in each secondary clarifier (three in total) is adjusted according to the sludge blanket level in the specified clarifier based purely on experience. For instance, if the sludge blanket level is above 10 feet, return sludge flow is increased by about 0.10MGD. Conversely, if the sludge blanket level is below 2 feet, return sludge flow is decreased by about 0.10 MGD. Shown in Figure 6.11, on the other hand, is the combined return sludge flow from all three secondary clarifiers.



Figure 6.10 Returned Sludge Flow Rate Data During the Entire Simulation Period from One of the Secondary Clarifiers



Figure 6.11 Combined Returned Sludge Flow Rate Data from All Three Secondary Clarifiers



Figure 6.12 Sludge Wastage Flow

Figure 6.12 shows the actual manipulation of the sludge wastage flow. Processing of waste sludge is operated at the Athens Wastewater Treatment Facility No.2 only four days a week, normally from 5:00am to 4:00pm, Monday through Thursday. The sludge wastage processing machines are stopped at times other than that specified. Also there exists a limitation on the sludge wastage capacity. When sludge builds up in the secondary clarifier and an elevated sludge wastage is desirable, the sludge-processing sector in the treatment plant may therefore not be able to accommodate this need.

Half of the influent flow was fed directly into the inner channel of aeration tank No. 2 with the other half fed into the inner channel of aeration tank No.1 at 3:00pm on February 16, 1998. On February 19, 1998 at 8:15am all the influent flow began to be channeled through the outer channel of aeration tank No.2 again. Thereafter normal operation resumed.

Shown in Figure 6.13 is the actual number of active aerators in the outer channel during the simulation period. There are six pairs of aerators in the outer channel of the aeration tank, and two each in the middle and inner channel. The aerators in the middle

and inner channels are always running together. The aerators in the outer channel are turned on/off according to the actual DO concentration. When DO concentration is over 0.5 mg/L, one or more aerators are turned off. If DO concentration is below 0.2 mg/L, one or more aerators are turned on until the maximum available number of aerators is reached.



Figure 6.13 Number of Active Aerators in the Outer Channel

6.3.2 Conventional Control

In Chapter 2, conventional control of the activated sludge process has been described. To evaluate the effectiveness of these algorithms, the following tasks have been conducted.

Algorithm I

As explained in Section 6.3.1, in actual operation the return sludge rate is controlled based on the sludge blanket level in the secondary clarifier based purely on experiences. The average value of the ratio between sludge return rate and influent flow rate, however, is about 0.28 even though it was not in fact so manipulated during the storm event. In this algorithm the sludge return rate (Q_R) is controlled in proportion to the influent flow rate Q_{in} , i.e., $Q_R = k1^*Q_{in}$. The following three cases are tested:

- $\Box \quad \text{Strategy 1: } Q_R = 0.25 * Q_{\text{in}}$
- $\Box \quad \text{Strategy 2: } Q_R = 0.35 * Q_{in}$
- $\Box \quad \text{Strategy 3: } Q_R = 0.42 * Q_{\text{in}}$

Algorithm II

Sludge wastage rate (Q_w) can be controlled using the sludge age. In actual operation the sludge wastage flow stayed zero during the storm event for the reason explained in Section 6.3.1. Conventional activated sludge has a sludge age of 3-4 days, and has good settling properties. However, at a sludge age greater than 6 day there is a reduction in settleability (Gray, 1990). The following three cases are therefore tested:

- \Box Strategy 4: Q_w is adjusted to give a sludge age of 3 days
- \Box Strategy 5: Q_w is adjusted to give a sludge age of 4 days
- **\Box** Strategy 6: Q_w is adjusted to give a sludge age of 12 days

6.3.3 Feedback Control

Although conventional feedback control may include what is called proportional (P), integral (I), and derivative (D) action (Olsson and Newell, 1999), only proportional feedback controllers are considered in this study. Again, as for the conventional controls in section 6.3.2, the sludge return rate (Q_R) and sludge wastage (Q_w) rate are considered as the only control variables.

The following equation is used to implement the feedback control of Q_R :

$$Q_{R,t} = Q_{R,t-1} + K_c * \left[-w1(\frac{NH_3}{NH_{3,std}} - s1) + w2(\frac{SS}{SS_{std}} - s2) - w3(\frac{TOC}{TOC_{std}} - s3) - w4(\frac{P}{P_{std}} - s4) - w5(\frac{NO_2}{NO_{2,std}} - s5) - w6(\frac{NO_3}{NO_{3,std}} - s6)\right]$$

where,

- Q_{R,t}, Q_{R,t-1}–Sludge return rates at times t and t-1 respectively;
- K_c–Proportional control gain; and
- si-Coefficients for the controlled effluent quality set-points relative to their standards for i=1,2,3,4,5, and 6.

This equation extends that proposed by Chen's (1993) by including TOC, P, nitrite and nitrate. According to these control laws, return sludge manipulation attempts to maintain system performance at some desired set-points under normal conditions. But in case the performance is abnormal, "emergency" action is then taken. In respect of effluent quality, it seeks to maintain NH₃ \rightarrow s1*NH_{3,std}, SS \rightarrow s2*SS_{std}, TOC \rightarrow s3*TOCstd, P \rightarrow s4*P_{std}, NO₂ \rightarrow s5*NO_{2,std}, and NO₃ \rightarrow NO_{3,std} with 0 \leq s_i \leq 1. The coefficients, s_i, are introduced to keep the effluent quality below the permitted discharge standard. If s_i=0, control operation is essentially in pursuit of the best possible performance of the process. The smaller the s_i, the more strictly the process is to be controlled, and this may therefore require more energy expenditure, which is undesirable in terms of operational costs. The following four options will be tested.

Algorithm III

- □ Strategy 7: $K_c = -5e-3$, s1 = s2 = s3 = s4 = s5 = s6 = 0.8
- □ Strategy 8: $K_c = -1e-4$, s1 = s2 = s3 = s4 = s5 = s6 = 0.8
- □ Strategy 9: $K_c = -1e-3$, s1 = s2 = s3 = s4 = s5 = s6 = 0.5

6.3.3.2 Feedback Control of Q_w

Feedback control of Q_w is based on a principle similar to that described in section 6.3.3.1 for Q_R . The control rules are thus applied to Q_w as shown in the following equation.

$$Q_{w,t} = Q_{w,t-1} + K_C * [w1(\frac{NH_3}{NH_{3,std}} - s1) - w2(\frac{SS}{SS_{std}} - s2) + w3(\frac{TOC}{TOC_{std}} - s3) + w4(\frac{P}{P_{std}} - s4) + w5(\frac{NO_2}{NO_{2,std}} - s5) + w6(\frac{NO_3}{NO_{3,std}} - s6)]$$

Obviously, the control gain K_c will be different. So we have the following options.

Algorithm IV

- **u** Strategy 10: Kc = -0.001, s1 = s2 = s3 = s4 = s5 = s6 = 0.8
- □ Strategy 11: Kc = -0.003, s1 = s2 = s3 = s4 = s5 = s6 = 0.8
- **u** Strategy 12: Kc = -0.003, s1 = s2 = s3 = s4 = s5 = s6 = 0.6

6.3.4 Advanced Control

In Chapter 2 we noted that operation of the activated sludge process can be enhanced by implementing step-feed control, especially when the system experiences abnormal short-term influent shocks (Funamizu et al., 1997). This and other options are now considered. Step-sludge and step-feed are both implemented only during the storm event.

6.3.4.1 Step Sludge

In this algorithm, the influent flow still enters the aeration tank from the outer channel, while the returned activated sludge flow is split between outer, middle, and inner channels. In other words, each channel receives a portion of the total returned sludge flow, and the sum of the returned sludge fed to each channel amounts to the total returned sludge flow pumped from the bottom of the secondary clarifiers.

Algorithm V

- **u** Strategy 13: w7 = 0.80, w8 = 0.20, w9 = 0
- **u** Strategy 14: w7 = 0.60, w8 = 0.20, w9 = 0.20
- **Given Strategy 15:** w7 = 0.10, w8 = 0.20, w9 = 0.70

where,

w7–Fraction of returned sludge entering the outer channel of the aeration tank;

w8–Fraction of returned sludge entering the middle channel;

w9–Fraction of returned sludge entering the inner channel.

6.3.4.2 Step Feed

In this algorithm, the returned sludge flow enters the aeration tank from the outer channel, while influent flow is split between outer, middle, and inner channels. In other words, each channel receives a portion of the influent flow. And the sum of influent flow fed to each channel amounts to the influent flow entering the aeration tank.

Algorithm VI

- **Given Strategy 16:** w10 = 1.00, w11 = 0, w12 = 0
- **Given Strategy 17:** w10 = 0.60, w11 = 0.20, w12 = 0.20
- **Given Strategy 18:** w10 = 0.20, w11 = 0.20, w12 = 0.60
- **Given Strategy 19:** w10 = 0, w11 = 0, w12 = 1.00

where,

w10-Fraction of influent flow entering the outer channel;

w11–Fraction of influent flow entering the middle channel;

w12–Fraction of influent flow entering the inner channel.

6.3.5 DO Set-point Control

The purpose of oxygen control is to keep DO profile in the aeration tank at a desired level such that biochemical reaction would not be downgraded. Many papers have reported on DO set-point control by adjusting air compressor flow rate (Buhr et al., 1984; Healey, 1989; Olsson and Newell, 1999). Model based DO set-point control by turning on and off surface aerators is achieved in a manner similar to airflow control as suggested by Olsson and Newell (1999).

The error between the DO set-point, S_{O,SP}, and the true DO value, S_O, is defined as:

$$e = S_{O,SP} - S_o \tag{6.1}$$

The controller should work so that the error will approach zero. Therefore it is prescribed that the error should obey the differential equation:

$$\frac{de}{dt} = -e \tag{6.2}$$

Then take the derivative of Equation 6.1 assuming that DO set-point is constant.

$$\frac{de}{dt} = \frac{d}{dt}(S_{O,SP} - S_O) = -\frac{dS_O}{dt}$$
(6.3)

Combining equation 6.2, equation 6.3, and DO dynamics listed in Table 4.3, the number of aerators needed to keep the desired DO level can be deduced. For instance, the

number of aerators needed in the outer channel (denoted as 'n1') as a result of the DO controller is derived as follows:

$$n1 = \left[e - \frac{-(Q_{above} + Q_{d1}) * S_{O2_upper}(1) + Q_{resus} * S_{O2_bottom}(1)}{V_{upper1}} - (1 - \frac{3.43}{Y_{SO}}) * \mathbf{m}_{SO} * \frac{S_{NH4_upper}(1)}{K_{NH4_SO} + S_{NH4_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_SO} + S_{O2_upper}(1)} \\ * \frac{S_{PO4_upper}(1)}{K_{PO4_SO} + S_{PO4_upper}(1)} * X_{SO_upper}(1) \\ - (1 - \frac{1.14}{Y_{ba}}) * \mathbf{m}_{ba} * \frac{S_{O2_upper}(1)}{K_{O2_ba} + S_{O2_upper}(1)} * \frac{S_{NO2_upper}(1)}{K_{NO2_ba} + S_{NO2_upper}(1)} \\ * \frac{S_{PO4_upper}(1)}{K_{PO4_ba} + S_{PO4_upper}(1)} * X_{ba_upper}(1) \\ - (1 - \frac{1}{Y_{H}}) * \mathbf{m}_{H} * \frac{S_{easy_upper}(1)}{K_{easy_H} + S_{easy_upper}(1)} * \frac{S_{O2_upper}(1)}{K_{O2_H} + S_{O2_upper}(1)} \\ * \frac{S_{NH4_upper}(1)}{K_{NH4_H} + S_{NH4_upper}(1)} * \frac{S_{PO4_upper}(1)}{K_{PO4_H} + S_{PO4_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_upper}(1)} \\ - (1 - \frac{1}{Y_{PAO}}) * \mathbf{m}_{PAO} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{NH4_pao} + S_{NH4_upper}(1)} \\ * \frac{S_{PO4_upper}(1)}{K_{rou} + S_{roo}} * \frac{S_{O2_upper}(1)}{K_{O2_PAO} + S_{O2_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{PAO_upper}(1)} * \frac{S_{NH4_upper}(1)}{K_{PAO_upper}(1)} * \frac{S_{PAA_upper}(1)}{K_{PAA_upper}(1)} *$$

$$\frac{S_{PO4_upper}(1)}{K_{PO4} + S_{PO4_upper}(1)} * \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{K_{PHA} + X_{PHA_upper}(1) / X_{PAO_upper}(1)} * X_{PAO_upper}(1)$$

$$+ g_{PHA} * q_{PP} * \frac{S_{O2_upper}(1)}{S_{O2_upper}(1) + K_{O2_PAO}} * \frac{S_{PO4_upper}(1)}{S_{PO4_upper}(1) + K_{PO4_PAO}}$$

$$* \frac{X_{PHA_upper}(1) / X_{PAO_upper}(1)}{X_{PHA_upper}(1) / X_{PAO_upper}(1) + K_{PHA}} * \frac{K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}{K_{IPP} + K_{MAX} - X_{PP_upper}(1) / X_{PAO_upper}(1)}$$

$$* X_{PAO_upper}(1)] / [const * (S_{O,SP} - S_{O})]$$

Although DO control is widely used and an accuracy of 0.2mg/L was reported 20 years ago (Ryder, 1972), the set-point of the DO level is quite system-specific and mostly in the range of 0.5-1.5 mg/L (Ryder, 1972). In this DO set-point control study, the following options will be tested:
Algorithm VII

- $\Box \quad \text{Strategy 20: } S_{O,SP} = 1.0$
- $\Box \quad \text{Strategy } 21: \text{ } \text{S}_{\text{O,SP}} = 1.5$
- $\Box \quad \text{Strategy 22: } S_{O,SP} = 5.0$

DO set-point as high as 5.0 mg/L is chosen to find out the possible scale of nitrification improvement and associated energy requirement.

For all the 21 control strategies listed above, the control actions are taken 2 hours before the storm events start, and ceased 2 hours after the storm events subside. No control is exercised before and after the event. The time before the event indicates 'normal operation', while the simulation window after the storm event could possibly show the impact on the treatment system in the aftermath of the storms. Also for each control strategy tested, other than step-feed and step-sludge, during the storm event all influent flow is still fed into the outer channel (not into the inner channel as in the base case) and the same for the return sludge flow for the ease of comparing different control strategies.

6.4 SIMULATION RESULTS AND EVALUATION OF CONTROL STRATEGIES

The results to be presented here have to be selective. The discussion is oriented towards comparison of the various control strategies and demonstration of the operational flexibility of the activated sludge process.

Table 6.1 gives summary statistics of the simulation results for the control strategies tested. Their detailed discussion is presented below.

6.4.1 Conventional Control

6.4.1.1 Control of Q_R in Proportion to Q_{in} (strategies 1 to 3)

Control of Q_R in proportion to Q_{in} has both advantages and disadvantages relative to the base case. From the results of strategies 1-3, it is apparent that ratio control of

return sludge rate is better at reducing the average effluent mass loads of ammonium, nitrate, TOC, and ortho-phosphate-P than the base case. Yet it increases the average effluent mass loads of SS (see Table 6.1).



Figure 6.14 TOC Concentration in the Secondary Clarifier Effluent



Figure 6.15 Heterotroph Concentration in the Upper Inner Channel



Figure 6.16 Fermentation Products Concentration in the Upper Inner Channel



Figure 6.17 Easily Biodegradable Substance Concentration in the Upper Inner Channel

Figure 6.14 shows the TOC concentration profiles in the secondary clarifier effluent under base-case and strategies 1-3 controls. Obviously, the effluent TOC

concentrations are consistently lower than the base case during the storm by implementing any of strategies 1-3. Figure 6.15 shows heterotroph concentration in the upper inner channel with different control strategies (strategy 1-3). Figure 6.16 shows fermentation product concentrations in the upper inner channel. And those curves shown in Figure 6.17 are easily biodegradable substance concentrations at the same location.

Even though heterotroph concentrations in the inner upper channel for the base case are higher from day 4.5 to day 6.5 relative to strategy 1-3 (Figure 6.15), concentrations of fermentation products and easily biodegradable substances at the same location are consistently higher (Figure 6.16 and Figure 6.17), which therefore causes higher effluent TOC mass load in the secondary clarifier effluent for the base case (Table 6.1). Worthy of mention is that TOC is modeled as the combination of easily biodegradable substance, fermentation products, and inert product, which last turns out to be zero from model calibration ('percentage-inert' in Table 5.1).



Figure 6.18 MLSS Concentration in the Upper Inner Channel



Figure 6.19 Sludge Blanket Level in One Secondary Clarifier



Figure 6.20 Suspended Solid Concentration in the Secondary Clarifier Effluent

However, caution should be taken in applying ratio control of Q_R as far as the behavior of the sludge blanket is concerned. The more the sludge is returned to the aeration tank, the larger the mixed liquor concentration in the aeration tank (Figure 6.18).

Such circumstances will clearly leave more sludge in the clarifier (Figure 6.19), so that the risk of sludge overflow is greater (Figure 6.20).

In summary, ratio control of return sludge rate could achieve a better performance in effluent mass loadings, and a good maintenance of the sludge in the aeration tank during storm events, but it could lead to deterioration of effluent SS and an increased risk of sludge overflow.

6.4.1.2 Sludge Age Control (strategies 4 to 6)

The effect of sludge age control through the wastage of sludge (Q_W) during the storm event does not prove to be a better strategy than base-case control in terms of effluent water quality indicators such as ammonium, nitrite, nitrate, and ortho-phosphate-P (Table 6.1).



Figure 6.21 Ammonium Concentration in the Secondary Clarifier Effluent



Figure 6.22 Nitrosomonas Concentration in the Inner Upper CSTR

Also the benefit of implementing different sludge age controls is not obvious. Figure 6.21 shows the ammonium concentration in the secondary clarifier effluent with different sludge ages. Figure 6.22 plots out Nitrosomonas concentration in the inner upper CSTR of the aeration tank. Clearly, Nitrosomonas concentration stays unchanged with different sludge age control strategies.



Figure 6.23 Sludge Blanket Level in One Secondary Clarifier



Figure 6.24 Suspended Solid Concentration in One Secondary Clarifier Effluent

Perhaps the most desirable feature of sludge age control for this case study is that sludge blanket level (Figure 6.23) and effluent suspended solids concentration (Figure 6.24) are lower compared to the base case, the reason being more sludge is wasted during the storm event compared to zero wastage in the base case.

In summary, sludge age control has little effect during a single rain event. However, it is effective in preventing sludge overflow and lowering effluent solids. The slow responses of the activated sludge system to wastage rate determine that it may be more suitable for long-term control (Vacari et al., 1989)

6.4.2 Feedback Control

6.4.2.1 Feedback Control of Q_R (strategies 7 to 9)

A simple feedback controller is used in this algorithm, and it can in general be concluded that feedback control of Q_R is not good at handling transient influent shocks. The reason is that for a feedback controller, performance error has to be detected first before control actions are implemented and such action is therefore applied too late to improve the performance of the activated sludge process.

Under this algorithm both nitrification and the effluent ortho-phosphate-P concentration are worse than for the base case, with some improvement in effluent SS and TOC concentration. Since DO concentration in the inner channel increases (Figure 6.25), elevated carbonaceous substance removal becomes possible. However, the elevated DO concentration in the inner channel is not sufficient to improve nitrification. The biomass concentrations for all three strategies are significantly reduced during the storm event although they recover thereafter (Figure 6.26). It seems that the feedback controller for Q_R tries to improve the effluent water quality at the expense of process stability. Therefore, feedback control of Q_R may not be a good control in the sense that effluent performance might be deteriorated.



Figure 6.25 DO Concentration in the Inner Upper Channel



Figure 6.26 MLSS Concentration in Inner Upper Channel

6.4.2.2 Feedback Control of Sludge Wastage Rate (strategies 10 to 12)

Compared to feedback control of Q_R , feedback control of Q_W achieves slightly better effluent performance in terms of maximum effluent water quality indicators and average effluent mass loadings. However, if compared to sludge age control strategies, feedback control of Q_w achieves the same control effect (Table 6.1).



Figure 6.27 Phosphorus-Accumulating Bacteria Concentration in Outer Upper Channel

The big surge between day 3 and day 6 in the base case is due to the fact that during that period of time influent flow was fed exclusively into the inner channel with only the returned sludge being fed into the outer channel.



Figure 6.28 Nitrobacter Concentration in Middle Upper Channel

Also noted is that biomass concentrations in each channel stay unchanged by implementing strategy 10-12. For example, Figure 6.27 shows phosphorus-accumulating bacteria concentrations in the outer upper CSTR, and Figure 6.28 shows Nitrobacter concentration in the middle upper CSTR.

6.4.3 Advance Control

All three CSTR elements for the aeration tank are used for step-sludge and stepfeed controls and their influences on system performance are now described.

6.4.3.1 Step Sludge (strategies 13 to 15)

In this algorithm, the returned activated sludge is split between three channels. With more and more sludge sent to the inner channel of the aeration tank, obvious deterioration of effluent quality in terms of ammonium is seen in Figure 6.29 with the simulation window starting right before the storm event. This is due to the fact that less Nitrosomonas is available to decompose ammonium. For example, Nitrosomonas concentration in the outer upper and bottom CSTRs are plotted out in Figure 6.30 and Figure 6.31 respectively. Note also that ammonium concentration is consistently higher than for the base case, even after the storm event subsides.



Figure 6.29 Ammonium Concentration in Secondary Clarifier Effluent



Figure 6.30 Nitrosomonas Concentration in Outer Upper CSTR



Figure 6.31 Nitrosomonas Concentration in Outer Bottom CSTR

Since more sludge is stored in the secondary clarifier, and with more sludge distributed into the inner channel, the sludge blanket thus builds up steadily as seen in Figure 6.32.



Figure 6.32 Sludge Blanket Level in Secondary Clarifier Effluent

In summary, step-sludge control can easily lead to sludge buildup if more sludge is distributed closer to the outlet of the aeration tank, so that it is not good for storm event control. So the closer returned sludge is kept to the head of aeration tank during the storm event, the better it is for the system performance. Step-sludge control is advantageous when the system is oxygen-limited at the inlet stage.

6.4.3.2 Step Feed (strategies 16 to 19)

As far as maximum effluent quality is concerned, step-feed configurations (strategies 16 to strategy 18) achieve worse effluent ammonium, nitrite and orthophosphate-P concentrations relative to the base case even thought none of them exceed effluent standards. Figure 6.33 shows ortho-phosphate-P concentration in the secondary clarifier effluent. The maximum ortho-phosphate-P concentration found during the storm event is higher for strategies 16 to 18 than for the base case. The reason is that in the base case only half of the influent was routed through the aeration tank No.2.



Figure 6.33 Ortho-Phosphate-P Concentration in Secondary Clarifier Effluent as a Result of Step Feed Control

In consideration that in case there is no tank for temporary storage of storm flow during the storm event, one more simulation is then run to find out the maximum water quality if all influent flow is put into the inner channel without diversion (strategy 19 in Table 6.1). The results show that step-feed control evidently improves the effluent water quality compared to strategy 19.

An attractive feature of step-feed control is that the sludge blanket level is properly kept at a safe level such that sludge overflow would not take place. Such behavior is highly desirable since it gives the system a better ability to withstand the effects of more intense and frequent storm events. Figure 6.34 shows the behavior of the sludge blanket level with step-feed control.



Figure 6.34 Sludge Blanket Level in Secondary Clarifier As A Result of Step Feed Control



Figure 6.35 Ortho-phosphate-P Concentration in the Crude Sewage and Outer Channel (Base Case)

DO concentration goes up towards the end of the bioreactor during a major storm and ortho-phosphate-P is released upstream where the MLSS is being stored as a consequence of the step-feed strategy being practiced at the Athens Wastewater Treatment Facility No.2 as seen in Figure 6.35, where the curve in red is for the base case. Obviously, ortho-phosphate-P concentration is higher in the outer channel than in the crude sewage during the storm event, which is presumed to result from the effect of ortho-phosphate-P release. This is certainly an operational complication of an otherwise a promising strategy. However, this complication might be attenuated by keeping sufficiently high DO concentration upstream during the storm events.

To summarize, step-feed manipulation is a promising strategy for storm event control due to the fact that it not only improves effluent performance (even though only effluent ortho-phosphate-P was shown in Figure 6.33), but also controls sludge blanket level at a safe level.

6.4.4 DO Set-Point Control

For the base case the numbers of operating aerator units in the outer, middle, and inner channel during the rain event are 10, 2, and 2 respectively. Figures 6.36-6.38 show the number of operating aerator units needed in the outer, middle, and inner channel respectively under various control strategies.



Figure 6.36 Number of Aerators in Outer Channel Under DO Set-point Control



Figure 6.37 Number of Aerators in Middle Channel Under DO Set-point Control



Figure 6.38 Number of Aerators in Inner Channel Under DO Set-point Control

Obviously, in order to have a high DO set-point any more aerators are needed. Effects of the various control strategies on effluent ammonium concentration are plotted out in Figure 6.39. Figure 6.40 shows DO concentration in the inner channel under various DO set-point control strategies.



Figure 6.39 Ammonium Concentration in the Secondary Clarifier Effluent



Figure 6.40 DO Concentration in the Inner Channel

By running more aerators during the storm event, effluent quality, for instance, ammonium can be improved to a certain extent, but obviously not significantly better than for the base case (Figure 6.39). But these many aerators needed do not actually exist in the treatment plant itself. Therefore, DO set-point control has to be balanced between effluent quality and economic expenditures.

No. of	Max	Maximum Concentration During the Storm Event (mg/L)						
Strategy	ammonium	nitrite	nitrate	TOC	ortho-phosphate-P	SS		
effluent standard	17.0	0.2	2.5	30.0	2.0	30.0		
0	5.91E+0	1.10E-1	3.01E+0	1.64E+1	3.51E-1	8.52E+0		
1	6.14E+0	1.22E-1	3.02E+0	1.02E+1	3.81E-1	6.55E+0		
2	6.00E+0	1.21E-1	3.02E+0	8.76E+0	3.79E-1	8.73E+0		
3	5.95E+0	1.23E-1	3.02E+0	8.12E+0	3.79E-1	1.04E+1		
4	6.05E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0		
5	6.06E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0		
6	6.06E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0		
7	6.07E+0	1.21E-1	3.02E+0	9.14E+0	3.79E-1	7.53E+0		
8	6.07E+0	1.21E-1	3.02E+0	9.97E+0	3.79E-1	6.82E+0		
9	6.07E+0	1.21E-1	3.02E+0	9.79E+0	3.79E-1	6.95E+0		

Table 6.1 Assessment of Operational Control Strategies of

 Activated Sludge Process During Storm Event

10	6.07E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0
11	6.07E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0
12	6.07E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0
13	5.78E+0	1.50E-1	3.02E+0	9.78E+0	3.79E-1	7.02E+0
14	5.83E+0	1.53E-1	3.02E+0	1.06E+1	3.79E-1	7.12E+0
15	5.98E+00	1.61E-1	3.02E+0	1.38E+1	3.78E-1	7.66E+0
16	6.07E+0	1.21E-1	3.02E+0	9.84E+0	3.79E-1	6.94E+0
17	6.25E+0	1.13E-1	3.02E+0	1.06E+1	3.75E-1	6.80E+0
18	6.73E+0	1.11E-1	3.01E+0	1.67E+1	3.69E-1	6.26E+0
19	7.38E+0	1.10E-1	3.01E+0	2.41E+1	3.64E-1	5.29E+0
20	6.13E+0	1.18E-1	3.02E+0	1.05E+1	3.79E-1	6.94E+0
21	6.11E+0	1.20E-1	3.02E+0	1.01E+1	3.79E-1	6.94E+0
22	6.01E+0	1.33E-1	3.02E+0	9.30E+0	3.79E-1	6.94E+0

No. of	Average Effluent Loadings (Kg/h)					
Strategy	ammonium	nitrite	nitrate	TOC	ortho-phosphate-P	SS
0	1.40E+0	2.89E-2	7.91E-1	2.22E+00	8.87E-2	2.16E+0
1	1.35E+0	3.46E-2	6.53E-1	1.21E+0	7.99E-2	1.82E+0
2	1.29E+0	3.49E-2	7.08E-1	1.06E+0	8.12E-2	2.30E+0
3	1.27E+0	3.47E-2	7.36E-1	9.90E-1	8.24E-2	2.63E+0
4	1.31E+0	3.41E-2	6.58E-1	1.15E+0	7.88E-2	1.87E+0
5	1.32E+0	3.42E-2	6.61E-1	1.15E+0	7.92E-2	1.88E+0
6	1.33E+0	3.45E-2	6.68E-1	1.16E+0	8.00E-2	1.90E+0
7	1.31E+0	3.49E-2	6.91E-1	1.10E+0	8.07E-2	2.04E+0
8	1.34E+0	3.46E-2	6.66E-1	1.18E+0	8.02E-2	1.89E+0
9	1.33E+0	3.47E-2	6.71E-1	1.16E+0	8.03E-2	1.92E+0
10	1.32E+0	3.44E-2	6.65E-1	1.16E+0	7.96E-2	1.89E+0
11	1.30E+0	3.38E-2	6.54E-1	1.13E+0	7.84E-2	1.85E+0
12	1.32E+0	3.43E-2	6.64E-1	1.15E+0	7.96E-2	1.89E+0
13	1.28E+0	4.02E-2	7.03E-1	1.15E+0	8.04E-2	1.93E+0
14	1.36E+0	4.31E-2	6.57E-1	1.27E+0	8.08E-2	1.96E+0
15	1.61E+0	4.39E-2	5.59E-1	1.84E+0	8.22E-2	2.07E+0
16	1.33E+0	3.47E-2	6.71E-1	1.17E+0	8.03E-2	1.90E+0
17	1.30E+0	2.94E-2	7.19E-1	1.39E+0	7.96E-2	1.85E+0
18	1.43E+0	2.49E-2	7.11E-1	2.24E+0	7.86E-2	1.70E+0
19	1.72E+0	2.28E-2	5.72E-1	3.27E+0	7.70E-2	1.40E+0
20	1.41E+0	3.23E-2	6.22E-1	1.25E+0	8.04E-2	1.90E+0
21	1.38E+0	3.42E-2	6.41E-1	1.21E+0	8.03E-2	1.90E+0
22	1.28E+0	3.86E-2	7.04E-1	1.11E+0	8.02E-2	1.90E+0

6.4.5 Evaluation of Control Strategies (Strategy 1-Strategy 22)

All of the 22 control strategies have thus been tested. We are now in a position to compare them and pick out the best and worst among them, excluding strategy 19, which is an extreme case and is introduced to support the discussion of Section 6.4.3.2. Table 6.2 lists the best and worst strategies as far as maximum nutrient concentrations in the

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secondary clarifier are concerned. Table 6.3 lists the best and the worst strategy as far as average effluent loadings are concerned.

	ammonium	nitrite	nitrate	TOC	ortho-	SS
					phosphate-P	
best strategy	No.13	No.18		No.3	No.18	No.18
(No.)						
worst strategy	No.18	No.15		No.18	No.1	No.3
(No.)						
difference	14%	46%		2%	9%	22%
relative to base						
case for worst						
strategy (%)						
over effluent	No	No		No	No	No
standards for						
worst strategy?						

Table 6.2 Comparison of Control Strategies Based on Maximum

 Nutrient Concentration in the Secondary Clarifier Effluent

Table 6.3 Comparison of Control Strategies Based on Average

 Effluent Loadings in the Secondary Clarifier Effluent

	ammonium	nitrite	nitrate	TOC	ortho-	SS
					phosphate-P	
best strategy (No.)	No.3	No.18	No.18	No.3	No.18	No.18
worst strategy (No.)	No.18	No.15	No.3	No.18	No.3	No.3
difference relative to base base for worst strategy (%)	2%	52%	7%	1%	7%	22%

As far as maximum effluent ammonium concentration is concerned (Table 6.2), strategy 18 (step-feed control) is ranked the worst (6.8 mg/L) because it is 14% higher than that in the base case (6.0mg/L). However, the effluent standard for ammonium is 17 mg/L. So even in the worst case, effluent ammonium concentration is not even half-way close to the effluent standard. As far as average effluent loadings are concerned (Table

6.3), strategy 18 is ranked worst for ammonium and TOC. However, these responses are only 2% and 1% respectively higher than the base case, which is not significant. Conversely, strategy 3 performs best in terms of average effluent loadings for ammonium and TOC. Nevertheless, more energy is consumed to achieve the positive results of this strategy. As seen in Figure 6.41, strategy 3 has a much higher return sludge flow rate during the storm than in the base case, which indicates more energy consumption in terms of pumping costs. Implementing step-feed control, however, does not require much extra energy input. Moreover, step-feed successfully prevents sludge overflow, which has been a significant concern in stormwater control.



Figure 6.41 Comparison of Return Sludge Flow

To conclude the discussion above, strategy 18 is judge to be the best strategy for rain event control, from both the perspective of effluent quality and economic considerations.

6.5 FURTHER CONSIDERATIONS

6.5.1 Effect of Alternative Effective Mixing Volume

In this section, more simulations will be run based on the best strategy (strategy 18 in Table 6.1) to explore the impact of alternative equally effective mixing volumes in each channel (from 50% effective mixing to approaching 100% effective mixing) on the system performance. The following three options will be tested (Table 6.4). The relevant statistics are computed in Table 6.5. In Table 6.5 'case 0' refers to the base case.

	Outer Channel		Middle	Channel	Inner Channel	
	Effective	Non-	Effective Non-		Effective	Non-
	Mixing	Effective	Mixing	Effective	Mixing	Effective
	(%)	Mixing	(%)	Mixing	(%)	Mixing
		(%)		(%)		(%)
Case 1	50	50	50	50	50	50
Case 2	80	20	80	20	80	20
Case 3	95	5	95	5	95	5

Table 6.4 Test Cases for Impact of Alternative Effective Mixing Volumes

Evidently by having ever larger effective mixing volumes in each channel, effluent water quality in terms of ammonium is significantly improved, and significantly better than strategy 18 in Table 6.1. To accompany this argument, Figure 6.42 shows ammonium concentration in the outer channel; Figure 6.43 is the DO concentration at the same location; and Figure 6.44 gives the corresponding Nitrosomonas concentration. Effluent water quality in terms of other nutrient concentrations is improved as well, except for nitrate. The reason might be due to the fact that elevated DO concentrations result in more nitrite being transformed to nitrate.



Figure 6.42 Ammonium Concentration in the Outer Channel



Figure 6.43 DO Concentration in the Outer Channel



Figure 6.44 Nitrosomonas Concentration in the Outer Channel

6.5.2 Effect of One Layer Model Structure

Further to cases 1-3, one more simulation will be run here to test out the best strategy (strategy 18 in Table 6.1) with no flow routed through the base of the channel (the lower set of three CSTRs), i.e., with all of the volume of the actual tank assigned to the upper CSTRs. The statistics of the model results are listed in Table 6.5 as case 4. Clearly, effluent water quality in terms of ammonium is further improved to a small extent, but the system performance in terms of nitrite and nitrate has deteriorated compared to case 3.

In order to achieve good nitrification in the case study plant, large effective mixing volume proves to be an extremely important issue. The revised two-layer model makes evaluation of system performance under better effective mixing volume possible. Some form of deliberate mixing control might cure this plant's inability to achieve good nitrification, such as using diffused air at the bottom of the tank to increase effective mixing volume and resuspension. Maier and Krauth (1998) reported that surface aeration systems did not nitrify as well as diffused systems due to DO shortage. Unfortunately,

this option cannot be tested out in the current model structure, because it is assumed in the model that the bottom layers are anoxic. If oxygen input is exerted into the bottom CSTRs, they become oxic and therefore the biochemical mechanisms in the bottom layers need to be changed completely accordingly.

No. of	Maximum Concentration During the Storm Event (mg/L)						
Case	ammonium	ammonium Nitrite nitrate TOC ortho-phosphate-P				SS	
effluent standard	17.0	0.2	2.5	30.0	2.0	30.0	
0	5.91E+0	1.10E-1	3.01E+0	1.64E+1	3.51E-1	8.52E+0	
1	4.84E+0	8.15E-2	4.82E+0	1.52E+1	3.57E-1	6.30E+0	
2	3.91E+0	8.37E-2	6.25E+0	1.39E+1	3.47E-1	6.33E+0	
3	3.67E+0	8.57E-2	6.59E+0	1.35E+1	3.44E-1	6.35E+0	
4	3.60E+0	8.68E-2	6.69E+0	1.35E+1	3.43E-1	6.35E+0	

 Table 6.5 Assessment of Further Test Cases

No.	Average Effluent Loadings (Kg/h)							
of	ammonium nitrite nitrate TOC (Ortho-phosphate-P	SS		
Case								
0	1.40E+0	2.89E-02	7.91E-01	2.22E+00	8.87E-2	2.16E+0		
1	1.09E+0	2.33E-2	9.62E-1	2.00E+0	7.89E-2	1.69E+0		
2	8.73E-1	2.30E-2	1.22E+0	1.78E+0	7.98E-2	1.67E+0		
3	8.21E-1	2.31E-2	1.31E+0	1.72E+0	8.00E-2	1.66E+0		
4	8.07E-1	2.33E-2	1.34E+0	1.71E+0	8.00E-2	1.66E+0		

6.6 CONCLUSIONS

The results of a detailed assessment of control strategies for the activated sludge process have been presented in this Chapter. The controlled event is a short-term event with storm runoff as the chosen example. The following conclusions can be drawn from these results:

- Of the tested strategies, the advanced controllers are found to be more powerful than the "conventional" and "feedback" ones for controlling the deleterious effects of the rain events;
- Of the tested control variables, step-feed can be declared to be the best for rain-event control;

Great effective mixing volume is essential for good nitrification.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 INTRODUCTION

This very last chapter is comprised of two parts. Firstly, the conclusions from major sections of the dissertation are presented. Secondly, recommendations for future work are presented.

7.2 CONCLUSIONS

Throughout the program of research presented attempts have been made to address the problems of modeling and operational control of the activated sludge process. The following is concluded from the results of the study:

- The scope and quality of the some 60-day data acquired from the Athens Wastewater Treatment Facility No.2 with the EPCL are found to be exceptional, compared to what has been reported in the literature, and thus provides a solid base for subsequent model and control strategy development;
- A dynamic multiple species model has been developed for simulating nitrogen, carbon, and ortho-phosphate-P removal in the activated sludge system. Significant contributions have been made in the characterization of solute and particulate transport in the activated sludge process, and simultaneous nitrification and denitrification in the aerated zones;

- The constituent hypotheses of the proposed model have first been tested against a 19day period data from the experiment at the treatment plant. The general conclusion of this assessment is that the model, as a whole, is acceptable for simulating the behavior of the activated sludge process. From sensitivity analyses of the model parameters, especially those associated with the hydraulic transport component, it is found that the chosen values for these parameters are appropriate;
- Following this preliminary assessment of its structure, the model has been validated against another set of data, covering some 18 days. Evaluation results demonstrate the model's capacity to simulate the system performance, particularly under extreme flow conditions;
- Using the model thus identified, a detailed analysis of control strategies for regulating the effects of storm events has been undertaken. In general, it is found that,
 - Of the tested strategies, the advanced controllers are found to be more powerful than the "conventional" and "feedback" ones for the rain events;
 - Of the tested control variables, step-feed can be declared to be the best for rain event control;
 - An effective mixing volume is essential to good nitrification.

7.3 RECOMMENDATIONS FOR FUTURE WORK

Based on the results of this study, the following recommendations for future work are made:

□ From the perspective of data collection, detailed information on aerator operation and sludge wastage processing should be recorded for more accurate model results;

- □ A tracer study should be undertaken to confirm the current 2-layer model configuration of the aeration tank;
- Detailed and sophisticated analyses based on the Kalman filtering-smoothing algorithms would also be helpful to confirm that it is essential to reconstruct the model into a 2-layer configuration. Using the signal extraction algorithms to separate trend and noise components from a periodic component, incisive insights into the two fundamental aspects of bioreactor reactor behavior, its mixing and biomass activities, together with their variation with time, could thereby be exposed;
- □ Improvement of the proposed model in terms of biological removal of orthophosphate-P is very desirable with increasing knowledge of this process in the future;
- For the proposed dynamic model, further refinement of the clarification and thickening sub-models is essential if any improvement in simulation performance is required. More detailed exploration of biochemical reactions during clarification, thickening and sludge recycle, will be desirable.
- □ There is scope for the wider application of formal search techniques for model parameter estimation, including in particular the increasing popular methods of controlled random search.

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APPENDIX I

QA/QC PROCEDURES FOR THE MONITORS

1. DATA DOWNLOADING

On a daily basis fresh data (detailed data & daily data) have to be FTPed from HP workstation onto the personal computer. Then data, especially detailed data for each individual instrument have to be checked for diagnostic purposes. Once malfunction of instrument is spotted from the diagnostic information, laboratory visit is necessary to correct the problem as soon as possible and then resume the normal operations.

2. NUMBERING OF THE TRAILERS AND INSTRUMENTS

The trailer housing ammonium monitor, TOC monitor, and suspended solids monitor is numbered as trailer 1, while the other one with orthophosphate monitor, NO_x monitor is numbered as trailer 2. All the instruments coming with EPCL, either on-board or out-board, are numbered in sequence as follows:

- □ Instrument #1: Respirometer 1 (trailer 1);
- □ Instrument #2: Data Logger 1 (trailer 1);
- □ Instrument #3: TOC/NH₃/ Low Suspended Solid Monitor (outboard);
- □ Instrument #4: Respirometer 2 (trailer 2);
- □ Instrument #5: Data Logger 2 (trailer 2);
- □ Instrument #6: PO₄ Monitor (trailer 2);
- □ Instrument #7: NO_x Monitor (trailer 2);
- □ Instrument #8: Sludge Blanket Level Sensor (outboard);
- □ Instrument #9: External Double DO and MLSS Sensors (outboard).

3. LOCATIONS OF SAMPLE FLOWS AND THEIR NUMBERING

During the comprehensive sample campaign from 1 February, 1998 to 6 April, 1998, the locations of sample flows directed to each trailer and the numbering are as follows:

Trailer 1

- Stream 1–Tail end of the outer channel in the aeration tank #2;
- Stream 2–Tail end of the middle channel in the aeration tank #2;
- Stream 3–Tail end of the inner channel in the aeration tank #2 before disposal into the secondary clarifier.

Trailer 2

- Stream 1–Tail end of the inner channel in the aeration tank #2;
- Stream 2–Tail end of the middle channel in the aeration tank #2;
- Stream 3–Final effluent of the secondary clarifier before its being discharged into the receiving water body.

4. DETAILED DATA CHECKING

The detailed data files have the extension of ".p*", where * represents the instrument number.

4.1 Respirometers

Check the "DO" plot to see if the cycles are doing exactly as it is set up. And also check to see if there is time gap between adjacent data files.


Figure 1. An Examplary 'absolute d.o." Curve of the Respirometer Detailed Data.

For example, if the respirometer is set up to run 11 DO cycles, one should be able to see that much cycles in every single 'absolute d.o.' plot as shown if Figure 1.

4.2 TOC/NH₃/ Low Suspended Solid Monitor

These three instruments are bundled together and numbered as instrument #3. Since all three instruments can be accessed with the same keypad on the face of ammonium monitor, if maintenance work is performed on one of the three instruments the operations of the other two have to be temporarily suspended.

4.2.1 Ammonium Monitor



Figure 2. Detailed Data of the Ammonium Monitor

Shown in Figure 2 is the diagnostic information for the ammonium monitor. One always has to make sure that 'Diagnostic pH' is over 11 such that all ammonium in the samples is liberated into ammonia gas to be measured by the ammonia probe. Also make sure that the 'Diagnostic temperature' stays constant at 40 degree centigrade.

The ammonium monitor performs self-calibration on two standard solutions on a regular basis before taking any samples, ensuring that any following sample analysis is sound. Calibration starts with the high standard solution (10 mg/L) and then goes to the low standard solution (1 mg/L). During the calibration cycle, ammonium concentration readings (mg/L) should stay constant and fairly close to the real standard solution concentrations as shown in Figure 2.

4.2.2 TOC Monitor



Figure 3. Detailed Data of the TOC Monitor

Figure 3 shows the diagnostic information for the TOC monitor. Two signals in this graph serve the diagnostic purposes. One is 'Oxidisable C (mg/l)', which is the concentration of carbon that can be oxidized by a very strong oxidant. The other signal is 'rip TOC (voltage)'. This monitor is calibrated against two standard solutions. One is deionized water (0 mg/L), the other solution has a carbon concentration of 200 mg/L. So the readings of "oxidisable C" should stay fairly constant at 0 and 200 mg/L. In the "rip TOC" curve the "flat plane" near the end should stay stable.

4.3 External Double DO and MLSS Sensors

As shown in Figure 4, the 'raw sat. DO' signal should be over 80% saturation, and the 'autocal. DO' signal should be automatically adjusted to somewhere around 100% saturation.



Figure 4. Detailed Data of the Double DO Sensors

5. DAILY DATA CHECKING

5.1 Respirometers



Figure 5. 'Stream Number' Plot of Respirometer Daily Data

The "stream number" plot as shown in Figure 5 is used to check whether there is problem in flow switching due to blockage in any of the lines. If stream 1 is the current sample flow, the curve should stay at 1.0. Likewise if stream 3 is the current sample flow, the curve should stay at 3.0. If the curve stays at levels other than 1 or 3 when it should be, blockage in sample lines needs to be cleared.





Figure 6. 'Amm Delta Cal' Plot of Ammonium Monitor Daily Data

As shown in Figure 6, the "ammonia delta cal" signal should be consistently over 50 mV.

5.3 Orthophosphate Monitor



Figure 7. Orthophosphate Monitor Daily Data for Diagnostic Purposes

The 'Last PO₄ Std. V', 'Last PO₄ 0% T', 'Last PO₄ Air V', 'Last PO₄ 100% T' readings as shown in Figure 7 should stay relatively constant.



5.4 NO_x Monitor

Figure 8. NO_x Monitor Daily Data for Diagnostic Purposes

The 'Last NO₂ Std. V', 'Last TON Std V', 'Last NO_x 0% T', 'Last NO_x Air V', 'Last NO_x 100% T' readings as shown in Figure 8 should stay relatively constant.

APPENDIX II

DAILY MAINTENANCE CHECKLIST OF THE INSTRUMENTS IN EPCL

1. SAMPLING SYSTEM

□ Sampling collection & pumping

Rinse the collection drums with high-pressure hose daily.

- Observe sample loop condition and monitor discharge rates daily
- □ Ultrafilters
 - Avoid buildup of debris in ultrafilter casings. When debris buildup is spotted, stop the line and take the ultrafilter out for a clean;

Note: Avoid regular purging of sample loops with compressed air. Try to avoid full-scale air blasts, since the ultrafilters are delicate.

• Monitor and record the suction pressure on each filtrate line & intervene before the pressures become excessive.

2. RESPIROMETERS

- □ Wash respirometer tanks to remove excess fouling inside the walls;
- □ Keep the float switches as clean as possible. Better test the overflow level switches periodically;
- Observe the filling rate of sample into the tank;
- □ Prevent mixer problems by cleaning any debris collected on the impeller;
- □ Clean and check the pH probe periodically.

3. TOC MONITOR

- □ Check reagents & standard solutions regularly, especially on Fridays;
- □ Look inside the enclosure for water leakages, which may indicate pump tube failures;
- □ Check deflection of "U" tube water.

4. NH₃ MONITOR

- □ Check reagents & standard solutions regularly, especially on Fridays;
- □ Check the free flow of sample, indicated by the ratio of air and sample in scavenge pump inlet tube;
- □ Pulsing of sample feed tube is indicative of sample starvation;
- □ Look in tray for evidence of pump tube failure.

5. TURBIDITY MONITOR

- Check that the sample delivery line has no entrained air;
- □ Clean the debubbler with large bottle brush.

6. PO₄ MONITOR

- □ Visually inspect for free delivery of sample & reagents;
- Check dilution water supply and replenish it if necessary;
- Check condition of deionizer cartridge.

7. NO_X MONITOR

The same as those listed for PO_{4} . Other than that, watch Cd column performance and replace it when necessary.

8. EXTERNAL DOUBLE D.O PROBES

Inspect daily for gross fouling.

Note: Try to avoid over-maintaining, but should not neglect it either.

9. EXTERNAL MLSS PROBE

The same as those listed for external double DO probes. Other than that, calibrate the external MLSS probe on a weekly basis.

10. SLUDGE BLANKET LEVEL SENSOR

Calibrate and inspect head of the sensor weekly. Make sure that the head is clean.