PROCESS MODELING AND ENERGY OPTIMIZATION OF A FULL SCALE ADVANCED BIOLOGICAL WASTEWATER TREATMENT PLANT IN ISTANBUL

by

Gıyasettin GÜNEŞ

BS. in Env.E. Niğde University, 1999

M.S. in Env.Sc., Boğaziçi University, 2005

Submitted to the Institute of Environmental Sciences in partial fulfillment of

the requirements for the degree of

Doctor

of

Philosophy

Boğaziçi University

2023

ACKNOWLEDGEMENTS

It was a great pleasure and privilege for me to have two advisors: Prof. Dr. Ayşen Erdinçler and Prof. Dr. Güçlü İnsel. During the time I was writing this thesis, I had a great time talking with them about scientific and life-related topics in productive and satisfying ways. Under their supervision, I always felt confident about how I should find the right answer to the problems that I faced.

Here, I would like to thank all my professors and researchers, which are the staff of Boğaziçi University, Institute of Environmental Science. I appreciate their kind cooperation and valuable help.

ABSTRACT

PROCESS MODELING AND ENERGY OPTIMIZATION OF A FULL SCALE ADVANCED BIOLOGICAL WASTEWATER TREATMENT PLANT IN ISTANBUL

Keywords: Activated Sludge Model, Moving Bed Biofilm Reactor, Reactive Primary Clarifier, Energy Optimization, Wastewater Treatment.

When standard design methods are applied directly to projects, they can lead to serious design errors. Due to the low nitrification growth rate in Istanbul, treatment plants generally cannot meet total nitrogen discharge standards. During the study, an innovative treatment process was discovered. The pilot plant with "Biofilm Nitrification - Contact Denitrification System" and the real plant were operated simultaneously and their performances were compared. In the pilot plant, the return sludge was sent to the reactive pre-sedimentation tank and 80% of the organic matter was removed without the use of chemicals. Another advantage of this system is that nitrification in the hybrid system is less affected by inhibition because it occurs in a separate biofilm reactor. Nitrogen removal performance was also improved. Since the organic matter settled in the preliminary clarifier is sent directly to the denitrification tank, it is not exposed to aerobic conditions and therefore there is no loss of organic matter. Sending the organic matter first to nitrogen removal and then to biogas production is another advantage of this process. Modeling studies have shown that a higher amount of biogas can be obtained from anaerobic digestion of this sludge. The hybrid system was found to have lower operating costs (i.e., 9% less air and 39% less energy required for mixing). It also requires 40% less volume than other BNR systems, resulting in significantly lower capital costs. Therefore, the new hybrid configuration was found to be a more efficient and attractive option for treating wastewater.

ÖZET

İSTANBUL'DA TAM ÖLÇEKLİ BIR İLERİ BİYOLOJİK ATIKSU ARITMA TESİSİNİN PROSES MODELLEMESİ VE ENERJİ OPTİMİZASYONU

Anahtar Kelimeler: Aktif Çamur Modeli, Hareketli Yatak Biyofilm Reaktörü, Reaktif Ön Çökeltme Tankı, Enerji Optimizasyonu, Atıksu Arıtma.

Standart tasarım yöntemleri doğrudan projelere uygulandığında, ciddi tasarım hatalarına yol açabilirler. Genellikle, İstanbul'da düşük nitifikasyon büyüme oranı nedeniyle arıtma tesislerinde, toplam azot deşarj standardı karşılanamamaktadır. Çalışma sırasında yenilikçi bir arıtma prosesi keşfedilmiştir. "Biyofilm Nitrifikasyon - Temaslı Denitrifikasyon Sistemi "ne sahip pilot tesis ile gerçek tesis, eş zamanlı olarak çalıştırılmış ve performansları karşılaştırılmıştır. Pilot tesiste, geri devir çamuru reaktif ön çökeltme tankına gönderilmiş ve organik maddenin %80'i kimyasal kullanılmadan çöktürülebilmiştir. Bu sistemin diğer bir avantajı da hibrit sistemdeki nitrifikasyon prosesinin ayrı bir biyofilm reaktöründe gerçekleştiği için inhibisyondan daha az etkilenmesidir. İlave olarak, azot giderim performansınıda artırmıştır. Ön çökeltme tankında çöken organik madde doğrudan denitrifikasyon tankına gönderildiğinden, aerobik koşullara maruz kalmamakta, bu nedenle organik madde kaybı da olmamaktadır. Bu prosesin diğer bir avantajı da organik maddenin önce azot giderimine sonra da biyogaz üretimine gönderilmesidir. Modelleme çalışmaları, bu çamurun anaerobik çürütülmesinden daha yüksek miktarda biyogaz elde edilebileceğini göstermiştir. Hibrit sistemin daha düşük işletme maliyetlerine sahip olduğu tespit edilmiştir (yani %9 daha az hava ve %39 karıştırma enerjisi gereksinimi). Ayrıca, diğer BNR sistemlerine göre %40 daha düşük hacim gerektirdiğinden yatırım maliyetlerini de önemli ölçüde azaltacaktır. Bu nedenle, yeni hibrit konfigürasyonun atık su arıtımı için daha verimli ve cazip bir seçenek olduğu görülmüştür.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS/ABBREVIATIONS

$\mathbf{Y_H}$	: Heterotrophic yield coefficient
$\mathbf{Y_{NH}}$	: Net heterotrophic yield coefficient
$\hat{\mu}_H$	: Maximum heterotrophic growth rate
μ_{Amax}	: Maximum autotrophic growth rate
$\mathbf{k_h}$	: maximum hydrolysis rate
$\mathbf{K_S}$	: Half saturation constant for heterotrophic growth
$\mathbf{k_h}$	: Maximum hydrolysis rate
$\mathbf{K_x}$	: Half saturation constant for hydrolysis
$\mathbf{K_O}$	: Oxygen half saturation constant for heterotrophs
$\mathbf{K_{OA}}$	: Oxygen half saturation constant for autotrophs
$\mathbf{S_s}$	: Readily biodegradable COD
$\mathbf{X_s}$	: Slowly biodegradable COD
$\mathbf{C_S}$	: Biodegradable COD concentration (COD_{BD})
$\mathbf{b_H}$	: Endogenous decay rate for heterotrophs
$\mathbf{b_A}$	: Endogenous decay rate for autotrophs
SBR	: Sequencing Batch Reactor
T	: Temperature
$\mathbf{Q_{in}}$	: Influent Flowrate
$\mathbf{S_O}$	: Dissolved oxygen concentration
$\mathbf{S_O\text{-}sp}$	: Dissolved oxygen set-point concentration
$\mathbf{S_{O\infty f}}$	: Dissolved oxygen saturation concentration under process condition
$\mathbf{K_La}$	: Volumetric oxygen transfer coefficient
$\mathbf{K_La_f}$	: Volumetric oxygen transfer coefficient under process condition
α	: Correction factor for process water K_La_f / clean water K_La
β	: Correction factor for process water $S_{O\infty f}$ / clean water $S_O{}^*{}_{\infty f}$
θ_X	: Total Sludge Age
θ_{XA}	: Aerobic sludge age
θ_h	: Hydraulic retention time
θ	: Arrhenius coefficient (temperature correction)
$\mathbf{N_X}$	: Nitrogen incorporated during heterotrophic growth

$\mathbf{N_{Ox}}$	: Oxidized nitrogen
$\mathbf{N_{DP}}$	: Denitrification potential
$\mathbf{S_{NH}}$	: Ammonia nitrogen
$\mathbf{S_{ND}}$	: Soluble organic nitrogen
$\mathbf{S_{NO}}$	: Nitrate nitrogen
$\mathbf{X_{ND}}$	: Particulate biodegradable nitrogen
$\mathbf{S_{PO4}}$	: Ortho-phosphate
$\mathbf{X_{I}}$	: Particulate Inert COD
$\mathbf{S_{I}}$	: Soluble Inert COD
$\mathbf{S_{F}}$	: Fermentable COD
$\mathbf{S_{A}}$	: Acetate COD
$\mathbf{S_{H}}$	: Rapidly hyrolyzable COD
$\mathbf{X_{H}}$	: Active heterotrophic biomass
$\mathbf{i_{NBM}}$	: Nitrogen fraction of biomass
$\mathbf{i_{NX?}}$	: Nitrogen fraction of particulate COD, $X_?$
$\mathbf{i_{NS?}}$	: Nitrogen fraction of soluble COD, $S_?$
$\mathbf{i_{PX?}}$	: Phosphate fraction of particulate COD, $X_?$
$\mathbf{i_{PS?}}$	: Phosphate fraction of soluble COD

1 INTRODUCTION

As economies and technologies have improved, the amount of wastewater from cities gradually increases around the world. Effective treatment of this increasing amount of wastewater has become a major social and environmental concern. Suspended solids, microbial pathogens and parasites, toxic materials, recalcitrant materials , biodegradable organic substances, volatile organic substances, nutrients (nitrogen and phosphorus) are the main pollutants found in wastewater (Metcalf and Eddy, 2014).

The proper operation and discharge of wastewater treatment plants are essential for protecting the environment and public health. Therefore, a good knowledge of the complex biological reactions in activated sludge systems is required for the proper design, operation, and upgrading of activated sludge plants (Moretti et.al., 2011).

The efficiency of a process depends largely on the characteristics of the influent, environmental factors, and how operating conditions change over time. Generally, activated sludge systems are designed to treat wastewater under steady-state conditions, but in practice, the systems often experience dynamic conditions (i.e., fluctuating influent flowrate and pollution loads, temperature, side stream loads, etc.) that can have a negative impact on the treatment performance and effluent quality. So, reliable process design and control are becoming more and more important to make sure that wastewater treatment is sustainable and cost-effective in different environments. It's essential to use computer-based models to understand how these complex biological reactions and dynamic factors interact, so that activated sludge plants can be better designed and/or upgraded. Through detailed model-based studies, activated sludge plants can achieve more efficient removal of carbon, nitrogen, and phosphorus

Several standardized methods have been widely used for the engineering design of activated sludge systems, including the ASM (Activated Sludge Model) method and the ATV (German Association for Water, Wastewater and Waste) 131E method, among others (Insel et.al., 2012). ATV-DVWK-A 131E is a German standard for the design of single-stage activated sludge plants used in the design and construction of wastewater treatment plants. Water Utility Administrations and

Municipalities in our country commonly use the ATV 131 standard for designing wastewater. Here are some potential drawbacks of using this standard for designing WWTP.

- The direct application of these standardized methods to wastewater treatment plant projects can result in serious design errors because they do not reflect local conditions.
- Whenever the original design values do not fall within the proposed ranges, the results of the standardized methods and the results in real situations are different. There are design and operating variations between big and smaller wastewater treatment facilities, and these differences are more obvious in larger plants (Insel et.al., 2012).
- The ATV-DVWK-A 131E standard is specific to single-stage activated sludge plants, which may not be appropriate for all wastewater treatment applications. This may limit its applicability in certain situations.
- The standard is based on established design principles and may not encourage innovation or creativity in wastewater treatment plant design. This may result in designs that are not optimized for maximum efficiency or sustainability.

The most important design parameter in these standardized methods is BOD_5, which shows how much organic matter is in the water coming in. According to the standard design of the activated sludge system, the BOD5 parameter does not effectively describe the link between biomass growth and organic carbon removal with respect to oxygen consumption (Insel et.al., 2012). To overcome the disadvantages mentioned above, WWTP design via chemical oxygen demand (COD) has become standard practice with advances in activated sludge modeling. Chemical oxygen demand (COD) is a commonly used method to characterize the amount of carbonaceous material in wastewater, including organic compounds that can be biodegraded in an activated sludge process. The advantage of using COD over other parameters such as TOC (total organic carbon) or BOD_5 (biochemical oxygen demand) is that it provides a more consistent and reliable measure of the organic strength of the wastewater (Barker et al., 1997).

Activated Sludge Model (ASM) is a group of mathematical models used to describe the biological and chemical processes that occur in activated sludge systems. An International Water Association (IWA) task group is coordinating research in this area. The models were developed by researchers to simulate the complex interactions that occur in activated sludge systems and to better understand the processes involved in wastewater treatment. ASMs are common and advantageous applications for the design, operation, and upgrade of activated sludge systems for organic carbon and nutrient removal (Sözen et.al., 2008). These models show accurately how the system works. They need accurate model-based characterization of the wastewater influent (such as COD fractionation), dynamic model calibration and verification, and accurate interpretation of the modeling studies'' results. When carefully calibrated with reference data for sludge production and effluent nutrients, ASM can also be used to design, update and optimize full-scale wastewater treatment plants (Grady et al., 1986, Hulsbeek et.al., 2002)

Biological models should not be confused with simulation programs (simulators). In real life, model simulations are faster than trial-and-error methods for figuring out what will happen if different things happen in different operating or design conditions. A simulator is a computer program that combines the biological model with models of other unit operations and must connect the units of a particular system according to the flow scheme. Depending on the intended use of the simulator, information on the inputs, configuration, and operating conditions of the process is required (Wilson A.W., et al., 1998).

One of the objectives of this study is the design and operation of a wastewater treatment plant, which was designed and operated with standard design methods such as ATV-131E (2000), by means of model simulations. Then, the design methods of the ATV-131E and the model simulations were compared with the actual operating data of the plant. The comparison of the design methods and model simulations with actual plant operational data can provide valuable insights into the WWTP performance of the system.

Modeling a full-scale advanced biological wastewater treatment plant in Istanbul using a simulation tool (SUMO biological system simulator) is one of the objectives of this thesis. Thus, a mathematical method to combine knowledge of process dynamics will be developed. The simplest possible description of the carbonaceous and nitrogenous treatment activities in the existing process is estimated and identified with reasonable accuracy using this mathematical method.

The adoption of new technology to meet higher drinking water and wastewater quality requirements has resulted in a rise in energy consumption in the treatment industry during the past 20 years. This has resulted in increased operational costs, especially for electricity, which has become a significant factor in addition to the initial investment costs of municipal and industrial wastewater treatment plants. As a result, municipalities and the private sector are facing rising wastewater and sludge treatment costs as treatment requirements continue to increase. In particular, the high operating costs resulting from long periods of operation are a significant concern when it comes to choosing wastewater treatment technologies and ensuring energy efficiency. Therefore, there is a growing interest in developing innovative and cost-effective wastewater treatment processes that can reduce energy consumption and operational costs while still achieving high-quality effluent. The hybrid treatment process discussed earlier is an example of such an innovative approach that can significantly reduce operating costs, energy consumption, and capital costs, making it a more attractive option for wastewater treatment.

There are many different types of equipment used in wastewater treatment plants that either consume too little or too much energy. In general, energy consumption per equivalent population per year is used to evaluate the energy requirements of wastewater treatment plants. As a general average, treatment plants serving 5,000 to 100,000 equivalent populations require 40–20 kWh per person per year. With the right automation, monitoring inside the plant, use of process controls, and choice of energy-efficient equipment, each person can save between 4 and 7 kWh of energy per year. In fact, wastewater treatment plants can be turned into systems that produce more energy than they use (Nowak, 2003). This can be done with the right planning and technology choices.

The other objective of this thesis is to optimize the energy consumption of the selected advanced biological wastewater treatment plant. Due to the rising energy costs in Turkey, saving energy in wastewater treatment plants has become a crucial concern. This study offers a new, cost-effective, sustainable, and integrated approach to solve design and operational difficulties associated with wastewater treatment in a circular economy. The aim of this study is to provide wastewater utilities with valuable information as a resource to enhance the energy performance of their facilities through upgrade, expansion, or new construction projects.

In this study, experimental studies have been carried out at three wastewater treatment plants in Istanbul. The plant where the pilot and modelling studies were conducted will be referred to as "**the WWTP**" in the following sections of the thesis.

2 LITERATURE REVIEW

Domestic, agricultural, and industrial discharges containing nitrogen and phosphate are known to cause eutrophication in receiving water bodies, which can severely limit their potential uses due to excessive diurnal algal activity. To mitigate the risk of eutrophication, the European Union has enforced discharge standards through the EEC Directive 91/271 (CEC, 1991) with respect to total nitrogen and phosphate, especially in sensitive areas.

Activated sludge system design and operation may be optimized for efficient and sustainable nutrient removal. To do this, engineers must have a full grasp of the complex biological features of activated sludge systems, particularly in relation to environmental factors that may impact the design and operation of activated sludge facilities (Keller et.al., 2001).

Understanding the nature of wastewater is essential to the design and operation of collection, treatment, and reuse facilities and to the engineering management of environmental quality. Therefore, the characteristics of the influent wastewater must be well understood in order to design the processes correctly. In addition, the biological processes (nitrification and denitrification) that occur in wastewater treatment plants must be known in detail. Furthermore, it is essential to understand the activated sludge operating parameters that affect plant performance. The following is a brief overview of these subjects.

2.1 Influent wastewater characteristics

Wastewater is characterized by its physical, chemical, and biological composition. Wastewater characterization includes the separation of influent organic material into degradable and inert fractions, total influent nitrogen to ammonia and organic nitrogen, etc. From one municipal wastewater to another, influent characteristics can vary, often significantly. The characteristics of the wastewater have a very significant effect on the performance of the system, especially for nutrient removal systems. However, wastewater flows, and influent pollutant concentrations are not constant or uniform but vary from hour to hour, day to day, month to month, and year to year. Understanding wastewater flow rate and its variability, both today and in the future is critical when designing a treatment plant.

Based on data on wastewater, treatment plant designs can be made. If relevant measurements or data are unavailable, an estimation related to wastewater characteristics should be made. In order to analyze the wastewater and generated sludge in WWTPs, scientists frequently use a variety of analytical techniques. When characterizing wastewater, proper sampling and analytical techniques are essential. Numerous of these analytical techniques were made especially for the WWTP. According to Randall et al. (1992), the characteristics of influent wastewater, including the ratios of biodegradable COD, COD/TKN, and COD/TP, determine how effectively nutrients are removed by activated sludge. For instance, elimination of 1.0 mg NO_3-N and 1.0 mg PO_4-P needs, respectively, 6.0 mg biodegradable-COD and 10 mg VFA-COD, according to practical experiences (Grady et al., 1996).

A crucial step in modeling is accurate (model-based) influent characterization because it has a direct impact on how responsive the model output is and how important some processes are. In addition, If some pollution parameters are not present in the influent wastewater, simplifying the model might be a practical choice. For instance, there is no need for the fermentation step to be included in the wastewater treatment process, if the influent contains no fermentable COD.

According to APHA (1998), standard techniques are used for total/filtered COD, Total Kjeldahl Nitrogen (TKN), Total Phosphorus (TP), Total Suspended Solids (TSS), etc. in conventional wastewater characterization. However, due to the enormous advancements in biotechnology and the growing model complexity, more thorough model-based wastewater characterization is required. In most studies, conventional wastewater characterization is translated into model input, which can be considered as another sub-modeling process (Figure 2.1). In this process, dedicated physical, analytical methods (i.e., STOWA procedure: Roeleveld et al., 2002) together with widely accepted respirometric tests are used to determine the model inputs in terms of biodegradable/inert COD, nitrogen and phosphate fractionations and active biomass in the influent (Vanrolleghem *et al.*, 1999; Sperandio *et al.*, 2000; Sollfrank *et al.*, 1991; Ekama *et al.*, 1986b).

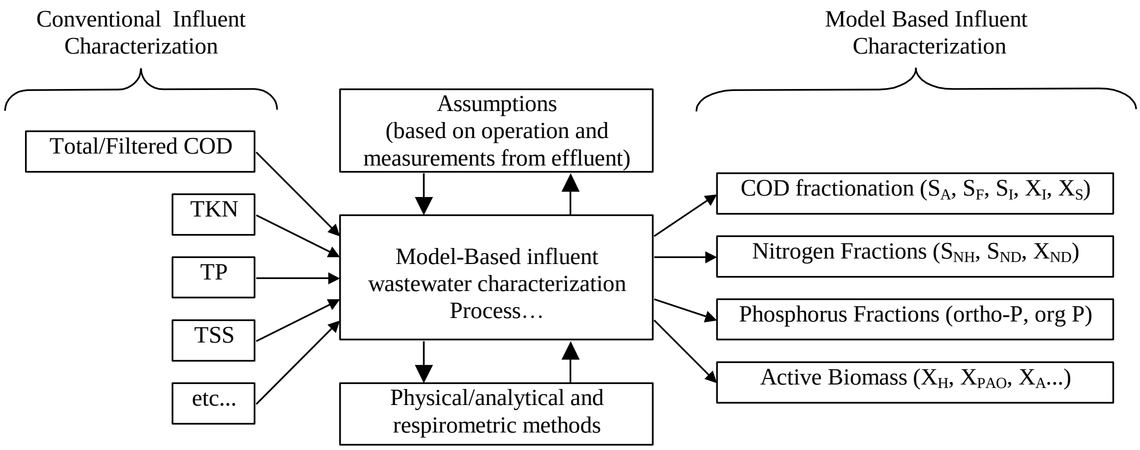


Figure 2.1. Characterization of inlet wastewater for the activated sludge (WERF, 1999).

Biodegradable COD (C_S) is divided into readily biodegradable COD (S_S) and slowly biodegradable COD (X_S). Readily biodegradable COD (S_S) is composed of acetate (VFA) COD (S_A) and fermentable COD (S_F). The X_S is hydrolyzed to fermentable COD (S_F) under anaerobic, anoxic and aerobic conditions. The fermentable COD (S_F) is then fermented to acetate COD (S_A) by heterotrophic activity under anaerobic conditions. It is important to note that in each ASM model, the fractions of COD and the processes in which these fractions are involved are different. Figure 2.2 shows an ASM2d-based COD fractionation.

Concentrations are often analyzed using both unfiltered and filtered effluent. In this regard, it is very important to draw attention to any confusion or lack of clarity in the reporting of "soluble" concentrations. In many cases, the term "soluble" in the context of wastewater treatment generally refers to material that passes through a 0.45 μm membrane filter (Grady et al., 1999). For the separation of soluble and particulate COD, membrane filters with a pore size of 0.45 m are commonly used. On the other hand, short-term or long-term respirometric methods are used for accurate estimation of readily and slowly biodegradable COD along with kinetic and stoichiometric model parameters (Figure 2.3).

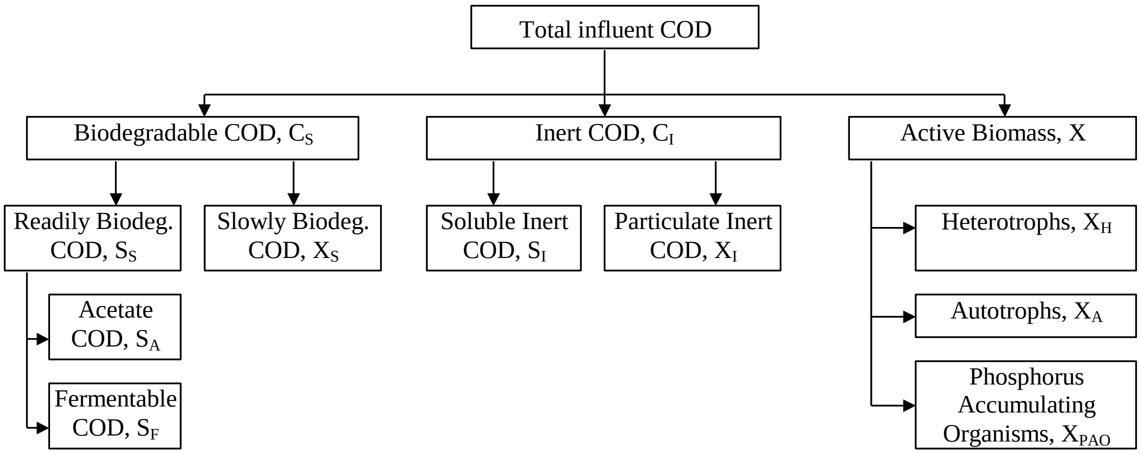


Figure 2.2. ASM2d based influent wastewater COD fractionation (Barker et.al., 1995)

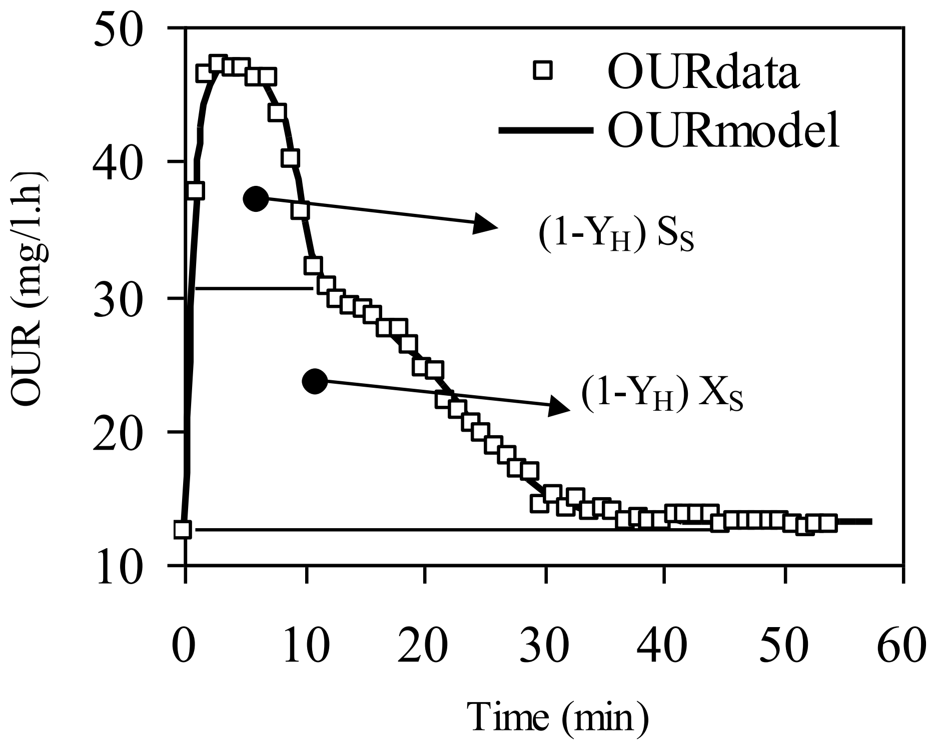


Figure 2.3. . Biodegradable COD fraction estimation using respirograms (Gujer et.al., 1995).

To simplify the model, assumptions can also be made that take into account the mode of operation of the activated sludge plant and the plant effluent analyses. For example, the rapidly hydrolyzable COD fraction (S_H) (Orhon et al., 1999a) can be incorporated as part of the slowly biodegradable form (X_S) if the system is operated in an extended aeration mode.

The active biomass fractions in influent wastewater are assumed to be negligible, and they can be added to the influent X_S according to Henze (1992). Alternately, the amount of heterotrophic biomass in wastewater can be measured by doing long-term respirometric tests on raw wastewater without adding any heterotrophic biomass (Sperandio et al., 2000).

The effluent soluble inert COD (S_I) is assumed to be equal to the influent inert soluble COD of the treatment plant if it is operated at an SRT at least higher than 3 days disregarding the inert soluble microbial products (S_P) due to endogenous biomass activity (Ekama *et al.*, 1986; Mamais *et al.*, 1993, Orhon *et al.*, 2001). In a practical way, 90% of the effluent COD can be regarded as the inert influent soluble COD (Siegrist et al., 1992) for domestic wastewater. The inert COD fractions can also be experimentally calculated via long-term COD measurements as described in Germirli *et al.* (1991). Another important issue is that the diurnal, weekly or seasonal changes in wastewater characteristics (load, water temperature, flow, rain and storm events etc.) should be considered if the actual performance of the system is to be evaluated (Metcalf and Eddy, 2003). Moreover, aside from modelling, the planning of sampling time and measurement frequency are of great importance in mimicking the actual behavior of the activated sludge plant (De Pauw *et al.,* 2004).

According to ATV-131E, one of the most widely known and used design standards in the world, the inflow COD ($C_{COD,IAT} = C_S$) of a biological reactor can be segregated into two fractions: soluble COD (S_S) and particulate COD (X_S). Each fraction comprises of biodegradable COD ($S_{COD,\ deg,\ IAT} = S_s$) and inert COD ($S_{COD,\ inert,IAT} = S_i$). The soluble inert COD fraction can be approximated to the soluble effluent concentration (ATV 131E, 2000).

The range of the inert soluble COD (S_i) is usually 0.05 to 0.1 times the inflow COD ($C_{COD,IAT} = C_S$). If data is not available, it is advised to approximate $S_{COD,inert,IAT}$ (S_i) as 0.05 times COD ($C_{COD,IAT} = C_S$) for municipal wastewater (ATV 131E, 2000).

The inert component of the particulate COD ($X_{COD,inert,IAT} = X_i$) can also be calculated as a fraction of the total particulate COD (Kayser, 2001). The value of A, which depends on the wastewater type and retention time in the primary settling tank, usually ranges from 0.2 to 0.35. For municipal wastewater, it is recommended to consider A as 0.25 (ATV 131E, 2000).

Considering the high sludge age, it can be assumed that both the biodegradable particulate substances ($X_{COD,deg,IAT}$) and the biodegradable soluble substances ($S_{COD,deg,IAT}$) have been completely converted. The slight increase in inert soluble COD as well as inorganic solids resulting from biodegradation are ignored for further consideration (ATV 131E, 2000). Figure 2.4 shows an ATV-131E based COD fractionation.

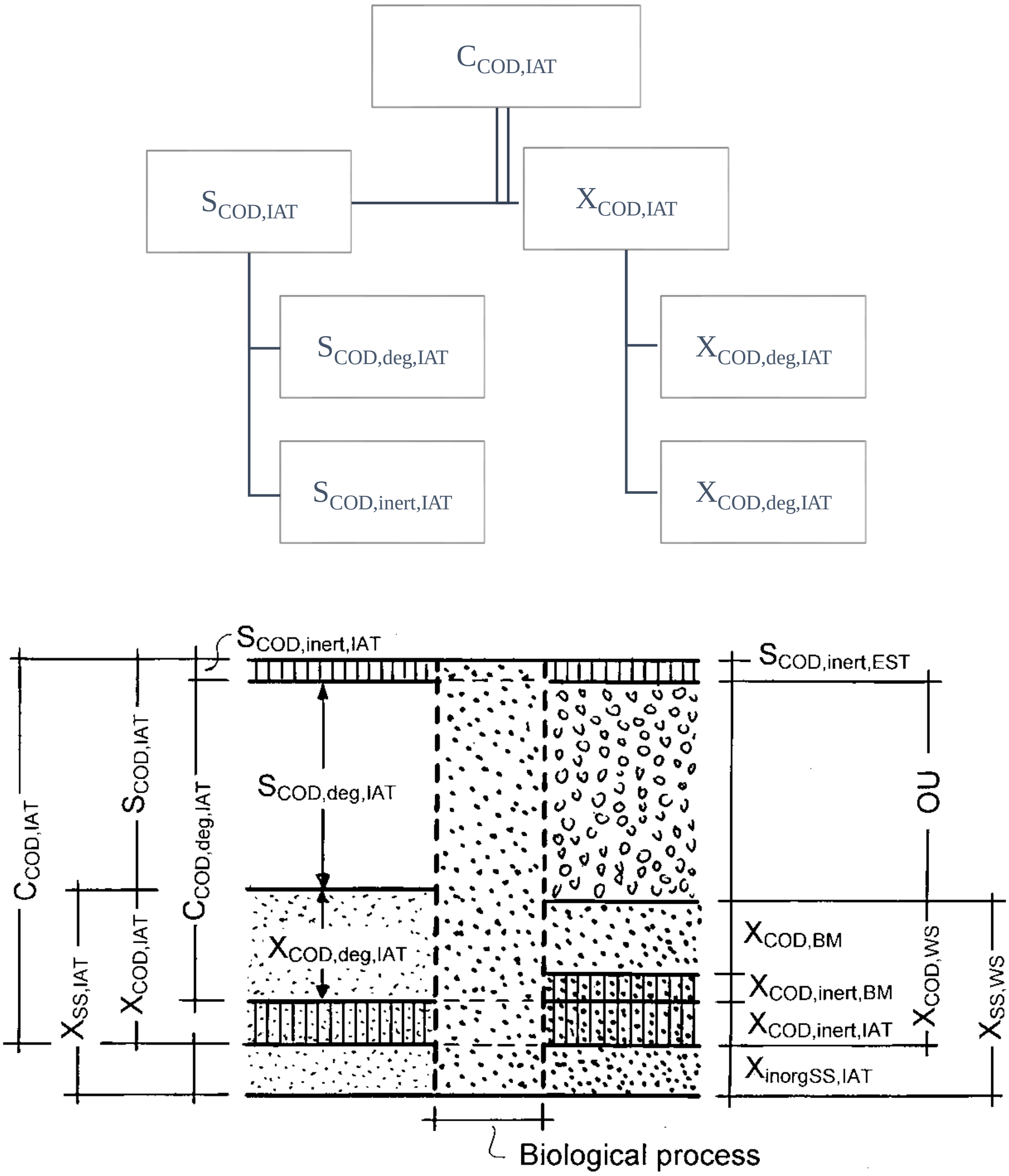


Figure 2.4. Influent wastewater characterization according to ATV-131E.

2.2 Biological Nutrient Removal

Biological Nutrient Removal (BNR) is a process used to remove nitrogen and phosphorus from wastewater before it is discharged to surface water or groundwater. The increase in the concentration of nutrient compounds (nitrogen and phosphorus) in the effluent from municipal wastewater treatment plants is the cause of eutrophication in surface waters. Summer algal blooms are a well-known example of eutrophication. They're a problem for ecosystems and humans. It results in the depletion of desirable flora and fauna due to the effects of a decrease in dissolved oxygen levels in the receiving water body, fish mortality, turbidity of the water, etc.

2.2.1 Nitrification

Two sequential reactions occur during nitrification. The first reaction is the oxidation of ammonia (NH_4-N) to nitrite (NO_2-N) by a group of bacteria known as Nitrosomonas, and the second reaction is the oxidation of nitrite (NO_2-N) to nitrate (NO_3-N) by another group of bacteria known as Nitrobacter. The oxidation of NH_4-N to NO_2-N requires 3.43 g O_2/g N_2 and the oxidation of NO_2-N to NO_3-N requires 1 to 1.3 gO_2/gN_2, based on the process stoichiometry. In summary, the complete oxidation of NH_4-N to NO_3-N consumes 4.57 g O_2/g N_2 (Metcalf and Eddy, 2003; Orhon et al., 1994).

An experimental respirogram that was obtained from a batch test that was initially fed with NH_4 is shown in Figure 2.5 (Petersen, 2000). Basically, the first plateau represents the oxidation of NH_4 to NO_2. The tailing of the second plateau indicates the end-point of NO_2 oxidation to NO_3 as a final step. However, it is sometimes difficult to observe the second plateau from the respirogram. The kinetics can often be simplified to one-step nitrification when NH_4 is oxidized directly to NO_3 (Petersen, 2000; Brouwer *et al.*, 1998). High nitrite concentrations in wastewater due to possible inhibition of the second step by environmental conditions have been reported in the literature (i.e., temperature, dissolved oxygen, pH).

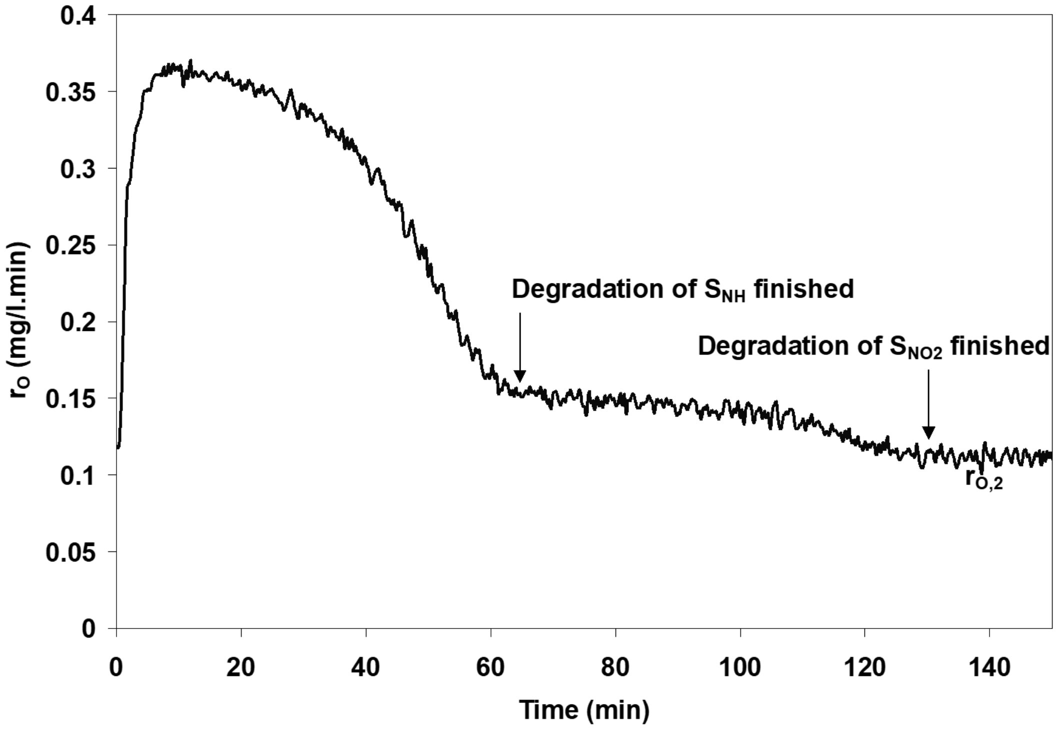


Figure 2.5. Nitrifiers respirometric batch test.

The rate of nitrification is affected by a number of environmental factors, such as the amount of dissolved oxygen, temperature, pH, and inhibitors (Orhon et al., 1994, Sözen et.al., 1998). It is important to note that nitrification is more susceptible to low dissolved oxygen levels than the activity of heterotrophs. In addition, the nitrifying bacteria can be inhibited by many different substances, especially those from industrial discharges (i.e., leather tanning, metal finishing, and textile effluents) (Orhon et al., 2000). The reaction rate constants for nitrifying bacteria are listed below in Table 2.1 (Henze *et al.*, 1995b).

Table 2.1. Constants of reaction rate for nitrifying bacteria at 20 °C.

Parameter	**Symbols**	**Unit**	**Quantity**		
			Nitrosomonas	Nitrobacter	Overall Process
Maximum specific growth rate	μ_{maxA}	d^{-1}	0.6-0.8	0.6-1.0	0.6-0.8
Half-saturation constant for NH_4	K_{NA}	gN/m^3	0.3-0.7	0.8-1.2	0.3-0.7

Half-saturation constant for oxygen	K_{OA}	gO_2/m^3	0.5-1.0	0.5-1.5	0.5-1.0
Maximum yield coefficient	Y_{maxA}	gVSS/g N*	0.10-0.12	0.05-0.07	0.15-0.20
Endogenous decay coefficient	b_A	d^{-1}	0.03-0.06	0.03-0.06	0.03-0.06

*per gram NO_3 formed

2.2.2 Denitrification

Denitrification, or the reduction of nitrate to nitrogen gas under anoxic conditions, is dependent on the formation of nitrate in the nitrification process under aerobic conditions. For total nitrogen removal, nitrification should occur first, and then denitrification should occur efficiently to achieve the desired effluent quality. The nitrate produced during nitrification is then used as an electron acceptor during denitrification, which occurs under anoxic conditions, and involves the reduction of nitrate to nitrogen gas by heterotrophic bacteria.

Efficient denitrification requires the presence of organic carbon as a source of energy for the denitrifying bacteria. Nitrification is an aerobic process and a consumer of alkalinity. Denitrification requires no aerobic conditions and produces NO_3-N as an alternative electron acceptor, reducing the overall oxygen requirement of the process. Denitrification also recovers some of the alkalinity consumed during nitrification. Therefore, in order to reduce the overall energy footprint and the addition of external alkalinity, denitrification should be incorporated where feasible. The potential disadvantage is the cost of adding external carbon (e.g., methanol) if the effluent does not contain sufficient amounts of readily biodegradable carbon to meet effluent total nitrogen limits.

According to separate Monod-type expressions, denitrification is affected by nitrate, carbon source, and dissolved oxygen concentrations. However, because the half-rate constants for nitrate and the carbon source (C_{BOD} or methanol) are very small, it is unlikely that either of these parameters will significantly affect the rate of denitrification.

2.3 Operational Parameters of Activated Sludge Plant

2.3.1 Hydraulics

Accurate characterization of the hydraulic properties and reactor configuration is crucial to replicate the actual design of the system. For instance, the plug-flow pattern of a reactor plays a critical role in shaping the distribution of oxygen along the channel and can induce denitrification by creating anaerobic zones in the reactor. Furthermore, it is essential to accurately determine the internal flow rates, as the recycled nitrified water will transport oxygen and nitrate across the anaerobic and/or anoxic reactors, which directly impacts the overall capacity of the system to remove total nitrogen and phosphate.

The oxidation ditch can be a good example since the internal recycle rate in the channel is much higher than that of influent flowrate. In theory, altering the internal recycle rate would also change the travel time of mixed liquor between aerators. As a result, the mixed liquor would encounter air on/off periods more frequently if the internal recycle rate is increased. Practically, it means that the cycle-time (T_C) is decreased by increasing the internal recycle which causes considerable change in the nitrate in the reactor. Hence, additional measurements should be performed for the determination of internal recycles, influent flowrate and sludge feeding-wastage patterns and sampling points (De Clercq *et al.*, 1999).

2.3.2 Sludge Retention Time (SRT)

The most difficult and important part of modeling is figuring out how much SRT is really there. The total solids retention time (SRT) is a critical operating parameter that directly impacts the solid balance in the system, as well as the concentration of active biomass. It is considered to be the most sensitive operating parameter in wastewater treatment processes (Metcalf and Eddy, 2014). From a modeling point of view, a minor change (or error) in the SRT can be compensated by the endogenous decay rates (b_H) and/or yield coefficients (Y_H) to close the balance of the solid over the system. In addition, the reaction rates governed by Monod expressions are the result of active biomass concentrations which are highly dependent on the SRT value. By utilizing solid and total phosphate balance calculations throughout the system, it becomes easier to estimate the actual sludge age of the system. (Rodrigo et.al., 1996). At higher sludge ages, the activity of biomass is reduced due to factors such as increased protozoan activity and the accumulation of inert materials (van Loosdrecht et al., 1999).

2.3.3 Air (oxygen) Transfer

The oxygen transfer in process conditions is represented by the volumetric oxygen transfer coefficient K_{La} in activated sludge models. K_{La} is a measure of the efficiency of oxygen transfer from the air to the liquid phase in the aeration tank (ASCE, 1991). It is affected by several factors, including the design of the aeration system, the dissolved oxygen (DO) concentration in the liquid phase, the temperature, the pressure, and the physical properties of the liquid, such as viscosity and surface tension. The oxygen content of a CSTR tank may be expressed as follows:

$$\frac{dS_O}{dt} = K_La_f\,(S_{O\infty f} - S_O) - (OUR_{nitrifiers} + OUR_{heterotrophs}) \qquad (2.1)$$

In activated sludge models, the oxygen transfer is typically quantified using the volumetric oxygen transfer coefficient, K_{Laf}, which is specific to the process conditions. Additionally, the oxygen saturation concentration under process conditions, $S_{O\infty f}$, and the actual oxygen concentration in the tank, S_O, are taken into account. The oxygen uptake rates (OUR) for nitrification and heterotrophic biomass are also considered. Accurately measuring oxygen input into the system is crucial because it impacts the activation of nitrification, carbon oxidation, and denitrification processes based on oxygen half-saturation constants (K_O). The K_O values are determined using Monod expressions, where lower values correspond to higher Oxygen Uptake Rates.

Consequently, this can be taken advantage of to activate the simultaneous nutrient removal process with appropriate aeration control providing limited oxygen levels (Naidoo *et al.*, 2002; Ritmann et al., 1985; Insel *et al.*, 2004; Munch *et al.*, 1996,). From a modelling point of view, the effect of K_La_f on nutrient removal can be compensated by manipulating the oxygen half-saturation constants for heterotrophs (K_O) and autotrophs (K_{OA}). The reaction rates can then be adjusted by reducing those parameters with an underestimated K_La_f. Hence, the K_La_f should be measured under process conditions. Measured (online/offline) oxygen data may be useful to estimate the magnitude of the process K_La. The power consumption and air flow rate give information to estimate the aeration intensity.

2.3.4 Sludge Settling

Final settling tanks are critical components in wastewater treatment systems because they have the ability to regulate the mass fraction of mixed liquor suspended solids (MLSS) based on various operating conditions, including RAS, surface loading, and retention time. Furthermore, if not managed effectively, these tanks can cause bulking, resuspension, and rising of the sludge blanket, leading to a decline in effluent quality. Additionally, these tanks can provide opportunities for denitrification, leading to further nitrate removal, and the potential release of secondary phosphate (Henze et al., 1993, Wouters-Wasiak et al., 1996).

It is essential to accurately define and incorporate the dynamics and contribution of final clarifiers into the modeling process for wastewater treatment systems. This is because final clarifiers play a significant role in the overall efficiency of the system, and any misrepresentation of their function or impact can lead to inaccurate results. Therefore, a thorough understanding of the performance and operation of final clarifiers is critical to the development of reliable models of wastewater treatment systems. On the other hand, primary settling tanks (PST) influence the distribution of particulate and soluble COD components prior to the biological reactor. However, depending on the retention time and sludge blanket in the PST, the hydrolysis and fermentation processes can be activated. As a result, a probable increase in VFA potential can be observed due to heterotrophic activity.

2.3.5 Process control

Automated control methods have become popular tools to help stabilize the nutrient removal process (i.e., Supervisory Control and Data Acqusition-SCADA systems). In continuous and batch-wise systems, the aeration, return, and internal cycles, as well as RAS, sludge wastages, chemical dosing, etc., have become key controllable factors (actuators) for system optimization (Copp *et al.*, 2002; Olsson et al., 1994). In this way, the control algorithms can have an impact on the performance of the system through changes in the composition of the mixed liquor. So, the controlling algorithms need to be put into the simulators so that the models are accurate.

2.3.6 Integrated BNR

Biological Nutrient Removal (BNR) is a process used in wastewater treatment to remove nutrients such as nitrogen and phosphorus from municipal wastewater. BNR processes typically involve the use of both aerobic and anaerobic tanks, along with specialized bacteria and other microorganisms to remove the nutrients. There are many Biological Nutrient Removal (BNR) processes widely used for nitrogen and phosphorus removal from municipal wastewater (Wentzel et.al. 1990),. Information on these processes is presented below.

2.3.7 Wuhrmann Process

Wuhrmann process pioneered the nitrogen removal technique as a means of enhancing the standard activated sludge process used in WWTPs (Applegate et al., 1980). The nitrogen removal process, which is more advanced than the Wuhrmann process, was a later development by Ludzack and Ettinger. The most obvious differences between the two processes are the carbon source for denitrification and the location of the denitrification tank.

In the Wuhrmann process (oxic-anoxic), the denitrification tank is located after the aeration tank. Therefore, the carbonaceous substrate is treated in the oxic tank. The denitrification rate is severely limited due to the lack of carbonaceous substrate. There is a long retention time in the denitrification tank, where organic matter from cell decomposition is used as a carbon source in the denitrification process. The addition of a supplemental carbon source to the anoxic zone and the integration of a small aerobic zone between the anoxic reactor and the final clarifiers for the removal of residual nitrogen gas and oxidation of residual organic matter are two improvements to the Wuhrmann process that would allow successful nitrogen removal. The Wuhrmann process is illustrated in Figure 2.6.

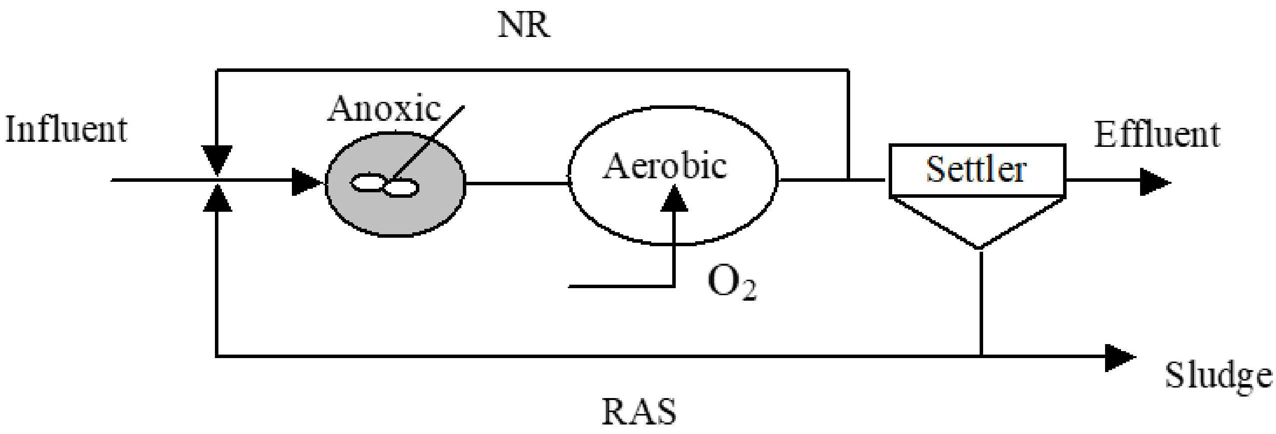


Figure 2.6. Flow diagram of Wuhrmann process.

2.3.8 Modified Lutzack Ettinger (MLE)

The other treatment plant configuration for nitrogen removal is the pre-denitrification plant, sometimes called the Modified Lutzack and Ettinger system (Barnard, 1973). The MLE process consists of an anoxic zone prior to the aerobic zone, which is a modification of the standard activated sludge process. These zones can be created by making the necessary modifications to the aeration system along a plug flow reactor, or they can be physically separated by arranging the reactors in series.

In order to remove nitrogen effectively from municipal wastewater, internal recirculation pumps return the nitrate-rich mixed liquor from the aerobic nitrification zone to the anoxic zone where it combines with the influent wastewater. The amount of nitrate concentration that can potentially be denitrified in the anoxic zone depends on the internal recirculation flow rates and the availability of influent BOD. Therefore, it is necessary to have a high internal recirculation rate (3-5 $Q_{influent}$) from the aerobic reactor through the pre-anoxic reactor (Kayser, 1999).

The MLE is insufficient for reducing final nitrogen concentrations to extremely low levels since denitrification can occur only with nitrate that has been recycled. The maximal denitrification potential is about 82% at a 500% recycling rate (Metcalf and Eddy, 2014). Effluent concentrations of TN are typically in the range of 5 to 8 mg/L (Barnard 2006).

Other factors, such as the availability of the carbon source, the kinetics of the process, and the size of the anoxic or aerobic zone, may limit the denitrification rate. In addition, the recycling of oxygen from the aerobic zone may have a negative effect on the rate of denitrification in the anoxic zone (Metcalf and Eddy, 2003). In practice, the anoxic zone can be divided into 3-4 stages in series. Utilizing the MLE technique to convert activated sludge facilities from carbon to nitrogen removal is a very adaptable configuration (Metcalf and Eddy, 2003). The modified Ludzack-Ettinger process is illustrated in Figure 2.7.

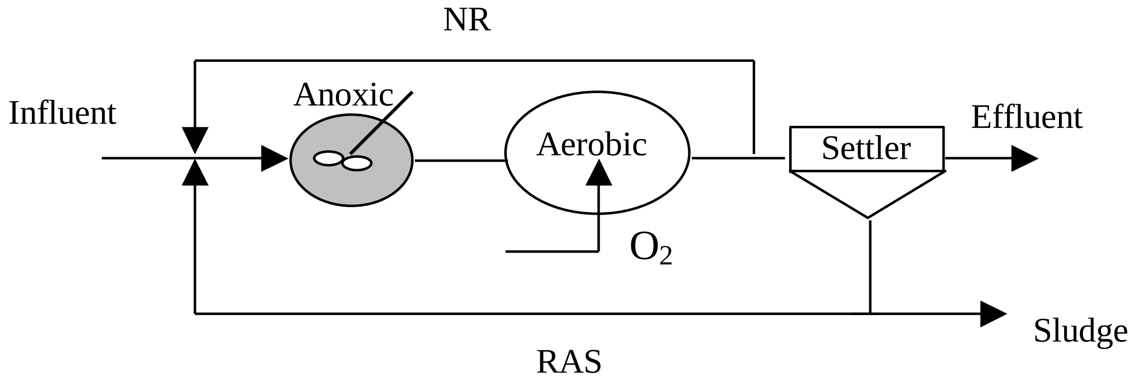


Figure 2.7. Flow diagram of modified Ludzack-Ettinger process.

2.3.9 Three Stage Phoredox (A2O) Process

Enhanced phosphorus and nitrogen removal from municipal wastewater can be achieved with A/O type systems, commonly known as Phoredox systems. The A2O process consists of an anaerobic zone, an anoxic zone, and an aerobic zone. This layout is the modification of the AO system designed for denitrification together with EBPR.

Similar to the MLE method, nitrates are recycled from the aerobic to the anaerobic zone via an internal recirculation stream. The anaerobic reactor's hydraulic retention period is set at roughly 1 hour (Metcalf and Eddy, 2014). On the other hand, the RAS is returned to the anaerobic zone and mixed with the raw wastewater. The nitrate in the return sludge can thus be removed simultaneously in these tanks. The concentration of nitrates in the internal recirculation sludge is decreased thanks to the anoxic zone, which improves the efficiency of the anaerobic process. In the same way that it happens in all biological systems that include nitrogen and phosphorus, some phosphorus is taken up by the PAO in the anoxic zone. The amount of time that the sludge spends in each zone needs to be sufficient to enable the removal of all of the phosphorus. In addition, it needs to be operated such that there is consistent removal of solids in order to prevent the release of phosphate caused by the respiration of endogenous PAO. Three Stage Phoredox process is illustrated in Figure 2.8.

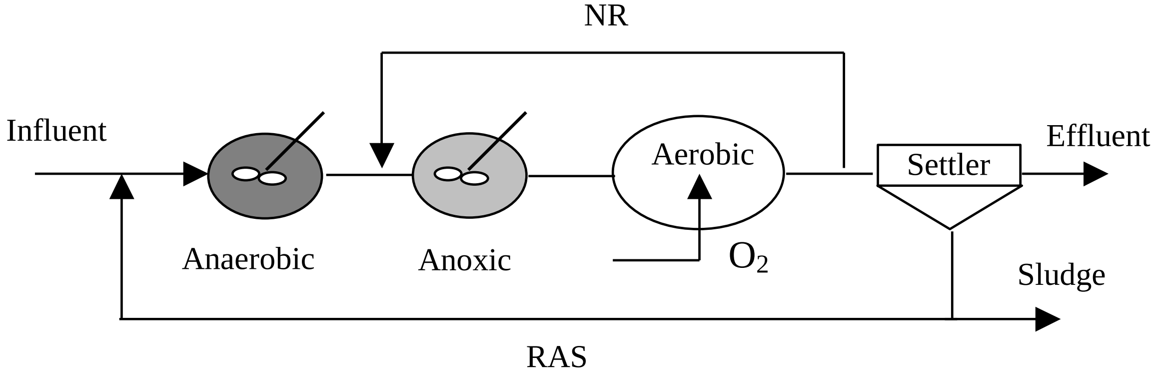


Figure 2.8. Flow diagram of three stage Phoredox process.

2.3.10 Bardenpho (4-5 Stage) Process

The 5-stage Bardenpho process is a modification of the 4-stage Bardenpho process, which is a type of biological nutrient removal (BNR) process used in wastewater treatment to remove both nitrogen and phosphorus from wastewater. Addition of an anaerobic compartment to promote acetate uptake for enhanced biological phosphorus removal. The 5-stage Bardenpho process is more effective at removing phosphorus from wastewater than the 4-stage Bardenpho process, as it promotes the growth of phosphorus-accumulating organisms in the anaerobic zone.

Different from the A2O layout depicted in Figure 2.8, the second anoxic stage removes the additional nitrate using the endogenous carbon source. In the second anoxic zone, additional carbon can be added using methanol or VFA if phosphorus uptake needs to be supported. The final aerobic stage is required for nitrogen gas stripping to prevent a possible failure of activated sludge settling. The 5-Stage Bardenpho process is illustrated in Figure 2.9.

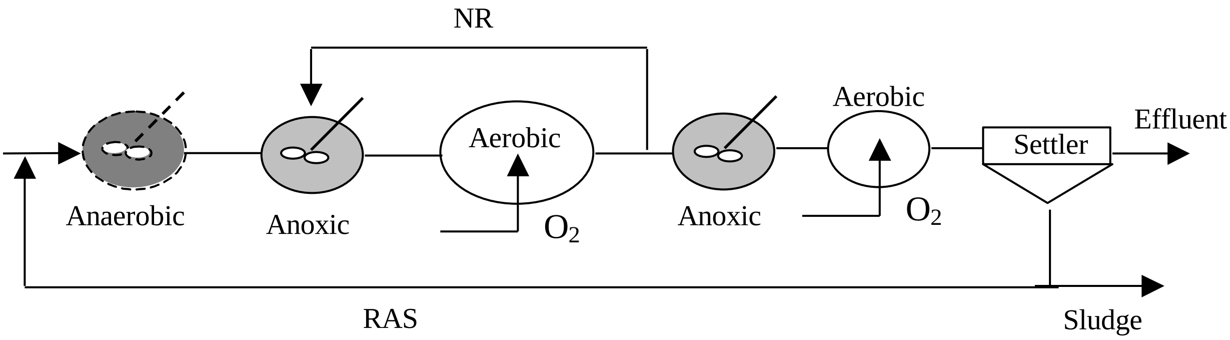


Figure 2.9. Flow diagram of Bardenpho process.

2.3.11 Step-Feed Process

The step-feed reactor configuration is arranged with anoxic and aerobic reactors in series. Return-activated sludge (RAS) enters the first anoxic zone. For nitrogen removal, the influent wastewater is distributed to the two denitrification zones.

In proportion to the flow split that enters each part, the biomass concentration is kept preferentially greater in the first section, and then it is kept lower in the second section and in the successive sections. The last part keeps the least concentration of biomass in inverse proportion to the flow; as a result, this section has the lowest MLSS concentration. Therefore, the rate of solids loading to the final clarifiers will be the lowest possible given the flow. Figure 2.10 provides a visual representation of the Step-Feed process. The Step-Feed process is illustrated in Figure 2.10.

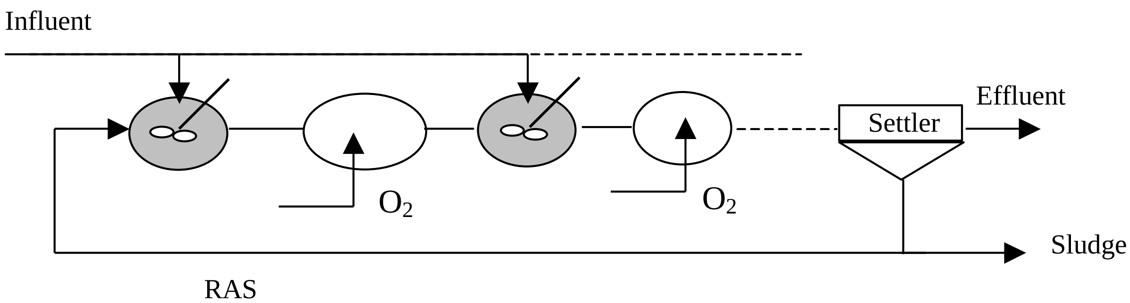


Figure 2.10. Flow diagram of Step-Feed process.

2.3.12 University of Cape Town (UCT) Process

The research group at the University of Cape Town (South Africa) was tasked with developing a method for the EBPR of low-strength wastewater, and they came up with the UCT process. Similar to the Phoredox process, the University of Cape Town (UCT) process has three distinct phases: anaerobic, anoxic, and aerobic.

Nitrates that were produced in the aerobic zone are transferred to the second anoxic zone with the assistance of an internal recirculation system. A second internal recirculation brings the effluent from the first anoxic zone back to the beginning of the anaerobic zone. To reduce the quantity of nitrate concentration entering the anaerobic zone, the RAS is returned to the head of the anaerobic zone. It is the goal of the design to maintain a high concentration of volatile fatty acids (VFAs) and phosphate-accumulating reactions while avoiding direct competition with denitrifying processes that use VFAs. No additional carbon sources are required if sufficient VFAs are present. Downstream chemical precipitation and filtration are needed to get the amount of phosphate in the water down to a very low level. The UCT process is shown in Figure 2.11.

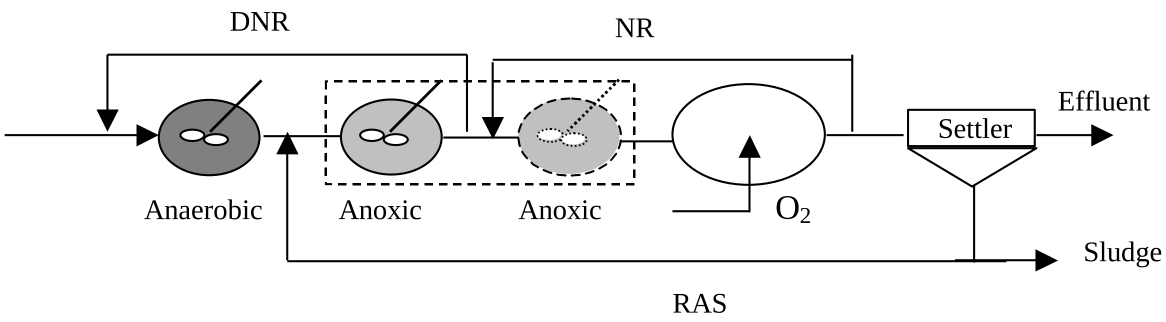


Figure 2.11. Flow diagram of University of Cape Town, UCT.

2.3.13 Modified University of Cape Town Process

Another variant of Phoredox, termed Modified UCT, features an anaerobic zone, then two anoxic zones, and lastly an aerobic zone, all of which arrive before the final clarifiers. The purpose of the two anoxic zones in series is to prevent nitrates from internal circulation and recirculation from entering the anaerobic zone. Nitrates formed in the aerobic zone are recycled back to the upstream of the anaerobic zone, and flow from the end of the first anoxic zone is also recycled back to the upstream of the anaerobic zone. Return activated sludge is directed to the upstream of the first anoxic zone. If a sufficient amount of VFA is present, then no additional carbon sources are required for denitrification. Downstream chemical precipitation and filtration are needed to get the amount of phosphate in the water to be very low.

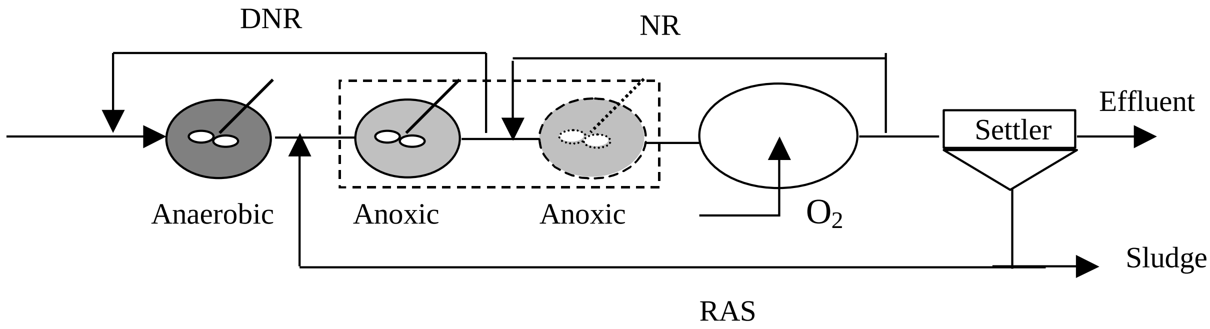


Figure 2.12. Flow diagram of modified University of Cape Town.

2.3.14 Virginia Initiative Plant (VIP) Process

Research on the VIP process has been done by Daigger et al (1988). Another form of the Phoredox process is the Virginia Initiative Process (VIP), which is quite similar to the modified UCT process. The distinguishing feature of VIP from A2O and UCT facilities is that all zones consist of at least 2 staged CSTR tanks in series. The internal recirculation (IR) and return activated sludge (RAS) are connected to the first compartment of the anoxic reactor as shown in Figure 2.13.

The anoxic reactor operates in a plug flow regime, which results in improved denitrification. As a result, the nitrate load returned to the anaerobic reactor is reduced. The VIP procedure is capable of being run at both a high rate and for an extended aeration mode. Sludge retention time (SRT) for anaerobic and anoxic compartments is typically between 1.5 and 3 days while hydraulic retention time (HRT) is typically between 1.0 and 1.5 hours (Metcalf and Eddy, 2014).

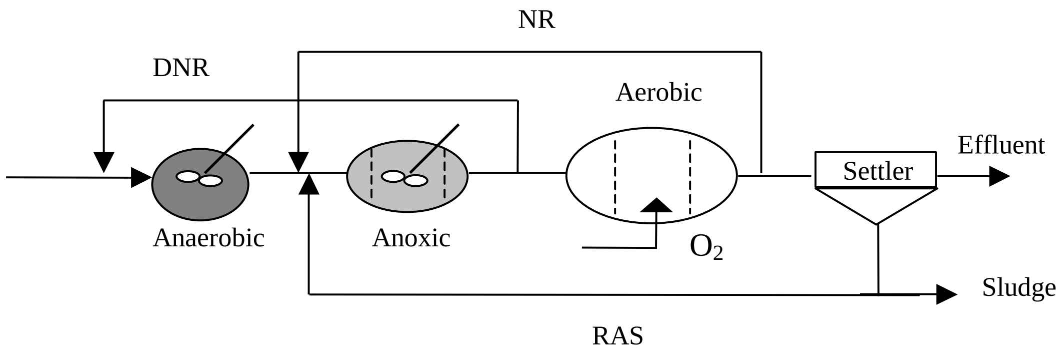


Figure 2.13. Flow diagram of VIP process.

2.3.15 Johannesburg Process

Johannesburg process is another alternative type of activated sludge plant that can remove nitrogen and phosphorus. Compared to UCT and MUCT, it is a first pre-anoxic reactor that receives the RAS. Nitrate is removed from the environment by the biomass's own metabolic processes (endogenous activity). The reduction of nitrate loading in the anaerobic reactor is made possible by careful consideration of the configuration and volume of the anoxic reactor. In contrast to the UCT method, the concentration of MLSS may be maintained at a greater level in the anaerobic reactor. Figure 2.14 provides a visual representation of the Johannesburg process.

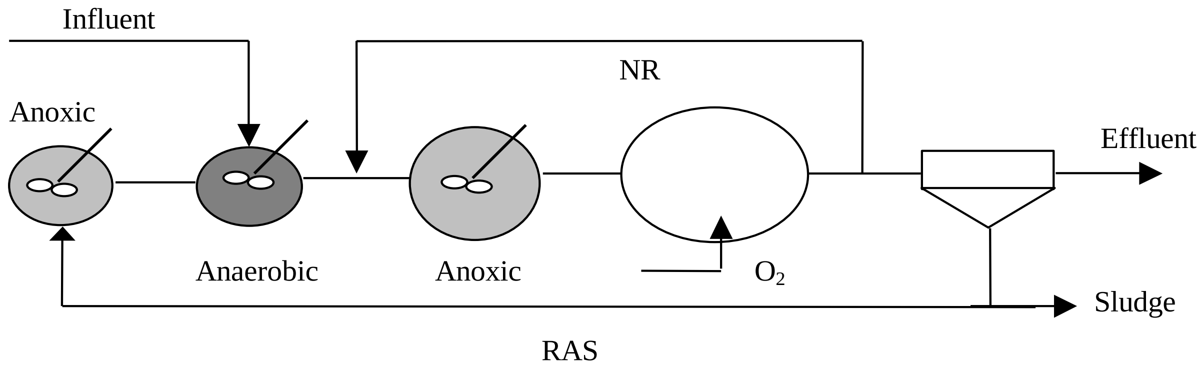


Figure 2.14. Flow diagram of Johannesburg process.

2.3.16 Biodenipho Process

The PID or Biodenipho process is very similar to the Biodenitro process, with the exception that an anaerobic tank is placed upstream of the two oxidation ditches, and these ditches are operated in cycles to induce denitrification and nitrification. Because of the cycle, the organic carbon that is present in the wastewater is utilized for the process of denitrification as well as the biological removal of phosphorus. If there is already an enough amount of carbon, then no more source is required. The RAS is directed into the anaerobic zone of the process. As with the biodenitro process, the system's footprint can be substantial. To get very low levels of phosphate, chemical precipitation and filtration may be needed further down the line. Figure 2.15 provides a visual representation of the Biodenipho process.

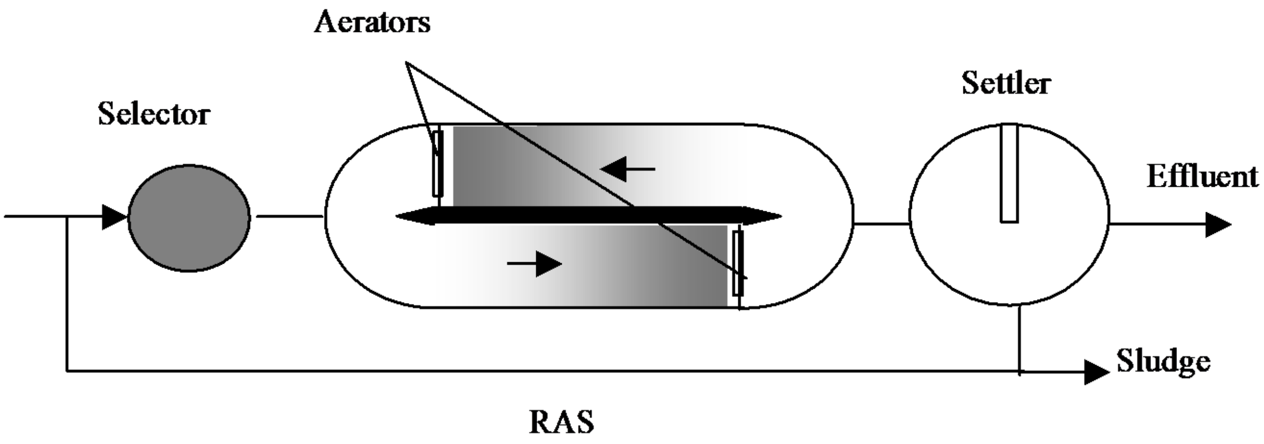


Figure 2.15. Flow diagram of Biodenipho process.

2.3.17 Blue Plains Process

The Blue Plains method was integrated into the Washington, DC WWTP's preexisting nitrifying activated sludge process. Within the aeration tank, a new area that was devoid of oxygen (anoxic) was produced. In comparison to the standard HRT of 3.3 hours throughout the whole basin, the actual HRT of this basin was just 0.8 hours. 13 days was chosen to be the target age of the sludge in the design. During this refit, the pre-existing system for return activated sludge did not see any modifications. Direct addition of methanol was done in order to get the desired nitrogen content of 7.5 mg/L in this newly formed anoxic zone (Kang et al. 1992; Sadick et al. 1998). Phosphorus can be removed from the water via tertiary filtering and the addition of ferric chloride. Figure 2.16 provides a visual representation of the Blue Plains process.

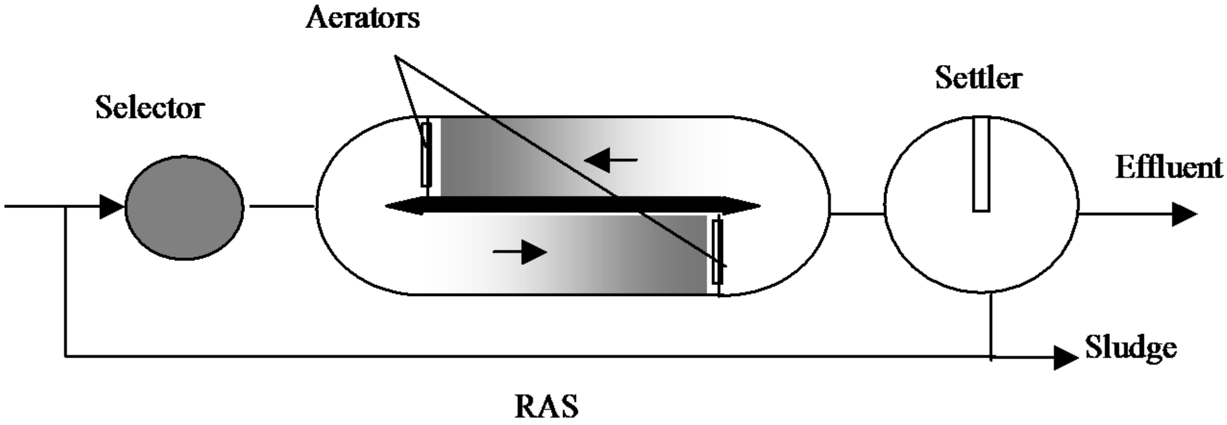


Figure 2.16. Flow diagram of Blue Plains process.

2.3.18 Westbank Process

The Westbank process is a modified version of the Bardenpho process, which has five stages. In this modified process, the second anoxic zone and reaeration zone are eliminated. Instead, a step-feed arrangement is used to distribute the primary effluent and the fermenter supernatant enriched with VFA to the anaerobic and anoxic zones zone (Onnis-Hayden et.al., 2018). Figure 2.17 illustrates this distribution. The process begins with a small pre-anoxic zone that minimizes the entry of DO and nitrates into the anaerobic zone, maximizing phosphorus release. The process then proceeds through an anaerobic zone, an anoxic zone, and an aerobic zone. The RAS is introduced to the anoxic zone.

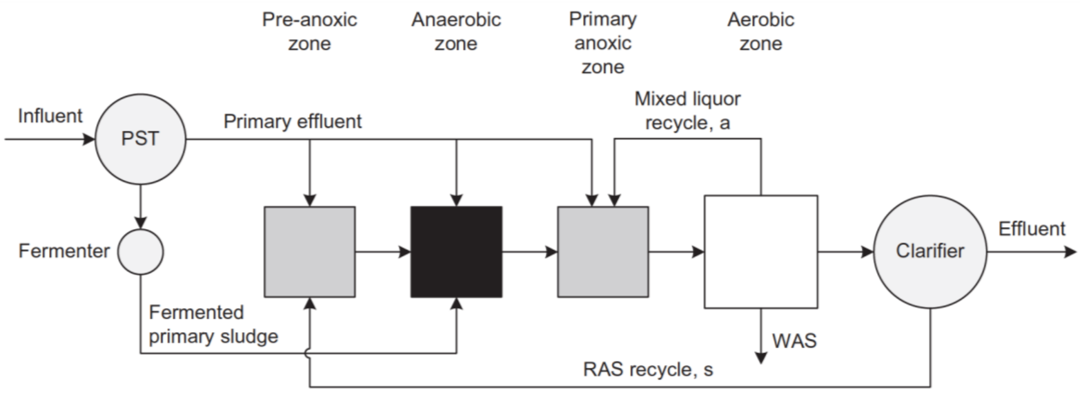


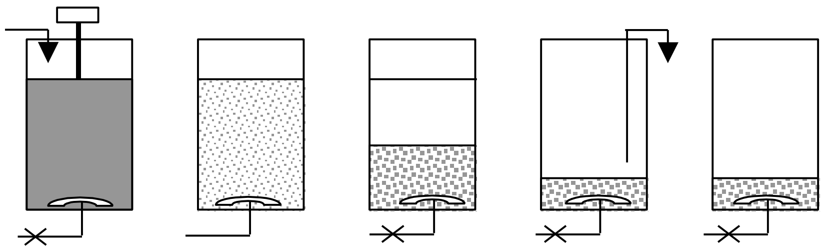
Figure 2.17. Flow diagram of Westbank process.

2.3.19 Sequencing Batch Reactor (SBR)

In general, SBRs are a viable option for treating wastewater in small communities (Wilderer et al., 2001). SBR tanks often come in groups of two or more. Each group has similar characteristics. The SBR tank has floats or level sensors, aeration diffusers, submerged mixers, influent and effluent valves, effluent decant lines, and effluent pumps .

An SBR treats a portion of the total daily wastewater flow in a batch process. A visual representation of the cycle of an SBR is shown in Figure 2.18. The order in which processes occur in an SBR is time-dependent. The total time spent in each phase contributes to the duration of a full process cycle (TC), which is repeated progressively. Generally, durations can be divided into the following categories: (1) fill/mix time, (2) aeration period, (3) settling period, (4) decanting period, and (5) idle period.

An SBR tank, which receives raw wastewater, performs several selected processes, including anaerobic, anoxic, or aerobic cycling. These cycles are controlled by a computer system (Kazmi et.al., 2001). The operator programs the cycles to achieve the desired effluent treatment goals. While influent is flowing into one SBR tank, the other is not receiving influent. It is performing final aeration (react), settling, decanting, or wasting. The SBR tank is ready to treat another batch of influent after it has decanted a portion of its total liquid volume. An SBR treatment facility has no secondary clarifiers or RAS pumping systems. It can meet very stringent effluent limitations with a small footprint.



1. Fill/Mix	2. Aeration	3. Settling	4. Decant	5. Idle

Figure 2.18. Flow diagram of SBR process.

2.3.20 BCFS® process

The BCFS process (biochemical phosphorus and nitrogen removal) is based on the modified UCT process. It was developed to achieve low nutrient effluent concentrations at relatively low influent COD/N and COD/P ratios (van Loosdrecht et al., 1998).

The BCFS process consists of five compartments (

Figure 2.19), a P-stripper from the anaerobic compartment, and three internal recycle (IR) streams. A contact tank is added for the removal of slowly biodegradable COD (XS) with nitrate, which is conveyed via return sludge (RAS). In addition, a mixing tank is introduced to promote simultaneous nitrification and denitrification (DO 0.5 mg/l), if required. In the BCFS process, longer sludge retention times are favorable for nitrogen removal.

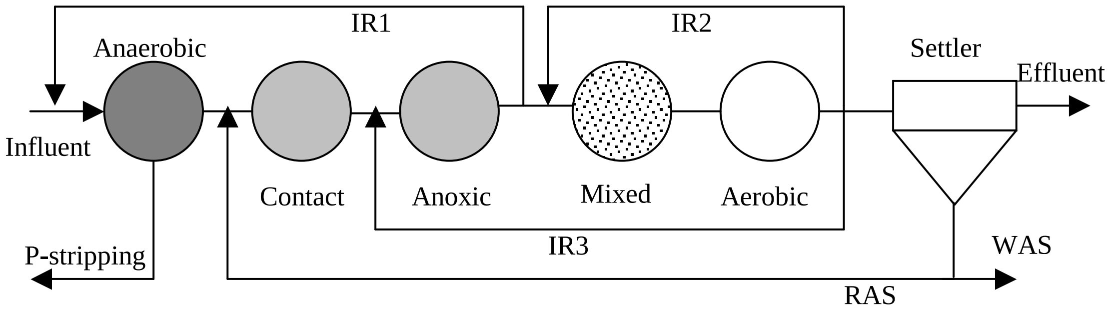


Figure 2.19. Flow diagram of BCFS® process.

2.3.21 Membrane Bioreactor

The membrane bioreactor (MBR) is a wastewater treatment system that includes anoxic and aerobic zones, followed by a membrane that removes solids from the mixed liquor sludge, thus replacing the secondary clarifiers typically used in conventional activated sludge systems. The membranes may be located either submerged in the final activated sludge tank or placed in a separate tank and operated in either an "inside-out" or "outside-in" mode. In the "inside-out" mode, only clean water is allowed to exit the membranes, while in the "outside-in" mode, only clean wastewater is allowed to enter the membranes.

In most cases, wastewater must be pumped through the membranes due to the high solid's concentration. However, under certain circumstances, gravity feeding can be used. By eliminating settling, MBRs can operate at higher mixed liquor suspended solids (MLSS) concentrations than a comparable activated sludge system. While activated sludge systems typically operate at MLSS concentrations of 2,000-5,000 mg/L, MBRs can operate at concentrations of 8,000-18,000 mg/L.

The membranes in an MBR are usually configured as either hollow fiber tubes or flat plates and can be either immersed in the final tank or operated as a separate, self-contained unit. When the membranes operate as a separate unit, an internal recycle system returns a portion of the solids retained by the membranes back to the anoxic zone. This ensures that the solid retention time is sufficient for complete degradation of the organic material, thus achieving high-quality effluent treatment. The process of an MBR is shown in Figure 2.20.

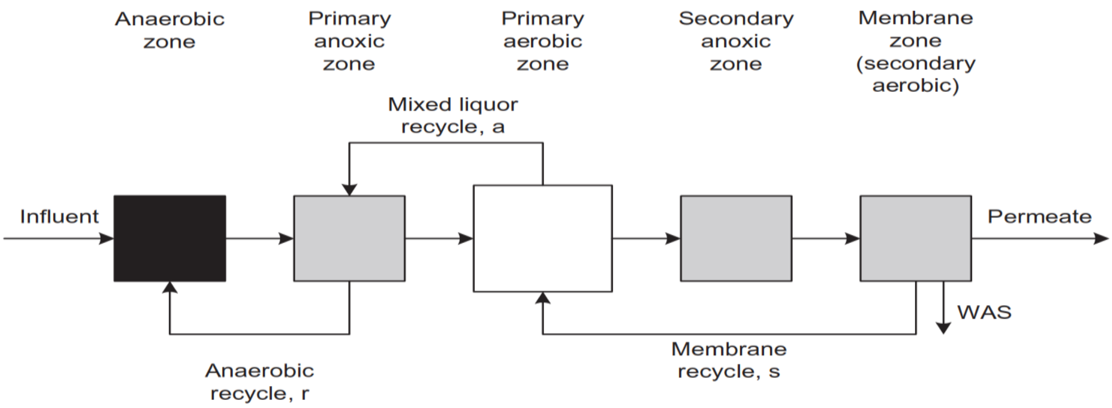


Figure 2.20. Flow diagram of MBR process.

2.3.22 Moving-Bed Biofilm Reactor

An MBBR system is comprised of small plastic media or carriers that are utilized in either anoxic or aerobic zones. Polyethylene is typically the preferred material for the plastic media, and its specific gravity is just below 1.0. The carriers are usually in the shape of cylinders or wheels and have internal and external fins, which provides a high surface area per unit volume, thereby promoting the growth of biofilm while protecting against shear forces. MBBR technology can be installed in an existing activated sludge tank to combine the benefits of both technologies, especially in situations where space is limited. The MBBR process is highly efficient in removing organic and nitrogen loads through high-rate biofilm processes.

MBBRs can be used in separate aerobic and anoxic zones. Slow-speed submersible mixers are used in the anoxic zones, while in the aerobic zones, coarse bubble diffusers are used to supply air since fine bubbles tend to coalesce in the plastic media. A screen holds the media in the tank. MBBR technology does not require recirculation and does not depend on suspended solids to enhance treatment. Selection factors for MBBRs include a moderate footprint, but the addition of a recycle stream would increase the footprint. Screens are the only necessary supporting elements to be constructed in the basins.

2.3.23 Integrated Fixed-Film Activated Sludge

Integrated Fixed-Film Activated Sludge (IFAS) is a hybrid wastewater treatment process that combines conventional activated sludge with a fixed-film system. In an IFAS system, a high surface area media, such as plastic or ceramic material, is added to the aeration tank, creating a surface on which bacteria can attach and grow.

Two types of media are used for attached growth: fixed and floating. Fixed media, such as rope or stranded material, remain in place while floating media, like sponges, packing material, or plastic media similar to those used in MBBRs, can either be free-floating or held in place by cages or screens.

Integrated Fixed-film Activated Sludge (IFAS) systems have higher treatment rates and produce sludges that settle better than activated sludge systems. The use of media in zones throughout the tank allows for both nitrification and denitrification to occur in separate anoxic and aerobic zones. To achieve very low nitrogen concentrations in the effluent, two anoxic zones may be necessary, with one potentially serving as a final polishing zone. Tight control of dissolved oxygen (DO) is possible, and selection factors for media retention include a moderate footprint using the existing activated sludge system. Construction within the aeration basin is required for media retention, but no additional piping or pumping is typically necessary as long as the flow pattern maintains sequential aerobic/anoxic conditions. Electrical costs may be reduced if additional pumping is not required for zones that are no longer aerated.

2.3.24 Sharon®/Anammox® process

In the Sharon (Single reactor system for High Ammonium Removal Over Nitrite) process, ammonium is converted to nitrite under aerobic conditions at low sludge age and high temperatures (≈1 day sludge age, 35^0C) by ammonium-oxidizing bacteria (Hellinga *et al.*, 1998; Jetten *et al.*, 1999). As a result of the short retention time and high temperature, the nitrite-oxidizers are washed out (STOWA, 2001) from the system and only nitrite is formed as an end product.

Table 2.2. Reactions of Sharon and Anammox processes

Sharon:	$0.5\ NH_4^+ + 0.75\ O_2 \rightarrow 0.5\ NO_2^- + 0.5\ H_2O + H^+$
Anammox:	$0.5\ NH_4^+ + 0.5\ NO_2^- \rightarrow 0.5\ N_2 + H_2O$
Overall process:	$NH_4^+ + 0.75\ O_2 \rightarrow 0.5\ N_2 + 1.5\ H_2O + H^+$

The Sharon reactor is operated such that ammonium is only partially converted to nitrite to promote the growth of ammonia-oxidizers. In the Anammox (ANaerobic AMMonia OXidation) process, the ammonia is oxidized if nitrite is available (STOWA, 2001). Finally, a combined Sharon-Anammox process was developed by van Dongen *et al.* (2001). The process stoichiometry is given in

Table 2.2.

The Sharon-Anammox Process can be an alternative to remove nitrogen if a sludge digester is available. For this process, a scheme is illustrated in Figure 2.21. One-stage ammonium removal process can also be achieved in a biofilm reactor with simultaneous nitrification and Anammox, so called 'CANON (Completely Autotrophic N-removal Over Nitrite)' process (Strous *et al.*, 2000).

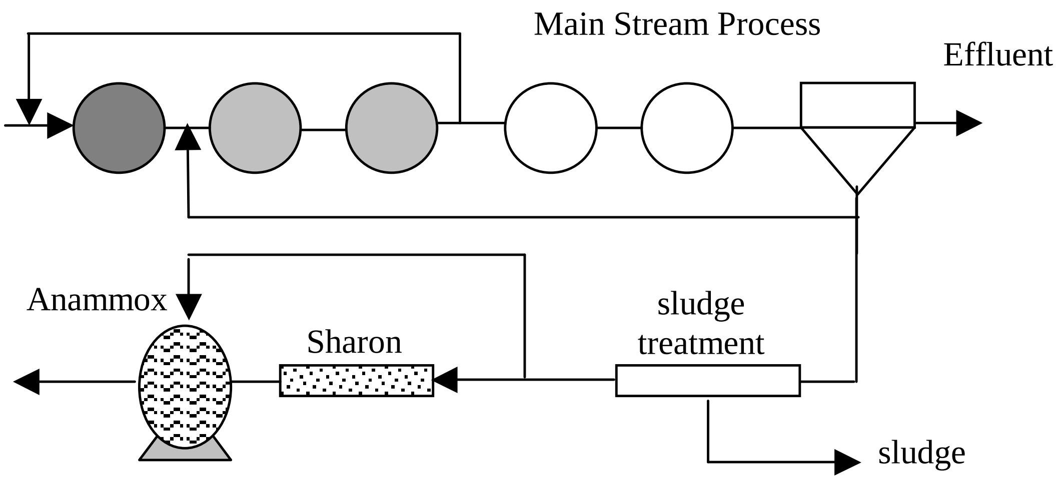


Figure 2.21. Coupling of Sharon-Anammox to main-stream processes.

2.4 Model Needs

Wastewater treatment plants are complex systems that rely on a variety of chemical, physical, and biological processes to produce the desired effluent. It is not always possible to predict how a change in any one variable will affect effluent quality due to the complexity of process behavior and the variety of wastewater characteristics, biological populations, and plant designs. There is a possibility that a plant design that works well in one type of influent and one type of climate may not work well in a different environment. Small-scale (pilot) or large-scale (full-scale) trials can help to see how different variables affect the outcome, but it may be too expensive or time-consuming to try every combination. This is where models come in, allowing us to model a process and estimate the effect that changing parameters will have on the effectiveness of the treatment.

Models can be used for many things. They can be used to design new wastewater treatment plants, design retrofits or upgrades to existing plants, find out how changes in operations might affect the concentrations of permitted pollutants in the effluent, find out how plants will respond to changes in the quality or flow of the influent, and train operators. It's important to choose a model carefully based on its intended use, as not every model can be used for every purpose.

The definition of "model" in their work is disputed by some researchers and writers. Although the term "model" is often used to describe a computer program that is used to solve a specific set of mathematical equations describing a process, it is sometimes used to describe the equations themselves (Makinia et.al., 2002). A 'model' is a simplified representation of a real-world object, situation, or process. Usually, a model shows one behavior or a few of the most important parts of a system. A numerical model is an attempt to represent a real situation (usually on a computer) with the help of math equations. Simulation is the process of putting a mathematical model into a computer program. Computer software is referred to as a "simulator" when it combines the biological model with models for other unit processes. Simulators are required to link the components of a given system in accordance with the flow scheme that is being used.

Numerical models typically need to be calibrated to one or more data sets before they can be used. Model validation comes after the other modeling steps. It is one of the most important steps in model studies because it ensures that the model can be used to predict how the system will behave in different situations. The main uses of numerical models are diagnostic (to make predictions about the future), diagnostic (to find out how something works), and educational (to help experts and ordinary people talk to each other or to teach future process engineers or operators) (Hug et al. 2009).

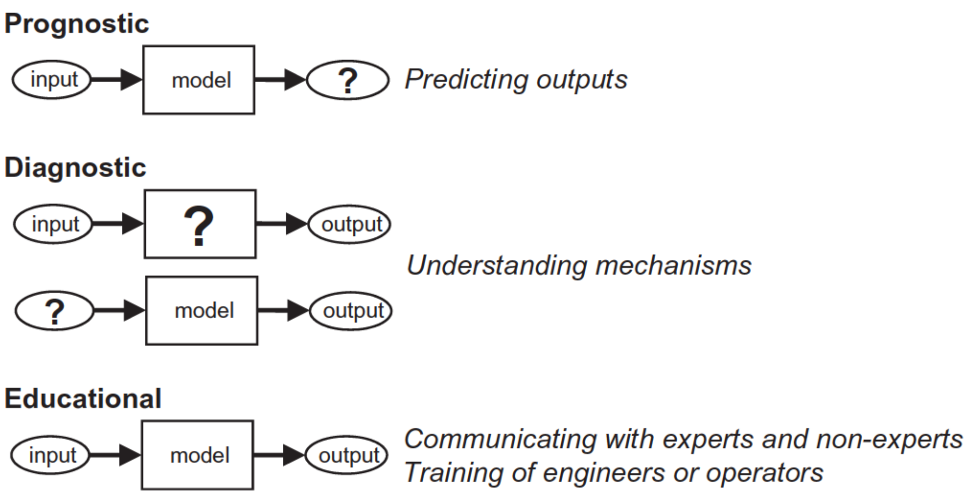


Figure 2.22. Purpose of modelling.

Based on hypotheses about the biological processes going on in the system, these mechanistic models include mathematical expressions that show how the biological interactions work. Obtaining reliable measurements (observations), selection of key characteristics and behaviors, use of simplified approximations and assumptions, accuracy of simulation output (calibration and validation), and predictability of predictions are the key issues in developing a successful model.

External factors (such as environmental and operating conditions) and interactions between complicated and still relatively unknown biological reactions make it difficult to understand the expected response of the system under specific conditions. It has also been used as an optimization tool to assess the consequences of changes in waste flows and loads, operational variations (such as changes in recycle flow rate and changes in internal recirculation), and proposed changes in plant configuration during full-scale operation. Modeling has also been useful in training operators. Simulation exercises using the model give the operator 'instant' experience of what to expect when inputs, plant configurations, and operating strategies change.

In recent decades, mathematical modeling of activated sludge systems has become a widely accepted tool for designing and operating plants, training process engineers and operators, and conducting research (Meijer et.al., 2001). However, only if the model predictions are reliable will the model results be useful in practice. Depending on the project objectives, the resources committed to the project, and the expertise available, the level of detail and quality required for simulation studies varies considerably. Inconsistent approaches and inadequate documentation make the quality of simulations difficult to assess and compare. Most are based on the work of the International Water Association (IWA) to standardize them.

2.5 Overview of Activated Sludge Models

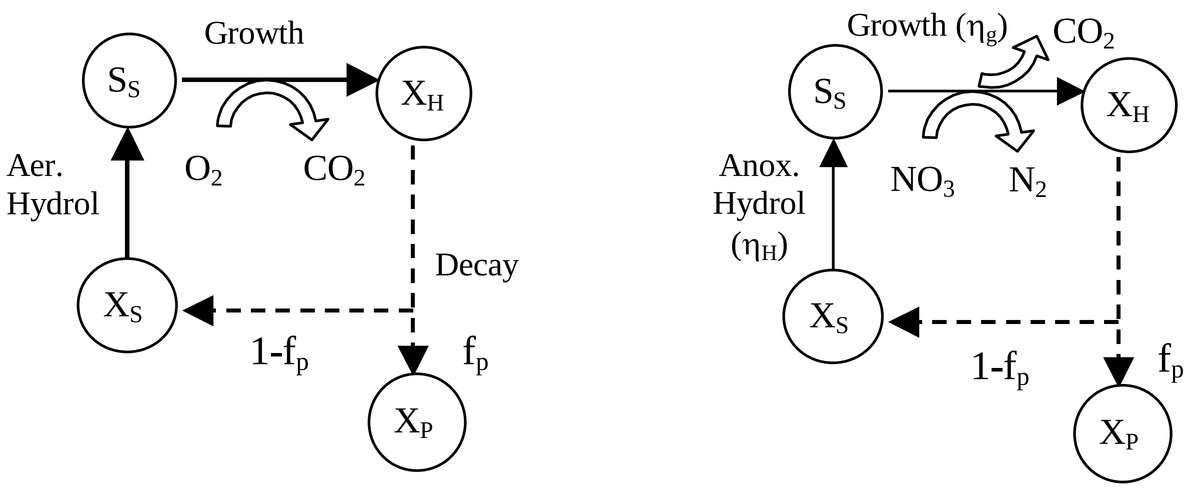
Wastewater treatment processes can be represented by mathematical models. These models combine theory and actual data. There is a separate model for each treatment process (Vanhooren et.al., 2003). Simple and frequently used model equations exist for clarification and settlement. However, modeling biological wastewater processes like activated sludge is extremely challenging.

A revolution in process modeling occurred as computer processing capacity increased. The first process model to be computerized was Busby and Andrews (1975). This implementation was a dynamic model using the "Continuous System Modeling Program" (CSMP). This model included a stored mass model, an active mass model, and a sedimentation model. in the 1980s , the University of Cape Town's research group proposed a general model for the activated sludge process (Dold et al., 1980). This model is concerned with the removal of carbon from domestic wastewater under aerobic conditions. One year later, the model developed by Dold et al. (1980) was expanded upon by Van Haandel (1981), and the nitrification and denitrification processes were incorporated into the model. The readily biodegradable COD is directly utilized by heterotrophic biomass in this model. On the other hand, the slowly biodegradable COD fraction (X_S) is subjected to 4 subsequent reactions, given as follows: (a) X_S is enmeshed in the flocs; (b) adsorbed in heterotrophic biomass; (c) hydrolyzed to hydrolysis products via surface saturation kinetics; and finally (d) utilized by heterotrophs, $X_{H.}$

Then, a task group was established in 1983 by the International Association on Water Pollution Research and Control (IAWPRC), which would later change its name to the International Water Association (IWA), with the goal of combining the various modeling ideas into a "consensus" model for activated sludge. Following the first round of discussion among the members of the group, a preliminary version of the "IAWPRC model" was developed (Grady et al., 1986). The preliminary model was subjected to a comprehensive evaluation by Dold and Marais (1986). It was proposed that certain modifications be made, particularly with regard to the modeling of the fate of organic nitrogen.

The activated sludge model (ASM1) was the first IWA model to be created in 1986. This model, later known as ASM1, was able to simulate biological carbon oxidation, nitrification, and denitrification (Henze et al., 1987). ASM1 consists of four major processes: i) biomass production; ii) biomass decay; iii) organic nitrogen ammonification; and iv) organic particulate matter hydrolysis. In the ASM1 model, the biological reactions are done by heterotrophic and autotrophic microorganisms.

ASM1 received widespread academic and consulting/designer attention and publicity. Heterotroph microorganisms (X_H) are able to make direct use of the readily biodegradable COD (S_S) when aerobic conditions are present. During the lysis of X_H with fraction f_P, an inert microbial product (X_P) is produced. On the other hand, the remaiASM1 is made up of four main processes: the production of biomass, the breakdown of biomass, the ammonification of organic nitrogen, and the hydrolysis of organic particulate mattering portion of "1- f_P" is changed into a substrate that degrades slowly (X_S). The slowly biodegradable COD is hydrolyzed into readily biodegradable COD that can be utilized by the ordinary heterotrophs. Figure 2.23 is a schematic diagram of ASM1 component transformations (Henze et al., 1987, Petersen et.al., 2002).



(a) aerobic (b) anoxic

Figure 2.23. Schematic diagram for ASM1 components.

In the ASM1 and Dold (1986) models, the nitrogen components were introduced with the aid of a "nitrogen module". Nitrate is used as a final electron acceptor in anoxic conditions. The biodegradable particulate organic nitrogen (X_{ND}) is hydrolyzed into soluble organic nitrogen (S_{ND}).

The reaction rate is controlled by a surface saturation-type reaction that occurs concurrently with the hydrolysis process. Ordinary heterotrophs convert soluble biodegradable nitrogen (S_{ND}) to ammonia nitrogen via a first-order reaction. Despite the fact that the ASM model became widely used in both academics and industry, it had a number of disadvantages. For example, the model assumed that temperature and pH were always constant, did not account for EBPR, and assumed that biological processes were independent of the carbon source.

The main difference between ASM2 and ASM1 is the presence of Phosphorus Accumulating Organisms (PAOs) in the activated sludge, which were not present in ASM1. Excessive Phosphorus Removal (EBPR) is a microbiological process that involves developing microorganisms within the mixed community that have the potential to take up more phosphorus than they need to grow. In a well operated wastewater treatment plant, this excess uptake of phosphorus makes it possible to reduce the total phosphorus concentration in the effluent to less than 1 mg/l. Polyphosphate granules, which are stored in a microbially-mediated process, are responsible for the removal of phosphorus and can take up to 60% of the cell volume and 38% of the phosphorus mass of the volatile suspended solids (Lotter et al., 1986; Wentzel et al., 1989a, 1989b). Figure 2.24 is a schematic diagram of ASM2/2d component transformations (Henze et al. 1995a).

ASM2 doesn't take into account the structure (composition) of individual cells. Instead, it only looks at the average properties of the biomass. This is important because the kinetic expressions used in Model No. 2 are not linear, so it may not be possible to predict average behavior from average properties.

The following five primary processes are taken into consideration in ASM3: i) the development of biomass, ii) the respiration of endogenous material, iii) the storage of readily biodegradable organic substrates, iv) the respiration of material that has been stored, and v) the hydrolysis of material (Henze et al., 1991). The primary difference between ASM1 and ASM3 is that ASM3 takes into account the significance of storage polymers in heterotrophic activated sludge processing. Biomass was supposed to be totally created by the external substrate present in ASM1, and oxygen consumption was clarified by the decomposition of biomass following depletion of the external substrate. All rapidly biodegradable organic substrates (Ss) are promptly turned into stored material (X_{STO}) in accordance with of ASM3 (Gujer et.al., 2000). These stored compounds give carbon and energy for plant development during the succeeding era of starvation.

Figure 2.25 is a schematic diagram of ASM3 component transformations (Henze et al. 1999).

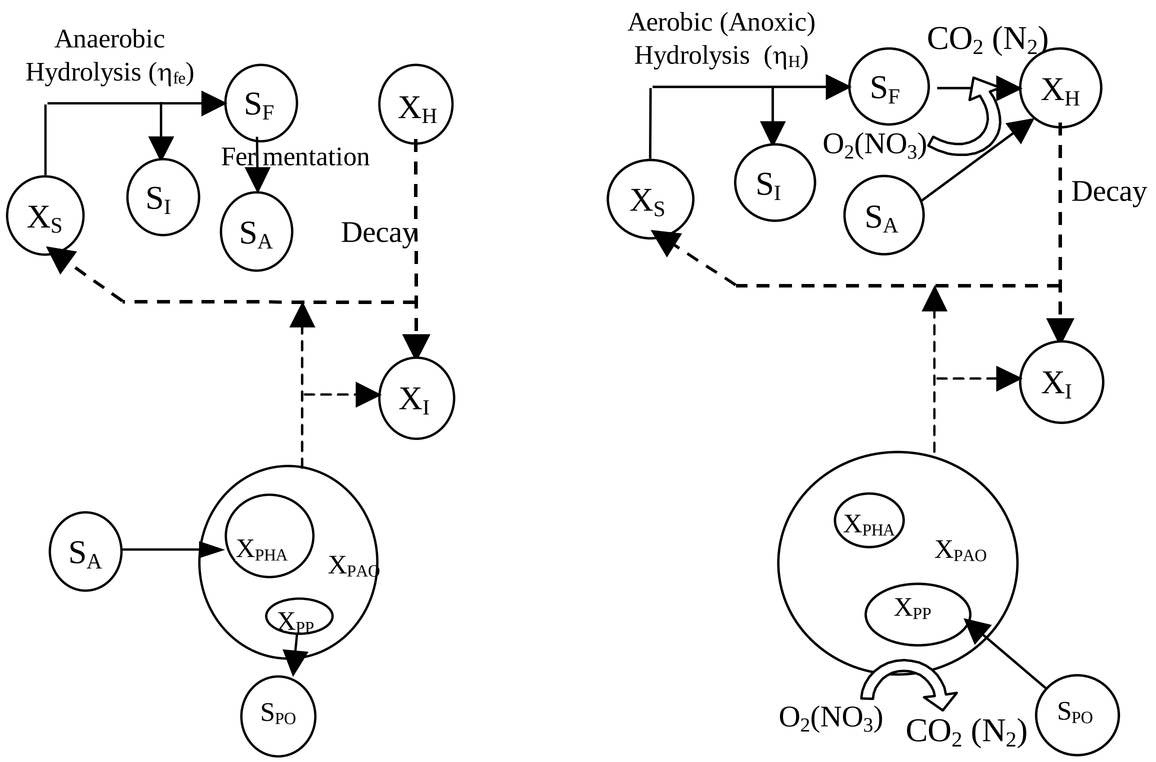


Figure 2.24. Schematic diagram for ASM2/2d.

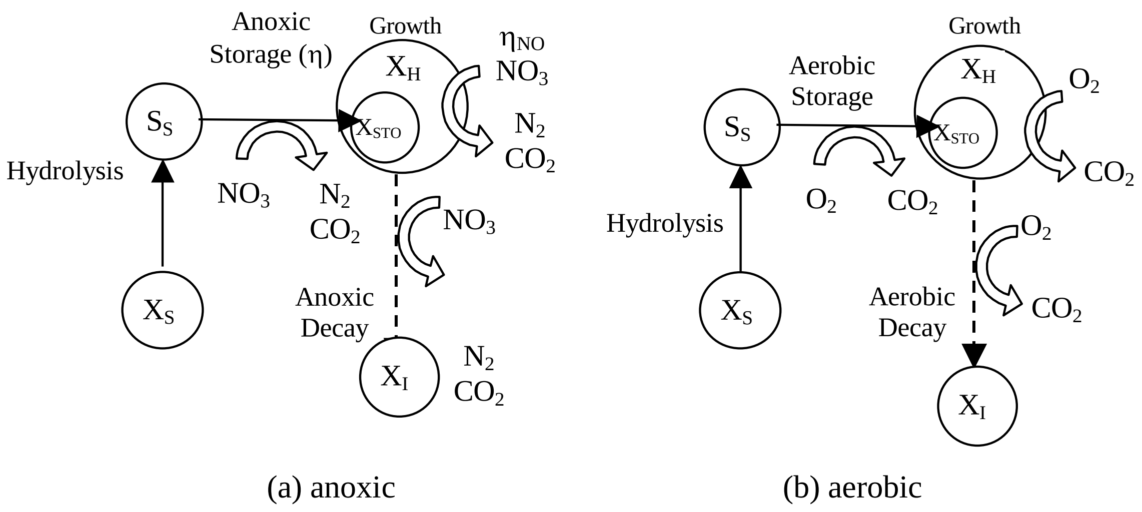


Figure 2.25. Schematic diagram of ASM3 for the components.

Several models have been created in an effort to enhance the ASM model. Delft University of Technology (TUDP) created the metabolic biological phosphorus model to take into consideration the metabolism occurring in PAOs during EBPR. Barker and Dold (1997) created a model (B&D) that includes varying rates of growth depending on the carbon source.

When selecting models, it is critical to consider and contrast the required operations with the plant's regular operating conditions. For example, if a plant wants to use chemical phosphorus removal, it can only use the ASM2 or ASM2d models. Each model also specifies the temperature and pH ranges in which it is applicable.

Each mathematical model for wastewater treatment is described in Table 2.3, along with the processes it can simulate and the source from which the model's equations, constraints, and optimal operating parameters may be obtained.

Table 2.3. Summary of wastewater treatment mathematical models.

Model Name	**Wastewater Treatment Unit Processes**	**Reference**
ASM1	Oxidation of carbon, nitrification, and denitrification	Henze et al. 1987
ASM2	Oxidation of carbon, nitrification, and denitrification, enhanced biological phosphorus removal, fermentation, chemical phosphorus removal	Henze et al. 1995
ASM2d	Oxidation of carbon, nitrification, and denitrification, enhanced biological phosphorus removal, fermentation, chemical	Henze et al. 1999
ASM3	Oxidation of carbon, nitrification, and denitrification,	Henze et al. 1999
ASM3 w/ BioP	Oxidation of carbon, nitrification, and denitrification, enhanced biological phosphorus removal	Reiger et al. 2001
TUDP	Oxidation of carbon, nitrification, and denitrification, enhanced biological phosphorus removal, fermentation	Brdjanovic et al. 2000

B&D	Oxidation of carbon, nitrification, and denitrification, enhanced biological phosphorus removal, fermentation	Barker and Dold 1997

In nutrient removal, fixed film methods are gaining popularity. The additional complexity of diffusion to and from the biofilm makes modeling these processes more challenging. For fixed film processes, there is no comparable model to the ASM models. However, in 2006 (Wanner et al., 2006), IWA published a reference including all of those model equations accessible for fixed film wastewater processes.

2.6 Process Modeling Stages

For WWTP modeling, there are some requirements for information on the proces''s inputs, configuration, and operating conditions. Figure 2.26 shows the requirements for putting up a wastewater treatment process simulation (Melcer et.al., 2003).

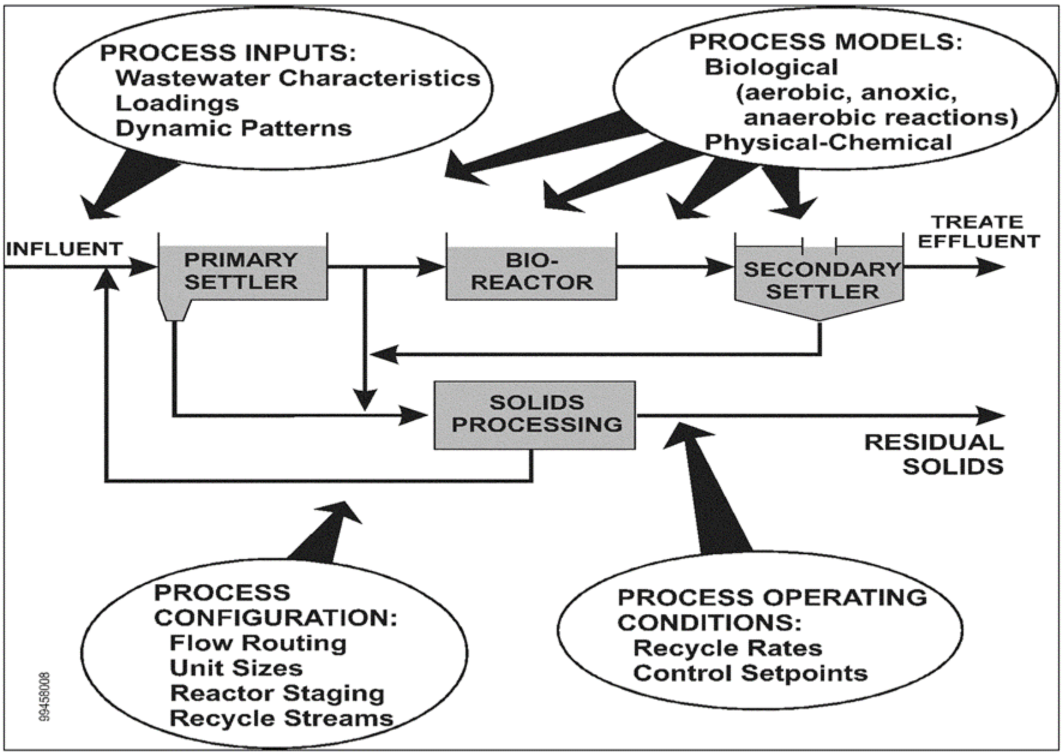


Figure 2.26. Essential requirements for wastewater treatment process simulation.

A generic method to developing a simulation model for a specific plant is showed in Figure 2.27 (Melcer et.al., 2003). The order of these methods are as follows:

:

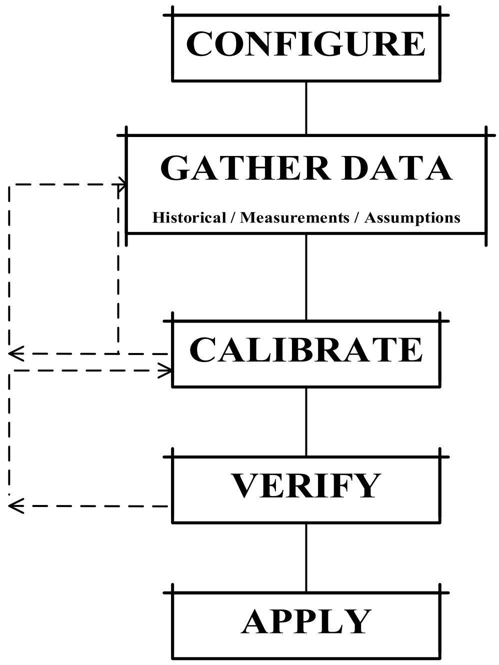


Figure 2.27. A generic method to developing a simulation model.

- The "physical design" of the treatment plant, such as unit sizes, flow routings, and known flowrates, and the modeling framework (biological kinetic coefficients, settling, DO transfer, etc.) are loaded into the simulator.
- For the purpose of calibrating the simulator to the plant under consideration, information is obtained on wastewater characteristics, plant performance, and operating conditions. These data may be gathered from historical plant operating records, new measurements to quantify unknowns, or assumptions for characteristics and variables that cannot be measured due to time or financial constraints.
- The "operational" or "controllable" properties of the treatment plant are often combined with the characteristics of the input wastewater during calibration, and some parameters are adjusted to suit a set of plant performance data. There may occasionally be some discrepancy in the model predictions after an initial calibration effort. Depending on the nature of the

problem, it may be necessary to either re-evaluate the data or obtain new information to resolve it.

- Verification is the application of the calibrated simulator to a different set of operating data than that used during calibration. During this phase of calibration, model parameters are changed for which direct measurements are either not possible or would take excessive effort.
- When the simulator is verified, it may be utilized for its intended purpose.

Steady state simulation analysis is appropriate for rough sizing of process units and for assessing sludge generation and sludge wasting amount. On the other hand, dynamic simulation is useful for determining minimum and maximum aeration requirements, the effects of fluctuation of carbonaceous and nitrogenous to the WWTP and evaluating the impact of stormflow or other transient loading conditions (De Pauw et.al., 1999).

When only the diurnal flow pattern and a constant COD and TKN concentration are used in simulator projections, the aeration needs are often overestimated. The peak daily flow rate does not always happen at the same time as the peak COD or TKN loading. If separate diurnal patterns aren't used, this could cause the aeration system and blowers to be bigger than they need to be.

2.6.1 Configuration of the Plant

In a simulator, the process layout, unit sizes, and entry points for input and recycling streams are entered to configure a plant. When setting up the bioreactor, which is modeled as a series of one or more fully mixed vessels, special care must be taken. Bioreactors need to be designed with a variety of factors in mind, including the number and size of totally mixed vessels used to simulate the mixing regimes, the length, width, and depth ratios of the various zones, and the aeration and non-aeration rates of the zones. If the simulator will be used to predict spatial changes in a bioreactor, these factors are especially important. As part of designing an aeration system, one example would be figuring out how much air a bioreactor needs as it gets longer. Predicting the removal of ammonia and nitrogen in a bioreactor is another example.

When setting up and using a simulator, it is very helpful that the influent stream can be split into different parts, each of which has its own characteristics.

Figure 2.28 shows a simulator setup for a plant where the influent stream has been split into four parts: a municipal contribution, an industrial contribution, an inflow and infiltration contribution, and a potential pre-fermenter addition contribution. By separating the influence in this way, you can look at the effect of each part on its own (Melcer et.al., 2003).

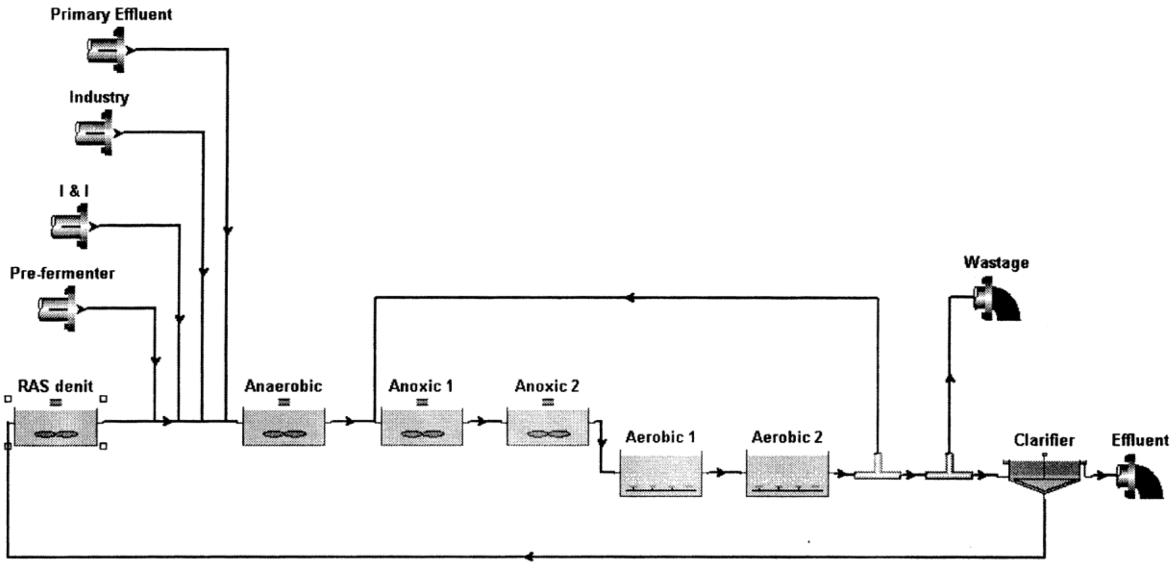


Figure 2.28. Example of influent stream in a simulator configuration.

2.6.2 Data Gathering

The user-supplied variables known as "model parameter" may be divided into 3 groups:

- wastewater characteristics
- parameters defining the process's reactions
- process inputs

When a user selects a model for a wastewater treatment process, they are not only choosing the model but also selecting the parameters that define the reactions. In some cases, users may need to adjust these parameters to fit their specific needs. The inputs required for the process include flow rates for each stream, process size, and recycling flows. Typically, these parameters are known values for existing facilities or design parameters. However, most models require these parameters to be broken down in detail to ensure accuracy.

Wastewater characteristics are typically divided into three categories: COD, nitrogen, and phosphorus. The COD is usually separated into four groups based on their biodegradability: slowly biodegradable, readily biodegradable, particulate nonbiodegradable, and soluble nonbiodegradable. Nitrogen is divided into four groups, including ammonia, soluble unbiodegradable TKN, biodegradable TKN, and particulate unbiodegradable TKN. Similarly, phosphorus is divided into four groups, including orthophosphate, soluble unbiodegradable phosphorus, organically bound biodegradable phosphorus, and particulate unbiodegradable phosphorus.

The characteristics of the wastewater can be expressed in concentrations, or some models require both total concentration and fractions to be given to ensure accurate results. Therefore, the user must provide detailed information about the wastewater characteristics to select the appropriate model and ensure effective wastewater treatment.

To meet the model input requirements, the influent wastewater needs to be sampled and analyzed in the lab more. At least, the fractions of each of the wastewater components must be determined. Furthermore, determining some biological reaction constants through laboratory experiments with biomass is necessary. For example, the nitrifier growth rate, denitrification rate, and phosphorus uptake rate, frequently vary from plant to plant and measuring them directly may yield better design results.

If a new sewage collection and treatment system is to be constructed, there will be no information on the amount and quality of wastewater. In this case, one must make assumptions about the amount of wastewater, its properties, and how it is loaded in order to use a simulation exercise. Calibration and verification of the simulator would need almost minimal action. In contrast, the design should have a substantial margin of safety due to the lack of knowledge of the actual wastewater flow and loading patterns. But, if a treatment plant needs to be built and there is already a sewage collection system in place, there is a chance to learn about how the water flows and what it is made of. In the two cases above, there is no way to get stoichiometric and kinetic information that can be used to calibrate the models in the simulator. Instead, assumptions must be made about these things.

There will be historical plant performance data if the project involves upgrading or retrofitting an already operating facility or using a simulator to make operational decisions or train operators. In addition, there will be the opportunity to gather additional data as desired for wastewater characterization and for the stoichiometric, kinetic, and other parameters used in the simulator.

In general, historical plant records are a good source of data for a simulator's rough calibration. Depending on the extent of the historical data available, in many instances it may be appropriate to design an intensive monitoring program that would:

- Obtain 24-hour composite samples of untreated wastewater, primary effluent, and secondary effluent for the purpose of analyzing a variety of parameters (e.g., COD, BOD, VSS, TSS, TKN, NH3, NO3, Total P and Soluble Reactive P, alkalinity, pH). Simultaneously, conduct measurements of recycling rates and characteristics, such as MLSS and MLVSS, within the process. This activity in data collection might last up to two weeks in order to collect adequate information for a thorough calibration.
- During the sample period mentioned above, there should be a grab sampling program that takes samples every hour or every two hours for 24 or 48 hours. This information is very important for figuring out how the loading patterns change throughout the day and how the system responds in terms of nitrification and ammonia breakthrough, phosphorus removal in EBPR systems, and peak aeration needs.

The duration of such a program and the number of parameters that are measured will depend on the site and what plant data is already available. This may seem like an expensive and time-consuming task, but the quality of the data used to calibrate the simulator will directly affect the accuracy of the predictions made when the simulator is calibrated. In the past, it has been said that the maximum specific growth rate of the nitrifiers (μ_{AUT}) is an exception. This parameter was said to vary greatly between systems that treated different types of wastewater. Values for μ_{AUT} (at 20°C) have been given that range from 0.2 to 1.0 d^{-1}. There's no doubt that nitrifiers are more sensitive to toxicity and inhibition than heterotrophs, and they will show more variation. Because of this, changes in μ_{AUT} may be linked to how much industrial waste is in the wastewater. The performance and design of systems for removing nutrients, as well as model projections, are significantly influenced by the nitrification rate. As a result, extra consideration should be given to μ_{AUT} measurement or estimation.

In examining the veracity of historical or other data, it's typically appropriate to evaluate ratios rather than absolute quantities. Concentration values vary day-to-day, but TKN/COD shouldn't. Ratios can assist discover outliers or invalid data. Typical residential COD/BOD_5 ratios are 2.0–2.2 for raw sewage and 1.9–2.1 for primary effluent. COD/VSS should be 1.42 to 1.48 mg per mg MLVSS. Significant variations from these levels imply suspicious data or an abnormal influent wastewater characteristic.

2.6.3 Calibration of the Simulator

Models are simplified descriptions of reality. Even if all of the input data to the simulator were perfect, the simulator would give an approximate prediction and not an exact match. To fix small differences when calibrating the simulator, it may be necessary to make small changes to certain parameters in the simulation models until the outputs predicted by the simulator match the performance of the plant.

The parameters that need to be changed should be those for which reliable data are not available from the data-gathering task described above and which have a big effect on the model's predictions. If a parameter affects model performance, this can be found out through sensitivity analysis and engineering knowledge and experience. When there are big differences between what was expected and what was seen, it means that the information that was put in was not very accurate. Most of the time, the part of a simulation exercise that takes the most time is calibration. This includes both measuring and estimating parameters.

Different calibration strategies have been used in various modeling studies. These strategies vary in terms of how the influent wastewater is characterized, how kinetic parameters are estimated, which parameters are chosen for calibration, which priorities are given priority within the calibration process, etc. No consistent technique was used to undertake the calibration study, making it difficult, if not impossible, to compare ASM calibrations and do internal quality checks. Concerns about the accuracy of activated sludge system models have prompted the recent proposal of systematic calibration techniques to aid in the calibration of full-scale models. BIOMATH, HSG, STOWA, and WERF are the four currently utilized calibration methods. Firstly, information about WERF, STOWA and HSG calibration methods is given. Then, detailed information about the BIOMATH calibration method used in this thesis is given.

2.6.4 The WERF model calibration protocol

This calibration protocol reflects North American (U.S. and Canada) practice of ASM calibration and is based on consultants' and researchers' experiences modeling full-scale activated sludge treatment facilities for a variety of uses. The proposed WERF protocol does not have a general structure or schedule that summarizes each step (Melcer et al., 2003).

The general approach of the WERF protocol is summarized as follows: The physical plant data, influent data, and plant performance data are provided in the first stage, which also involves setting up the plant configuration in the simulator. Additional information on the WWTP under investigation is acquired in the second stage. This stage entails gathering previous data, taking new measurements (both full-scale and lab-scale), and explaining the underlying assumptions in full detail. The calibration phase, which is the third stage, is carried out differently depending on the calibration level (a tiered approach). Model validation is the fourth phase. Finally, the model is prepared for full-scale implementation after successful validation.

2.6.5 The STOWA model calibration protocol

The BOD and physical-chemical measurements are used in the STOWA calibration protocol to describe the influent wastewater. Although findings from physical-chemical approaches are repeatable and reliable, the BOD method results in a significant amount of ambiguity when determining the inert particle fraction ($f_{P)}$ (Roeleveld et al., 2002). To extend the STOWA protocol, Weijers (1999) has investigated the use of model-based interpretation of the BOD profiles using a simplified ASM1 model. More research on the repeatability of the BOD method might significantly enhance this aspect of the STOWA process.

The protocol doesn't say anything about biomass characterization, which is figuring out how much autotrophic and heterotrophic biomass was in the WWTP at the beginning. Also, the design of dynamic measurement campaigns, which is the most expensive part of a model calibration study, isn't talked about in detail, nor are there any guidelines or comments about them.

The STOWA protocol is the only one that gives an idea of how long each step of the calibration process will take. The protocol is based on a number of WWTPs that have been studied, so it may not work for all systems. Settling and biological characterization are talked about in detail, but hydraulic characterization of the aeration tanks gets less attention.

2.6.6 The HSG model calibration protocol

This protocol provides general guidelines for documenting and conducting a calibration study that can be applied universally. Its objective is to establish a reference standard for model calibration and validation studies that meet the highest requirements. The goal is to create a systematic documentation of the entire calibration process, with no feedback loops in the scheme, implying that all calibration steps are straightforward and do not require internal checks.

In contrast, the HSG guidelines do not focus on standardizing the calibration process itself, but rather on ensuring that all crucial factors are considered to guarantee a high-quality simulation study. While this allows modelers more flexibility in determining the experimental methodologies to be used, it may raise questions about the systematization of the calibration procedure. Not imposing certain experimental methods may conflict with the ultimate aim of the protocol, which is to create a standard for the overall calibration study. Furthermore, new practitioners may not be adequately guided by the protocol in selecting appropriate experimental methods for parameter determination and influent characterization.

The responsibility of selecting the parameters to be calibrated is also left to the practitioners. However, sensitivity analysis is recommended to identify the most sensitive parameters, which can assist in selecting parameter subsets for calibration. Nevertheless, it is necessary to have quantitative criteria to rank the parameters, as proposed in studies by Weijers et al. (1997) and Brun et al. (2002).

2.6.7 The BIOMATH model calibration protocol

There are four main steps and 12 modules in the BIOMATH activated sludge model calibration protocol (Vanrolleghem et al., 2003).

The first step is to determine the goal(s) of the modeling study. Next, a decision is made as to what information needs to be collected from the activated sludge (AS) plant in order to achieve the modeling study objective. Depending on the overall assessment of whether the objectives are met, some of the modules may be skipped.

Detailed information on the activated sludge plant is gathered in the second step. This step includes mass transfer (hydraulic and oxygen transfer), biological, settling and influent characterization. In addition, the experimental or laboratory-scale study incorporates the Optimal Experimental Design (OED) technique. For the mass transfer, settling, and biological models, steady-state modeling is achieved by averaging influent and operational parameters (e.g., ASM).

In the third step, the activated sludge model is fully calibrated using the dynamic influent data and parameter values obtained from full-scale or laboratory experiments.At the final stage, decisions will be made upon eventual re-iteration of a number of the modules. The suggested procedure for modeling the treatment plant is refined based on an earlier protocol published by Petersen. (2000).

2.6.8 Verification of the Calibrated Simulator

Once the simulator has been calibrated, the calibration is verified using one or more additional sets of plant performance data obtained under various loading situations. The modeling engineer will decide on a case-by-case basis whether to accept the verification's conclusions. To fine-tune the calibration of the simulator, it may be essential to iterate between verification and calibration.

2.6.9 Application of the Simulator

A calibrated simulator can be used for a variety of tasks. Steady-state analysis can be used to set the basic design sizes and operating conditions for treatment process components. Dynamic analysis can be used to size the aeration system for the maximum loading conditions that are wanted. For process analysis, troubleshooting, and operational decisions, both steady state and dynamic analysis can help answer "what if?" questions. A simulator is often useful for figuring out if an existing facility can meet a certain quality limit for treated effluent. Lastly, for operator training, both steady state and dynamic analysis can be used to show the effects of different operator decisions and to give an idea of how carbon, nitrogen, phosphorus, and solids change over time and space in the treatment plant.

2.6.10 Activated Sludge Plant Simulators

In the research, design and operation of wastewater treatment systems, simulators have a huge potential advantage. At present there are a number of simulators available that run a combination of the models that are available. Typically, simulators offer a graphical user interface that allows the user to configure the unit processes of the plant. Most of the simulators allow the user to select from a number of models that are appropriate for the unit processes to be represented.

Choosing the right simulator is critical. Each simulator runs its own unique collection of models. In addition, the simulator user not only selects the required process, but also configures the flow rates, recycle streams, and influent wastewater characteristics. The user can either manually enter values for parameters, such as kinetic constants for biological growth and stoichiometric constants for processes or use predefined values. The simulator solves the system of equations to predict the effluent characteristics throughout the plant based on the process layout, input parameters, and selected model. Eight major WWTP simulators have been commercially available around the world. These are listed in Table 2.4. The simulators that are now accessible include SUMO, GPSX, EFOR, STOAT, BioWin, ASIM, SIMBA, WEST, AQUASIM, and AQUIFAS.

Table 2.4. Activated sludge simulators surveyed.

Simulator	Vendor	Location
ASIM	EAWAG (Swiss Federal Institute for Environmental Science &Technology)	Switzerland
SUMO	Dynamita	France
GPS-X	Hydromantis, Inc.	Canada
BioWin	EnviroSim Associates Limited	Canada
SIMBA	IFAK-System GmbH	Germany
EFOR	DHI, Inc.	Denmark
WEST	DHI	Belgium
STOAT	WRc Group	United Kingdom

In this study, Sumo software is used to simulate wastewater processes. Sumo is a powerful, open-source, multi-purpose simulation environment that has been developed for environmental modeling, in particular for the modeling of municipal and industrial wastewater treatment plants. There is a wide range of BNR plant configurations that can be simulated in Sumo.

Sumo models are written in an Excel-based open process source code language called SumoSlang (Dynamita, 2021).Depending on the simulation mode, Sumo can simulate traditional biokinetic models in dynamic or steady-state, mixed equilibrium kinetic models, and direct algebraic models. Sumo is shipped with internally researched and developed whole plant models as well as focus models (e.g. focusing on sulfur, high rate plants, and the fate of nitrogen and greenhouse gases). The seven most well-known published models are also included in the Sumo Museum for the removal of N and P.

Sumo models are executable via multiple interfaces. The most common is Sumo", an intuitive graphical user interface. (Figure 2.29)

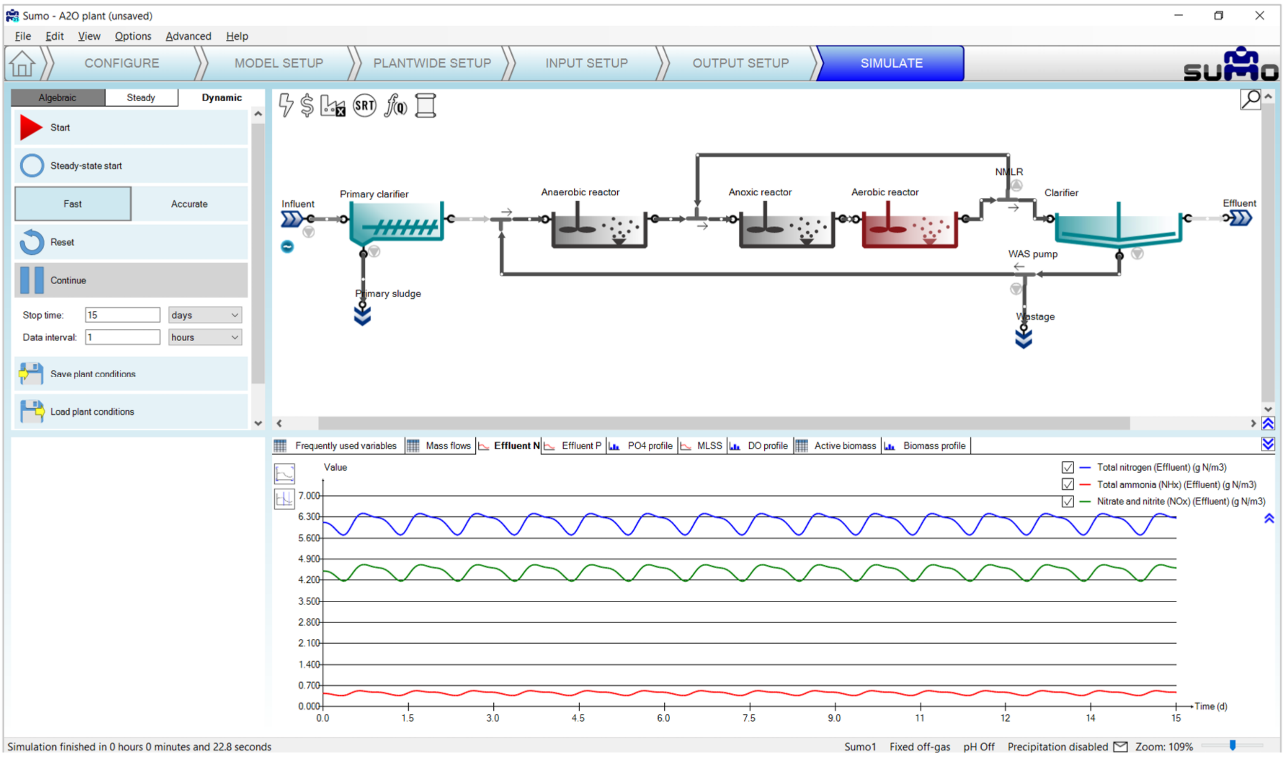


Figure 2.29. The Sumo graphical user interface.

2.7 Energy Requirement of Municipal Wastewater Treatment Plant

The large amounts of energy and chemicals required to operate wastewater treatment plants are rapidly depleting natural resources. To ensure efficient use of resources and sustainability of treatment performance, energy, and chemical consumption should be minimized. Therefore, the most effective solution would be a detailed evaluation of wastewater treatment plants in terms of operation and initial investment costs, as well as a detailed evaluation in terms of resource requirements, especially at the project stage (Insel, 2009; Güneş et al., 2010). Life cycle assessment (LCA) is an increasingly used tool in the tendering of wastewater treatment plants. It can be used in the design phase to formulate the environmental impact analysis as a function of time as well as the operating energy costs during the operating phase of the treatment plant (ISO14040; 2006; Remy et al., 2011; Kinnear et al., 2010).

According to statistical data obtained from different countries, it can be seen that as the number of nutrient removal wastewater treatment plants increases, the total amount of energy used in the treatment plants also increases within the rate of use across the country. Energy is used at every stage, from the collection of municipal wastewater to the discharge of the effluent in compliance with the limits of the receiving environment. Therefore, designing and operating wastewater treatment plants that meet effluent standards and consume less energy has become increasingly important (Nuhoğlu, 2012).

In order to make the cost of the technology and operation of a wastewater treatment plant affordable, the selection of technology, equipment, and process control should be evaluated in detail in terms of integrated waste management approach and operating costs, and their alternatives should be considered in the design phase. The energy consumption of wastewater treatment plants varies as a function of local conditions, the level of service provided, the type of treatment selected, and the type of sludge removal important (Nuhoğlu, 2012).

The ratio of energy consumption in wastewater treatment plants to energy consumption in general is 0.7% in Germany (Mauer, 2009) and 0.1-0.3% in the USA (Stillwell, 2010). Annual energy consumption per capita for municipal treatment is in the range of 30-50 kwh/PE in Austria (Queensland Govt., 2005) and 30-60 kwh/PE in Germany, of which 15-40 kwh/PE is biological.

According to the analysis, in Germany, in the population of 1000 to 5000 inhabitants, the consumption of energy for the treatment of wastewater can be as high as 150 kWh per PE (Kolish et all.,2009). The annual consumption of electrical energy in a conventional activated sludge system is approximately 0.4 kWh/m3 or 30 to 40 kWh per PE (Mauer,2009). Table 2.5 shows municipal WWTP electric energy consumption data (Geilvoet ,2010).

Table 2.5. Municipal WWTP electric energy consumption data.

Processes	Mean	Range	Unit
Total plant	27	20-34	kwh / PE-a
Aeration tank	15	10-20	kwh / PE-a
Dewatering	0,13	0,04-0,27	kwh / kg ds
Transport	0,016	0,0028-0,0333	kwh/m^3 km

According to a study supported by the German Federal Environment Agency, based on population, the ratio of electricity consumption for wastewater treatment in settlements with up to 1000 inhabitants is 75 kWh/PE, while the same ratio decreases to 32 kWh/PE in settlements with more than 100,000 inhabitants. Figure 2.30 shows both ratios graphically (Kolisch et al.,2009).

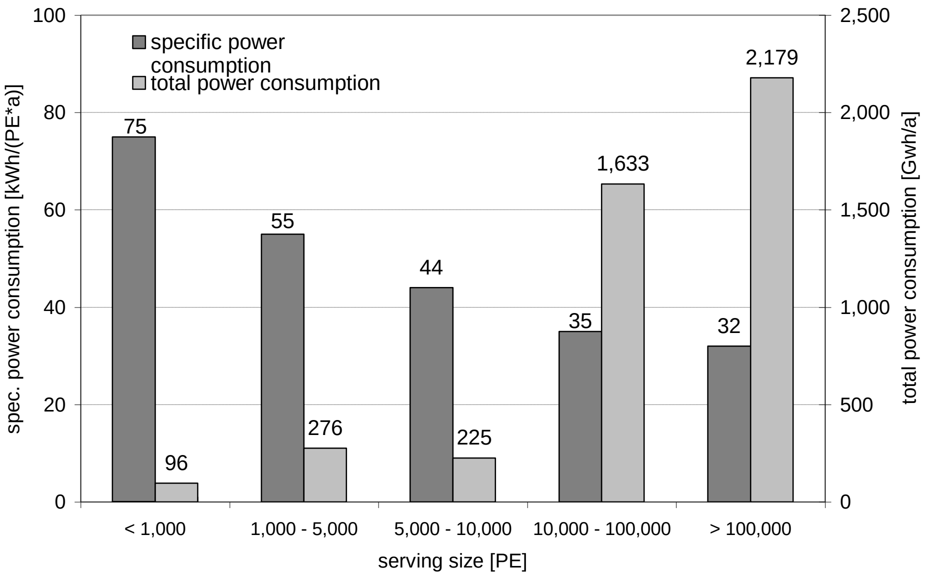


Figure 2.30. Specific and absolute power consumption of sewage plants in Germany.

The energy demand mainly represents the power needed for the treatment units. Pumping, especially when considering the entire process of collecting and transporting wastewater to the treatment plant, is an energy-intensive process. When designing wastewater treatment plants, it is important to ensure that the area where the plant is to be built has suitable hydraulic conditions. Significant energy savings can be made during the operation of the treatment plant if the wastewater is transported by gravity or with less pumping power important (Nuhoğlu, 2012).

Aeration provides oxygen to the bacteria to treat and stabilize the wastewater. Oxygen is needed by the bacteria to allow biodegradation to occur. Aeration is the most critical component of an activated sludge wastewater treatment system (WERF, 1996). The level of wastewater treatment and energy consumption are directly affected by a well-designed aeration system. It is the most energy-consuming part of the wastewater treatment process, accounting for 50% to 60% of the energy consumption of the entire wastewater treatment plant (WERF, 1996).

The heat required for digestion is obtained by generating electricity from biogas through cogeneration. Alternatively, it can be obtained by burning of biogas in a boiler. On the other hand, more energy is required for indirect processes (e.g. production, transport) if chemicals are used for processes such as dewatering, phosphorus removal. To understand the direct and indirect energy consumed by WWTPs, the energy consumed by using chemicals needs to be taken into account important (WERF, 1996, Nuhoğlu, 2012).

The secondary treatment process generally consumes the largest amount of energy. The typical distribution of energy consumption in an activated sludge treatment plant is shown in Figure 2.31. This distribution would remain relatively constant (Ast et al., 2008) regardless of the size of the plant.

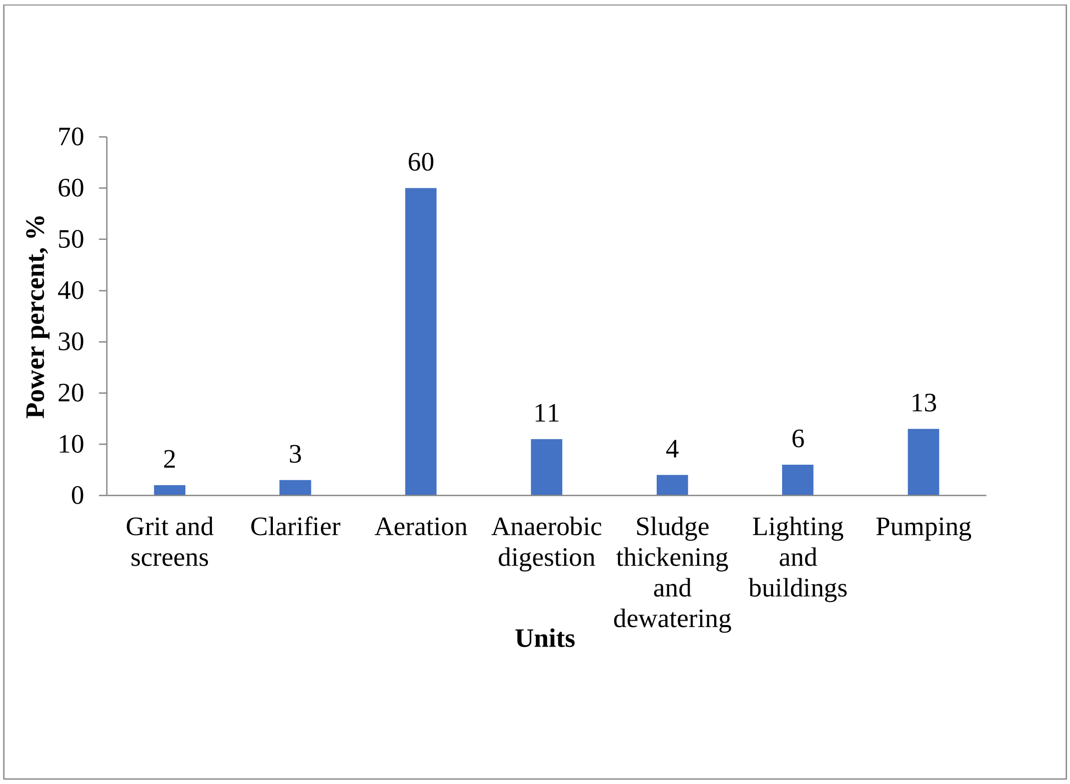


Figure 2.31. Electricity requirement for typical activated sludge facilities.

According to the data of the Turkish Statistical Institute (TUIK), 65% of the population of our country lives in big cities. The annual energy demand for operation can be calculated to be 922,000,000 kWh, assuming that the average annual cost of operation per person is 20 kWh/person/year (Insel, 2011). Even an improvement of 1%, which can be easily achieved in the operation, will result in a significant cost saving.

In 2012, there were 24 wastewater treatment plants in Istanbul. There were 9 pre-treatment plants. The remaining 15 plants were biological or advanced biological treatment plants. In these WWTPs, 60% of the wastewater volume was pretreated, and 39% of the wastewater volume was treated using advanced biological treatment. Only 1 % of the amount of wastewater was run biological treatment (Güneş, 2010). The energy consumption of these plants was about 5% of the total energy consumption in Istanbul. 13% of the energy consumed is used for wastewater treatment plants.

The pre-treatment plants in Istanbul consist of coarse and fine screens, pumping stations, grit removal, deep sea discharge and, in general, odor removal systems. Pumps are the major energy consumers in wastewater pretreatment plants. Ozone generators are used in the odor removal unit, which is also the other major energy consuming equipment. The energy consumption for belonging to the pretreatment plants in Istanbul are given in Table 2.6 and Figure 2.32 (Gunes et al., 2010).

Table 2.6. Consumed energy for pretreatment WWTP in Istanbul.

Plant	**Population**	**Average flowrate (m^3/d)**	**Energy cons. (kwh-a)**	**Energy cons. (kwh/m3)**	**Energy cons. (kwh/PE.a)**
Yenikapı	3,000,000	750,185	20.526.017	0,08	6,84
Baltalimanı	3,000,000	315,940	21.854.272	0,19	7,28
K.Çekmece	1,400,000	162,040	4.400.779	0,08	3,14
B.Çekmece	620,000	80,117	2.961.881	0,10	4,78
Kadıköy	2,230,000	556,782	17.621.107	0,09	7,90
Küçüksu	1,400,000	31,990	3.877.527	0,33	2,77
Paşabahçe	2,000,000	24,147	897.718	0,10	0,45
Kumbaba	184,000	14,680	548.607	0,10	2,98

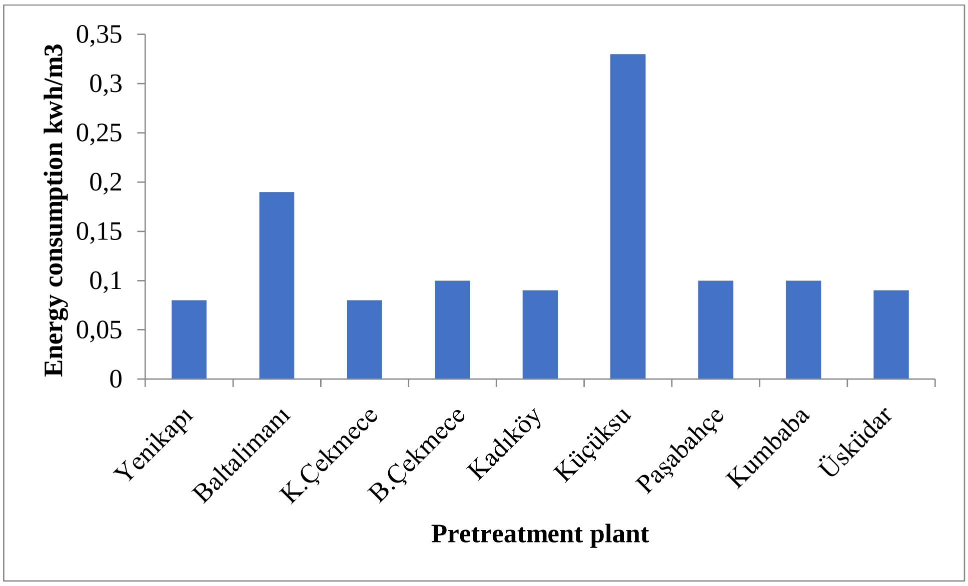


Figure 2.32. Energy consumption of pretreatment WWTPs in Istanbul (Gunes et al., 2010).

Biological treatment applications include pre-treatment, aeration tanks, final sedimentation, and sludge treatment units. The wastewater treatment plant in Gümüşyaka is an activated sludge system with a sequencing batch reactor (SBR). The energy consumption values of the biological pretreatment plants in Istanbul are shown in Table 2.7 and Figure 2.33 (Güneş, 2010).

Table 2.7. Consumed energy for biological WWTP in Istanbul.

Plant	**Population**	**Average flowrate (m^3/d)**	**Energy cons. (kwh-a)**	**Energy cons. (kwh/m3)**	**Energy cons. (kwh/PE.a)**
Bahçeşehir	26,000	6,863	900,984	0.36	34.65
Çanta	15,000	3,567	394,660	0.3	26.31
Ömerli	2,000	447	119,613	0.73	59.81
Gümüşyaka	6,800	2,107	130,214	0.17	19.15

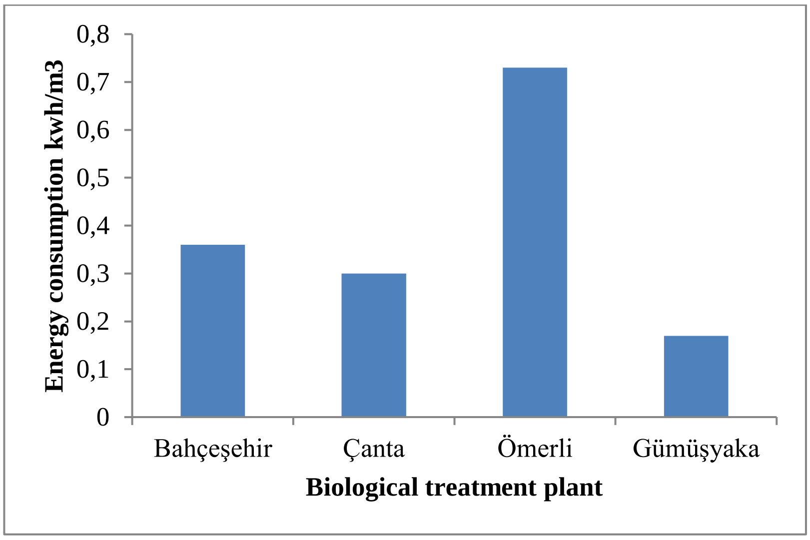


Figure 2.33. Energy consumption of biological WWTPs in Istanbul.

The advanced biological treatment plants in Istanbul consist of pre-treatment, anaerobic, anoxic, and aerobic tanks, secondary sedimentation, sludge treatment units, odor removal, and energy production units. Three of these advanced biological treatment plants (Ataköy, Tuzla, Paşaköy) have cogeneration units for electricity and heat with different capacities. The energy consumption of these plants is shown in Table 2.8 and Figure 2.34 (Günes, 2010).

Table 2.8. Consumed energy for advanced biological WWTP in Istanbul.

Plant	**Population**	**Average flowrate (m^3/d)**	**Energy cons. (kwh-a)**	**Energy cons. (kwh/m3)**	**Energy cons. (kwh/PE.a)**
Ataköy	2,400,000	315,000	28,998,052	0.25	12.08
Tuzla	4,500,000	317,464	30,068,784	0.26	6.68
Paşaköy	2,500,000	113,689	17,696,464	0.43	7.08
Akalan	800	145	57,101	1.08	71.38
Belgrat	200	32	22,688	1.95	113.44
Örencik	1000	123	58,643	1.31	58.64
Kömürlük	500	134	37,886	0.77	75.77
Sahilköy	2000	95	73,236	2.12	36.62
Yeniköy	800	109	25,641	0.64	32.05
Terkos	7,000	1,582	132,642	0.23	18.95

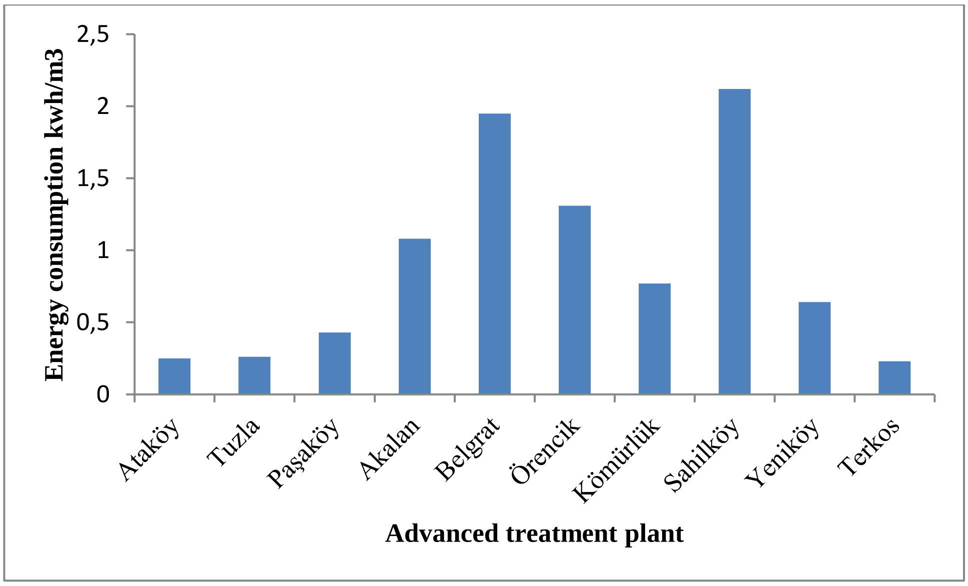


Figure 2.34. Energy consumption of advanced biological WWTPs in Istanbul .

The traditional activated sludge (CAS) method has a significant downside in that it has a high energy consumption rate, which accounts for roughly 60 percent of the overall operating expenses (Yang et al., 2020). Conventional treatment systems are especially sensitive to rising energy prices because of the substantial amount of energy they consume. Additionally, the utilization of traditional activated sludge systems results in the production of significant quantities of waste biological sludge, which necessitates the incurring of extra costs for treatment and/or disposal (Boguniewicz-Zablocka et al., 2020, Mauer et.al., 2009).

At the moment, wastewater treatment plants prefer a system that gets rid of nutrients and uses old-fashioned primary sedimentation. Only about 25–30% of the organic matter that goes into the plant is sent to the anaerobic digestion process. In recent years, more attention has been paid to reactive primary clarifiers with the goal of carbon diversion to improve process performance in terms of removing nitrogen and/or making biogas (Güneş et al., 2019). In the reactive primary clarifier, organic matter is redirected by bio-adsorption of carbonaceous matter on active biomass and separation by solids settling. The fact that organic carbon and nutrients are separated at the beginning of the system is a promising improvement over single sludge systems. People are paying a lot more attention to High-Rate Activated Sludge Systems (HRAS) for carbon management, like the A/B (Adsorption/Bio-oxidation) process, because they can reroute carbon to improve the overall performance of the process (Jimenez et al., 2015). A unique thing about the A-stage is that 85–90% of the organic matter is taken out of the main stream without aeration by using the adsorption property of HRAS. Activated sludge's bio-flocculation capacity aids in organic carbon capture for biogas generation under low sludge age circumstances; nevertheless, high rate systems still have difficulties with biomass separation during the final clarifying stage (Guven et al., 2019; Jimenez et al., 2015; Van Winckel, 2014). In contrast to the high rate "A-system," the raw influent is mixed with a percentage of return activated sludge before being sent through a reactive primary clarifier (Ekama, 2015; Güneş et al., 2019; Hu et al., 2001). Thus, soluble biodegradable COD, which is necessary for EBPR, is not consumed, but rather retained by phosphorus-accumulating organisms (PAOs). There is still room for development in those systems in order to obtain higher process performance while maintaining stringent discharge restrictions.In this regard, hybrid systems combined with biofilm processes and reactive clarifiers might be a potential option for cost-effective and dependable process modifications.

Moving bed biofilm reactor (MBBR) and integrated fixed film activated sludge (IFAS) hybrid systems have gained popularity because they enhance nutrient removal efficiency over conventional suspended activated sludge systems (Ødegaard et al., 2014). MBBR was developed for the purpose of biologically treating various types of wastewater, and it has been effectively utilized for the purpose of treating wastewater with a refractory nature in both domestic and industrial wastewater (Leyva-Díaz et al., 2017; Tang et al., 2021). Hybrid systems are easy to use, small, can be updated easily, have a small footprint, low capital costs, and are stable when the load changes. MBBR technology can be easily put into place by adding carriers to an existing activated sludge system. No extra building is needed. (Singh et al., 2018). Activated sludge processes have been combined with different commercial carrier technologies, such as Bio-2, Linpor®, Captor®, and HybasTM, for use in MBBR or IFAS (Randall and Sen, 1996).

Reactive primary clarifiers are helpful because they trap and redirect organic matter before oxidizing it in later treatment phases (Güneş et al., 2019; Hu et al., 2001). The process efficiency of hybrid systems lets them work at low sludge retention times and keep the biomass on the carrier for longer, which could also make biogas recovery more efficient.The comparison of the three sludge sources from suspended biomass (i.e., the activated sludge process) and attached biomass (i.e., the moving bed bioreactor (MBBR) and packed bed biofilm (PBBR) reactors) showed that the attached growth processes were easier to control in the aerobic processes because they would leave a smaller footprint because they had a shorter hydraulic retention time (HRT)(Azizi et al., 2018). Hybrid activated sludge systems have been used in experiments to remove both nutrients and specific pollutants (Bassin et al. 2012; Castro et al., 2017).

2.8 Information on the WWTP which is the subject of the study

In this study, experimental studies have been carried out at three wastewater treatment plants in Istanbul. The plant where the pilot and modelling studies were conducted will be referred to as "**the WWTP**" in the following sections of the thesis.

The WWTP treats wastewater from an Istanbul catchment area. It discharges into the Sea of Marmara.The WWTP consists of coarse screen, inlet pump station, fine screen, grit removal unit, pre-sedimentation tanks, bio-P units, nitrification (N) /denitrification (DN) tanks, final sedimentation tanks, sludge thickening, anaerobic mesophilic sludge digestion, sludge dewatering, and thermal sludge drying units (Process Design Report, 2007). The activated sludge process consists of 3 lines and is designed to have an average daily wastewater treatment capacity of 390,000 m^3/day.

The WWTP was designed as a single-sludge, step-activated sludge system according to the German design criteria (ATV-131E, 2000). Prior to biological treatment, 50% of the raw wastewater entering the plant is subjected to pre-sedimentation with a hydraulic retention time of 1 hour. The remaining 50% of the wastewater flow bypasses the pre-sedimentation tanks and is combined with the wastewater from the pre-treatment effluent. After pre-sedimentation, 56% of the effluent flow is directed to Cascade-1 and the remaining 44% is directed to Cascade-2. Nitrate recirculation occurs within each cascade.

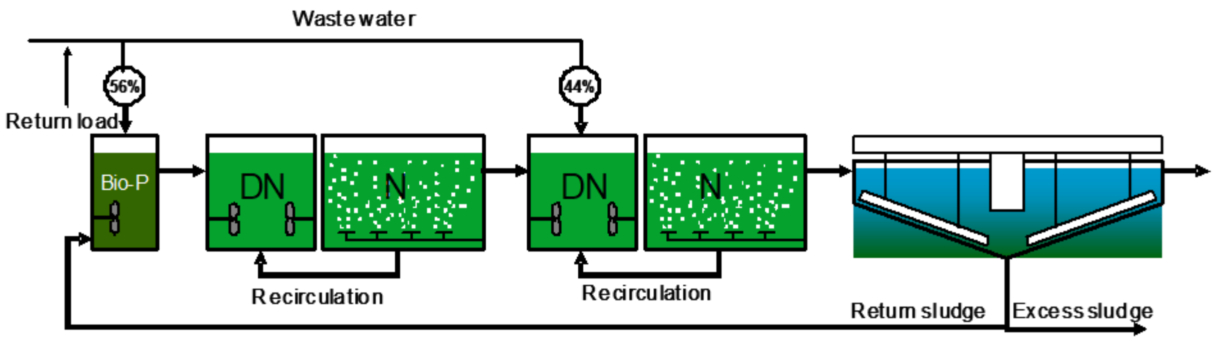


Figure 2.35. System configuration of the WWTP.

Table 2.9 is a summary of the average and maximum hourly flow rates and the inlet wastewater characterization used in the design (Process Design Report, 2007). The inlet COD concentration is 600 mg/L and the corresponding TKN is 60 mg/L. As a result, the input COD/TKN ratio can be calculated as 10. The total nitrogen required to be provided at the outlet is 10 mg N/L and the removal rate for nitrogen should be at least 84%. On the other hand, the inlet total phosphorus concentration was accepted as 8 mgP/L in the design. The accepted concentration for total phosphorus (TP) is 2 mg P/L in the design report. However, in the Regulation on the Treatment of Municipal Wastewater, this value is 1 mgP/L. Accordingly, the total performance for TP unit must be at least 88%.

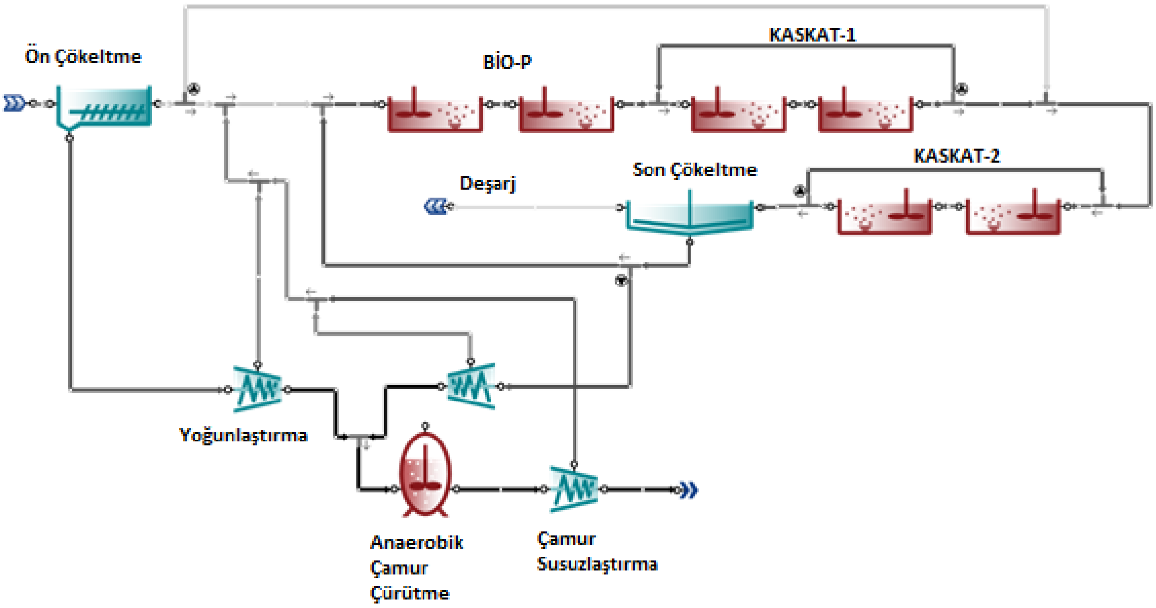


Figure 2.36. Process flow diagram of the WWTP

Table 2.9. Design values of the WWTP.

Parameter	**Unit**	**Raw Wastewater**	**Discharge Limit**
Total COD, C_T	mgO_2/L	600	<125
Biochemical Oxygen Demand, BOD_5	mgO_2/L	300	25
Suspended Solids, AKM	mg/L	500	35
Total Kjeldahl's Nitrogen, TKN	mgN/L	60	*10
Total Phosphorus, TP	mgP/L	8.0	3
Maximum Hourly Flow Rate, $Q_{h,max}$	m^3/hour	16250	-
Average Flow Rate, $Q_{h,\ avg}$.	m^3/hour	21250	-

The dimensions used in the design report where the German standard (ATV-131E, 2000) is preferred are summarized in Table 2.10 (Process Design Report, 2007). From the design and operational data, it is understood that the aerobic and total sludge ages of the system were selected as 4.93 days and 8.80 days, respectively. In the design report, at the aerobic sludge age calculated for nitrification (4.93 days), it was assumed that nitrification was complete and all of the ammonium nitrogen (NH_4-N) was converted to nitrate nitrogen (NO_3-N).

Table 2.10. Design information of the WWTP.

Design Parameter	**Unit**	**Value**
Minimum design temperature	°C	15
Total sludge age, SRT	day	8.80
Aerobic sludge age, SRT_a	day	4.93
Total N/DN reactor volume, V_T	m3	240,000
Anoxic volume ratio, V_{DN}/ V_T	%	44
1. Internal recirculation ratio, IR_1	[-]	4.29
2. Internal recirculation ratio,, IR_2	[-]	2.00
First cascade MLSS concentration, X_{MLSS1}	mg/L	5.67
Second cascade MLSS concentration, X_{MLSS2}	mg/L	4.37
Peak factor for carbon,, f_C	-	1.2
Peak factor for nitrogen, f_N	-	2.0
Maximum hourly oxygen demand, OU_h	kgO_2/hour	10,840
Average air flow rate, Q_{air}	Nm^3/ hour	213,440
Excess sludge, P_{XT}	kgSS/day	132,450

According to the Regulation on Urban Wastewater Treatment (2006), in order to meet the discharge limit for total nitrogen (TN<10 mgN/L), the total aeration tank volume is 240,000 m^3 and the anoxic volume is 105,600 m^3 (V_{DN}/V= 44%). After the biological treatment system, the separation of activated sludge from the treated water is provided with the help of 12 circular final settling tanks with a diameter of 44 meters.

3 MATERIALS AND METHODS

As it is known, the activated sludge process is used in municipal wastewater treatment plants, and biological nitrogen removal is realized by nitrification and denitrification processes. Nitrification is a process in which ammonium nitrogen (NH_4-N) is converted to nitrate nitrogen (NO_3-N) with the help of autotrophic microorganisms. In denitrification, heterotrophic microorganisms play a role, using the biodegradable organic matter in the wastewater to convert nitrate nitrogen (NO_3-N) into harmless nitrogen gas (N_2).

Nitrification takes place in an oxygenated environment, while denitrification takes place in an oxygen-free (anoxic) environment. To ensure biological nitrogen removal, nitrate produced in the nitrification stage is directed to the anoxic phase for denitrification. Autotrophic bacteria performing nitrification are the group of bacteria most affected by environmental factors (temperature, pH, inhibitors, alkalinity, etc.) and are known as the limiting factor in the biological nitrogen removal process (Randall et al., 1992; Henze et al., 2008). Therefore, the inability to produce nitrate (NO_3-N) as a result of the problem in the nitrification process causes the wastewater treatment plant to be unable to remove nitrogen at the desired efficiency.

Using the operational data of "the WWTP" for the years 2012-2017, the design criteria (ATV-131E, 2000) were analyzed. By conducting activated sludge activity experiments, the extent of the nitrogen removal problem of the plant was revealed. Finally, the plant capacity required for nitrogen removal in the plant was calculated considering the actual conditions. In order to evaluate the total nitrogen removal performance of the WWTP, the approach summarized below was adopted.

- evaluation of historical data of the WWTP
- determination of effects of primary sedimentation to the WWTP
- evaluation of nitrogen removal performance
- laboratory experiments for biological and kinetic characterization
- performance evaluation aided by process simulation
- determining the required process volumes

3.1 Evaluation of Historical Data of the WWTP

The influent pollutant concentrations of some WWTPs and in Istanbul were measured. The variation in the pollutant loads of COD, TKN, TP, TSS parameters for the years 2012-2016 were compared with the values of the plant design. At the same time, the nitrogen removal efficiency of the plant was studied on a seasonal basis. The effect of temperature on the process efficiency was evaluated. Table 3.1 shows the 2011 influent values of three operating wastewater treatment plants in Istanbul and the concentrations indicated in the literature. The plants named as WWTP 1 and WWTP 2 are located on the Asian side and are advanced biological treatment plants. The plant named as WWTP is the advanced biological treatment plant where the pilot plant was operated during the thesis study.

Table 3.1. Influent concentrations of Europe, America and some WWTPs in Istanbul

Parameter		**Literature**	**Güneş, 2011**		
mg/L	**Henze et.al., 2001**	**Metcalf & Eddy, 2014**	**WWTP-1**	**WWTP-2**	**the WWTP**
COD	500-1200	339-1016	155-874	182-1230	264-1180
BOD_5	230-560	133-400	76-452	87-790	132-850
TN	30-100	23-69	17-81	23-95	26-97
Ammonium Nitrogen	20-75	14-41	-	12-70	13-65
Phosphorus	6-25	3,7-11	1-9	3-15	3-16
TSS	250-600	130-389	87-544	154-1018	147-1075
VSS	200-480	101-304	48-401	102-742	89-832

Figure 3.1 shows the hourly variation of COD, TSS, Total Nitrogen (TN), and Total Phosphorus (TP) parameters at the inlet of “the WWTP” in Istanbul (dry air flow rate). It has been observed that the concentrations of pollutants in the wastewater entering the plants are similar to the values reported in the literature.

According to experiments with samples taken hourly from the composite influent in the WWTP, the concentrations of the pollutants and the variation of the flow rate show different trends. During the day, the highest influent is received between 7:00 a.m. and 10:00 a.m., while the highest TN and COD concentrations are received at 12:00 p.m. and 22:00 p.m., respectively. (Figure 3.1b and Figure 3.1c). Therefore, the peak conditions experienced by the plant for COD and TN are at different times. Temporal variation in characterization is important for proper design and process control architecture in biological nitrogen removal. TN and TP concentrations follow the same trend throughout the day as shown in Figure 3.1c and Figure 3.1 d (İnsel et al., 2021).

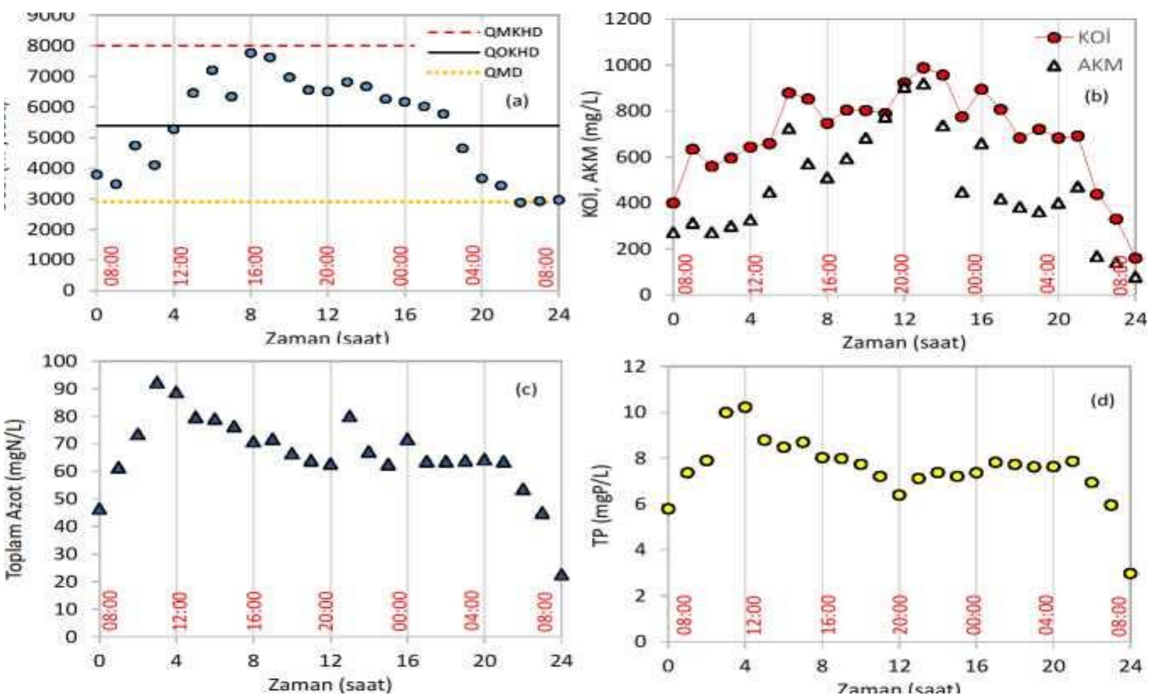


Figure 3.1 Hourly inflow and pollutant concentrations of the WWTP)

3.2 Determination of Effects of Primary Sedimentation to the WWTP

In wastewater treatment plants, primary sedimentation tanks prior to biological treatment, the removal efficiencies obtained for SS and COD parameters vary depending on the hydraulic retention time. Therefore, it is necessary to know the concentration of pollutants in the wastewater entering the biological treatment units for suitable process design in wastewater treatment plants.

The WWTP has 2 pre-sedimentation tanks. The primary sludge that is settled here is pumped to the sludge treatment units. The tanks are designed for hydraulic retention time between 0.50 and 1 hour.

The flowrate is divided into two lines for safe denitrification and nitrification. It is planned to send 50% of the raw water directly to the biological treatment and 50% of the remaining raw water to the pre-sedimentation tanks during the normal operating time. In addition, depending on the operating conditions, distribution of the wastewater flowrate can be adjusted.

Table 3.2. Flow distribution of pre-sedimentation tanks and biological treatment units.

	Percentage of wastewater entering pre-sedimentation tanks	**Percentage of wastewater entering biological treatment units**
Normal operation period / Case 1	50%	50%
Case 2	67%	33%
Case 3	0%	100%
Case 4	100%	0%

The data presented in Figure 3.2 were obtained from the experiments carried out on 8 composite samples taken from the WWTP. Therefore, for the correct design of the biological process, it is important to determine the effect of primary sedimentation on flow-weighted composite samples collected in plants where primary sedimentation tanks are planned. Especially in combined sewer systems, the organic content (VSS/TSS) in the influent is lower due to the high content of sand/silt (inorganic matter). On the other hand, the organic content of primary sedimentation sludge in combined systems is also lower than in separate systems. In combined sewer systems, the organic content of influent wastewater is usually in the range of 25-75% (Fan et al., 2015; Wentzel et al., 2002; Yan et al., 2014).

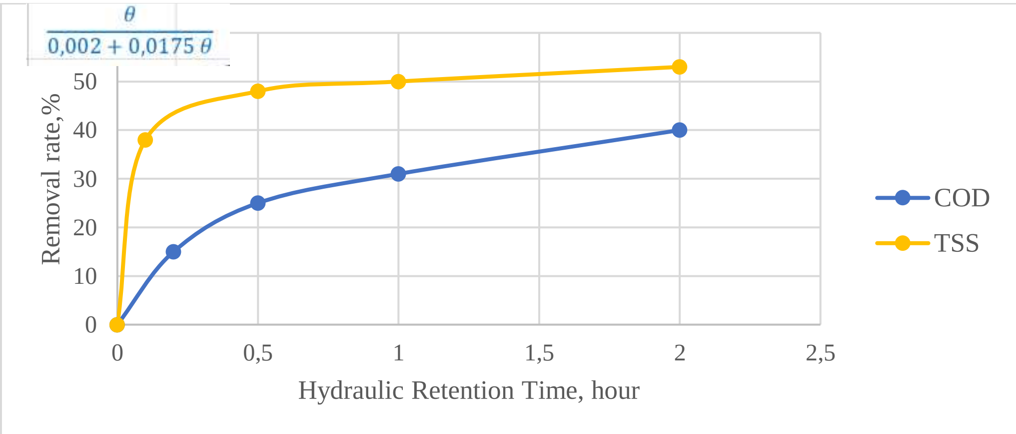


Figure 3.2. Removal rates of TSS and COD due to HRT in the WWTP.

3.3 Evaluation of Nitrogen Removal Performance

The time-dependent variation of nitrogen concentrations in the WWTP influent for the last 5-year period is analyzed. In Table 3.3, the average flow (m^3/day) and pollutant loads (kg/day) used in the design and exposed to the plant between 2012-2016 were analyzed.

Table 3.3. Average flow rates and pollutant loads of the WWTP.

Parameter	**Unit**	**Design**	**2012**	**2013**	**2014**	**2015**	**2016**
Avg. Flow Rate	m^3/day	*390000*	328650	389280	402200	400341	359520
COD	kgCOD/day	*234000*	218925	238205	224118	214785	208773
SS	kgSS/day	*195000*	141793	150646	156334	153813	139299
TN	kgN/day	*23400*	21077	25072	22341	20727	21653
TP	kgP/day	*3120*	2994	2966	2858	1941	1808

It is understood that the hydraulic capacity of the plant was fully utilized and the average COD, SS, TN and TP loads used in the design were not exceeded. In the WWTP design, the lowest wastewater temperature was selected as 15°C for the sludge age for the nitrification process. Under these conditions, daily ammonium nitrogen (NH_4-N) and total nitrogen (TN) measurements at the plant discharge for one year and changes in process temperature are presented in Figure 3.3 (İnsel et al., 2021).

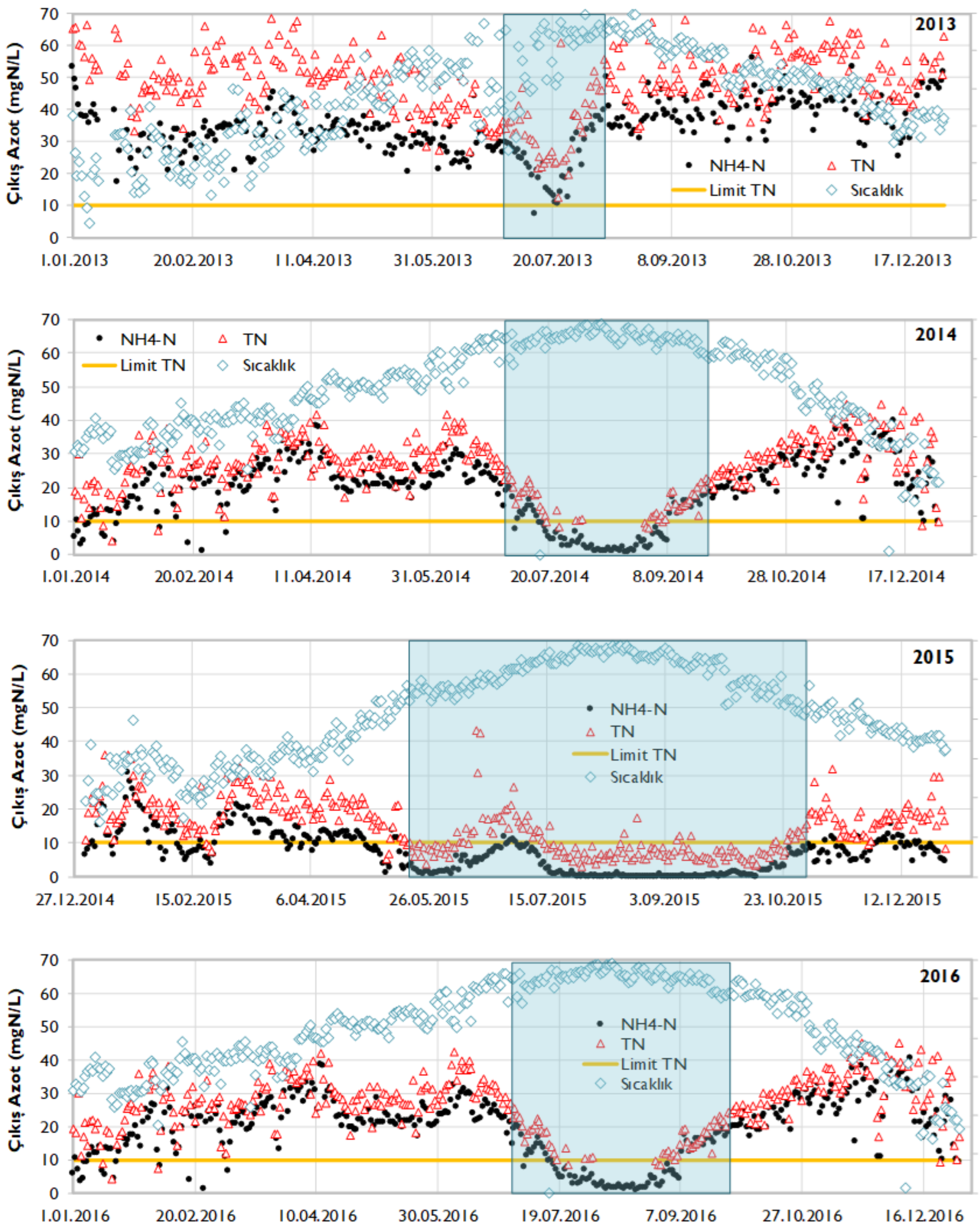


Figure 3.3. Daily total nitrogen and phosphorous discharge of the WWTP

Total nitrogen effluent concentrations are shown on the left axis and temperature of the wastewater on the right axis. When Figure 3.3 is analyzed, it is seen that when the variable process temperature increases according to the seasons and reaches 20-22°C and exceeds this temperature, a decrease in the effluent nitrogen concentrations of the plant is reported.

In particular, it is seen that TN<10 mgN/L standard can be achieved for certain periods in summer months. On the other hand, nitrogen removal deteriorates as the water cools down again in autumn and winter. It is understood that ammonium nitrogen (NH_4-N) remains in the discharge water in periods when the temperature is low, and the activated sludge system cannot realize Total Nitrogen (TN) removal due to the inadequacy of the nitrification process. As a result, it became necessary to measure the actual process rate of nitrification and then the experimental determination of the growth rate of nitrifying bacteria was organized. The obtained nitrification rate was compared with the literature and the parameters used in the design.

3.4 Laboratory experiments for biological and kinetic characterization

Accurate characterization of the pollutants entering a wastewater treatment plant is critical for the design and operation of biological processes. Accurate determination of influent characteristics is only possible if the sample analyzed is representative of the entire influent. Composite samples, which are flow-weighted mixtures of samples collected at regular intervals throughout the day, are typically used for this purpose. The samples used in the experiment were taken on June 7, 2011, from the composite sampling devices in the wastewater treatment plant.

Some experimental studies were conducted for biological and kinetic characterization of the inlet wastewater at the WWTP between June and July 2011. Table 3.4 describes the conducted laboratory experiments, the location of those experiments, the methods used for measurements, and the duration of those experiments.

Table 3.4. Details on the experimental study.

Experimental Characterization	Test Location	Method	Experiment Period
COD fractions	ITU Environmental Biotechnology lab .	Ekama et al., 1986b; Spanjers et al., 1995; Ubay et al., 1998, İnsel et al ., 2003a;	1 day
Nitrification rate	the WWTP	Melcer et al ., 2003	8 - 20 days
Denitrification rate	ITU Environmental Biotechnology lab .	Henze , 1987	1 day
Phosphorus release/uptake rate	the WWTP	Henze et al ., 2002	1 day
Sludge settling velocity test	the WWTP	Pitman , 1980, Takacs et al ., 1991	1 day

3.4.1 COD Fractions

The chemical oxygen demand is commonly used to characterize carbonaceous material in municipal wastewater for modeling purposes (COD). It is vital to estimate the fractions of COD that are biodegradable and/or inert (non-degradable) because all modeling and design calculations are based on biodegradable organic matter. The biodegradable COD component was calculated using respirometric measurements and modeling studies that measured the respiration of heterotrophic bacteria under laboratory settings using composite wastewater samples from the WWTP. (Ekama et al., 1986b; Spanjers et al., 1995; Ubay et al., 1998) For this aim, 1 L of activated sludge from the WWTP was aerated until the internal respiration level was determined using a respirometer. Figure 3.4 depicts the fractionation of total influent COD into the various fractions utilized in nutrient removal system design and modeling.

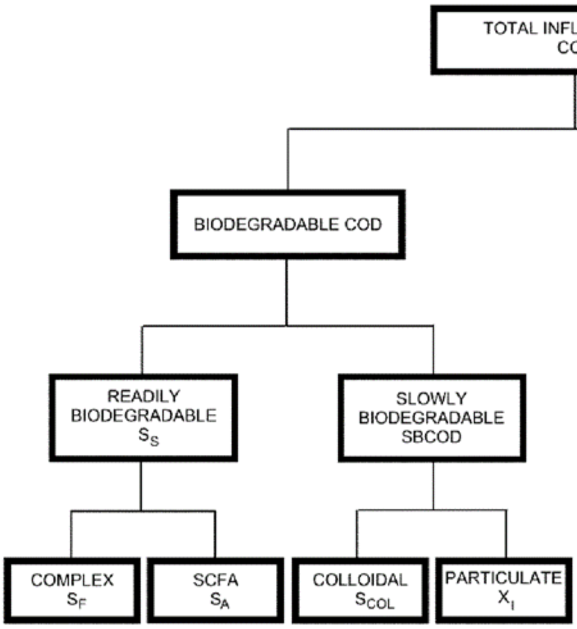


Figure 3.4. COD fractions.

Endogenous respiration level represents the biomass activity at the absolute starvation level when there is no substrate in the medium (wastewater) in which the biomass taken from the treatment plant, and the substrate accumulated in the biological sludge and in the cell is also consumed. Depending on the character of the wastewater in which the tested biomass is found and the growth phase in which the biomass is found, the time to reach the intrinsic respiration level varies. For this reason, the respiratory level (MPV) was monitored during the experiment, depending on the respirometer. The level at which the Oxygen Uptake Rate curve remains constant for a long time at the lowest level it descends is considered the intrinsic respiration level. In the experiment, when the internal respiration level was reached, 1 L of raw wastewater was added and the Oxygen Uptake Rate (OUR) measurement was carried out with the help of a respirometer.

It was aimed to determine the fraction of COD that can be separated from the area under the respirometric profile. Due to the applied loading rate and low hydrolysis rates, related components were determined with the help of modeling study (Ekama et al., 1986; Spanjers et al., 1995, Muller et al., 2002). In the evaluation of the whole respirometric profile, the dynamic modeling and system identification method was applied as described in Insel et al. (2003).

As a result of the experiment, the COD fractions, growth rate and hydrolysis rates were determined by parameter estimation. AQUASIM program developed by Zurich Technical University (ETH) was used for automatic parameter estimation (Reichert et al., 1998). VFA measurements in inlet wastewater using SCR-101H organic acid specific column at UV210 nM It was carried out by Schimadzu brand HPLC. Comparison of COD fractions of influent wastewater is given in Table 3.5 (Güneş, 2011). Also, the matrix used to determine COD fractions is given in Appendix B.

Table 3.5. Comparison of % COD fractions of influent wastewater.

Country	**Sı**	**Ss**	**Xı**	**Xs**
USA	5,0	16,0	13,0	66,0
Denmark	7,6	20,3	13,0	58,7
South Africa	5,0	20,0	13,0	62,0
Spain	8,5	18,3	24,9	48,3
Sweden	15,0	27,0	17,0	41,0
Switzerland	4,0	10,0	20,0	66,0
Italy	6,0	15,0	8,0	71,0
Germany	6,4	18,6	11,3	63,7
Netherlands	6,0	26,0	39,0	28,0
İstanbul				
WWTP-1	8,5	7,0	7,0	77,5*
WWTP-2	6,0	7,0	8,0	79,0*
The WWTP	11,4	10,0	7,0	71,6*

'About 30% of Xs in dissolved form

Table 3.6. Kinetic and stoichiometric parameters for the WWTP.

Parameter	**Notation**	**Unit**	**Temperature Correction Factor (0)**	**The WWTP (T=20°C)**	**Literature * (T=20°C)**
Nitrification Bacteria					
Yield coefficient	Y_A	g COD/g NH_4-N	-	0.24	0.24
Maximum growth rate	$P_{A,max}$	1/d	1.072	0.54	0.4-1.2
Endogenous rate	b_A	1/d	1,029	0,12	0,10-0,17
Self-saturation constant	K_{NH}	g N/m^3	-	1,0	0,5-1,5
Heterotrophic Bacteria					
Yield coefficient (aerobic)	Y_H	g COD/g COD	-	0,.66	0.60-0.68
Yield coefficient (anoxic)	Y_{HD}	g COD/g COD	-	0.54	0.5-0.6
Maximum growth rate	$P_{H,max}$	1/d	1,045	4.2	2.5-6.0
Half-saturation constant	K_s	g COD/m^3	-	17	5-20
Slow hydrolysis max. rate	k_h	1/d	1,030	1.0-1.5	0.5-3.0
Half-saturation constant for hydrolysis	K_x	g COD/m^3	-	0.06	0.02-0.15
Endogenous rate	b_H	1/d	1,020	0.24	0.15-0.25
The inert fraction of biomass	f_E	-	-	0.20	0.15-0.20

3.5 Experimental Determination of Nitrification Rate

The nitrifier growth rate is arguably the most important parameter in the design of activated sludge systems, especially for nutrient removal systems. Estimating the growth rate of nitrifying bacteria (μ_{AUT}) is therefore extremely important when designing of BNR. This is also true if process simulation is to deliver reliable results.

Two different experimental methods were used to measure the growth rate of nitrifying bacteria (μ_{AUT}) in the WWTP. The first one is the "High F/M (Nutrient/Microorganism) Experiment" which is conducted under batch conditions and the measurements are performed by measuring nitrite and nitrate (S_{NO}) (Melcer et al., 2003). The second experiment is based on Oxygen Uptake Rate (OUR) measurements under low F/M conditions and batch conditions (Spanjers et al., 1993). In both experiments, the measurement of the growth rate of nitrifying bacteria (μ_{NITO}) is obtained by modeling the experimental data. The theoretical background and experimental results of the experimental methods are summarized below.

3.5.1 High F/M Batch Test (Experiment-1)

The high F/M assay is one of the most widely used methods for determining the net growth rate (μ-b). A relatively low concentration of nitrifying mixed liquor is spiked with ammonia in the high F/M batch testThe basis of this method for estimating μ_{AUT} is that the initial nitrifier concentration is low. The growth rate of nitrifiers during the test is exponential.

In the experiment, the effluent from the wastewater treatment plant was used as dilution water and the experiment was conducted with a small amount of inoculum sludge from the aerobic reactor. At the start of the test, diluent and seed were added to the reactor, mixing and aeration were started, and reagents were added.

The initial volume of the experiment on the nitrification rate of the WWTP was 4 liters and the inoculum sample taken from the aerobic sludge was 70 mL. By adding ammonium chloride (NH_4Cl), the initial concentration of ammonium nitrogen (NH_4-N) was adjusted to 50 mgN/L. There was no temperature adjustment in the experiment. The water temperature was measured to be 27.5±0.3 °C throughout the experiment. The dissolved oxygen (DO) concentration did not fall below 5 mg O_2/L during the experiment and was not a limiting factor for nitrification. The pH value was maintained at 7.2 by the addition of CO_2 gas. In addition, at the beginning of the experiment, $NaHCO_3$ equivalent to 350 mg $CaCO_3$/L was added for alkalinity.

In the test, a small volume of nitrifying mixed liquor was added to a diluent (treatment plant secondary effluent in this case) to result in an initial seed concentration in the test of 30 to 35 mg VSS/L. Ammonium chloride was added in an amount that resulted in an initial ammonia-nitrogen concentration of about 120 mg/L. The pH of each test was checked with a pH probe about 3 times per day, and 0.5 g to 1.0 g of sodium bicarbonate was added if the pH dropped to 7.2. Aeration was controlled by a DO concentration controller, which cycled aquarium pumps on and off to maintain a DO concentration setpoint between 4 mg/L and 6 mg/L. Over a period of time, say 2 days, the increase in nitrite/nitrate concentration was monitored. The experimental setup prepared in the WWTP laboratories is shown in Figure 3.5.

(a) (b) (c)

Figure 3.5. High F/M nitrification experimental setup (the WWTP).

In the experiment, the time-dependent change in the oxidized nitrogen (SNO) concentration is calculated using the following formula (Melcer vd., 2003):

$$S_{NO,1} = S_{NO,0} + \frac{\mu_{NITO} \cdot X_{NITO}}{Y_{NITO} \cdot (\mu_{NITO} - b_{NITO})} \cdot [e^{(\mu_{NITO} - b_{NITO}) \cdot t - 1}] \qquad (3.1)$$

In the formula,

$S_{NO,1}$: concentration of the oxidized nitrogen at the time "t" (mgN/L)

$S_{NO,0}$: initial oxidized nitrogen concentration (mgN/L)

μ_{NITO} : maximum growth rate of nitrifying bacteria (day-1)

b_{NITO} : endogenous respiration rate of nitrifying bacteria (day-1)

Y_{NITO} : yield rate of nitrifying bacteria (gCOD/gN)

$X_{NITO,0}$: initial concentration of active nitrifying bacteria (mgCOD/L)

Nitrate NO3-N was 2.5 mg/L at the beginning of the experiment and 51 mgN/L at the end of the experiment (day 2). Figure 3.6 shows the results of the experiment. The oxidized nitrogen (NOx) was calculated using equation (1) above.

Ms Excel program was used to determine the maximum growth rate of nitrifying bacteria. At a process temperature of 27.5 °C, the maximum growth rate of the nitrifying bacteria was calculated as μ_{AUT} = 1.20 day^{-1}.

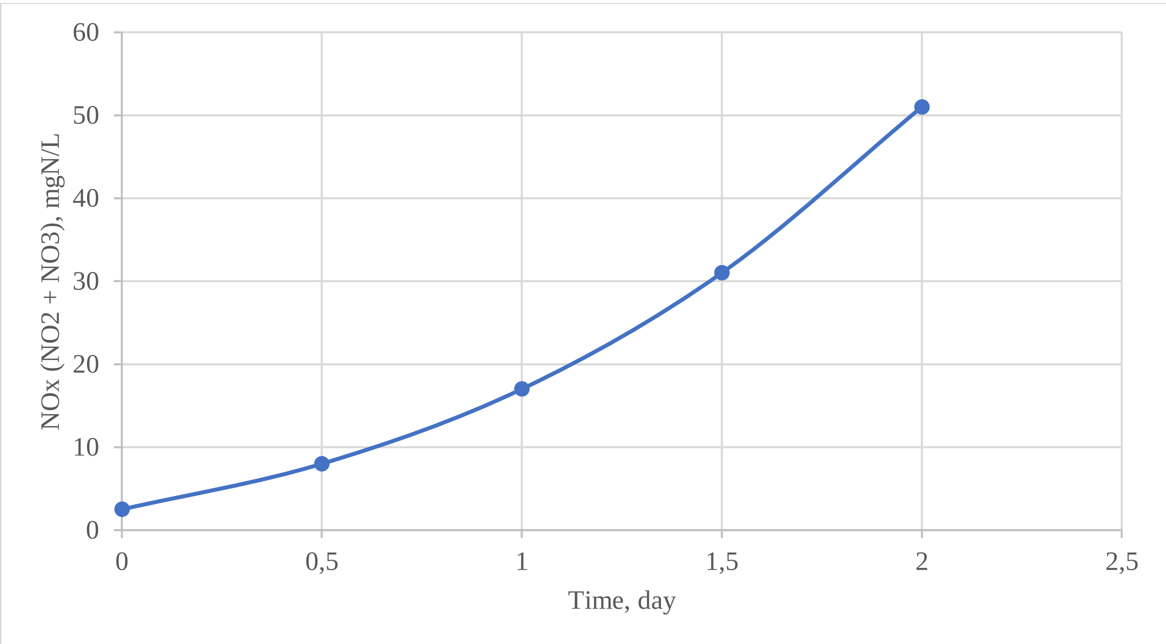


Figure 3.6. High F/M experiment nitrate measurements and calculation results

3.5.2 Respirometric Based Kinetic Experiments (Experiment-2)

Kinetic evaluation on the basis of Oxygen Uptake Rate (OUR) measurement is now widely used in the design of wastewater treatment plants (Insel et al., 2003b; Orhon et al., 2002; Spanjers et al., 1995; Spanjers et al., 2011). In this context, in parallel to the high F/M experiment, a nitrification rate experiment based on respirometric measurement was carried out. (Figure 3.7).

The activated sludge sample from the aerobic tanks of the WWTP was collected on July 2O, 2011. The sample was aerated for 24 hours. The reason for aeration of the sample was to avoid interference of slow biodegradable organic matter accumulation in the plant with Oxygen Uptake Rate (OUR) measurements.

As mentioned above, the MLSS concentration was measured to be 4450 mg/L in the activated sludge sample. Before starting the respirometric measurement, 1.5 liters of activated sludge was diluted with 1 liter of effluent from the treatment plant. Then, aeration was started. Respirometric measurements were performed with Applitek Ra-Combo 1000 (Belgium). The sampling frequency was one Oxygen Uptake Rate, OUR (mg O_2/L/h) sample per minute.

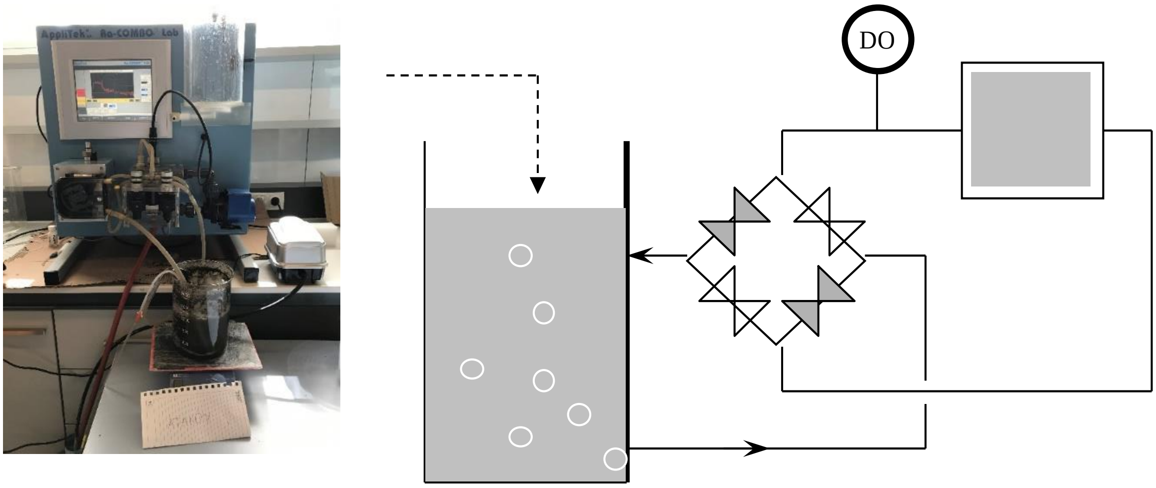


Figure 3.7. Continuous respirometer device and working principle.

After the Oxygen Uptake Rate (OUR) measurements reached a stable level (26 mg/L/h), ammonium chloride (NH_4Cl) was added to provide initial nitrogen concentrations of 7.5 mgN/L and 4 mgN/L, respectively. The experimental procedure was carried out as described by Spanjers and

Vanrolleghem (1995). The total duration of the experiment was 180 min. The DO profiles obtained are shown in Figure 3.8. The dissolved oxygen concentration was measured above 5 mgO2/L during the experiment. The experiment was conducted at room temperature (24°C). In addition, at the beginning and end of the experiment, the pH was measured to be around 7.5.

Ammonium chloride (NH_4Cl) was initially added to 2.5 L of oxygen-saturated sludge. In the first stage, OUR increased from 26 mg/L/h to 60 mg/L/h in 30 minutes. Oxygen consumed indicates that nitrification is occurring. Subsequently, due to the depletion of ammonium nitrogen, the OUR measurements returned to the previous level after 100 minutes. The second addition of NH_4Cl resulted in a similar OUR profile. The experiment was completed in less time because the nitrogen dose in the second addition was lower than the previous one.

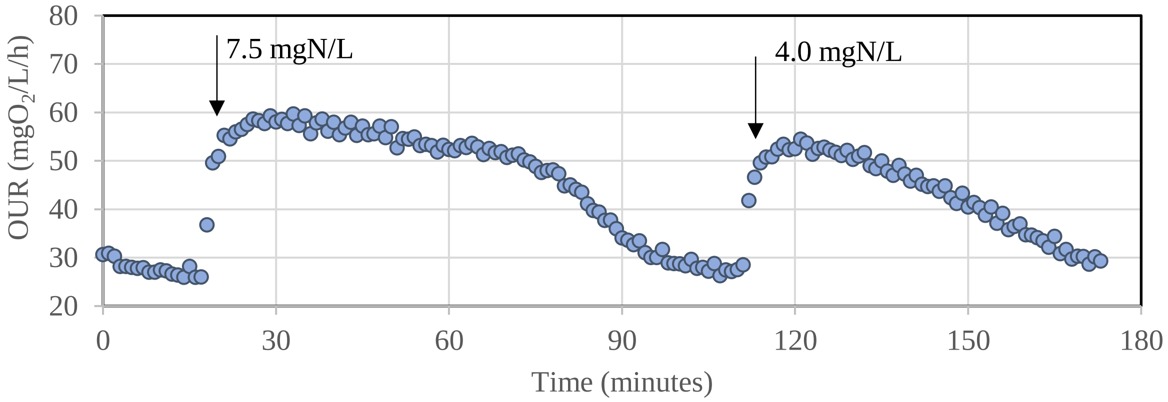


Figure 3.8. Oxygen Uptake Rate (OUR) measurement of the WWTP.

The mathematical model based on the Activated Sludge Model No.1 (Henze et al., 1987) was used to evaluate the obtained OUR profiles. In the model, the coefficients of maximum nitrification rate (μNITO) and half saturation constant (KNITO) were estimated (equation 4).

$$\frac{dS_{O2}}{dt} = +\frac{4.57-Y_{NITO}}{Y_{NITO}} \boldsymbol{\mu_{NITO}} \frac{S_{NH}}{\boldsymbol{K_{NITO}}+S_{NH}} X_{NITO} - (1-f_E) \cdot b_{NITO} \cdot X_{NITO} \qquad (3.2)$$

Other kinetic coefficients used in the simulations were taken from Table 3.6. In determining the kinetic coefficients, an attempt was made to make the calculated OUR graph resemble the experimentally obtained OUR profile (Figure 3.9). The program AQUASIM (Reichert et al., 1998) was used for the simulations and the least squares method and the automatic SIMPLEX algorithm were used for parameter estimation.

From the modeling studies conducted using respirometric data, the maximum growth rate of nitrifying bacteria was determined to be μ_{NITO}=0.83 day^{-1} and the half saturation coefficient to be K_{NITO}=0.7 mgN/L for two consecutive sets of experiments at 24°C process temperature.

Accordingly, if the temperature correction factor for growth (Θ) is taken as 1.072, the maximum growth rate corresponding to 20°C can be calculated as μ_{NITO}=0.5 day^{-1}. In the experiments, the endogenous respiration rate (b_{NITO}) for nitrifying bacteria at 20°C process temperature was taken as 0.17 day^{-1}. This parameter was obtained by model calibration of the large-scale plant in the next stage and is consistent with the kinetic values proposed by Melcer et al. (2003).

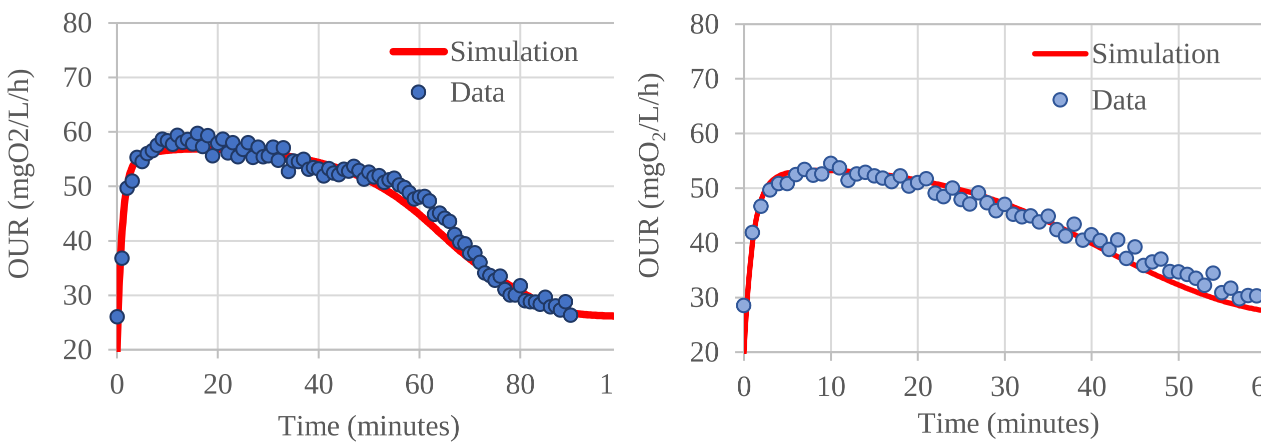


Figure 3.9. Modeling of OUR profiles in determining nitrification kinetics.

The temperature correction factor for the endogenous respiration rate of nitrification (Arrhenius) was accepted as Θ=1.035. The above-mentioned experimental results are summarized in Figure 3.9 together with the results of previous studies. It can be seen that 7 different temperature dependent experiments for the measured net growth rates (μ_{NITO}-b_{NITO}) are on the same trend line. Thus, the coefficients found in this study are consistent with the literature (Sözen et al., 1996; 2008; Insel, 2014).

In the design of the WWTP, the German design method (ATV-131E, 2000) was applied. For this purpose, the maximum growth rate of nitrifying bacteria (μNITO) at 20°C is between 1.0-1.6 day-1

(Wichern et al., 2003) according to the kinetic evaluation of the WWTPs of Gümmerwald, Neumünster, Lage, Koblenz, Hildesheim and Duderstadt in Germany where this method was applied.

The maximum growth rate (µNITO) of nitrifying bacteria in the WWTP is 0.5 day-1 (20°C). Compared to wastewater treatment plants in Germany, the growth rate is much lower (Henze et al., 2008; Wichern et al., 2003, Melcer et al., 2003). It has been stated that industrial discharges in the basin may be the reason for the low activity (Insel, 2014). Due to the low nitrification rate, much higher aerobic sludge ages are needed than the value calculated in German design standard (ATV-DVWK-A 131E, 2000) in the WWTP design report.

The graph of the minimum (aerobic) sludge age calculated by Insel (2021) using the experimental results and parameters obtained for the WWTP is shown in Figure 3.10. As can be seen from the design documents, according to ATV-131E (2000), the aerobic sludge age required for 15 °C (SRTa) was chosen to be 4.93 days. However, the process temperature must reach at least 22°C for nitrification to begin at the 4.93 day sludge age selected in the process calculation, as shown by the red line in Figure 3.11.

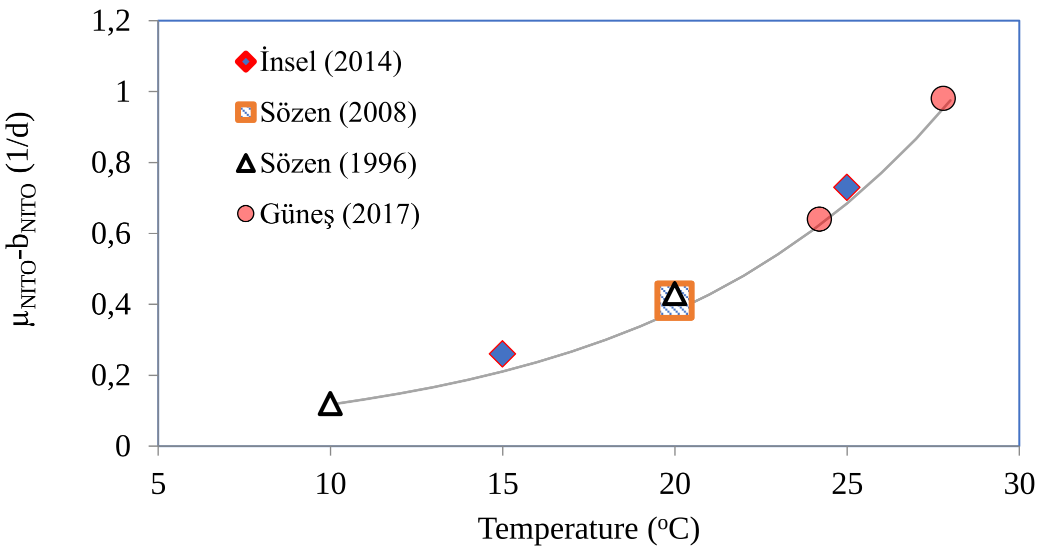


Figure 3.10. Temperature-dependent variation of net growth rate of nitrifying bacteria.

According to this, under conditions where the input load remains unchanged, nitrification should be expected to start when the water temperature reaches 22°C in the WWTP. This result is consistent with operational data (Figure 3.11) for the above years. The reason why nitrogen removal takes place in winter months and nitrification takes place in summer months in the WWTP is that the nitrification rate is sufficient in summer months (T>22°C) and insufficient in fall/winter seasons.

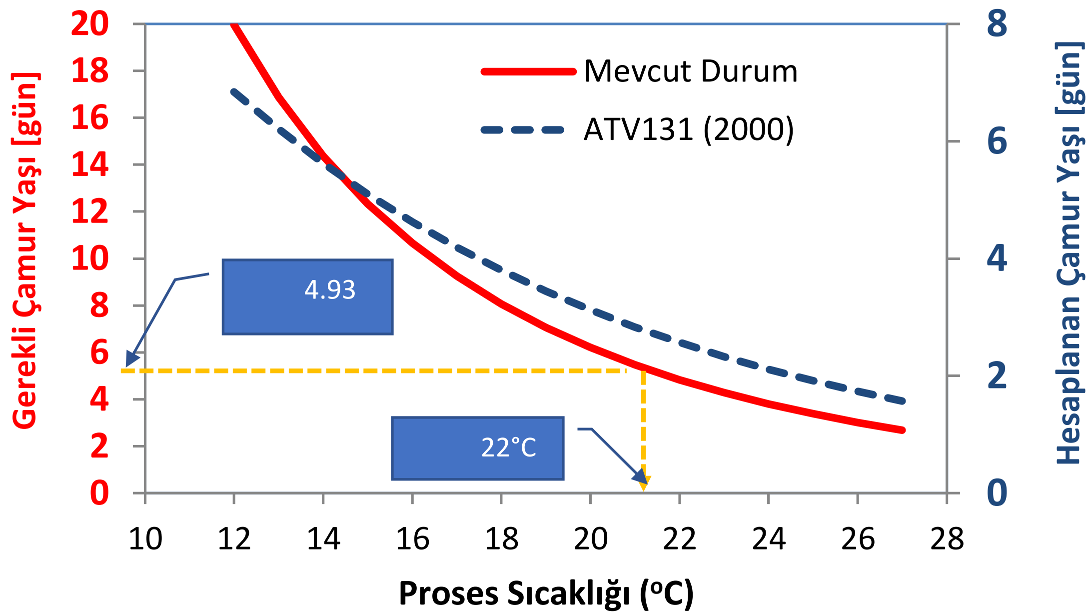


Figure 3.11. Minimum sludge age by temperature for nitrification.

3.6 Process simulation assisted performance evaluation

Nowadays, process simulators are widely used in wastewater treatment plant performance evaluation, process control and optimization. In particular, the effect of environmental factors such as dynamic pollution load and temperature variation on process efficiency can be easily analyzed.

Computer-aided evaluation of the nitrogen removal performance of the operation of the WWTP was carried out for July-August 2011 under dynamic input loads and increasing process temperature. Measurements of ammonium nitrogen (NH_4-N) and nitrate nitrogen (NO3-N) concentrations in the effluent were compared with the simulation results. In the model simulations, kinetic coefficients and temperature correction expressions related to nitrification were introduced into the process simulator. The kinetic parameters for removing organic matter and denitrifying were taken from the previous study (Insel, 2014).

The configuration of the WWTP was introduced into the simulation program as shown in Figure 3.12. The design data, volumes and pump capacities given in

Table 2.9 and Table 2.10 were introduced into the licensed SUMO program of Dynamita (www.dynamita.com). To reduce the simulation time, 3 lines were combined and calculations were performed on a single configuration. Daily average influent and conventional wastewater characterization required for dynamic simulation were also loaded. The information on COD and nitrogen fractions of the raw wastewater of the WWTP was adapted from previous studies (Insel, 2014; Okutman Taş et al., 2009). In the simulations, the processes were formulated as temperature dependent and daily data for the months of July-August 2011 were used. Other aspects that have been considered in the simulations are summarized below:

- the WWTP consists of pre-sedimentation tanks, process tanks (cascade-1, cascade-2), final sedimentation tanks, mechanical sludge thickening unit, anaerobic sludge digestion unit and sludge dewatering unit. The return pollutant loads (side stream water) inside the WWTP were considered in the calculations.
- Each process tank is a full batch reactor. It is divided into 4 main zones based on diffuser distributions and oxygen measurements within the reactor.
- The anaerobic digestion of the sludge takes place under mesophilic conditions. 30% VSS removal is achieved.
- The average MLSS concentration is approximately 8000 mg/L in Cascade-1 and 5000 mg/L in Cascade-2. Excess sludge is removed from the return sludge line. About 200 m3/day of primary sludge is discharged from the pre-sedimentation tank.
- The simulation period was carried out between the 1st of July and the 30th of July, when the temperature was 18-26 °C.
- First, steady state condition of the plant was realized, and then the calculations were carried out under dynamic loads and temperature variation. The steady state condition and dynamic simulations were performed as described in Vanrolleghem et al. (2003).

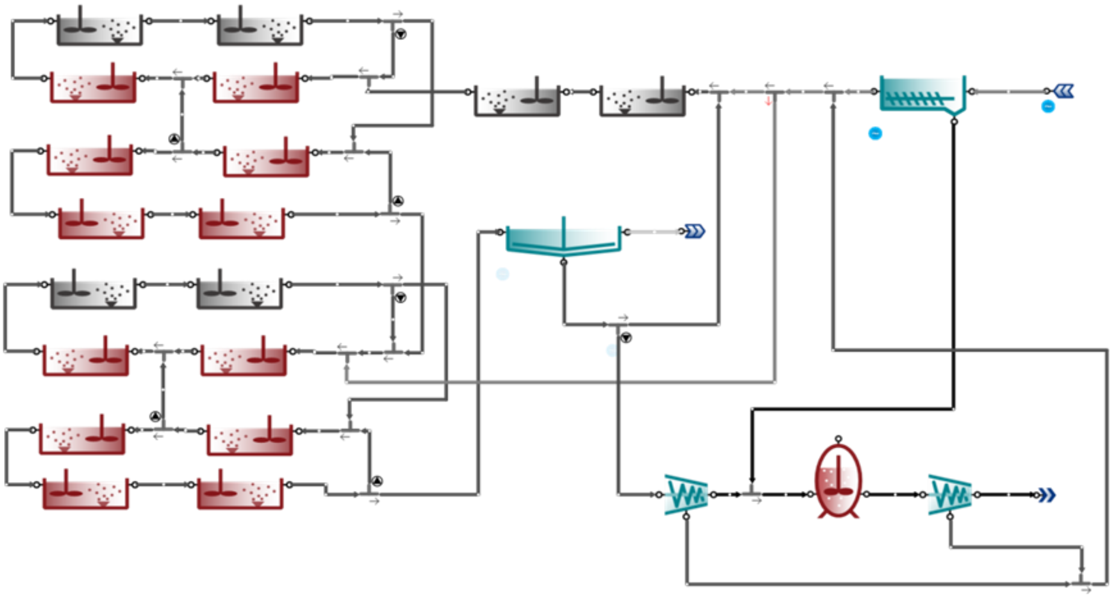


Figure 3.12. The WWTP configuration (SUMO Program).

As mentioned above, the steady-state simulation was performed for the average daily load in the first step. The dynamic simulation was then performed under the process temperature given in Figure 3.13 and the variable pollutant loads given. Figure 3.14 shows that the dynamic simulation results obtained for ammonium (NH_4-N) and nitric nitrogen (NO_3-N) were consistent with the daily measured values.

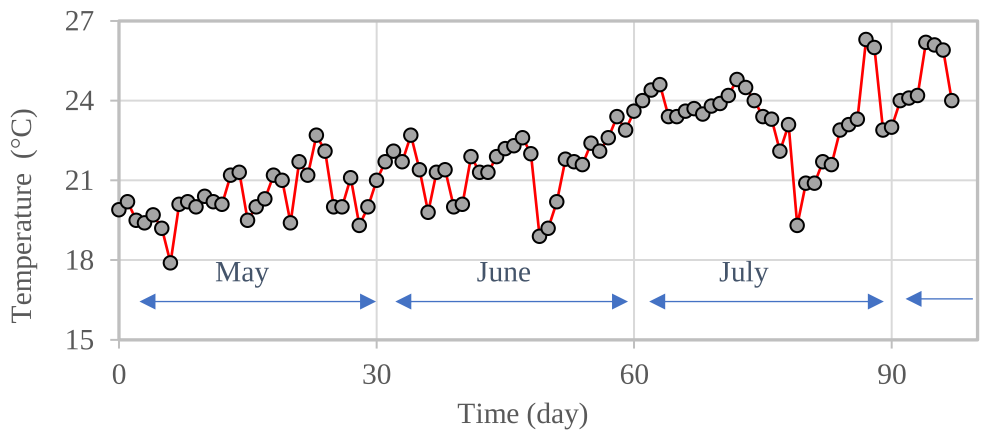


Figure 3.13. Daily temperature change in the process tank.

As mentioned above, nitrification started on day 40 when the process temperature exceeded 22°C. Therefore, the accuracy of the growth rates and temperature correction factors obtained from the kinetic experiments were verified using large-scale system data.

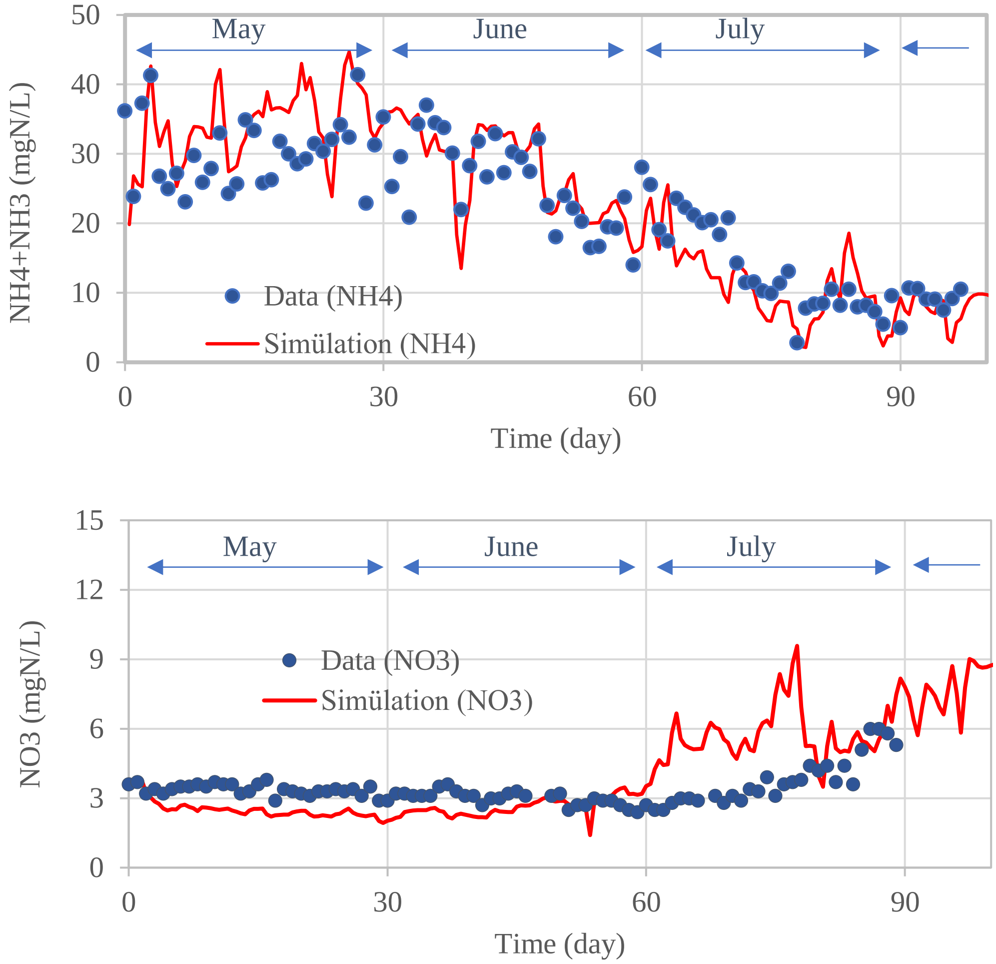


Figure 3.14. TN discharge concentrations of the WWTP.

In order to calculate the process volumes required for nitrogen removal in the WWTP, real conditions were considered, and recalculations were performed according to the standard method and model. In addition, the expected effluent nitrogen components were calculated for the plant designed at low (15°C) and high (25°C) temperatures.

3.7 Determination of required process volumes

The volumes required to achieve the total nitrogen (TN<10 mgN/L) limit at the lowest design temperature (15°C) were calculated with the help of simulation using all the kinetic parameters measured for the WWTP. In the simulation, model parameters were used which were specifically found in the WWTP with experimental studies. For comparison with the conventional single-sludge activated sludge system, the 5-stage Bardenpho design was selected in the project design (Figure

3.15). There are several alternatives in the selection of the configuration and the aim was to determine the total volume of the nitrification and denitrification in all the alternatives. Sludge treatment and anaerobic stabilization processes were assumed at design values. Pollutant parameters in all side stream flows were included in the calculations.

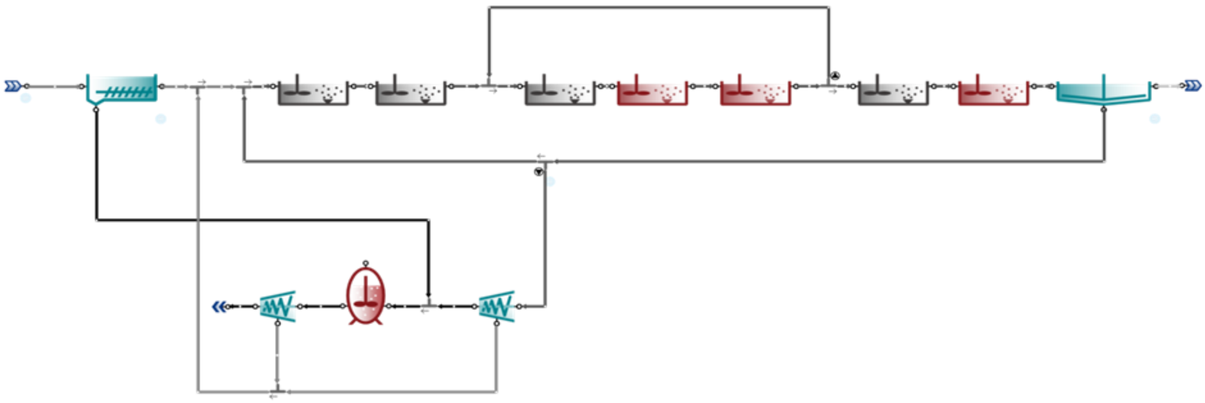


Figure 3.15. The WWTP configuration (5 Stage Bardenpho).

Table 3.7 summarizes the volumes required for the WWTP to meet the TN limit. The volumes calculated according to the project criteria in the calculation taking into account the actual kinetic rates were as follows

The volume capacity requirement was found to be 675,000 m3 instead of 240,000 m3. The safe aerobic sludge age (SRTa) of the activated sludge system was determined to be 18 days. The anoxic volume (VDN) required for the denitrification process is 236,000 m3. The corresponding sludge age is 28 days. In summary, the total nitrogen (TN) parameter remained below the discharge limit (TN<10 mg/L) in the solution of the system at 15 and 25°C process temperature (Figure 3.15). However, it is known that the efficiency of removing excess biological phosphorus (EBPR) can decrease with increasing sludge age (Insel et al., 2016; Onnis-Hayden et al., 2013). When selecting processes with higher sludge ages, this should be considered.

Table 3.7. Conceptual design of the WWTP.

Design Parameters	**Unit**	**Value**
Minimum temperature	°C	15
Total Sludge Age, SRT	day	28
Aerobic Sludge Age, SRT_a	day	18
Total volume of the aeration tank, V_T	m^3	675,000
Anoksic volume, V_{DN}	m^3	236,000
V_{DN}/V_T	%	35
Internal recirculation rate, IR	[-]	4.3

3.8 Biofilm Nitrification - Contact Denitrification System and Method

The present invention relates to a biofilm nitrification-contact denitrification system and method in which environmental technology and water recovery technologies are used and provides advanced removal of organic carbon, nitrogen, and phosphorous from wastewater.

The background of this invention is that the ammonia nitrogen present in wastewater is oxidized to nitrate nitrogen by nitrification through the activities of microorganisms in an aerobic environment. The process comprises the oxidation of ammonia nitrogen to nitrite ($N0_2$-N) by Nitrosomonas bacteria and the oxidation of nitrite up to the nitrate ($N0_3$-N) ions by the Nitrobacter bacteria. Denitrification is the transformation of oxidized nitrogen forms ($N0_2$-N $N0_3$-N) into molecular nitrogen via bacteria activity. As a common practice, nitrification and denitrification are carried out in single sludge systems by selecting suitable process conditions in the activated sludge systems.

In fluidized bed bioreactor (FBBR) systems, a media (usually plastic) is maintained in the reactor to increase the capacity of activated sludge systems. This media is used to retain more biomass in the reactor, thus increasing the capacity without requiring additional volume. The media used for FBBR are added to the single sludge systems wherein nitrification and denitrification processes take place together. Nitrification and denitrification processes are provided with the aid of attached biofilm growing on the surface of the media.

The competition between bacteria responsible for nitrification and organic carbon causes the washout of nitrification bacteria due to its comparably low yield and growth rate. Since the bacteria which carry out the nitrification process are in the same environment as the heterotrophic bacteria that carry out organic carbon removal, they cause oxidation of organic carbon under aerobic conditions leading to the loss of denitrification capacity. Particularly, in urban wastewaters wherein the nitrification rate is very low, this situation results in the loss of nitrogen removal efficiency or the selection of large reactor volumes.

High internal (nitrate) recirculation ratios are required in order to provide denitrification in conventional single sludge systems. Biological nitrogen removal in conventional systems requires recirculation of 4-5 times the inlet wastewater flow rate back to the anoxic tank (head of the bioreactor). This increases the operating costs due to the pumping costs. Furthermore, oxidation of the organic matter to C02 under aerobic conditions also negatively affects the potential of obtaining biogas from activated sludge via anaerobic digestion. In current practice, (in single-sludge systems), it is not possible to control nitrification and denitrification processes separately which is the major drawback. Nitrogen and phosphorus-containing fractions do not allow water reuse for irrigation purposes.

The above-mentioned problems have been overcome by means of the biofilm nitrification-contact denitrification system and method of the present invention. The objective of the present invention is to provide a biofilm nitrification-contact denitrification system and method which provide an advanced level of organic carbon, nitrogen, and phosphorous removal from wastewater. The hybrid process contains activated sludge compartments (selector, biofilm, denitrification, aerobic tanks etc.) integrated with a submerged biofilm reactor. The reactor conditions as well as process configuration enable to maintain certain reactions such as bio-flocculation, hydrolysis/fermentation, nitrification/denitrification, anoxic P removal, aerobic P removal and separation.

Another objective of the present invention is to provide a biofilm nitrification-contact denitrification system and a method that eliminates the nitrate internal recirculation required for denitrification and enables a reduction of the footprint of wastewater treatment plants. A further objective of the present invention is to provide a biofilm nitrification-contact denitrification system and method which enable nearly all of the settleable organic (biodegradable) matters to be used in the denitrification process.

Figure 3.16 is the schematic view of the biofilm nitrification - contact denitrification system.

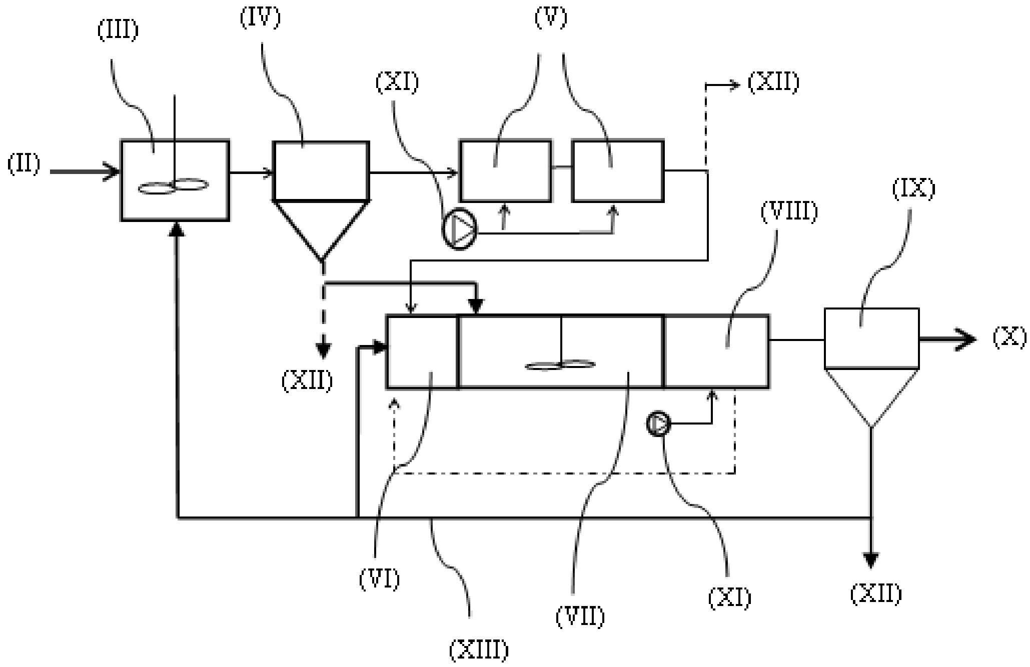


Figure 3.16. Schematic view of the patented biological process.

(I. Biofilm nitrification - Contact denitrification system; II. Wastewater Inlet, III. Selector Unit, IV. Intermediate settling tank, V. Aerobic Biofilm Tank,, VI. Deoxygenation (DeOx) Tank, VII. Contact Denitrification Tank, VIII. Aerobic Tank, IX. Final Sedimentation Tank, X. Treated Water Discharge, XI. Aeration/Mixing Unit, XII. Waste Sludge Discharge, XIII. Return Sludge Line)

3.8.1 Detailed Description of the Invention

A biofilm nitrification - contact denitrification system (I), which provides advanced level of nitrogen and phosphorous removal from wastewaters, basically comprises

- at least one wastewater inlet (II) through which the raw wastewater is delivered to the system,
- at least one selector unit (III) wherein the settleable organic substances in wastewater are mixed with the biomass,

- at least one intermediate settling tank (IV) wherein particulate organic substances are settled via biofloculation,
- at least one aerobic biofilm tank (V) wherein nitrification process is carried out,
- at least one DeOx tank (VI) which enables to reduce high dissolved oxygen concentration originating from the aerobic biofilm tank (V), at least one contact denitrification tank (VII) wherein denitrification process is carried out,
- at least one aerobic tank (VIII) which enables the nitrogen gas that is released as a result of denitrification process to be flushed out from the system and the residual ammonia nitrogen and/or dissolved organic substance in wastewater to be oxidized,
- at least one final settling tank (IX) which enables the treated water to be separated from the biomass,
- at least one treated water discharge (X) which is located at the outlet of the final settling tank (IX),
- at least one aeration/mixing unit (XI) which enables optimization of oxygen concentration in the aerobic tank (VIII) and the aerobic biofilm tank (V), and homogenous distribution of oxygen,
- at least one waste (excess) sludge discharge (XII) through which waste sludge produced in the system is removed,
- at least one return sludge line (XIII) which enables a part of the sludge settled in the system to return to the selector unit (III) and another part of it to return to the DeOx tank (VI). In a preferred application of the invention, a media is used to reduce the volume of the aerobic biofilm tank (V) and the contact denitrification tank(VII). Additionally, complete mixing of the DeOx tank (VI) and the contact denitrification tank (VII) is carried out by the mixers,
- Volume of the aerobic biofilm tank (V), in which nitrification is carried out, is selected depending on the specific surface area (m^2 surface/m^3 media volume) and unit nitrogen loading rate (gram N/ m / day) of the filler.

Figure 3.17. Pilot plant of Biofilm Nitrification - Contact Denitrification System.

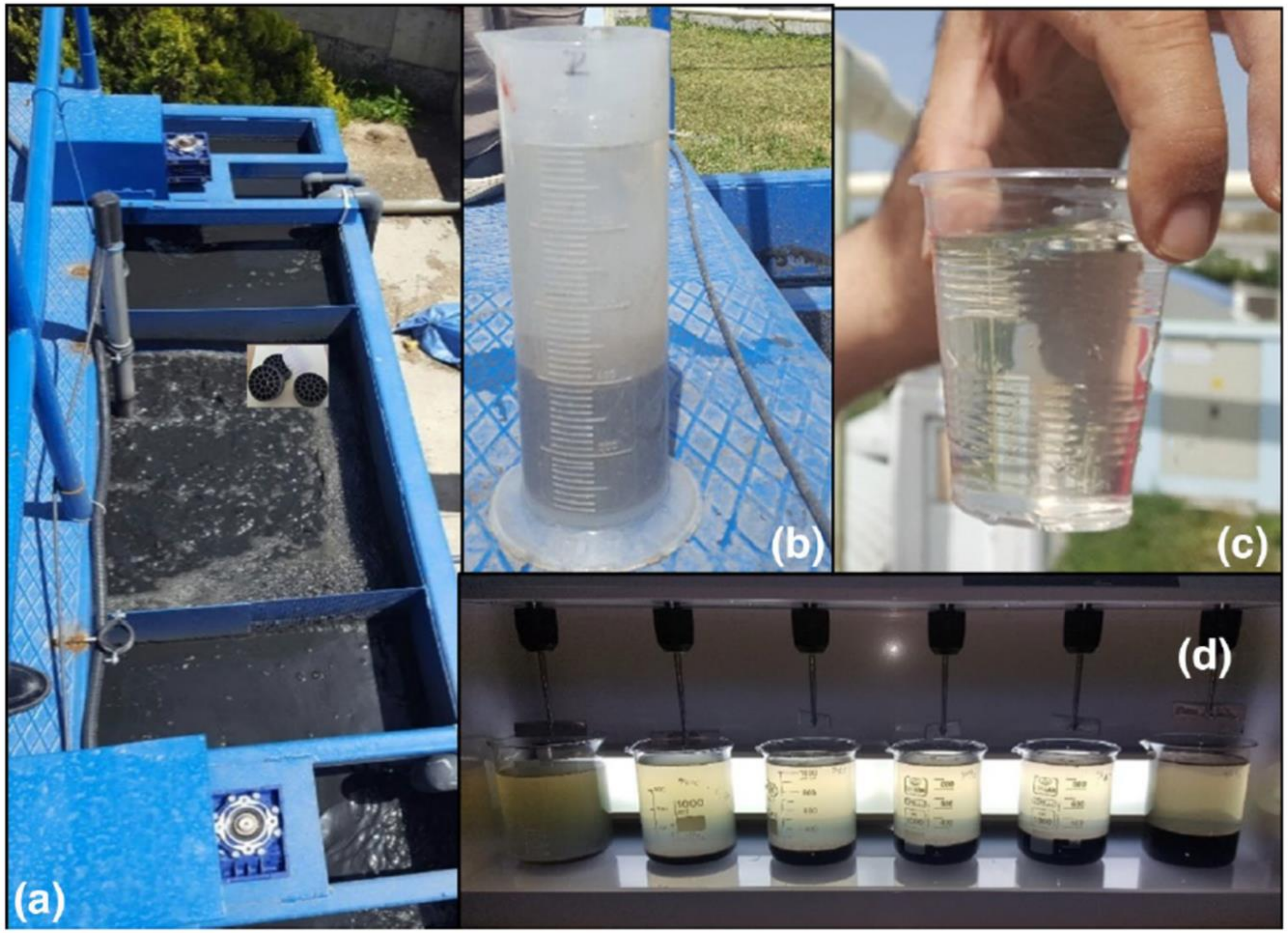


Figure 3.18. Pilot plant of Biofilm Nitrification .

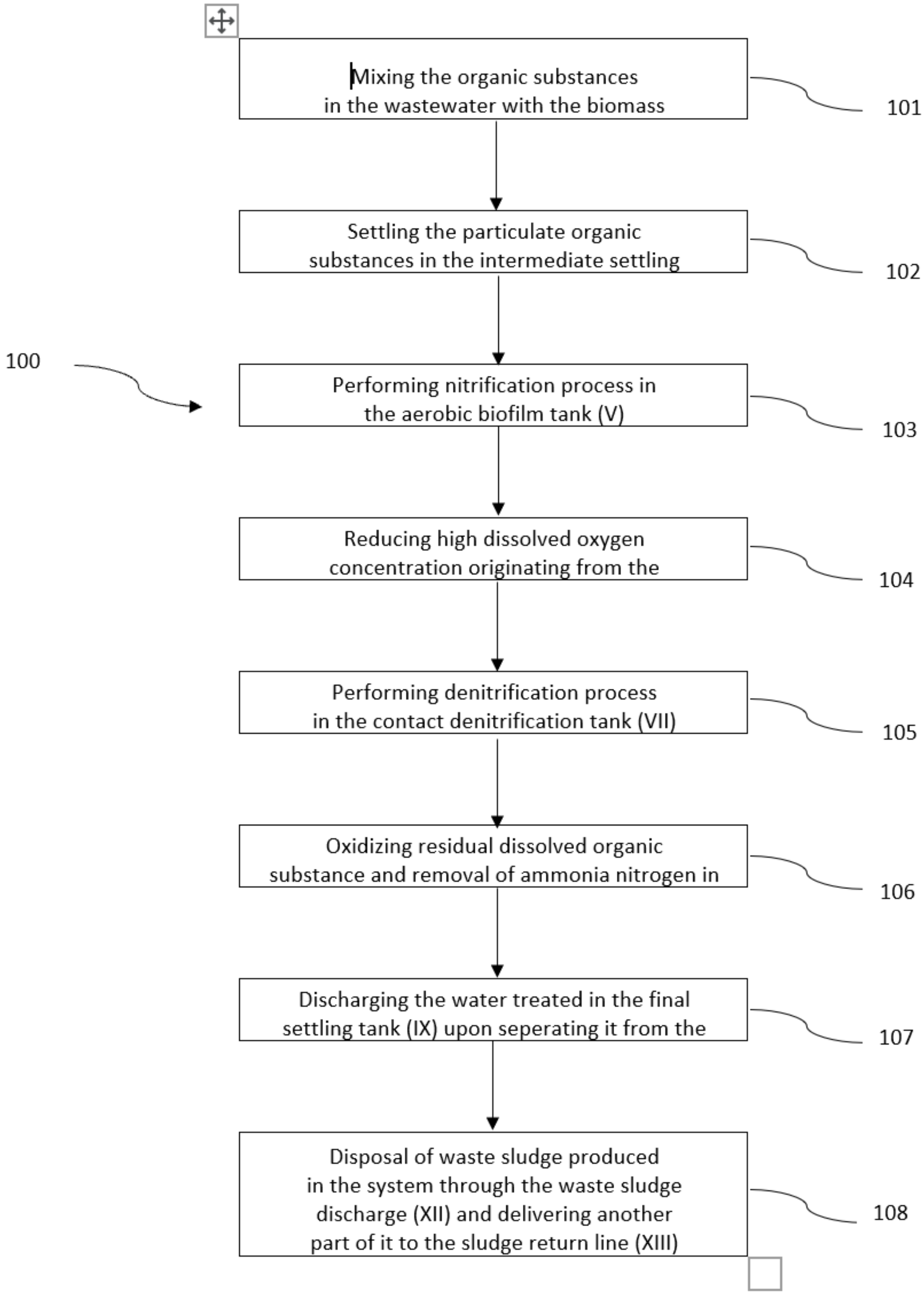


Figure 3.19. Flow diagram of patented biological process.

3.8.2 Treatment Steps of the Invention

A biofilm nitrification - contact denitrification method (100), which provides advanced level of nitrogen and phosphorous removal from wastewaters, basically comprises the steps of;

- mixing the organic substances in the wastewater with the biomass in the selector unit (III) (101),
- settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation (102),
- performing nitrification process in the aerobic biofilm tank (V) (103), reducing high dissolved oxygen concentration originating from the aerobic biofilm tank (V) in the DeOx tank (VI) (104),
- performing denitrification process in the contact denitrification tank (VII) (105),
- oxidizing residual dissolved organic substance and ammonia nitrogen in wastewater in the aerobic tank (VIII) (106),
- discharging the water treated in the final sedimentaion tank (IX) upon separating it from the biomass (107),
- disposal of waste sludge produced in the system through the waste sludge discharge (XII) and delivering another part of it to the return sludge line (XIII) (108).

A novel hybrid biofilm pilot-plant (a biofilm nitrification – contact denitrification system) configuration was developed to reduce the large footprint requirement of conventional wastewater treatment plant due to insufficient nitrification. The primary objective was to improve the nitrogen removal efficiency by reducing the size of the system and by increasing the denitrification potential. With this purpose, a separate moving bed biofilm reactor (MBBR) for nitrification and a post denitrification tank was built at treatment plant site. The configuration of the pilot-plant is given in Figure 3.16. System was designed to achieve enhanced nutrient removal based on the stream and solids separation. A Bio-P tank followed by a primary sedimentation tank were built where carbonaceous compounds were mostly entrapped/incorporated into biosolids. The supernatant of the sedimentation tank, having lower COD and higher ammonia content was directed into a fixed-film nitrification tank while COD rich solids and nitrified stream were diverted into denitrification tank followed by an aerobic tank for phosphorus removal (Gunes et al., 2011).

3.8.3 Details of Pilot Plant

The novel hybrid biofilm configuration (Figure 3.16) was expected to meet certain following objectives:

(a) to reduce the size of nitrification volume,

(b) to separate the carbon rich solids from wastewater to support the denitrification potential,

(c) to eliminate high internal nitrate, recycle and pumping requirements.

The raw wastewater (7 m^3/day) is introduced first into an acidification tank and then into the primary sedimentation tank. The return sludge (RAS) and influent wastewater are mixed in Bio-P tank (with an active volume of 1.1 m^3) where organic matter is pre-fermented and subsequently stored in the biomass as PHAs to support phosphorus removal. The mixed liquor (RAS and acidified wastewater) is sent further to the primary clarifier for bio-flocculation where organic matter is entrapped on to the flocs. The supernatant of the sedimentation tank, rich in ammonia and poor in COD, was directed into fixed-film nitrification tank (1.4 m^3). COD rich solids and nitrified stream were diverted into denitrification tank (1.0 m^3). Denitrification tank is further followed by a post aerobic tank to eliminate remaining COD and to strip dinitrogen gas (N^2) prior to secondary sedimentation. In final clarification, the treated water is separated from the biomass and the return sludge is sent back to the Bio-P tank.

Aquaflex biofilm media (350 m^2/m^3 specific area) with a filling ratio of 30% was used in MBBR tank. The Endress-Hauser probes for dissolved oxygen (Oxymax COS61D), MLSS (Turbimax CUS51D) and ORP/pH (Orbipac CPF81D) were used during the system operation. Single stage centrifuge blower with the capacity of 200 Nm^3/hour is used for aeration. The DO concentrations in MBBR tank and aerobic tank were adjusted to 2 and 5 mg/L, respectively. The oxygen is transferred by means of 9” Aquaflex, EPDM diffusers. The complete mixing in unaerated tanks were provided by vertical agitators.

The mixed liquor suspended solids (MLSS) in anoxic and aerobic reactors were adjusted to 4500 mg/L by wasting sludge from the RAS line. The RAS ratio was applied as 80%, which is the same as the full-scale system.

A bio-flocculation experiment was organized to investigate the capacity of organics that can be entrapped in settled sludge. With this aim, different volume fractions of wastewater and return sludge (WW-RAS: 90-10%; 80-10%, 70-30%, 60-40% and 50-50%) were mixed for 3 hours using a Jar-test device followed by 2 hours of settling. In this respect, optimum WW-RAS ratio could be determined. From the design point of view, this approach reduces the solids (RAS) loading rate of primary clarifier. Based on the results of the bio-flocculation test, simulation study was performed to compare conventional and hybrid biofilm system. The PO_4-P and total COD were also measured in the supernatant. All conventional measurements were conducted as prescribed in the APHA (1995)]. The steady state performance of pilot system under 3 different process temperatures (10, 20, 25 °C) was monitored and simulated by using design/operation parameters as mentioned above.

4 RESULTS AND DISCUSSIONS

4.1 Simulation Based Process Design to Scale-up the Pilot System

By integrating the results of the kinetic characterization and pilot plant performance from this study, a simulation-based process design was performed for both the conventional BNR and the innovative hybrid biofilm system. The aim is to evaluate and compare the environmental footprint, quality of effluent, and air requirements for treating municipal wastewater with a flow rate of 100,000 m3/day. (26.4 MGD).

To optimize the BNR process, it is important to determine the appropriate operating parameters, such as the aerobic sludge age and the anoxic volume. The aerobic sludge age is the length of time that microorganisms spend in the aerobic phase of the process, while the anoxic volume is the amount of wastewater that is treated in an environment without oxygen. Therefore, the selection of the appropriate aerobic sludge age was based on nitrification kinetic tests. Nitrification kinetic tests involve measuring the rate of ammonia oxidation to nitrate under different conditions to determine the optimal conditions for nitrification. Once the aerobic sludge age was determined, the appropriate anoxic volume was selected based on the denitrification potential (NDP) obtained from nitrate utilization tests (NUR). Nitrate utilization tests involve measuring the rate of nitrate utilization under different conditions to determine the optimal conditions for denitrification (Güneş et al., 2019).

In this study, using actual kinetic information obtained from biological characterization experiments, a similar approach was tested with ATV-131E (2000). After choosing the optimal aerobic sludge age for conventional BNR using nitrification kinetic tests, the anoxic volume was selected based on the denitrification potential (NDP) determined from nitrate utilization tests (NUR). To avoid nitrate limitation caused by the high COD/TKN ratio (i.e., 10.5), the simultaneous nitrification and denitrification (SNdN) process was selected. In addition, the hydraulic retention time was determined to be 0.8 hours when determining the Bio-P volume for the conventional activated sludge system (Güneş et al., 2019).

To improve the hybrid biofilm system, minor modifications were made based on the pilot plant operation results and the bio-flocculation experiment findings. The main idea was to reduce the return of activated sludge to the Bio-P unit. This was done to reduce the solids load of the primary clarifiers by maximizing the bio-flocculation. In this way, 30% of the RAS was diverted to the Bio-P system. The remaining portion was diverted to the inlet of the anoxic reactor of the hybrid system. The RAS rates were all selected on the basis of 80% of the inlet flow rate. The steady-state performance simulations for both the conventional and the hybrid system configurations have been performed using SUMO (Güneş et al., 2019).

4.2 COD Fractionation of Raw Influent and Primary Clarifier Effluent

The biodegradation characteristics of primary clarifier effluent vary depending on the type and application of the primary clarifier. Primary clarifiers are used to remove the particulate organic fraction from most municipal wastewater prior to biological treatment. For the first time in the literature, a detailed COD fractionation of reactive primary clarifier effluent is presented in this thesis.

Respirometric batch tests were used to determine the COD fractions of raw wastewater (RWW) and reactive primary clarifier effluent. The results are shown in Figures 4.1 and 4.2. While primary clarifier effluent COD fractionation has been extensively studied in the literature (Henze et al., 2000; Okutman et al., 2001), few studies characterize reactive primary clarifier effluent in detail.

The COD fractions of the wastewater were analyzed using the OUR profile and model estimation (Figure 4.1) according to the ASM1 model, which defines three different types of COD fractions: readily biodegradable (SS), rapidly hydrolysable (SH), and slowly hydrolysable (XS). The first plateau of the OUR profile, which was around 115 mg/L.h (Figure 4.1), indicated the rapid consumption of the easily biodegradable COD (SS) after the wastewater was added to the biomass in the endogenous phase. The second plateau represented the degradation of the secondary COD (SB), which has a much lower degradation rate compared to the easily biodegradable COD (SS). Finally, the last plateau corresponds to the degradation of the slowly hydrolysable COD (XS) (Güneş et al., 2019).

According to the respirometric tests conducted in this study, the raw wastewater used had a total COD (**C_T**) of 534 mg/L. Total COD was composed of various COD fractions including 16%

total inert (**C_I**), 84% biodegradable (C_S), 6% readily biodegradable (**S_s**), 12% rapidly hydrolysable (SB), and 67% slowly hydrolysable (XS1) (refer to Figure 4.2) (Güneş et al., 2019).

.

Based on the experimental studies, it appears that the raw wastewater used in the study has a total COD (C_T) of 534 mg/L, with 16% of the COD being composed of total inert COD (C_I) and 84% being total biodegradable COD (C_S). The total biodegradable COD (C_S) is further diveded of 6% readily biodegradable COD (S_{VFA}), 12% rapidly hydrolysable COD (S_B), and 67% slowly hydrolysable COD (X_S) fractions, as shown in Figure 4.2. Additionally, the study found that 87% of the total COD (CT) was caught in the reactive primary clarifier, which is significantly higher than the findings of previous studies such as Okutman et al. (2001) and Gupta (2018), who reported primary clarifiers catching 49% and 36% of total COD, respectively. Furthermore, the Rotating Belt Filter (RBF) proposed as a substitute for the primary clarifier by Gupta (2018) had a much lower efficiency of 17% in terms of total COD (C_T) capture, as shown in terms of total COD (C_T) capture (Güneş et al., 2019).

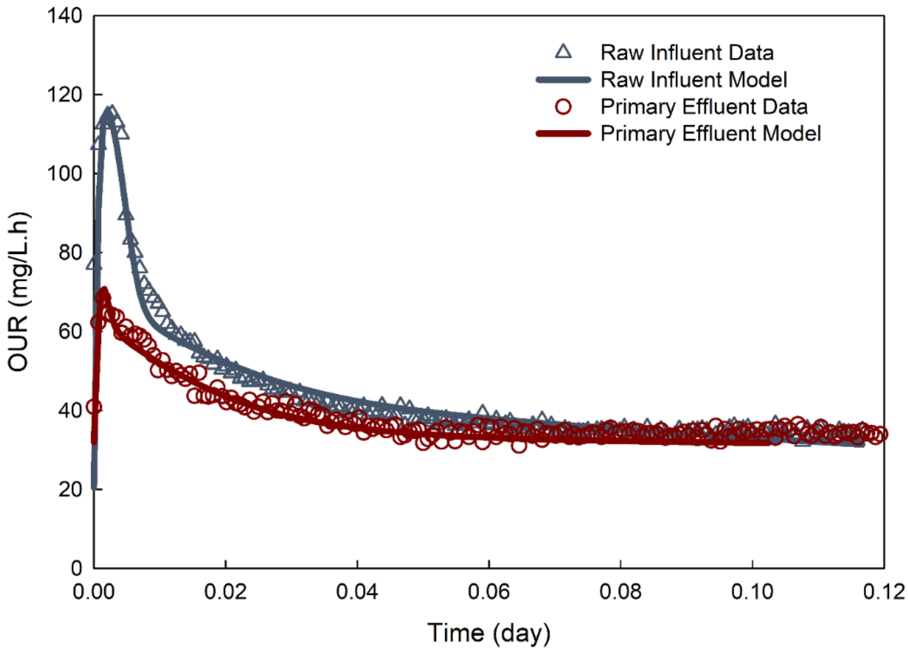


Figure 4.1. Respirograms of the raw waste water and the effluent of the reactive primary clarifier.

According to the respirometric tests, the slowly hydrolysable COD fraction (X_S) was the most significant COD component caught in the reactive primary clarifier, accounting for 96% of the total COD (C_T) component. This fraction is essential for the efficient utilization of organics through redirection, as shown in Figure 4.2. In contrast, previous studies conducted by Okutman et al. (2001) and Gupta (2018) reported removal efficiencies of primary clarifiers in terms of slowly hydrolysable COD (X_S) to be 64% and 62%, respectively (Table 4.1). The reactive primary clarifier, in contrast, demonstrates significantly higher performance than conventional primary clarifiers due to the adsorption capability of the biological sludge (Güneş et al., 2019). In the proposed configuration, the organic matter captured in the reactive primary clarifier is directed into the anoxic tank as the primary carbon source for denitrification. As a result, the degradation kinetics of the hydrolysable COD will govern the rate and efficiency of denitrification and/or anaerobic digestion. As per the experimental studies, the respirometric studies have confirmed that the readily biodegradable COD fraction (S_{VFA}) was fully utilized and taken up by the biomass in the Bio-P stage (Table 4.1). This fraction is easily biodegradable and can be directly utilized by microorganisms for their cellular metabolism (Güneş et al., 2019). However, the rapidly hydrolysable fraction of the organic matter was either not utilized in the Bio-P tank or not captured in the reactive primary clarifier (RPC) (Table 4.1). This fraction is not suitable for direct cellular use and has a soluble nature that makes it difficult for bio-sorption or sedimentation. It is possible that this fraction may require further treatment or processing to make it more accessible for microbial utilization or capture (Güneş et al., 2019).

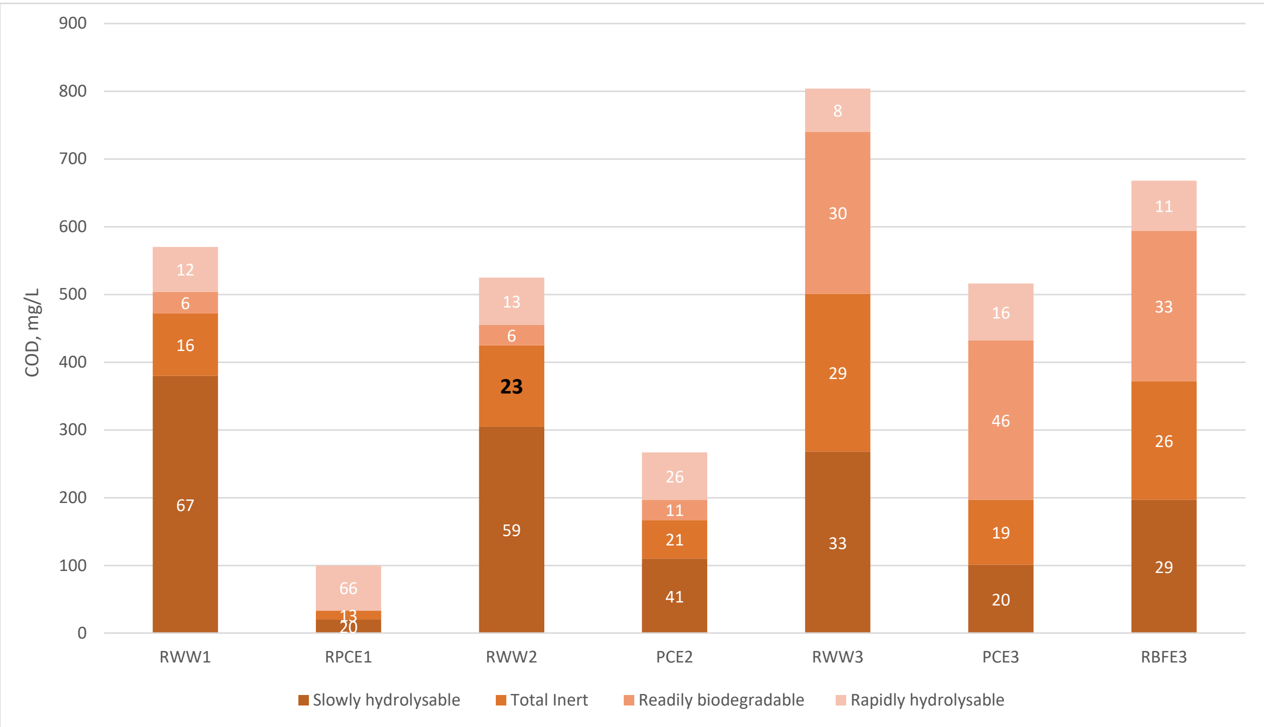


Figure 4.2. Removal efficiencies of Reactive Primary Clarifier (RPC), Primary Clarifier (PC) and Rotating Belt Filter Effluent (RBFE) based on COD fractions.

Table 4.1. Removal efficiencies of Reactive Primary Clarifier (RPC), Primary Clarifier (PC) and Rotating Belt Filter Effluent (RBFE) based on COD fractions.

	Removal, %			
COD fractions	**RPC[1]**	**PC[2]**	**PC[3]**	**RBF[3]**
Total COD, C_{T1}	87	49	36	17
Total Inert COD, C_{I1}	90	53	59	25
Biodegradable COD, C_{S1}	86	48	25	14
Readily biodegradable COD, S_{VFA}	97	0	2	7
Rapidly hydrolysable COD, S_B	24	0	-	-
Slowly hydrolysable COD, X_{S1}	96	64	62	26

[1] This study, [2] Okutman et al, 2001, [3] Gupta, 2018

4.3 Pilot Plant Operation and Mass Balances

The performance of the pilot plant together with the calculated mass balances are given in Figure 4.3. The results presented for the pilot plant operation are belong to the summer season (20°C). The pilot plant receives wastewater with an average influent COD and TKN concentration of 534 mg/L and 72 mg N/L, respectively. The effluent COD concentration after the primary reactive clarifier was measured as 71 mg/L, indicating that about 87% of the total influent COD could be captured in RPC via bio-flocculation and gravity sedimentation before MBBR (Table 4.1) (Güneş et al., 2019).

The soluble COD concentration was reduced from 226 to 56 mg/L (Figure 4.3) during Bio-P stage, indicating that soluble COD was fermented and presumably stored in the cells as biopolymers (PHA measurements are not available). The complete nitrification was achieved in the MBBR leading to an effluent NH_4-N concentration of 0.5 mg/L (Figure 4.3). TN and TP removal efficiencies were obtained as 86% and 89%, respectively. Observed high nutrient removal performance points out that organic matter captured in the Bio-P tank and RPC system was available for both denitrification and phosphorus uptake (Güneş et al., 2019).

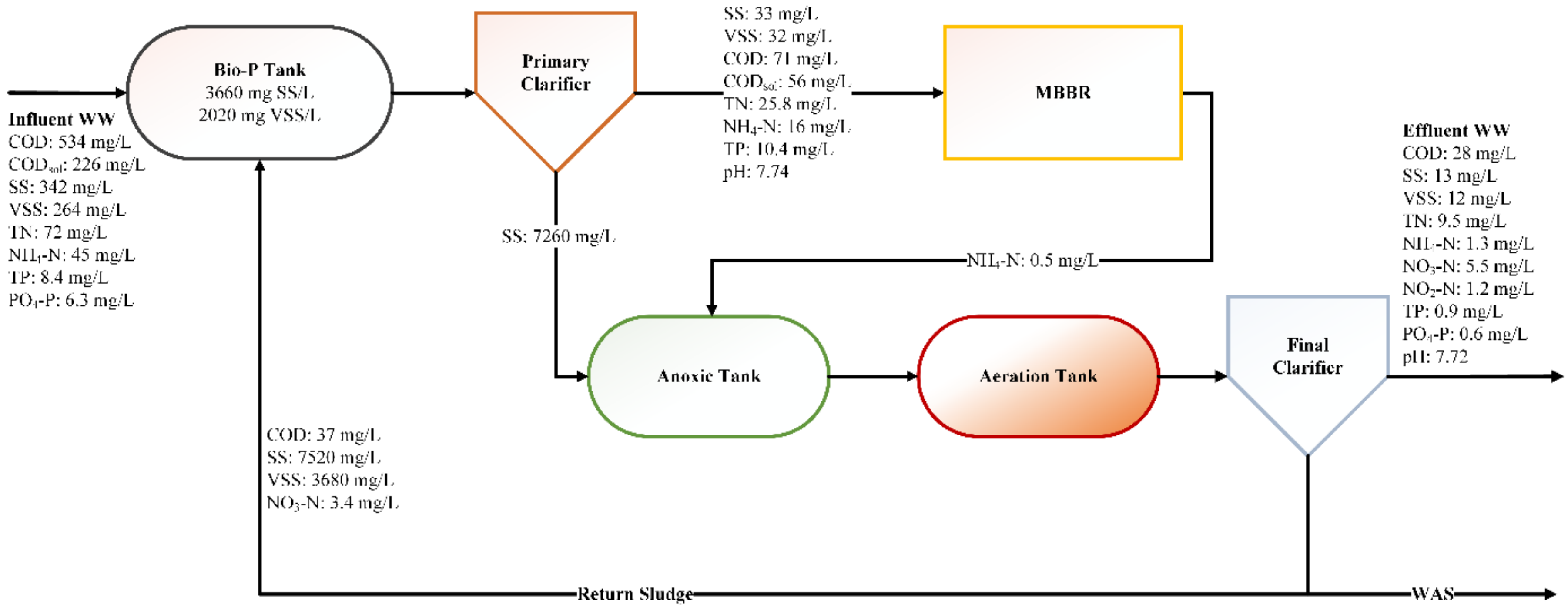


Figure 4.3. Pilot plant layout and the mass balance obtained in the hybrid configuration.

4.4 Simulation Results

The process design for both hybrid configurations and the conventional BNR system was conducted based on the EU discharge regulations (EEC, 1991), which set the limits for TN and TP as 10 mg N/L and 1 mg P/L, respectively. Table 4.2 provides the required dimensions and operational specifications for all systems. The effluent COD concentrations were approximately 45 mg/L in all configurations, consistent with the pilot plant operation. Simulation calculations revealed that phosphate uptake was more efficient in the hybrid configurations (0.3 mg PO_4-P/L) compared to the BNR system (0.5 mg PO_4-P/L) due to the more efficient use of organic matter through RPC. While the hybrid system provided a relatively low effluent TN concentration, the effluent ammonia (NH_4-N) concentration was slightly high in both configurations (5.6 mg N/L) compared to the conventional BNR system (1.8 mg N/L). This was because the RPC sludge was directly fed into the anoxic reactor, bypassing the nitrification MBBR, resulting in ammonia carryover (The simulation calculations revealed that in order to achieve complete nitrification, the BNR system required a minimum aeration volume of 70,000 cubic meters. The maximum nitrification rate in this system was calculated to be 62.8 grams of nitrogen per cubic meter per day, based on the given formula (5,367 – 1,287 kg NH_4/d /65,000 m3= 62.8 g N/m^3.d) On the other hand, the MBBR system was able to fully nitrify in a smaller aeration volume of 30,000 cubic meters, with a total nitrification rate of 140 grams of nitrogen per cubic meter per day (4,350-159/30,000 = 140 g N/m^3.d) (Güneş et al., 2019).

Although the ammonia mass flow rates to both systems were roughly equal at 4,125-4,350 kilograms of nitrogen per day, the MBBR had a higher specific nitrogen loading rate of 0.48 grams of nitrogen per square meter per day. This value was calculated by taking into account the specific surface area of the media used in the MBBR, which was 500 square meters per cubic meter, and a filling ratio of 60%. The maximum specific nitrification rate achieved in the MBBR system was found to be 0.47 grams of nitrogen per square meter per day. The maximum nitrification rate in biofilms was found to be higher, with reported values of 75 to 150 grams of nitrogen per square meter per day for dissolved oxygen concentrations of 3 and 6 milligrams per liter, respectively. These values were reported in previous studies (Odegaard, 2005b, Sen and Randall, 2008). However, according to an experimental study conducted by Forrest et al. (2016), the nitrification rate of biofilm was found to be around 1.5 grams of nitrogen per square meter per day at low organic loading rates, which is considered to be within a safe limit. Overall, the MBBR system appears to be more efficient in achieving nitrification at a smaller scale with a higher total nitrification rate, while the BNR system may require more space but can achieve complete nitrification (Güneş et al., 2019).

The study found that the nitrification rates were significantly higher in the proposed Moving Bed Biofilm Reactor (MBBR) configurations compared to the conventional Biological Nutrient Removal (BNR) process. This was due to two main factors. This was due to two main factors. Firstly, the efficient capture of organic matter in the Recirculating Packed Carrier (RPC) sludge led to a reduction of about 80% in the total Chemical Oxygen Demand (COD) load to the MBBR in Configuration 1 and 86% in Configuration 2 when compared to BNR. As a result, there was about a 78% advantage in the Oxygen Uptake Rate (OUR) for nitrification (as shown in Table 4.3) (Güneş et al., 2019).

This was due to two main factors. Firstly, the efficient capture of organic matter in the Reactive Primary Clarifier (RPC) sludge led to a reduction of about 80% in the total Chemical Oxygen Demand (C_T) load to the MBBR in Configuration 1 and 86% in Configuration 2 when compared to BNR. As a result, there was about a 78% advantage in the Oxygen Uptake Rate (OUR) for nitrification (as shown in **Hata! Yer işareti başvurusu geçersiz.**). Secondly, the MBBR attained a much higher biomass concentration of 40,000 grams of Volatile Suspended Solids (VSS) per cubic meter, which was about 25 times higher than the biomass concentration of 1,600 grams of VSS per cubic meter that was obtained in BNR. This was possible because the sludge age in MBBR was independent of the reactor hydraulics. The higher biomass concentration allowed the nitrifiers to be highly active and to grow without the risk of being washed out, which could occur at lower biomass concentrations. Therefore, the combination of efficient organic matter capture in RPC sludge and the attainment of much higher biomass concentration in MBBR led to the higher nitrification rates observed in the MBBR configurations compared to conventional BNR (Güneş et al., 2019).

Table 4.3). The effluent nitrate concentration was also higher in the BNR system (6.1 mg NOx-N/L vs 1 mg NOx-N/L) due to the low biodegradable COD concentration entering the anoxic tank compared to the hybrid system. Simulation results indicated that the effluent quality obtained for both hybrid configurations was very similar to the pilot plant operation results, demonstrating that the pilot system was accurately simulated (Güneş et al., 2019).

According to the analysis performed, it was found that the proposed hybrid systems would require a total reactor volume of 62,000 cubic meters, while the conventional biological nutrient removal (BNR) system would require a total reactor volume of 102,000 cubic meters. This means that the hybrid system offers a significant advantage over the conventional BNR system, as it represents a nearly 40% reduction in total volume. This reduction in volume can lead to lower capital and operating costs, making the hybrid system more economically competitive (Güneş et al., 2019).

Furthermore, the hydraulic retention time (HRT) for the hybrid configurations were 14.6 hours, while the HRT for the conventional BNR system was 24.5 hours. The required surface area for the primary sedimentation tank was slightly smaller for the hybrid configurations than for the conventional BNR system. On the other hand, the overall surface area requirements for the clarifiers in both the hybrid systems and the conventional BNR system were found to be almost equal. This implies that both systems would require a similar amount of space for the clarification stage, which is an important factor to consider when designing a wastewater treatment plant. Overall, these findings suggest that the proposed hybrid systems have the potential to be a more cost-effective and efficient alternative to conventional BNR systems (Table 4.2) (Güneş et al., 2019).

Table 4.2. Dimensions and required installations for treatment plant units for hybrid configurations and conventional BNR system.

Process Unit	**Unit**	**Plant Configuration**	
		Hybrid	**Conventional**
Total Bio-P volume	m^3	6,000	7,000
Primary clarifier surface	m^2	2500	2000
MBBR/Aerobic1volume	m^3	30,000	65,000
Aerobic 2 volume	m^3	8,000	5,000
Total aerobic reactor	m^3	38,000	70,000
Anoxic reactor volume	m^3	17,000	25,000
Total reactor volume	m^3	61,000	102,000
Total biofilm area	m^2	9,000,000	-
Internal Recirculation	m^3/ho	-	12,500
Anaerobic Digester	m^3	12,000	12,000
Total clarifier surface area	m^2	6,800	7,000

4.5 Nitrification Performance

The simulation calculations revealed that in order to achieve complete nitrification, the BNR system required a minimum aeration volume of 70,000 cubic meters. The maximum nitrification rate in this system was calculated to be 62.8 grams of nitrogen per cubic meter per day, based on the given formula (5,367 – 1,287 kg NH_4/d /65,000 m3= 62.8 g N/m^3.d) On the other hand, the MBBR system was able to fully nitrify in a smaller aeration volume of 30,000 cubic meters, with a total nitrification rate of 140 grams of nitrogen per cubic meter per day (4,350-159/30,000 = 140 g N/m^3.d) (Güneş et al., 2019).

Although the ammonia mass flow rates to both systems were roughly equal at 4,125-4,350 kilograms of nitrogen per day, the MBBR had a higher specific nitrogen loading rate of 0.48 grams of nitrogen per square meter per day. This value was calculated by taking into account the specific surface area of the media used in the MBBR, which was 500 square meters per cubic meter, and a filling ratio of 60%. The maximum specific nitrification rate achieved in the MBBR system was found to be 0.47 grams of nitrogen per square meter per day. The maximum nitrification rate in biofilms was found to be higher, with reported values of 75 to 150 grams of nitrogen per square meter per day for dissolved oxygen concentrations of 3 and 6 milligrams per liter, respectively. These values were reported in previous studies (Odegaard, 2005b, Sen and Randall, 2008). However, according to an experimental study conducted by Forrest et al. (2016), the nitrification rate of biofilm was found to be around 1.5 grams of nitrogen per square meter per day at low organic loading rates, which is considered to be within a safe limit. Overall, the MBBR system appears to be more efficient in achieving nitrification at a smaller scale with a higher total nitrification rate, while the BNR system may require more space but can achieve complete nitrification (Güneş et al., 2019).

The study found that the nitrification rates were significantly higher in the proposed Moving Bed Biofilm Reactor (MBBR) configurations compared to the conventional Biological Nutrient Removal (BNR) process. This was due to two main factors. This was due to two main factors. Firstly, the efficient capture of organic matter in the Recirculating Packed Carrier (RPC) sludge led to a reduction of about 80% in the total Chemical Oxygen Demand (COD) load to the MBBR in Configuration 1 and 86% in Configuration 2 when compared to BNR. As a result, there was about a 78% advantage in the Oxygen Uptake Rate (OUR) for nitrification (as shown in Table 4.3) (Güneş et al., 2019).

This was due to two main factors. Firstly, the efficient capture of organic matter in the Reactive Primary Clarifier (RPC) sludge led to a reduction of about 80% in the total Chemical Oxygen Demand (C_T) load to the MBBR in Configuration 1 and 86% in Configuration 2 when compared to BNR. As a result, there was about a 78% advantage in the Oxygen Uptake Rate (OUR) for nitrification (as shown in **Hata! Yer işareti başvurusu geçersiz.**). Secondly, the MBBR attained a much higher biomass concentration of 40,000 grams of Volatile Suspended Solids (VSS) per cubic meter, which was about 25 times higher than the biomass concentration of 1,600 grams of VSS per cubic meter that was obtained in BNR. This was possible because the sludge age in MBBR was independent of the reactor hydraulics. The higher biomass concentration allowed the nitrifiers to be highly active and to grow without the risk of being washed out, which could occur at lower biomass concentrations. Therefore, the combination of efficient organic matter capture in RPC sludge and the attainment of much higher biomass concentration in MBBR led to the higher nitrification rates observed in the MBBR configurations compared to conventional BNR (Güneş et al., 2019).

Table 4.3. Oxygen Uptake Rates calculated for hybrid configurations and BNR.

Symbol	**Hybrid Configuration 1**	**Hybrid Configuration 2**	**BNR**	**Unit**
Total Oxygen Uptake Rate (OUR) in the reactor	929	936	23,2	mg O_2/L/h
Total Carbonaceous oxygen uptake rate (COUR)	228	213	11,8	mg O_2/L/h
Total Nitrification oxygen uptake rate (NOUR)	701	723	11,3	mg O_2/L/h

4.6 Nutrient Removal Performance

The process in hybrid wastewater treatment configurations involves combining wastewater with RAS and introducing the mixture to Bio-P tanks. In these tanks, fermentation occurs, leading to P-release and PHA storage. The resulting mixture is composed of a high concentration of solids, with Config1 and Config 2 having concentrations of 3,962 g and 3,480 TSS/m^3, respectively. Afterwards, the mixture enters the RPC, where carbonaceous matter is trapped on active biomass through bio-adsorption. The microorganisms break down the carbonaceous matter, and the solids in the mixture are separated through solids settling. The settling efficiency in the RPC is high, with a rate of 87% in terms of total COD (chemical oxygen demand) compared to the conventional primary clarifier. This means that the solids are able to settle to the bottom of the tank efficiently. The settling efficiency was explained in the previous section (Güneş et al., 2019).

Table 4.4 shows the simulation calculations around Bio-P, RPC, and MBBR, anoxic and aerobic tanks for proposed configurations (a) as well as BNR system (b) that shows the fate of organic matter, nitrogen and phosphorus. In hybrid configurations, total anaerobic retention time of 2.81 h was attained in Bio-P tanks together with RPC, which enables fermentation, in consequence PHA storage and poly-phosphate (PP) release (Güneş et al., 2019)..

The polyphosphate (PP) stored in the biomass directed via RAS line was released in Bio P tanks (98 and 115 mg PO_4-P/L, for Config 1 and Config 2, respectively), however biomass still contain significant amount of PP (20 and 22 mg PP/L for Config1 and Config 2). VSS escaped with RPC supernatant (1,044 and 814 g VSS/m^3, Config1 and Config2, respectively), storing significant amount of PHA (78 and 55 g COD/m^3), was directed into MBBR where a substantial level of P uptake (36 and 25.3 g P/m^3) as well as an efficient ammonia oxidation (96%) was obtained (Table 4.4) (Güneş et al., 2019).

Captured organics in the RPC sludge was easily fermented by biomass directed by RAS line and VFA was produced. Consequently, PHA content of RPC sludge in Config1 and Config2 was reached a maximum of 384 and 371 g/m^3. Meanwhile, PP content of biomass was completel~~y~~ exhausted (0.17 g/m^3) and biomass became P-starved in RPC settling zone. Since biomass in RPC sludge was extremely P starved and rich in PHA storage, an efficient phosphorus uptake (51 g P/m^3) was possible under anoxic conditions. Additional PP uptake was occurred in the aeration tank (Aerobic 2) which contributes complete P removal and satisfies effluent quality (Güneş et al., 2019).

In contrast, phosphorus removal in BNR was depended on influent VFA concentration which could be a great problem for the wastewaters poor in VFA. In addition, low readily biodegradable COD content in anoxic tank influent is a limitation for denitrification in BNR (Table 4.4). Whereas in proposed configurations, system was operated as post denitrification where nitrate came from MBBR was denitrified with mainly use of stored PHA, VFA, and slowly biodegradable COD.

4.7 Biogas Production Performance

The anaerobic digester in Configuration 1 received waste sludge from the RAS line, which was carefully controlled using a controller unit to divert RAS and maintain a MLSS concentration of 4000 mg/L in the anoxic tank. On the other hand, Configuration 2 had RPC sludge that was divided between the anaerobic digester and anoxic tank. A detailed simulation calculation of organic matter components in terms of COD fractions around the anaerobic digester is presented in Table 4.5 (Güneş et al., 2019).

The biogas production rates of the hybrid for Config 1 and Config 2 (11,480 and 9,040 m^3/d at NTP) were significantly higher compared to BNR (6,030 m^3/d at NTP). It is worth noting that even though the input total COD (C_T) mass flow to the anaerobic digester in BNR was higher (25,807 kg/d) than in Config 1 (22,549 kg/d) and slightly lower than in Config 2 (28,378 kg/d), the difference in biogas production was substantial (Güneş et al., 2019).

The unique design of the hybrid configuration is the main cause of the contrast in biogas production, as it allows for the capture of organic matter in RPC and its conversion into easily biodegradable forms (as shown in Table 4.5), leading to greater biogas generation. As expected, RPC sludge, which is abundant in PHA, provides an excellent source for biogas production. Specifically, in Config 1 and Config 2, around 545 and 1350 g COD/m^3 of PHA equivalent were introduced into the AD, respectively (Güneş et al., 2019).

Configuration 2 showed the highest biogas production (11,480 m3/d at NTP), which was achieved by distributing RPC sludge between the anoxic tank and AD for optimal nutrient removal and biogas formation. In contrast, BNR sludge had a low PHA content and a significant portion of slowly biodegradable COD (8,318 kg/day) (Güneş et al., 2019). This suggests that the composition and availability of organic fractions in the substrate can affect the biogas yield in wastewater treatment plants, as reported in previous studies (Girault et al. 2012; Mottet et al., 2010; Nguyen et al., 2020).

The accumulation of cellular residues and suspended inert materials is a common occurrence in biological treatment systems that have long solids retention times (SRTs), which can result in lower anaerobic biodegradability in the anaerobic digesters (Ucisik and Henze, 2008; Mottet et al., 2010). In contrast, the total SRT of the proposed hybrid configuration was significantly lower than the SRT of the BNR system, which was only 18 days. Therefore, the higher biogas yield observed in the hybrid system as compared to the BNR system can be attributed to two factors: i) the redirection of carbon through pre-fermented RPC sludge, and ii) the operational characteristics of the sludge, which are related to the SRT.

Table 4.4. Simulation calculations around tanks for proposed configurations (a), BNR (b) based on organic matter, nitrogen and phosphorus.

(a) Symbol	Influent	Bio-P		RPC Overflow		MBBR Bulk		RPC Sludge		Anoxic In		Anoxic		Aerobic 2		Unit
	Config1-2	Config1	Config2	Config1	Config2	Config1	Config2	Config1	Config2	Config1	Config2	Config1	Config2	Config1	Config2	
Filtered chemical oxygen demand	290	61	74	47	57	36	36	98	117	52	51	36	36	36	36	g COD/m3
Readily biodegradable (non-VFA)	75	2,2	3	0,3	0,3	0,9	0,9	0,3	0,3	0,8	0,8	0,9	0,9	1,3	1,3	g COD/m3
Volatile fatty acids (VFA)	70	24	37	11	21	0	0	63	81	16	15	0	0	0	0	g COD/m3
Slowly biodegradable substrate	259	398	394	187	170	85	77	908	1047	295	295	251	250	212	213	g COD/m3
Stored polyhydroxyalkanoates (PHA)	0	118	87	73	55	29	21	384	371	120	95	91	68	72	53	g COD/m3
Stored glycogen (GLY)	0	2,7	1,4	1,4	0,7	0,7	0,4	7,1	4,5	2,3	1,3	1,8	1,2	1,4	1	g COD/m3
Endogenous decay products	0	135	118	66	53	72	58	340	349	140	140	141	141	143	143	g COD/m3
Stored polyphosphate (PP)	0	20	22	0	0	36	25	0	0	27	33	51	52	54	60	g P/m3
Total nitrogen	55	185	164	106	93	103	90	417	419	183	177	164	159	163	158	g N/m3
Total ammonia (NHx)	41	30	33	31	34	1	1	34	38	10	8	9	7	6	5	g N/m3
Nitrate and nitrite (NOx)	0	0	0	0	0	25	28	0	0	19	20	0	1,5	1,0	2,6	g N/m3
Total phosphorus	8	98	115	70	76	70	76	181	252	98	132	98	132	98	132	g P/m3
Orthophosphate (PO4)	5,3	28,5	28,4	44,8	46,9	8,0	18,2	56,2	60,4	20,3	23,8	1,0	6,0	0,2	0,3	g P/m3

(b) Symbol	Influent	After PS	BioP2	Anox1 In	Anoxic 1	Aerobic	Anoxic 2	Aerobic 2	Unit
Filtered chemical oxygen demand	290	282	71	48,9	36	36	36	36	g COD/m3
Readily biodegradable (non-VFA)	58	56	3	2	1	1	1	1	g COD/m3
Volatile fatty acids (VFA)	87	85	33	12,1	0	0	0	0	g COD/m3
Slowly biodegradable substrate	259	178	259	172,7	168	122	108	100	g COD/m3
Stored polyhydroxyalkanoates (PHA)	0,1	0,1	34,2	16,5	14,6	6,2	3,4	2,5	g COD/m3
Stored glycogen (GLY)	0,1	0,1	1,3	0,7	0,7	0,3	0,2	0,1	g COD/m3
Endogenous decay products	0	1	373	382,0	382	387	389	390	g COD/m3
Stored polyphosphate (PP)	0	0	33	44,8	46	52	55	56	g COD/m3
Total nitrogen	55	55	142	133,9	130	129	123	123	g COD/m3
Total ammonia (NHx)	41,3	43,7	27,0	11,7	11,3	2,7	3,2	1,7	g COD/m3
Nitrate and nitrite (NOx)	0,0	0,2	0,0	6,6	2,8	10,4	5,0	6,3	g COD/m3
Total phosphorus	8	8	101	101,2	101	101	101	101	g COD/m3
Orthophosphate (PO4)	5,3	5,8	15,5	6,3	5,0	1,0	0,6	0,4	g COD/m3

Table 4.5. Simulation calculations for organic matter components around the anaerobic digester.

Symbol	Digester Input			Digester Output			Unit
	Config1	Config 2	BNR	Config1	Config 2	BNR	
Ordinary heterotrophic organisms (OHO) mass flow	9789	11502	6678	2685	3233	1402	kg/d
Carbon storing organisms (CASTO) mass flow	4221	4026	1386	164	159	57	kg/d
Particulate unbiodegradable organics mass flow	5066	6021	6014	5137	6100	6047	kg/d
Slowly biodegradable substrate mass flow	1603	3806	8318	210	291	3387	kg/d
Volatile suspended solids (VSS) mass flow	15814	19410	17876	7883	9394	10374	kg/d
Total suspended solids (TSS) mass flow	30300	36531	36948	22963	26825	30422	kg/d
Endogenous decay products mass flow	1081	1269	2914	891	1050	2393	kg/d
Stored glycogen (GLY) mass flow	10,9	16	4	38	29	3	kg/d
Stored polyhydroxyalkanoates (PHA) mass flow	544,8	1350	22	45	52	26	kg/d
Volatile Fatty Acids (VFA) Mass Flow	0	50	7	52	64	65	kg/d

4.8 Operational Parameters

Table 4.6 presents the calculated operational parameters of the different wastewater treatment systems. The required total air flowrate under average loading was calculated for each system, with the hybrid configuration 1 requiring 28,482 Nm3/hour and the hybrid configuration 2 requiring 27,263 Nm3/hour. In both cases, it was found that 83% of the total airflow needs to be diverted to the MBBR module, while the remaining 17% is required for final aeration. On the other hand, the conventional BNR configuration required a total air flowrate of 35,342 Nm3/hour.

The proposed hybrid configurations require less air compared to the conventional BNR system, with a reduction of 19-23% in total oxygen requirement. Additionally, the hybrid system uses 35% less electricity overall and has a 55% reduction in mixing energy requirement due to smaller reactor volume (unit mixing energy: 5 W/m^3). Simulation results also showed that the hybrid system produces 22% more biogas, resulting in more energy being obtained from sludge digestion.

Table 4.6. Operational parameters for hybrid and conventional BNR systems.

Parameter	**Unit**	**Configuration**		
		Config.1	**Config.2**	**BNR**
Average air requirement, Q_{Air}	Nm^3/hour	28,482	27,263	35,342
Mixing energy requirement*	kWh/day	305	305	510
Daily biogas production	m^3/day	9,040	11,476	6,930
Solid retention time	days			18

*based on unit mixing intensity of 5 W/m^3

4.9 Cost Analysis for System Configurations

The United States Department of Energy estimated that wastewater treatment plants require 1-3% of a country's total electrical energy output (US DOE, 2014). MBBR systems are considered a sustainable option for wastewater treatment plant configurations due to their economic benefits. However, there is a need to optimize their energy consumption (Singh et al., 2018). The economic evaluation of the capital expenditure (CAPEX) and the operational expenditure (OPEX) for two process configurations has been determined.

Merely the electromechanical and civil works during the construction phase are considered in the calculation of CAPEX. Other expenses such as taxes, excavation-foundation costs, land expropriation, and levies are not included in the estimated capital investment costs. As the hybrid configuration and conventional BNR systems have the different volumes, but their initial investment costs are also the same. Figure 4.4. provides a comparative overview of the CAPEX for both systems. It shows that the proposed hybrid system has a lower CAPEX of $27.9M compared to the conventional BNR system, which costs $30.0M. This translates to a CAPEX advantage of about 7%. Additionally, Figure 4.5. shows the CAPEX ratios for the hybrid and conventional systems.

In the BNR system, concrete and steel fabrication accounted for 36.46% of the CAPEX, while in the hybrid system it was only 26.46%. The cost of support media for MBBR, which is approximately $4.5 million according to the supplier, is a significant cost component and represents approximately 16% of the CAPEX for the hybrid system. This cost is included in the mechanical work, which accounts for 40.2% of the total hybrid system CAPEX. Although the CAPEX advantage of the hybrid system over the conventional BNR system appears small ($2.1M), it is a significant amount, especially for developing countries. In addition, the proposed hybrid configuration has the added advantage of requiring significantly less land area due to the reduction in total reactor volume. This is a critical decision factor, especially for plants with space limitations and large urban treatment plants with space constraints.

.

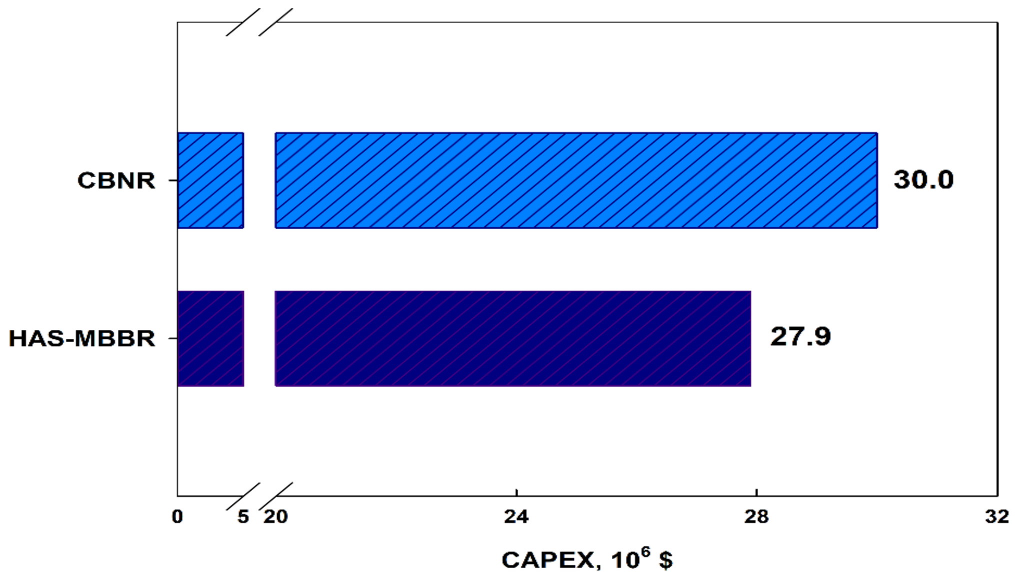


Figure 4.4. Total CAPEX for configurations as $\$10^6$.

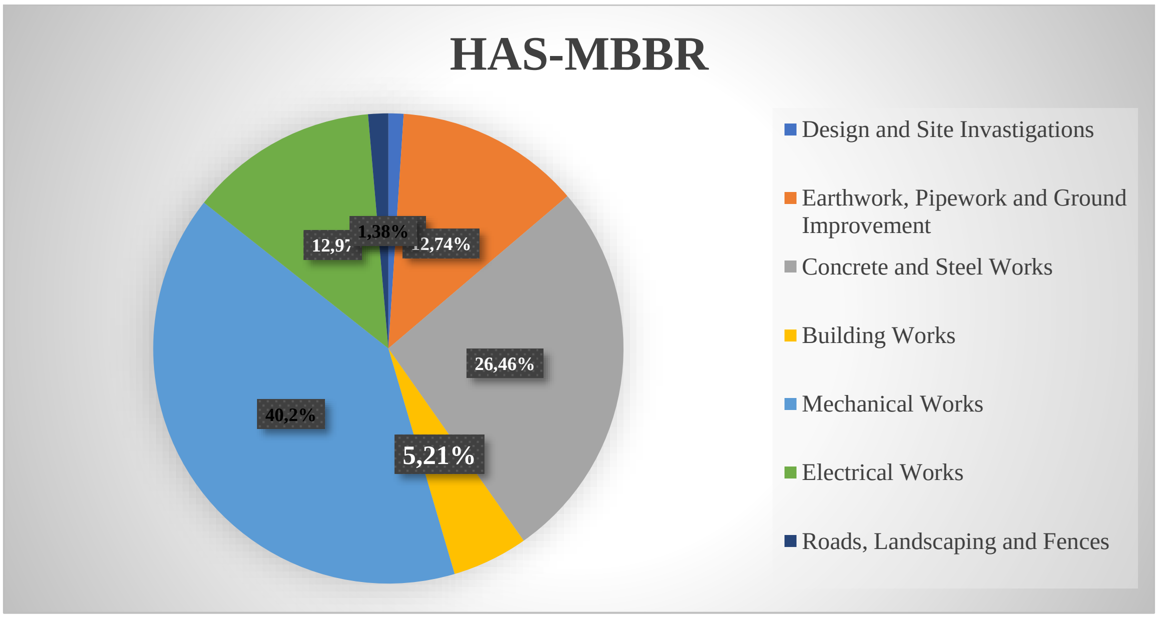


Figure 4.5. HAS-MBR capital expenditure (CAPEX) breakdown.

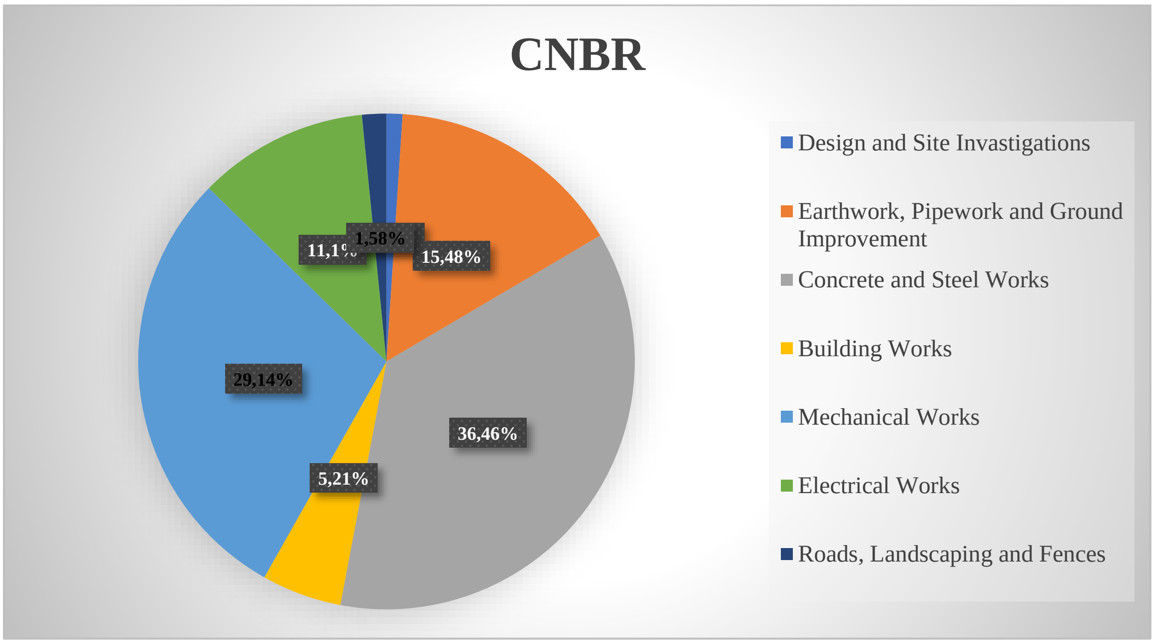


Figure 4.6. Conventional BNR capital expenditure (CAPEX).

The estimation of OPEX takes into account the energy consumed by mixing, internal recirculation,, aeration, and other processing units, as well as the energy produced from anaerobic digester biogas, for both the proposed hybrid configuration and the conventional BNR systems. This information can be found in Table 4.7, which provides the operational cost for the hybrid system.

When calculating OPEX, it was assumed that the cost of electricity is \$0.01 per kilowatt-hour, and the cost of sludge removal is \$50 per ton. Moreover, it was assumed that each cubic meter of biogas contains the calorific equivalent of 6 kWh, out of which 3.3 kWh of usable electricity is generated when the biogas is converted into electricity using a biogas-powered electric generator. The rest of the energy is thought to be converted into heat, which may be used for a variety of purposes, including heating and sludge drying. The hybrid configuration's energy budget is compared to the BNR's in Table 4.7.

Table 4.7. Operational cost for hybrid system.

Component	**Unit**	**Amount**	**Unit**	**Configuration 1 \$/day**	**Hybrid Configuration 2 \$/day**	**Conventional BNR \$/day**
(a) Consumption						
Aeration	kW/m^3	0.025	\$/kw	1,709	1,636	2,121
Mixing*	kW/m^3	0.005	\$/kw	305	305	510
Internal Recirculation	kW/m^3	0.0017	\$/kw	0	0	51
Other Units	kW/m^3	0.008	\$/kw	3,543	3,785	4,320
(b) Production						
Electricity from biogas	kW/m^3	3.3	\$/kw	2,983	3,787	2,287
(c) Environmental tax						
Sludge Disposal***	\$/ton	50	\$/day	1,148	1,341	1,521
TOTAL			\$/day	3,722	3,280	6,236

*based on unit mixing intensity of 5 W/m^3

**CHP conversion efficiency 55%

***Dry solids content of sludge 90%

The BNR system has the greatest energy consumption cost (\$0.047/m3) since the process requires more electricity to operate. In contrast, the hybrid system has a lower overall energy consumption cost than the BNR system (\$0.019/m3). This is mostly due to the higher energy output from biogas, the decreased aeration and mixing needs, and the absence of internal recirculation system. Based on the 15-year OPEX calculation, the conventional BNR system's energy need is roughly \$16.2M. In addition, the hybrid configuration's higher biogas production (40%) is equivalent to about \$2.07 million in revenue over a 15-year period. The hybrid configuration is a financially viable solution for wastewater treatment, particularly for WWTPs with low nitrification rates and having difficulties with TN discharge standard, as determined by the examination of several financial criteria. Similar findings were reported for energy recovery via biogas production in a full-scale, five-stage Bardenpho reactor(4023 kWhd-1) and IFAS-BNR (4152 kWhd-1) plants (Bashar et al., 2018)

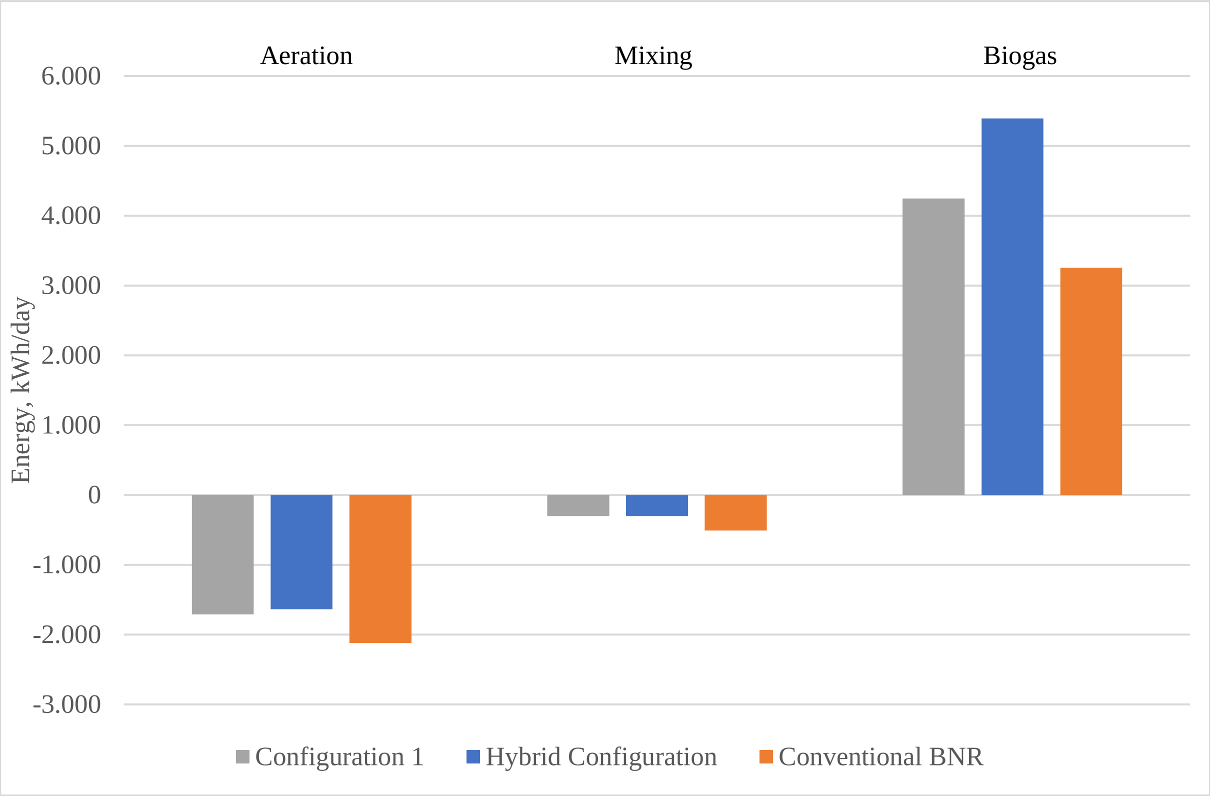


Figure 4.7. Energy budget for hybrid and conventional BNR systems.

5 CONCLUSIONS

In the context of sustainability, wastewater treatment plants play a critical role as a strategic service that can potentially recover resources and energy while reducing energy costs and emissions. The focus is on improving the quality of acceptable and reusable effluent. The main idea in such considerations is mainly energy reduction and resource recovery in an effective way while converting negative environmental impact of wastewater treatment plants into positive. With the circular economy approach, wastewater should not be considered as a potential problem, instead it should be handled as a potential resource (e.g., energy, nutrients, water, and organics) to be recycled and recovered. Therefore, the management of wastewater treatment should be implemented for the technological investments to encourage and economically address/support the resource and energy recovery. In order to minimize overall environmental footprint, it is important to achieve energy efficient treatment plant operation.

One of the objective of the study is to evaluate the effectiveness and accuracy of the ATV-131E design method and model simulations for designing and operating a wastewater treatment plant. ATV-DVWK-A 131E is a German standard for the design of single-stage activated sludge plants used in the design and construction of wastewater treatment plants. Water Utility Administrations and Municipalities in our country commonly use the ATV 131 standard for designing wastewater. By comparing the design methods and simulations with actual plant operational data, the study aims to determine whether the design and operation of the plant are consistent with the design parameters and predicted performance.

In this study, the maximum growth rate of nitrifying bacteria (μ_{AUTO}) at 20°C a in the WWTP was measured to be 0.5 day^{-1} (20°C). Compared to wastewater treatment plants in Germany, the growth rate of nitrifying bacteria (μ_{AUTO} = 1-1.6 day^{-1}) is much lower. It has been stated that industrial discharges in the basin may be the reason for the low activity.

This result highlights an important issue in wastewater treatment plant design. Standardized methods may not always be applicable to local conditions, which can lead to design errors. In this specific case, the growth rate of nitrifying bacteria in a wastewater treatment plant was found to be lower than what is typically observed in Germany, possibly due to industrial discharges in the area. This lower growth rate has implications for the design of the wastewater treatment plant, as it means that higher aerobic sludge ages are needed than what would be calculated using the German design standard. This emphasizes the need for careful consideration of local conditions when designing and operating wastewater treatment plants, as one-size-fits-all solutions may not be appropriate.

The innovative hybrid wastewater treatment plant combine multiple wastewater treatment technologies to create a more efficient and effective treatment process. The objective of this thesis is to evaluate a pilot-scale hybrid system using simulation, which aims to improve both nutrient removal and biogas production, while implementing an efficient organic carbon recovery method. The hybrid system is compared to a conventional BNR system for performance evaluation.

The proposed configuration for the hybrid system focused on capturing organic carbon to enhance the processes of carbon-dependent denitrification, phosphorus removal, and anaerobic digestion, while also improving nitrification efficiency by reducing organic loading through flow separation and application of MBBR. A key advantage of this hybrid system is that it does not consume soluble COD, unlike high rate activated sludge systems (HRAS) such as the A-system used for enhanced biological phosphorus removal (EBPR). Here are some advantages of the innovative hybrid wastewater treatment plant:

- One of the advantages of the hybrid system is that nitrification is carried out in a separate volume and suitable conditions are created for nitrifying autotrophic bacteria by using bio-media. By combining multiple treatment technologies, hybrid plants can achieve higher levels of treatment efficiency and produce higher quality effluent. The existing WWTP and the pilot plant were operated simultaneously in this study. In the WWTP designed according to the standard methods, nitrification problems were observed, and it was found that it was not able to meet the TN discharge standard at certain temperatures. In the hybrid wastewater treatment plant, nitrification occurred, and the TN discharge standard was met.
- The hybrid plant can be designed to accommodate varying influent flows and compositions, making them more flexible than traditional single-stage treatment plants. The nitrification in separate volumes, the fact that the autotrophic bacteria do not have to compete with the heterotrophic bacteria, and the high concentration of autotrophic bacteria were less affected

by the environmental conditions than in other BNR systems. It is recommended to use this process in municipal wastewater treatment plants with highly variable influent flows and pollution concentrations.

- The calculation of total reactor volumes for a hybrid wastewater treatment system and a conventional system indicates that the hybrid system requires a smaller total reactor volume compared to the conventional system. The total reactor volumes for the proposed hybrid system and conventional systems were determined through calculations and found to be 62,000 m3 and 102,000 m3, respectively. This indicates that the hybrid system requires nearly 40% less volume than the conventional system to achieve the same level of wastewater treatment. Thus, the hybrid system requires a smaller total reactor volume, it may require less infrastructure and equipment, leading to lower capital costs for construction and installation. In areas where there is a shortage of sufficient space, the use of a hybrid system is recommended. In addition, it is important to implement this system on a large scale in our country as it significantly reduces initial investment costs.
- Rising energy costs can increase the operating costs of wastewater treatment plants, which can strain the budgets of municipalities or private operators. This can lead to reduced funding for maintenance, repairs, or upgrades, which can compromise the treatment efficiency and environmental compliance of the plant. The analysis of the total oxygen requirement for the proposed hybrid configurations and the conventional BNR system revealed that the hybrid systems require a lower amount of air for the processes. Specifically, the hybrid configurations showed a reduction in air requirement by 19-23% when compared to the traditional BNR system. This reduction in air requirement can lead to lower operational costs and energy consumption, thus providing a significant advantage to the hybrid system. Mixing energy requirement was also reduced by 55% in hybrid system due to smaller volume of the reactors. Furthermore, hybrid system consumes 35% less overall electricity than the conventional system. This was confirmed by the results of pilot tests and simulations. The innovative hybrid wastewater treatment plant has lower operating costs than conventional treatment plant because it is designed to use the most appropriate technology for each stage of the treatment process..
- The Hybrid plant can reduce the environmental impact of wastewater treatment by using less energy, producing less sludge, and emitting fewer pollutants. In the hybrid system, 12% less sludge is generated compared to the BNR system. The specific design of the hybrid

configuration is the reason for this difference. The RPC captures organic matter and converts it to easily biodegradable forms, resulting in higher biogas production.

- Modeling studies have shown that the hybrid system can generate a higher amount of biogas, which is a clean and renewable energy source. The calculated biogas production in the hybrid configuration (11,476 m^3/d) was significantly higher than that produced in the traditional Biological Nutrient Removal (BNR) system (6,930 m^3/d). This is because in the hybrid system, organic matter settled in the preliminary clarifier is sent directly to the denitrification tank, where it is not exposed to aerobic conditions and can be efficiently converted into biogas through anaerobic digestion. Additionally, the hybrid system produces less sludge, which further increases the efficiency of the anaerobic digestion process and results in a higher yield of biogas. The increased biogas production is not only environmentally beneficial but also economically attractive as it can be used as a renewable energy source, reducing energy costs for the wastewater treatment plant.
- The hybrid plant can be designed to treat a wide range of wastewater types and compositions, making them suitable for various applications such as industrial wastewater treatment, municipal wastewater treatment, and decentralized wastewater treatment systems.

The use of this novel hybrid configuration can provide an effective and efficient solution for wastewater treatment, and can be a good alternative to traditional wastewater treatment plant configurations. Moreover, it can be used to upgrade existing treatment plants to improve their performance, reduce operating costs, and increase their overall efficiency.

REFERENCES

Arslan Alaton, I., Insel, G., Eremektar, G., Germirli Babuna, F., & Orhon, D., 2006. Effect of textile auxiliaries on the biodegradation of dyehouse effluent in activated sludge. Chemosphere, 62(9), 1549-1557. https://doi.org/10.1016/j.chemosphere.2005.06.010

Applegate, C.S., Wilder, B. and Deshaw, J.R., 1980. Total nitrogen removal in a multi-channel oxidation systems, J. Wat. Pollut. Cont. Fed., 52(3), 568-577.

APHA, 1998. Standard Methods for the Examination of Water and Wastewater, 20th Eds. American Public Health Association, Washington D.C., U.S.A.

APHA, 1995. Standard Methods for the Examination of Water and Wastewater, 19th Eds., American Public Health Association, Washington D.C., USA.

ASCE , 1991. ASCE Standard: Measurement of Oxygen Transfer in Clean Water, ISBN 0-87262-430-7, New York, NY.

ATV-131, 2000. Dimensioning of Single Stage Activated Sludge Plants. GFA Publishing Company of ATV-DVWK Water, Wastewater and Waste, Hennef, Almanya; ISBN: 3-935669-82-8.

Ast T., Cantwell, J., DiBara M., Hatcher C., Turgeon J., Wizniak M O., 2008. Benchmarking Wastewater Facility Energy Performance Using ENERGY STAR Portfolio Manager, Presented at WEFTEC 2008

Azizi S., Kamika I., Tekere M., 2018. Evaluation of the digestibility of attached and suspended growth sludge in an aerobic digester for a small community. Water 10(2), 161. https://doi.org/10.3390/w10020161

Barker, P.S. and Dold, P.L., 1997. General model for biological nutrient removal activated-sludge systems: model presentation, Water Env. Res., 69(5), 969-984.

Barker, P.S. and Dold, P.L., 1995. COD and nitrogen mass balances in activated sludge systems. Wat. Res., 29(2), 633-643.

Barnard, J.L., 1973. Biological denitrification. Wat. Poll. Cont., 72(6), 705-720.

Bashar R., Gungor K., Karthikeyan K.G., Barak P., 2018. Cost effectiveness of phosphorus removal processes in municipal wastewater treatment. Chemosphere, 197, 280-290. https://doi.org/10.1016/j.chemosphere.2017.12.169

Bassin J.P., Kleerebezem R., Rosado A.S., van Loosdrecht M.C.M., Dezotti M., 2012 Effect of different operational conditions on biofilm development, nitrification, and nitrifying microbial population in moving-bed biofilm reactors. Environ Sci Technol 46(3), 1546–1555. https://doi.org/10.1021/es203356z

Boguniewicz-Zablocka J., Klosok-Bazan I., Capodaglio A.G., 2020. Sustainable management of biological solids in small treatment plants: overview of strategies and reuse options for a solar drying facility in Poland. Environ. Sci. Pollut. Res. 28, 24680–24693. https://doi.org/10.1007/s11356-020-10200-9

Brouwer, H., Klapwijk, A. and Keesman, K.J., 1998. Identification of activated sludge and wastewater characteristics using respirometric batch experiments, Wat. Res., 32(4), 1240-1254.

Brun, R., Kühni, M., Siegrist, H., Gujer, W. and Reichert, P., 2002. Practical identifiability of ASM2d parameters-systematic selection and tuning of parameter subsets, Wat. Res., 36, 4113-4127.

Castro D.F., Bassin J.P., Dezotti M., 2017. Treatment of a simulated textile wastewater containing the Reactive Orange 16 azo dye by a combination of ozonation and moving-bed biofilm reactor: evaluating the performance, toxicity, and oxidation by-products. Environ. Sci. Pollut. Res. 24:6307–6316. https://doi.org/10.1007/s11356-016-7119-x

Christensson M., Walender T., 2004. Treatment of municipal wastewater in a hybrid process using a new suspended carrier with large surface area. Water Sci. Technol., 67(12), 2677-2684. https://doi.org/10.2166/wst.2004.0843

CEC, 1991. Council Directive concerning urban wastewater treatment (91/271/EC) Official Journal L135/40.

Copp, J.B., Spanjers, H. and Vanrolleghem, P.A., 2002. Respirometry in control of the activated sludge process: Benchmarking control strategies. IWA Scientific and Technical Report No:11, London, UK.

De Pauw, D.J.W., Sin, G., Insel, G., Van Hulle, S., Vandenberghe, V. and Vanrolleghem, P.A., 2004. Discussion of 'Assessing Parameter Identifiability of Activated Sludge Model No. 1' by Pedro Afonso and Maria da Conceicao Cunha. J. Environ. Eng., 128, 748-754. J. Environ. Eng.-ASCE, 130(1), 110-112.

De Pauw, B., Coen, F., Vanderhaegen, B. and Vanrolleghem, P.A., 1999. Calibrating simple models for mixing and flow propagation in wastewater treatment plants, Wat. Sci. Tech., 39(4), 61-69.

Dold, D.L. and Marais, G.v.R., 1986. Evaluation of the general activated sludge model proposed by the IAWPRC Task Group, Wat. Sci. Tech., 18(6), 63-89.

Dold, P.L., Ekama, G.A. and Marais, G.v.R., 1980. A general model for the activated sludge process, Prog. Wat. Tech., 12(6), 47-54.

Dynamita, France. www.dynamita.com/the-sumo/ (2021).

EEC (European Economic Community), 1991. The council of 21 May 1991 concerning urban wastewater treatment (91/271/EEC). Off. J. European Communities L135, 40–52.

Ekama G.A. 2015. Recent developments in biological nutrient removal. Water SA 41 (4), 515-524. https://doi.org/10.4314/wsa.v41i4.11

Ekama G, Dold P, Marais GvR. 1986. Procedures for determining influent COD fractions and the maximum specific growth rate of heterotrophs in activated sludge systems. Water Sci Technol 18(6):91–114. https://doi.org/10.2166/wst.1986.0062

Ekama, G.A., Dold, P.L. and Marais G.v.R., 1986b. Procedures for determining influent COD fractions and the maximum specific growth rate of heterotrophs in activated sludge systems. Wat. Sci. Tech., 18, 91-114.

Federation, W. E., 2013. Wastewater treatment process modeling (MOP31) (2nd ed.). McGraw Hill Professional

Forrest D., Delatolla R., Kennedy K., 2016. Carrier effects on tertiary nitrifying moving bed biofilm reactor: An examination of performance, biofilm and biologically produced solids. Environ. Technol. 37(6), 662-671. https://doi.org/10.1080/09593330.2015.1077272

Germirli, F., Orhon, D. and Artan, N., 1991. Assessment of the initial inert soluble COD in industrial wastewaters, Wat. Sci. Tech., 23(4-6), 1077-1086.

Girault R., Bridoux G., Nauleau F., Poullain C., Buffet J., Steyer J.P., Sadowski A.G., Beline F., 2012. A waste characterisation procedure for ADM1 implementation based on degradation kinetics. Water Res. 46 (13), 4099–4110. https://doi.org/10.1016/j.watres.2012.04.028

Grady, C.P.L.Jr., Daigger, G.T. and Lim, H.C., 1999. Biological Wastewater Treatment, Marcel Dekker Inc., Newyork, U.S.A.

Grady, C.P.L.Jr., Smets, B.F. and Barbeau, S., 1996. Variability in kinetic parameter estimates: a review of possible causes and a proposed terminology, Wat. Res., 30(3), 742-748.

Grady, C.P.L., W. Gujer, M. Henze, G.v.R. Marais, and T. Matsuo., 1986. A model for single-sludge wastewater treatment systems. Water Sci. Tech. 18(6):47-61.

Gujer, W., Henze, M., Mino, T. and van Loosdrecht, M.C.M., 2000. Activated Sludge Model No.3., in: Activated Sludge Models ASM1, ASM2, ASM2d and ASM3, Henze, M., Gujer, W., Mino, T., van Loosdrecht, M.C..M. Eds. IWA Scientific and Technical Report No.9. IWA London.

Gujer, W., Henze, M., Mino, T. and van Loosdrecht, M.C.M., 1999. Activated sludge model No 3., Wat. Sci. Tech., 39(1), 183-193.

Gujer, W. and Larsen, T.A., 1995. The implementation of biokinetics and conservation principles in ASIM, Wat. Sci. Tech., 31(2), 257-266.

Gupta M., 2018. Microsieving as a Primary Treatment for Biological Nitrogen Removal from Municipal Wastewater. Ph.D. Thesis, The University of Western Ontario, Canada.

Guven H., Ozgun H., Ersahin E., Dereli R.K., Sinop I., Ozturk I., 2019. High-rate activated sludge processes for municipal wastewater treatment: the effect of food waste addition and hydraulic limits of the system. Environmental Science and Pollution Research 26, 1770-1780. https://doi.org/10.1007/s11356-018-3665-8

Güneş G., Hallaç E., Özgan M., Ertürk A., Okutman-Tas D., Çokgor E., Güven D., Takacs I., Erdinçler A., Insel G., 2019. Enhancement of nutrient removal performance of activated sludge with a novel hybrid biofilm process. Bioproc. Biosystems. Eng. 42 (3), 379-390. https://doi.org/10.1007/s00449-018-2042-9

Güneş G., Demir A., Abamor H.E., 2010. İstanbul'da evsel atıksu arıtma tesislerinde enerji yönetimi, IWES, İstanbul

Hellinga, C., Schellen, A.A.J.C., Mulder, J.W., Van Loosdrecht, M.C.M and Heijnen, J.J., 1998. The SHARON process: an innovative method for nitrogen removal for ammonium-rich wastewater. Wat. Sci.Tech., 37, 135-42.

Henze M., van Loosdrecht M.C.M., Ekama G.A., Brdjanovic D., 2008. Biological Wastewater Treatment Principles, Modelling and Design, IWA Publishing, Londra, İngiltere, ISBN: 1843391880.

Henze, M., Aspegren, H., Jansen, J.L., Nielsen, P.H. and Lee, N., 2002. Effect of solids retention time and wastewater characteristics on biological phosphorus removal, Wat. Sci. Tech., 45(6), 137-144.

Henze, M., Gujer, W., Mino, T., and van Loosdrecht, M.C.M., 2000. Activated Sludge Models: ASM1, ASM2, ASM2d and ASM3, Scientific and Technical Report No:9, IWA Publishing, London.

Henze, M., Gujer, W., Mino, T., Matsuo, T., Wentzel, M. C., Marais., G. v. R. and van Loosdrecht, M. C. M., 1999. Activated sludge model No.2d, ASM2d, Wat. Sci. Tech., 39(1), 165-182.

Henze, M., Gujer, W., Mino, T., Matsuo, T., Wentzel, M.C. and Marais, G.v.R., 1995a. Activated sludge model No.2., IAWPRC Task Group on Mathematical Modelling for Design and Operation of Biological Treatment, IAWQ, London.

Henze, M., Harremoës, P., Jansen, J.C. and Arvin, E., 1995b. Biological and Chemical Processes, in Wastewater Treatment, Springer-Verlag, Berlin.

Henze, M., Gujer, W., Mino, T., Matsuo, T., Wentzel, M.C. and Marais, G.v.R., 1995c. Wastewater and biomass characterization for the activated sludge model no.2: Biological phosphorus removal. Wat. Sci. Tech., 31(2), 13-23.

Henze, M., Dupont, R., Grau, P and De la Sota, A., 1993. Rising sludge in secondary settlers due to denitrification, Wat. Res., 27(2), 231-236.

Henze, M., 1992. Characterization of wastewater for modelling of activated sludge processes, Wat. Sci. Tech., 25(6), 1-15.

Henze, M. and Mladenovski, C., 1991. Hydrolysis of particulate substrate by activated sludge under aerobic, anoxic and anaerobic conditions, Wat. Res., 25(1), 61-64.

Henze, M., Grady C.P.L.Jr., Gujer, W., Marais, G.v.R. and Matsuo, T., 1987. Activated sludge model No.1, IAWPRC Science and Technical Report No. 1, IAWPRC, London.

Hulsbeek, J.J.W., Kruit, J., Roeleveld, P.J., van Loosdrecht, M.C.M., 2002. A practical protocol for dynamic modelling of activated sludge systems, 45(6), 127-136.

Hu Z., Wentzel M., Ekama G., 2001. External nitrification in biological nutrient removal activated sludge systems. Water Science and Technology 43, 251-260. https://doi.org/10.2166/wst.2001.0055

Insel G., Artan N., Cokgor E., Okutman-Tas O., Guven D., Zengin Balci G.E., Pala-Ozkok I., Ozyildiz G., 2021. Wastewater Management Standardization Project: Experimental Characterization and Dynamic modeling of wastewater treatment systems, Water and Sewerage Administration of Istanbul, Research Project, İstanbul.

Insel G., 2014. İSKİ Ataköy Advanced Biological Wastewater Treatment Plant nitrogen removal performance evaluation report, Technical Report, ITU Faculty of Civil Engineering, No. 4083, October 2014, Istanbul.

Insel, G., B. Güder, G. Güneş, and E. Ubay Cokgor. "Are standard wastewater treatment plant design methods suitable for any municipal wastewater?", Water Science & Technology, 2012.

Insel, G., Dülekgürgen, E., Sözen, S., Orhon, D., 2009. Conceptual basis for the appropriate design of biological wastewater treatment systems: drawbacks of existing prescriptions

Insel, G., Sin, G., Lee, D.S. and Vanrolleghem, P.A., 2004. A calibration methodology and model-based system analysis for nutrient removing SBRs, 3rd IWA International conference on SBR, February 22-26th, Noosa, Queensland, Australia.

Insel, G., Orhon, D. and Vanrolleghem, P.A., 2003a. Identification and modelling of aerobic hydrolysis mechanism-application of optimal experimental design, J. Chem.Tech. Biotech., 78(4), 437-445.

Insel, G., De Pauw, D., Lee, D.S. and Vanrolleghem, P.A., 2003b. Dynamic calibration of an intermittently aerated carrousel plant-Report No1, Technical Report, Gent University, Applied Mathematics, Biometrics and Process Control.

Insel, G., Sin, G., Lee, D.S. and Vanrolleghem, P.A., 2003b. A calibration methodology and model-based system analysis for nutrient removing SBRs, 3rd IWA International conference on SBR, Queensland, Australia.

Jetten, M.S.M., Strous, M, de Pas-Schoonen, K.T., Schalk, J., van Dongen, G.J.M. van de Graaf, A.A., Logemann, S., Muyzer, G., van Loosdrecht , M.C.M, Kuenen, J.G., 1999. The anaerobic oxidation of ammonium, FEMS Microbial. Rev., 22, 421-437

Jimenez J., Miller M., Bott C., Murthy S., De Clippeleir H., Wett B., 2015. High-rate activated sludge system for carbon management-Evaluation of crucial process mechanisms and design parameters. Water Res. 87, 476-482. https://doi.org/10.1016/j.watres.2015.07.032

Kayser R., Activated Sludge Process, 2001, Wiley.

Kayser R., 1999. Activated Sludge Process, 13th Section IN: Rehm J and Reed G, Biotechnology, 2nd Edition, Wiley-VCH, Vol. 11a, Weinheim, Germany.

Kazmi, A.A., Fujita, M. and Furumai, H., 2001. Modeling effect of remaining nitrate on phosphorus removal in SBR, Wat. Sci. Tech., 43(3), 175-182.

Keller, J., Watts, S., Battye-Smith, W. and Chong, R., 2001. Full-scale demonstration of biological nutrient removal in a single tank process,. Wat. Sci. Tech., 43(3), 355-362.

Kolisch G., Osthoff T., Hobus I., Hansen J., 2009. Increasing the Energy Efficiency of Sewage Plants - Experiences of Energy Analyses carried out in Germany,

Leyva-Díaz J.C., Monteoliva-García A., Martín-Pascual J., Munio M.M., García-Mesa J.J., Poyatos J.M., 2020. Moving bed biofilm reactor as an alternative wastewater treatment process for nutrient removal and recovery in the circular economy model. Bioresour. Technol. 299, 122631. https://doi.org/10.1016/j.biortech.2019.122631

Lotter, L.H., M.C. Wentzel, R.E. Loewenthal, G.A. Ekama, and G.v.R. Marais., 1986. A study of selected characteristics of Acetinobacter spp. isolated from activated sludge in anaerobic/anoxic/aerobic and aerobic systems. Water SA 12(4):203-208.

Makinia, J., Swinarski, M. and Dobiegala, E., 2002. Experiences with computer simulation at two large wastewater treatment plants in northern Poland, Wat. Sci. Tech., 45(6), 209-218.

Mamais, D., Jenkins, D. and Pitt, P., 1993. A rapid physical chemical method for the determination of readily biodegradable soluble COD in municipal wastewaters, Wat. Res., 27, 195-197.

Mauer C., Husmann M., Sahlstedt K., Seitter J. and Simsheuser C., 2009. Improved Energy Savings and Recovery on Wastewater Treatment Plants by the use of a standardized Optimization Procedures or Simulation Studies, Germany

Meijer, S.C.F., van Loosdrecht, M.C.M. and Heijnen J.J., 2001. Metabolic modeling of full-scale biological nitrogen and phosphorus removing WWTPs, Wat. Res., 35(11), 2711-2723.

Melcer H., Dold P., Jones RM., Takacs I., Stensel HD., Wilson AW., Sun P., Bury S., 2003. Methods for wastewater characterization in activated sludge modeling, Technical Report, Water Environment Research Federation, Alexandria, ABD. ISBN: 1-893664-71-6.

Metcalf and Eddy, 2014. Wastewater Engineering, Treatment and Reuse, McGraw Hill, New York.

Moretti, Charles J., Dipesh Das, Brian T. Kistner, Harvey Gullicks, and Yung-Tse Hung. 2011. "Activated Sludge and Other Aerobic Suspended Culture Processes" Water 3, no. 3: 806-818. https://doi.org/10.3390/w3030806

Mottet A., François E., Latrille E., Steyer J.P., Déléris S., Vedrenne F., Carrère H., 2010. Estimating anaerobic biodegradability indicators for waste activated sludge. Chem. Eng. J. 160, 488–496. https://doi.org/10.1016/j.cej.2010.03.059

Muller, A., Wentzel, M.C., Loewenthal, R.E. and Ekama, G.A., 2003. Heterotrophic anoxic yield in anoxic aerobic activated sludge systems treating municipal wastewater, Wat. Res., 37(10), 2435-2441.

Munch, E.V., Lant, P. and Keller, J., 1996. Simultaneous nitrification and denitrification in bench-scale sequencing batch reactors, Wat. Res., 30(2), 277-284.

Naidoo, K., Ndlovu, V., Mjadu, L., Goatley, K. and Kerdachi, D.A., 2002. The exceptional simultaneous removal of carbon, nitrogen and phosphorus in a simple activated sludge treatment system at Kingsburgh Watewater Treatment Works, Water SA., WISA Proceedings, 67-72.

Nguyen L.N., Kumar J., Vu M.T., Mohammed J.A.H., Pathak N., Commault A.S., Sutherland D., Zdarta J., Tyagi V.K., Nghiem L.D., 2020. Biomethane production from anaerobic co-digestion at wastewater treatment plants: a critical review on development and innovations in biogas upgrading techniques. Science of The Total Environment 142753.
https://doi.org/10.1016/j.scitotenv.2020.142753.

Nowak,O., 2003. Benchmarks for the energy demand of nutrient removal plants, Water Science and Technology, Volume 47, Number 12, 125-132

Nuhoğlu A., System design for organic carbon and nutrient removal from sewage based on energy recovery, Ph.D. thesis, İstanbul Technic University, 2012.

Ødegaard H., Christensson M., Sørensen K., 2014. Hybrid systems. In: Jenkins D, Wanner J (eds) Activated Sludge - 100 years and counting. IWA Publishing, London, UK.

Okutman Tas D., Karahan O., Insel G., Övez S., Orhon D., Spanjers H., 2009. Biodegradability and denitrification potential of settled chemical oxygen demand in domestic wastewaters, Water Environment Research, 81(7), 715-727.

Okutman D., Ovez S., Orhon D., 2001. Hydrolysis of settleable substrate in domestic sewage. Biotechnology Letters 23, 1907–1914. https://doi.org/10.1023/A:1013737901624

Olsson, G. and Jeppson, U., 1994. Establishing cause-effect relationships in activated sludge plants-what can be controlled? Proceedings of Fac. Landbouww., Gent University, 59/4a.

Onnis-Hayden, A., Srinivasan, V., Tooker, N., Li, G., Wang, D., & Gu, A. Z., 2018. Survey of Full Scale Side-Stream EBPR Facilities and Comparison with Conventional EBPR:Process Stability, Kinetics and Microbial Ecology. Preprints.https://doi.org/10.20944/preprints201808.0241.v1

Onnis-Hayden A., Majed N., Drury D., Risso L., McMahon K., April Z., 2013. Effect of Sludge Residence Time on Phosphorus Removal Activities and Populations in Enhanced Biological Phosphorus Removal (EBPR) Systems Proceedings of the Water Environment Federation, WEFTEC2013: Session 4 through Session 9, pp. 121-132(12).

Orhon, D., Okutman, D. and Insel, G., 2002. Characterization and biodegradation of settleable organic matter for domestic wastewater,. Water SA, 28(3) 299-305.

Orhon, D., Babuna, F.G. and Insel, G., 2001. Characterization and modelling of denim-processing wastewaters for activated sludge, J. Chem. Tech. Biotech., 76(9), 919-931.

Orhon, D., Ates Genceli, E. and Sözen, S., 2000. Experimental evaluation of the nitrification kinetics for tannery wastewaters, *Water SA*, 26(1), 43–50.

Orhon, D., Taşlı, R. and Sözen, S., 1999a. Experimental basis of activated sludge treatment for industrial wastewaters – the state of the art, *Wat. Sci. Tech.*, 40(1), 1-11.

Orhon, D. and Artan, N., 1994. Modelling of Activated Sludge Systems, Technomic Press, Lancaster PA.

Petersen, B., Gernaey, K., Henze, M. and Vanrolleghem, P.A., 2002. Evaluation of an ASM1 model calibration procedure on a municipal-industrial wastewater treatment plant, *J. Hydroinformatics*, 4(1), 15-38.

Petersen, B., 2000. Calibration identifiability and optimal experimental design of activated sludge models, *PhD Thesis,* Applied Mathematics Biometrics and Process Control (BIOMATH) Department, Belgium.

Randall C.W., Sen D., 1996. Full-scale evaluation of an integrated fixed-film activated sludge (IFAS) process for enhanced nitrogen removal. Water Sci. Technol., 33(12), 152–162. https://doi.org/10.1016/0273-1223(96)00469-6

Randall, C.W., Barnard, L.B. and Stensel, H.D., 1992. Design and Retrofit of Wastewater Treatment Plants for Biological Nutrient Removal, Technomic Publishing Inc., Lancaster, PA.

Reichert, P., Ruchti, J. and Simon, W., 1998. AQUASIM 2.0, Swiss Federal Institute for Environmental Science and Technology (EAWAG), CH-8600 Duebendorf, Switzerland.

Rice, E. W., Bridgewater, L., 2012, Standard methods for the examination of water and wastewater, American Public Health Association, American Water Works Association, & Federation, W. E., 2012. Amer Public Health Assn.

Rittmann, B.E. and Langeland, W.E., 1985. Simultaneous denitrification with nitrification in single-channel oxidation ditches, J. Wat. Poll. Cont. Fed, 57(4), 300-308.

Rodrigo, M.A., Seco, A., Penya Roja, J.M., Ferrer, J., 1996. Influence of sludge age on enhanced phosphorus removal in biological systems, Wat. Sci. Tech., 34(1-2), 41-48.

Roeleveld, P.J. and van Loosdrecht, M.C.M., 2002. Experiences with guidelines for wastewater characterization in the Netherlands, Wat. Sci. Tech., 45(6), 77-87.

Siegrist, H. and Tschui, M., 1992. Interpretation of experimental data with regard to the activated sludge model no. 1 and calibration of the model for municipal wastewater treatment plants, Wat. Sci. Tech., 25(6), 167-183.

Singh A., Kamble S.J., Sawant M., Chakravarthy Y., Kazmi A., Aymerich E., Starkl M., Ghangrekar M., Philip L., 2018. Technical, hygiene, economic, and life cycle assessment of full-scale moving bed biofilm reactors for wastewater treatment in India. Environ. Sci. Pollut. Res. 25, 2552–2569. https://doi.org/10.1007/s11356-017-0605-y

Sollfrank, U. and Gujer, W., 1991. Characterization of domestic wastewater for mathematical modelling of the activated sludge process, Wat. Sci. Tech., 23(4-6), 1057-1066.

Sözen S., Dulkadiroglu H., Yucel A.B., Insel G., Orhon D., 2019. Pollutant footprint analysis for wastewater management in textile dye houses processing different fabrics", Journal of Chemical Technology & Biotechnology.

Sözen S., Orhon D., Ubay Cokgor E., Gorgun E., Insel G., Karahan O, Yağcı N, Okutman Taş D., Dülekgürgen E., Doğruel S., Ölmez Hancı T., Zengin Balcı GE, Çığgın AS., Pala İ., Katipoğlu T., Eldem N., Ünal A., 2008. İSKİ Rehabilitation of Wastewater Treatment Plants Project, ITU, Faculty of Civil Engineering, Department of Environmental Engineering, Istanbul.

Sözen, S., Ubay Çokgör, E., Orhon, D. and Henze, M., 1998. Respirometric analysis of activated sludge behaviour-II. Heterotrophic growth under aerobic and anoxic conditions, Wat. Res., 32(2), 476-488.

Sözen, S., Orhon, D. and San, H.A., 1996. A new approach for the evaluation of the maximum specific growth rate in nitrification. Wat. Res., 30(1), 43–50.

Spanjers H., Vanrolleghem P., Olsson G., Dold P., 2011. Respirometry in Control of the Activated Sludge Process: Principles, International Water Association (IWA), Technical Report No.7, ISBN 1900222043, London, England

Spanjers, H., Vanrolleghem, P.A., Olsson, G. and Dold, P.L., 1998. Respirometry in Control of the Activated Sludge Process Principles, IAWQ Scientific and Technical Report No. 7, London, UK.

Spanjers, H. and Vanrolleghem, P.A., 1995. Respirometry as a tool for rapid characterization wastewater and activated sludge. *Wat. Sci. Tech.*, 31(2), 105-114.

Spanjers, H., 1993. Respirometry in activated sludge, *PhD Thesis*, Wageningen Agricultural University, Wageningen, the Netherlands.

Sperandio, M. and Paul, E., 2000. Estimation of wastewater biodegradable COD fractions by combining respirometric experiments in various S0/X0 ratios, *Wat. Res.*, 34(4), 1233-1244.

STOWA, 2001. The combined Sharon-Anammox process, STOWA report. IWA Publishing.

Strous, M., 2000. Anammox and Nitrification. In: Microbiology of Anaerobic Ammonium Oxidation, Ph.D. thesis, 63-81, the Netherlands.

Ubay Çokgör, E., Sözen, S., Orhon, D. and Henze, M., 1998. Respirometric analysis of activated sludge behaviour. Assesment of readily biodegradable substrate, Wat. Res., 32(2), 461-475.

Ucisik A.S., Henze M., 2008. Biological hydrolysis and acidification of sludge under anaerobic conditions: The effect of sludge type and origin on the production and composition of volatile fatty acids. Water Res. 42, 3729–3738. https://doi.org/10.1016/j.watres.2008.06.010

Wentzel, M.C., Ekama, G.A., Dold, P.L. and Marais, G.v.R., 1990. Biological excess phosphorus removal-steady state process design, Water SA, 16, 29-48.

Wentzel, M.C., Dold, P.L., Ekama, G.A. and Marais, G.v.R., *1989a Enhanced polyphosphate organism cultures in activated sludge systems, Part III, Kinetic model, Water SA, 15(2), 89-102.*

Wentzel, M.C., Ekama G.A., Loewenthal, R., Dold P. and Marais, G.v.R., 1989b. Enhanced polyphosphate organism cultures in activated sludge systems, Part II: Experimental behavior, *Water SA*, 15(2), 71-88.
Weijers, S.R., 1999. Online BOD tests for the determination of biodegradable COD for calibrating activated sludge model no.1, Wat. Sci. Tech., 39(4), 177-184.

Weijers, S.R. and Vanrolleghem, P.A., 1997. A procedure for selecting the most important parameters in calibrating the activated sludge model no.1 with full-scale plant data, Wat. Sci. Tech., 36(5), 69-79.

W.E.R.F., 1999. *Methods for wastewater characterization in activated sludge modeling*, Report No: 99 WWF 3, Alexandria, USA.

W.E.R.F., 1996. Aeration-a wastewater treatment process, Report No: 68, Water Environment Research Federation, Newyork, USA.

Wichern M., Lübken M., Blömer R., Rosenwinkel KH., 2003. Efficiency of the Activated Sludge Model no.3 for German wastewater on six different WWTPs, Water Science and Technology, 47(11), 211-218.

Wilson, A.W., and Dold P.L., 1998. General methodology for applying process simulators to wastewater treatment plants. In Proc. 71st Annual Conference of the Water Environment Federation, Orlando, FL. Alexandria, VA: Water Environment Federation.

van Dongen, U., Jetten, M.S.M. and van Loosdrecht, M.C.M.., 2001. The SHARON-Anammox process for treatment of ammonium rich wastewater., 44(1), 153-160.

van Haandel, A.P.C, Ekama, G.A. and Marais, G.v.R., 1981. The activated sludge process 3-single sludge denitrification, Wat. Res., 15, 1135-1152.

Vanhooren, H., Meirlaen, J., Amerlinck, Y., Claeys, F., Vangheluwe, H. and Vanrolleghem, P.A., 2003. WEST: Modeling biological wastewater treatment, J. Hydroinformatics, 5, 27-50.

van Loosdrecht, M.C.M. and Henze, M., 1999. Maintanence, endogenous respiration, lysis, decay and predation, Wat. Sci. Tech., 39(1), 107-117.

van Loosdrecht, M.C.M., Brandse, F.A. and de Vries, A.C., 1998. Upgrading of wastewater treatment processes for integrated nutrient removal-the BCFS® process, Wat. Sci. Tech., 37(9), 209-217.

Vanrolleghem, P.A., Insel, G., Petersen, B, Sin, G., De Pauw, D., Nopens, I., Dovermann, H., Weijers, S. and Gernaey, K., 2003. A comprehensive model calibration procedure for activated sludge models, WEFTEC 2003, 76th Annual Technical Exhibition and Conference, October 11-15, 2003, Los Angeles, California U.S.A.

Vanrolleghem, P.A., Spanjers, H., Petersen, B., Ginestet, P. and Takacs, I., 1999. Estimating (combinations of) Activated Sludge Model No.1 parameters and components by respirometry, Wat. Sci. Tech., 39(1), 195-214.

Van Winckel T., 2014. Development of high-rate activated sludge processes for energy-efficient wastewater treatment. MSc. Thesis, Gent University, Belgium.

Yang X., Wei J., Ye G., Zhao Y., Li Z., Qiu G., Li F., Wei C., 2020. The correlations among wastewater internal energy, energy consumption and energy recovery/production potentials in wastewater treatment plant: An assessment of the energy balance. Sci. Total Environ. https://doi.org/10.1016/j.scitotenv.2020.136655

APPENDIX A : PATENT FILE of BIOFILM NITRIFICATION - CONTACT DENITRIFICATION SYSTEM AND METHOD

Details of the invention can be found at the link below.

https://patents.google.com/patent/WO2016122426A1/en?inventor=didem+okutman&oq=didem+okutman

(12) INTERNATIONAL APPLICATION PUBLISHED UNDER THE PATENT COOPERATION TREATY (PCT)

(19) World Intellectual Property Organization
International Bureau

(43) International Publication Date
4 August 2016 (04.08.2016)

WIPO | PCT

(10) International Publication Number
WO 2016/122426 A1

(51) International Patent Classification:
C02F 3/30 (2006.01) *C02F 101/30* (2006.01) *C02F 1/20* (2006.01) *C02F 3/08* (2006.01) *C02F 101/10* (2006.01) *C02F 3/06* (2006.01) *C02F 101/16* (2006.01) *C02F 3/12* (2006.01)

(21) International Application Number: PCT/TR2015/050300

(22) International Filing Date: 31 December 2015 (31.12.2015)

(25) Filing Language: English

(26) Publication Language: English

(30) Priority Data:
2015/01081 30 January 2015 (30.01.2015) TR

(71) Applicant: ISTANBUL TEKNIK UNIVERSITESI [TR/TR]; ITU Rektorluk Binasi Ayazaga Kampusu, Maslak, 34469 Istanbul (TR).

(72) Inventors: INSEL, Hayrettin Guclu; ITU Ayazaga Kampusu Insaat Fakultesi, Cevre Muhendisligi Bolumu, Istanbul (TR). COKGOR, Emine; Istanbul Teknik Universitesi Insaat Fakultesi, Cevre Muhendisligi Bolumu, Maslak, 3446 Istanbul (TR). GUNES, Giyasettin; Cevizlidere Caddesi Cevizlidere Mahallesi, Dostlar Apartmani No:58 Daire:6, Cankaya/Ankara (TR). OKUTMAN TAS, Didem; Darussafaka Mahallesi Karikaturistler Sitesi, B-blok D:15, Tarabya, Istanbul (TR).

(74) Agent: ANKARA PATENT BUREAU LIMITED; Bestekar Sokak No:10, Kavaklidere, Ankara (TR).

(81) Designated States *(unless otherwise indicated, for every kind of national protection available)*: AE, AG, AL, AM, AO, AT, AU, AZ, BA, BB, BG, BH, BN, BR, BW, BY, BZ, CA, CH, CL, CN, CO, CR, CU, CZ, DE, DK, DM, DO, DZ, EC, EE, EG, ES, FI, GB, GD, GE, GH, GM, GT, HN, HR, HU, ID, IL, IN, IR, IS, JP, KE, KG, KN, KP, KR, KZ, LA, LC, LK, LR, LS, LU, LY, MA, MD, ME, MG, MK, MN, MW, MX, MY, MZ, NA, NG, NI, NO, NZ, OM, PA, PE, PG, PH, PL, PT, QA, RO, RS, RU, RW, SA, SC, SD, SE, SG, SK, SL, SM, ST, SV, SY, TH, TJ, TM, TN, TR, TT, TZ, UA, UG, US, UZ, VC, VN, ZA, ZM, ZW.

(84) Designated States *(unless otherwise indicated, for every kind of regional protection available)*: ARIPO (BW, GH, GM, KE, LR, LS, MW, MZ, NA, RW, SD, SL, ST, SZ, TZ, UG, ZM, ZW), Eurasian (AM, AZ, BY, KG, KZ, RU, TJ, TM), European (AL, AT, BE, BG, CH, CY, CZ, DE, DK, EE, ES, FI, FR, GB, GR, HR, HU, IE, IS, IT, LT, LU, LV, MC, MK, MT, NL, NO, PL, PT, RO, RS, SE, SI, SK,

[Continued on next page]

(54) Title: BIOFILM NITRIFICATION - CONTACT DENITRIFICATION SYSTEM AND METHOD

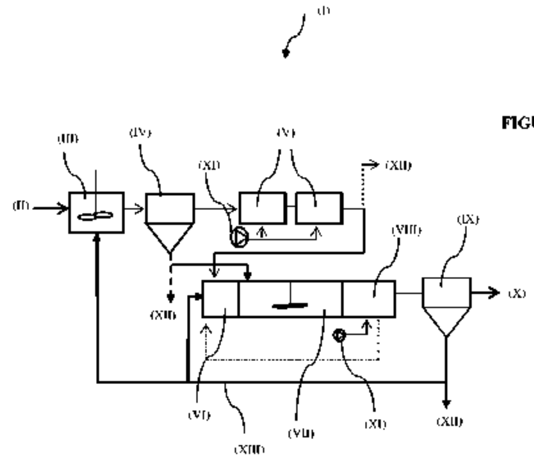


FIGURE 1

(57) Abstract: The present invention relates to a biofilm nitrification - contact denitrification system (I), which provides advanced level of organic carbon, nitrogen and phosphorous removal from waste waters, and basically comprises at least one wastewater inlet (II) through which the raw waste water is delivered to the system, at least one selector unit (III) wherein the settleable organic substances in wastewater are mixed with the biomass, at least one intermediate settling tank (IV) wherein particulate organic substances are settled via biofloculation, at least one aerobic biofilm tank (V) wherein nitrification process is carried out, at least one deoxygenation tank (VI) which enables to reduce high dissolved oxygen concentration originating from the aerobic biofilm tank (V), at least one contact denitrification tank (VII) wherein denitrification process is carried out, at least one aerobic tank (VIII) which enables the nitrogen gas that is released as a result of denitrification process to be removed from the system and the residual ammonia nitrogen and/or dissolved organic substance in waste water to be oxidized, at least one final sedimentation tank (IX) which enables the treated water to be separated from the biomass, at least one treated water discharge (X) which is located at the outlet of the final settling tank (IX), at least one aeration/mixing unit (XI) which enables optimization of oxygen concentration in the aerobic tank (VIII) and the aerobic biofilm tank (V), and homogenous distribution of oxygen, at least one waste sludge discharge (XII) through which waste sludge produced in the system is removed, at least one return sludge line (XIII) which enables a part of the sludge settled in the system to return to the selector unit (III) and another part of it recycled back to the deoxygenation tank (VI).

WO 2016/122426 A1

SM, TR), OAPI (BF, BJ, CF, CG, CI, CM, GA, GN, GQ, GW, KM, ML, MR, NE, SN, TD, TG).

Published:

— *with international search report (Art. 21(3))*

BIOFILM NITRIFICATION - CONTACT DENITRIFICATION SYSTEM AND METHOD

DESCRIPTION

Field of the Invention

The present invention relates to a biofilm nitrification - contact denitrification system and method in which environmental technology and water recovery technologies are used and which provide advanced removal of organic carbon, nitrogen and phosphorous from waste waters.

Background of the Invention

Nitrification is oxidation of ammonia nitrogen in the waste waters to nitrate nitrogen which is an oxidized nitrogen form in an aerobic environment by the activities of the microorganisms. The process comprises oxidation of ammonia nitrogen to nitrite (NO_2) by Nitromonas bacteria and oxidation of nitrite up to the nitrate (NO_3) ions by the Nitrobacter bacteria. Denitrification is transformation of oxidized nitrogen forms (NO_2, NO_3) into molecular nitrogen via bacteria activity. As a common practice, the nitrification and denitrification are carried out in single sludge systems by selecting the suitable process conditions in the activated sludge systems.

In Fluidized Bed Bioreactor (FBBR) systems, a media (generally plastic) is maintained in the reactor in order to increase the capacity of activated sludge systems and to keep more biomass in the reactor and thus to increase the capacity without requiring additional volume. Generally more biomass is kept in the aerated and/or unaerated reactors by the addition of those filling media. The media used for FBBR are added to the single sludge systems wherein nitrification and denitrification processes take place together. Nitrification and denitrification processes are provided with the aid of attached biofilm growing on the surface of the media.

The competition between bacteria responsible for nitrification and organic carbon, causes washout of nitrification bacteria due to its comparably low yield and growth rate. Since the bacteria which carry out the nitrification process are at the same environment with the heterotrophic bacteria that carry out organic carbon removal, they cause oxidation of organic carbon under aerobic conditions leading to the loss of denitrification capacity. Particularly, in urban wastewaters wherein nitrification rate is very low, this situation result in the loss of nitrogen removal efficiency or the selection of large reactor volumes.

High internal (nitrate) recirculation ratios are required in order to provide denitrification in conventional single sludge systems. Biological nitrogen removal in conventional systems requires recirculation of 4-5 times the inlet wastewater flowrate back to the anoxic tank (head of bioreactor). This increases the operation costs due to the pumping costs. Furthermore, oxidation of the organic matter to CO_2 under aerobic conditions also negatively affects the potential of obtaining biogas from activated sludge via anaerobic digestion. In current practice,, (in single-sludge systems), it is not possible to control nitrification and denitrification processes separately which is the major drawback. Nitrogen and phosphorus containing fraction does not allow water reuse for irrigation purposes.

The above mentioned problems have been overcome by means of the biofilm nitrification – contact denitrification system and method of the present invention.

Summary of the Invention

The objective of the present invention is to provide a biofilm nitrification - contact denitrification system and method which provide advanced level of organic carbon, nitrogen and phosphorous removal from waste waters.

Another objective of the present invention is to provide a biofilm nitrification - contact denitrification system and method which eliminate the nitrate internal recirculation required for denitrification and enable to reduce the footprint of wastewater treatment plants.

A further objective of the present invention is to provide a biofilm nitrification - contact denitrification system and method which enable nearly all of the settleable organic (biodegradable) matters to be used in denitrification process.

Detailed Description of the Invention

A biofilm nitrification - contact denitrification system and method developed to fulfill the objective of the present invention are illustrated in the accompanying figures (Figure 1 and Figure 2), in which;

Figure 1 is the schematic view of the biofilm nitrification - contact denitrification system.

Figure 2 is the steps of the biofilm nitrification - contact denitrification method.

The components given in Figure 1 are assigned reference numbers as follows:

I. Biofilm nitrification - Contact denitrification system
II. Wastewater Inlet
III. Selector Unit
IV. Intermediate Settling Tank

V. Aerobic Biofilm Tank

VI. Deoxygenation (DeOx) Tank

VII. Contact Denitrification Tank

VIII. Aerobic Tank

IX. Final Sedimentation Tank

X. Treated Water Discharge

XI. Aeration/Mixing Unit

XII. Waste Sludge Discharge

XIII. Return Sludge Line

The method steps given in Figure 2 are assigned reference numbers as follows:

100. Contact denitrification method

101. Mixing the settleable organic matter with biomass in the selector tank (III),

102. Settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation,

103. Performing nitrification process in the aerobic biofilm tank (V),

104. Reducing the high dissolved oxygen concentration in the deoxygenation tank (VI),

105. Performing denitrification process in the contact denitrification tank (VII),

106. Oxidizing residual ammonia nitrogen and/or dissolved organic substance in wastewater in the aerobic tank (VII),

107. Discharging the treated water in the final settling tank (IX) upon separating it from the biomass,

108. Disposal of waste (excess) sludge produced in the system through the waste sludge discharge (XII) and delivering part of it to the return sludge line (XIII),

A biofilm nitrification - contact denitrification system (I), which provides advanced level of nitrogen and phosphorous removal from waste waters, basically comprises

- at least one wastewater inlet (II) through which the raw waste water is delivered to the system,
- at least one selector unit (III) wherein the settleable organic substances in wastewater are mixed with the biomass,
- at least one intermediate settling tank (IV) wherein particulate organic substances are settled via biofloculation,
- at least one aerobic biofilm tank (V) wherein nitrification process is carried out,
- at least one DeOx tank (VI) which enables to reduce high dissolved oxygen concentration originating from the aerobic biofilm tank (V),
- at least one contact denitrification tank (VII) wherein denitrification process is carried out,
- at least one aerobic tank (VIII) which enables the nitrogen gas that is released as a result of denitrification process to be flushed out from the system and the residual ammonia nitrogen and/or dissolved organic substance in wastewater to be oxidized,
- at least one final settling tank (IX) which enables the treated water to be separated from the biomass,
- at least one treated water discharge (X) which is located at the outlet of the final settling tank (IX),
- at least one aeration/mixing unit (XI) which enables optimization of oxygen concentration in the aerobic tank (VIII) and the aerobic biofilm tank (V), and homogenous distribution of oxygen,
- at least one waste (excess) sludge discharge (XII) through which waste sludge produced in the system is removed,
- at least one return sludge line (XIII) which enables a part of the sludge settled in the system to return to the selector unit (III) and another part of it to return to the DeOx tank (VI).

In a preferred application of the invention, a media is used to reduce the volume of the aerobic biofilm tank (V) and the contact denitrification tank(VII). Additionally, complete mixing of the DeOx tank (VI) and the contact denitrification tank (VII) is carried out by the mixers.

Volume of the aerobic biofilm tank (V), in which nitrification is carried out, is selected depending on the specific surface area (m^2 surface/m^3 media volume) and unit nitrogen loading rate (gram N/ m^2/ day) of the filler.

In a preferred embodiment of the invention, a fluidized bed bioreactor and/or fixed bed bioreactor can be used in the aerobic bioreactor tank (V) wherein nitrification process is carried out.

In order to prevent the nitrogen (N_2) gas released as a result of the denitrification process taking place in the contact denitrification tank (VII) from hindering settling process, an aerobic tank (VIII) is incorporated to the system to remove N_2 gas from the system and to oxidize residual dissolved organic substance and ammonia nitrogen in wastewater.

In a preferred embodiment of the invention, the final settling tank (IX), which enables the treated water to be separated from the biomass, is also designed as membrane bioreactor (MBR) so as to remove all of the suspended solids (biomass) from wastewater. In this respect, high degree of removal efficiencies are obtained. Especially in the processes wherein water reclamation/recovery is planned, the MBR system can also be introduced to current configuration.

In a preferred application of the invention, an additional return sludge line (XIII) is connected from the aerobic tank (VIII) to the DeOx tank (VI) for denitrification of residual nitrogen which is transformed into oxidized nitrogen form.

A biofilm nitrification - contact denitrification method (100), which provides advanced level of nitrogen and phosphorous removal from waste waters, basically comprises the steps of

- mixing the organic substances in the wastewater with the biomass in the selector unit (III) (101),
- settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation (102),
- performing nitrification process in the aerobic biofilm tank (V) (103),
- reducing high dissolved oxygen concentration originating from the aerobic biofilm tank (V) in the DeOx tank (VI) (104),
- performing denitrification process in the contact denitrification tank (VII) (105),
- oxidizing residual dissolved organic substance and ammonia nitrogen in wastewater in the aerobic tank (VIII) (106),
- discharging the water treated in the final sedimentaion tank (IX) upon separating it from the biomass (107),
- disposal of waste sludge produced in the system through the waste sludge discharge (XII) and delivering another part of it to the return sludge line (XIII) (108).

Raw wastewater is mixed (III) with the flow coming from the return (activated) sludge line (XIII) in the selector unit. Thus the organic matter such as volatile fatty acid and fermentable organic matters in the wastewater are utilized for biological nitrogen (denitrification) and excess biological phosphorous removal.

The particulate organic substances in the wastewater which are mixed in the selector unit (III) with the sludge flow coming from the return sludge line (XIII) can be settled in the intermediate settling tank (IV) with high efficiency. Settling efficiency and retention time can (alternatively) be adjusted by the addition of chemicals. It is aimed that the settleable (separable) organic substances can be separated from the waste water as much as possible and can be retained in the intermediate settling tank (IV).

In a preferred embodiment of the invention, in the step of settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation (102), a coagulant is added when necessary in order to increase settling and phosphorous removal efficiency and rate.

After settling, the organic content and suspended solid substance concentration is low, however ammonia and phosphorous content is high; is obtained from the intermediate settling tank (IV). The wastewater, whose ammonia and phosphorous content is high, is directed to the aerobic biofilm tank (V) wherein mainly the oxidation of ammonia nitrogen (NH_4-N) to nitrite/nitrate nitrogen (NO_2-N, NO_3-N) that are oxidized nitrogen forms - which is known as nitrification process - is carried out. Thus, nitrification bacteria are enabled to reproduce on the media in the aerobic biofilm tank (V). In this tank (V), oxygen concentration can be adjusted by aeration/mixing unit (XI) and homogenous mixture of the reactor is enabled (if necessary, by using mixers). At the outlet of the aerobic biofilm tank (V), a wastewater flow having a high content of oxidized nitrogen forms (NO_2, NO_3) and phosphorous is obtained; thus it is possible to use a fraction of this stream that have high nutrient (N, P) contents as irrigation just after polishing step (after post filtration process, etc.).

A more effective nitrogen removal can be provided by denitrifying the oxidized nitrogen obtained from the aerobic biofilm tank (V) by using the settled biomass and organic substance coming from the intermediate settling tank (IV). Denitrification process is performed in the contact denitrification tank (VII).

Another preferred embodiment of the invention is applying stabilization and/or dewatering to the waste sludge produced in the system. According to the properties of the sludge, biogas can be obtained by installation of anaerobic sludge digestion process.

The biofilm nitrification – contact denitrification system and method of the present invention provide many advantages to the technologies used in the concerned field. First of all, the invention enables the wastewater treatment plant to fit into a smaller area. On the other hand, it provides advantage of space at urban areas where nitrification rate is low. Nitrate internal recirculation required for denitrification is also eliminated.

In the present invention, it is enabled that all of the separable and settleable organic substance is used in denitrification process. Fermentable dissolved organic substance can be used in denitrification and Enhanced Biological Phosphorus Removal (EBPR).

The biofilm nitrification – contact denitrification system of the present invention allows to control nitrification and denitrification processes separately and enables modification of full scale treatment processes that requires upgrade..

Organic carbon is required for heterotrophic microorganisms in denitrification process. In the present invention, by optimizing use of organic carbon required for denitrification, it is possible to obtain biogas from the excess organic carbon in the process via anaerobic sludge stabilization. Heat and electrical energy can be obtained by biogas utilization in gas engines.

In addition to all of these, increasing resistance of nitrification bacteria against low temperatures and toxic substances by means of biofilm processes is one of the major advantages of the invention.

CLAIMS

1. A biofilm nitrification - contact denitrification system, which provides advanced level of organic carbon, nitrogen and phosphorous removal from waste waters, basically **characterized by**
   - at least one waste water inlet (II) through which the crude wastewater is delivered to the system,
   - at least one selector unit (III) wherein the settleable organic substances in waste water are mixed with the biomass,
   - at least one intermediate settling tank (IV) wherein particulate organic substances are settled via biofloculation,
   - at least one aerobic biofilm tank (V) wherein nitrification process is carried out,
   - at least one deoxygenation tank (VI) which enables to reduce high dissolved oxygen concentration originating from the aerobic biofilm tank (V),
   - at least one contact denitrification tank (VII) wherein denitrification process is carried out,
   - at least one aerobic tank (VIII) which enables the nitrogen gas that is released as a result of denitrification process to be removed from the system and the residual ammonia nitrogen and/or dissolved organic substance in waste water to be oxidized,
   - at least one final settling tank (IX) which enables the treated water to be separated from the biomass,
   - at least one treated water discharge (X) which is located at the outlet of the final settling tank (IX),
   - at least one aeration/mixing unit (XI) which enables optimization of oxygen concentration in the aerobic tank (VIII) and the aerobic biofilm tank (V), and homogenous distribution of oxygen,
   - at least one waste sludge discharge (XII) through which waste sludge produced in the system is removed,

- at least one return sludge line (XIII) which enables a part of the sludge settled in the system to be recycled back to the selector unit (III), and another part of it to return back to the deoxygenation tank (VI).

2. A biofilm nitrification - contact denitrification system (I) according to Claim 1, **characterized in that** a media is used to reduce the volume of the aerobic biofilm tank (V) and the contact denitrification tank (VII).

3. A biofilm nitrification - contact denitrification system (I) according to Claim 1, **characterized in that** a fluidized bed bioreactor and/or fixed bed bioreactor is used in the aerobic bioreactor tank (V) wherein nitrification process is carried out.

4. A biofilm nitrification - contact denitrification system (I) according to Claim 1, **characterized in that** the final settling tank (IX), which enables the treated water to be separated from the biomass, is in the form of a membrane bioreactor (MBR) so as to achieve high efficiency.

5. A biofilm nitrification - contact denitrification system (I) according to Claim 1, **characterized in that** it comprises at least one return sludge line which is connected from the aerobic tank (VIII) to the deoxygenation tank (VI) for denitrification of residual nitrogen which is transformed into oxidized nitrogen form.

6. A biofilm nitrification - contact denitrification method (100), which provides advanced level of nitrogen and phosphorous removal from wastewaters and is implemented by the system defined in Claim 1, basically **characterized by** the steps of

- mixing the settleable organic substances in the waste water with the biomass in the selector unit (III) (101),

- settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation (102),
- performing nitrification process in the aerobic biofilm tank (V) (103),
- reducing the high dissolved oxygen concentration in the deoxygenation tank (VI) (104),
- performing denitrification process in the contact denitrification tank (VII) (105),
- oxidizing residual ammonia nitrogen and/or dissolved organic substance in waste water in the aerobic tank (VII) (106),
- discharging the water treated in the final settling tank (IX) upon separating it from the biomass (107),
- disposal of a waste sludge produced in the system through the waste sludge discharge (XII) and delivering a part of it to the recirculating sludge pipe (XIII) (108).

7. A biofilm nitrification - contact denitrification method (100) according to Claim 6, **characterized in that,** in the step of settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation (102), a coagulant is added in order to increase settling rate and efficiency.

8. A biofilm nitrification - contact denitrification method (100) according to Claim 6, **characterized in that** the sludge removed through the waste sludge discharge (XII) is sent to the stabilization and/or dewatering process.

FIGURE 1

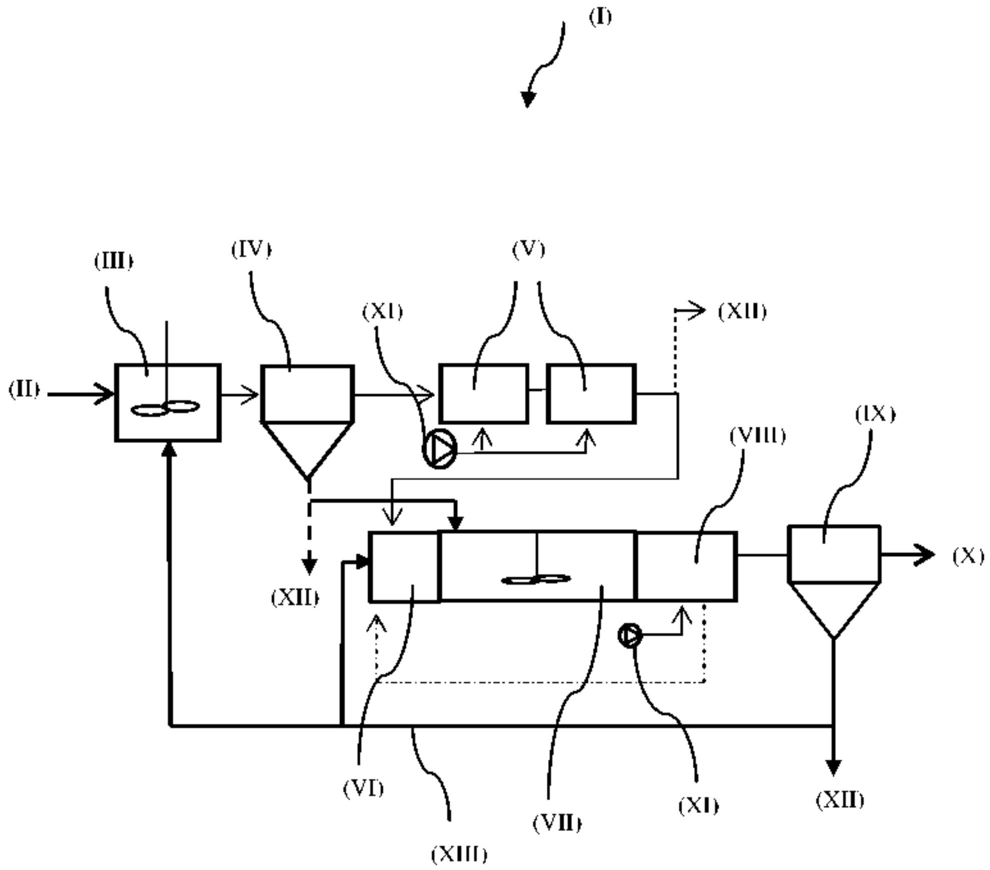
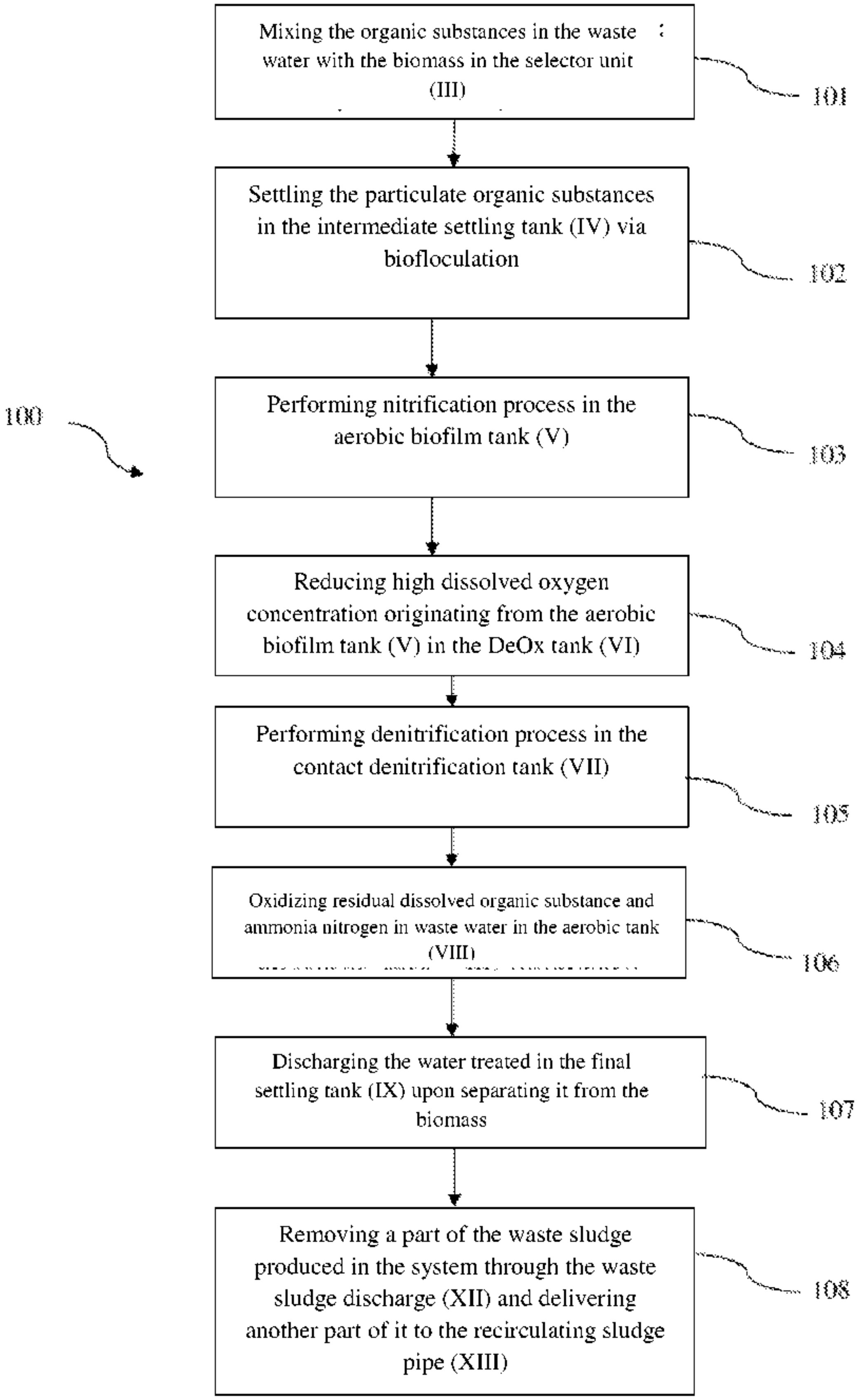


FIGURE 2


Mixing the organic substances in the waste water with the biomass in the selector unit (III)
101
Settling the particulate organic substances in the intermediate settling tank (IV) via biofloculation
102
100
Performing nitrification process in the aerobic biofilm tank (V)
103
Reducing high dissolved oxygen concentration originating from the aerobic biofilm tank (V) in the DeOx tank (VI)
104
Performing denitrification process in the contact denitrification tank (VII)
105
Oxidizing residual dissolved organic substance and ammonia nitrogen in waste water in the aerobic tank (VIII)
106
Discharging the water treated in the final settling tank (IX) upon separating it from the biomass
107
Removing a part of the waste sludge produced in the system through the waste sludge discharge (XII) and delivering another part of it to the recirculating sludge pipe (XIII)
108

INTERNATIONAL SEARCH REPORT

International application No
PCT/TR2015/050300

A. CLASSIFICATION OF SUBJECT MATTER
INV. C02F3/30
ADD. C02F1/20 C02F101/10 C02F101/16 C02F101/30 C02F3/08
C02F3/06 C02F3/12

According to International Patent Classification (IPC) or to both national classification and IPC

B. FIELDS SEARCHED

Minimum documentation searched (classification system followed by classification symbols)
C02F

Documentation searched other than minimum documentation to the extent that such documents are included in the fields searched

Electronic data base consulted during the international search (name of data base and, where practicable, search terms used)

EPO-Internal, WPI Data

C. DOCUMENTS CONSIDERED TO BE RELEVANT

Category*	Citation of document, with indication, where appropriate, of the relevant passages	Relevant to claim No.
X Y	KR 100 364 622 B1 (HALLA ENGINEERING & IND DEV CO [KR]) 16 December 2002 (2002-12-16) paragraphs [0015], [0025], [0033] - [0066]; figures 1-4	1,6,7 2-5,8
X	KR 100 901 484 B1 (ENBIO21 CO LTD [KR]) 8 June 2009 (2009-06-08) paragraphs [0015] - [0037]; figure 1	1,6
X	CN 203 333 449 U (UNIV ZHONGKAI AGRI & ENG) 11 December 2013 (2013-12-11) paragraphs [0003] - [0015]; figure 1	1,6
Y	DE 44 09 435 A1 (OTT PETER [DE]; KOCH REINHARD [DE]; PEUKERT VOLKMAR DR [DE]) 8 September 1994 (1994-09-08) claim 1; figure 1	2

-/--

[X] Further documents are listed in the continuation of Box C.

[X] See patent family annex.

* Special categories of cited documents :

"A" document defining the general state of the art which is not considered to be of particular relevance

"E" earlier application or patent but published on or after the international filing date

"L" document which may throw doubts on priority claim(s) or which is cited to establish the publication date of another citation or other special reason (as specified)

"O" document referring to an oral disclosure, use, exhibition or other means

"P" document published prior to the international filing date but later than the priority date claimed

"T" later document published after the international filing date or priority date and not in conflict with the application but cited to understand the principle or theory underlying the invention

"X" document of particular relevance; the claimed invention cannot be considered novel or cannot be considered to involve an inventive step when the document is taken alone

"Y" document of particular relevance; the claimed invention cannot be considered to involve an inventive step when the document is combined with one or more other such documents, such combination being obvious to a person skilled in the art

"&" document member of the same patent family

Date of the actual completion of the international search	Date of mailing of the international search report
7 March 2016	04/04/2016

Name and mailing address of the ISA/	Authorized officer
European Patent Office, P.B. 5818 Patentlaan 2 NL - 2280 HV Rijswijk Tel. (+31-70) 340-2040, Fax: (+31-70) 340-3016	Fiocchi, Nicola

1

INTERNATIONAL SEARCH REPORT

International application No
PCT/TR2015/050300

C(Continuation). DOCUMENTS CONSIDERED TO BE RELEVANT

Category*	Citation of document, with indication, where appropriate, of the relevant passages	Relevant to claim No.
Y	US 4 431 543 A (MATSUO YOSHITAKA [JP] ET AL) 14 February 1984 (1984-02-14) column 10, line 26 - line 55; figure 2	3
Y	US 2007/235386 A1 (BARNES DENNIS J [US]) 11 October 2007 (2007-10-11) paragraphs [0005] - [0018]; figure 2	4
Y	US 2014/116957 A1 (WOO KWANG JAE [KR] ET AL) 1 May 2014 (2014-05-01) paragraphs [0053], [0089]; figure 5	5
Y	US 2013/213883 A1 (JOSSE JUAN CARLOS [US] ET AL) 22 August 2013 (2013-08-22) paragraph [0029]; figure 1	8
A	KR 100 640 940 B1 (ENBIO21 CO LTD [KR]) 6 November 2006 (2006-11-06) the whole document	1-8
A	EP 0 346 013 A1 (ORANGE WATER & SEWER [US]) 13 December 1989 (1989-12-13) the whole document	1-8
A	DE 36 19 229 A1 (SEKOULOV IVAN PROF DR ING [DE]) 10 December 1987 (1987-12-10) the whole document	1-8

1

INTERNATIONAL SEARCH REPORT

Information on patent family members

International application No
PCT/TR2015/050300

Patent document cited in search report	Publication date	Patent family member(s)	Publication date
KR 100364622 B1	16-12-2002	NONE	
KR 100901484 B1	08-06-2009	NONE	
CN 203333449 U	11-12-2013	NONE	
DE 4409435 A1	08-09-1994	AT 164821 T	15-04-1998
		CN 1143942 A	26-02-1997
		CZ 9602709 A3	12-03-1997
		DE 4409435 A1	08-09-1994
		DK 0751913 T3	18-01-1999
		EP 0751913 A1	08-01-1997
		ES 2117389 T3	01-08-1998
		PL 316524 A1	20-01-1997
		RU 2136613 C1	10-09-1999
		WO 9525697 A1	28-09-1995
US 4431543 A	14-02-1984	FR 2503690 A1	15-10-1982
		JP S5839599 B2	31-08-1983
		JP S57171495 A	22-10-1982
		US 4431543 A	14-02-1984
		ZA 8202462 A	30-03-1983
US 2007235386 A1	11-10-2007	NONE	
US 2014116957 A1	01-05-2014	CN 103502157 A	08-01-2014
		US 2014116957 A1	01-05-2014
		WO 2012176974 A1	27-12-2012
US 2013213883 A1	22-08-2013	AU 2011288881 A1	28-02-2013
		CA 2807881 A1	16-02-2012
		EP 2603464 A1	19-06-2013
		US 2013213883 A1	22-08-2013
		US 2016023935 A1	28-01-2016
		WO 2012019310 A1	16-02-2012
KR 100640940 B1	06-11-2006	NONE	
EP 0346013 A1	13-12-1989	DE 68904598 D1	11-03-1993
		DE 68904598 T2	19-05-1993
		EP 0346013 A1	13-12-1989
		ES 2031810 T1	01-01-1993
		GR 3007279 T3	30-07-1993
		JP H0286895 A	27-03-1990
		US 4874519 A	17-10-1989
DE 3619229 A1	10-12-1987	NONE	

APPENDIX B : MATRIX DETAIL USED IN THE MODELING STUDY

Table B.1 The matrix used to determine COD fractions

Component, i →	1	2	3	4	5	6	7	8	9	Process Rate
Process, j ↓	S_I	S_S	X_I	X_{S1}	X_{S2}	X_H	X_A	X_P	S_O	$ML^{-3}T^{-1}$
Growth		$-\frac{1}{Y_H}$				1			$-\frac{(1-Y_H)}{Y_H}$	$\hat{\mu}_H \cdot \frac{S_S}{K_S + S_S} \cdot \frac{S_O}{K_{O,H} + S_O} \cdot X_H$
Rapid Hydrolysis, X_{S1}		1		-1						$k_{h1} \frac{X_{s1}/X_H}{K_{XS} + X_{s1}/X_H} \cdot \frac{S_O}{K_{O,H} + S_O} \cdot X_H$
Slow Hydrolysis, X_{S2}		1			-1					$k_{h2} \frac{X_{s2}/X_H}{K_{XX} + X_{s2}/X_H} \cdot \frac{S_O}{K_{O,H} + S_O} \cdot X_H$
Endogenous Resbiration				1- f_P	1- f_P	-1		f_P		$b_H \cdot X_H$
Parameter, ML^{-3}	COD	COD	COD	COD	COD	Cell COD	Cell COD	COD	O_2	