# Performance Evaluation and Model-Based Optimization of Membrane Bioreactors: The Case of Addis Ababa Package Treatment Plant

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# Performance Evaluation and Model-Based Optimization of Membrane Bioreactors: The Case of Addis Ababa Package Treatment Plant

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A thesis submitted to the School of Graduate Studies of Addis Ababa University in Partial fulfillment of the Degree of Master of Science in Water Supply and Environmental Engineering.

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# Certification

The undersigned certifies that he has read the thesis entitled**: Performance Evaluation and Model-Based Optimization of Membrane Bioreactors: The Case of Addis Ababa Package Treatment Plant** and hereby recommend for acceptance by the Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science.

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**Dr Agizew Nigussie** (Advisor)

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Date

# *ABSTRACT*

*Addis Ababa Water and Sewerage Authority invested 750 million ETB for installing membrane bioreactor units in the city. This new technology is expected to double the wastewater treatment capacity of the city. Evaluating the performance of the units is necessary in estimating the likelihood of meeting this expectation and predicting the sustainability of the treatment plants. This research explores performance of the installed plants and suggests optimal operational strategies to enhance the denitrification capacity of the plant while minimizing energy cost. Measures of performance were effluent quality and membrane filtration processes. In order to evaluate the effluent quality, samples were collected both in dry and wet seasons. The MBR module was evaluated using recorded data of Transmembrane Pressure (TMP), flux & permeability of five-month operation. Activated sludge model no1 based simulation was used to determine the optimal operation strategy. To ensure energy reduction while improving the treated water quality; the suitability of intermittent aeration was explored using the developed model. The average COD removal efficiency was 97%. The ammonium effluent concentration was always lower than 0.5 mg/L. However, absence of anoxic tank resulted in poor denitrification with consequently increase nitrate effluent concentration over 100 mg/l. The treated wastewater was found to be completely clear of fecal coliform; making the treated water suitable for reuse. The MBR filtration showed stable but low permeability condition. The average operational TMP was less than 20mbar. However, flux and permeability plot showed the plant was operating with decreasing trend over time and maximum of 80LMH/bar. This suggests the need for frequent cleaning or construction of a primary tank. Results from the ASM1 simulation indicated that intermittent aeration with aerobic-anoxic cycle of 43/54 min was the optimum duration. The proposed operational strategy increased the total nitrogen removal efficiency from 42% to 71% without affecting COD removal. These results indicate that creating optimal aerobic/anoxic conditions within the existing reactor is the most competitive solution to upgrade the MBR treatment plant.*

***Keywords****: Membrane BioReactor, Activated sludge model, AQUASIM, Cyclic aeration, Response surface optimization*

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# Abbreviations and Symbols

| | |
|---|---|
| AAWSA | Addis Ababa Water and Sewerage Authority |
| AF | Aerated Fraction |
| ANOVA | Analysis of Variance |
| ASM | Activated Sludge Model |
| BOD | Biochemical Oxygen Demand |
| CAPEX | Capital Expenditure |
| CAS | Conventional Activated Sludge |
| CCD | Central Composite Design |
| CIP | Cleaning-In-Place |
| COD | Chemical Oxygen Demand |
| COP | Cleaning-Out-of-Place |
| CTR | Cycle Time Ratio |
| DO | Dissolved Oxygen |
| EPA | Environmental Protection Agency |
| HRT | Hydraulic Retention Time |
| IWA | International Water Association |
| LMH | Liters per $m^2$ per hour |
| MBR | Membrane Biological Reactor |
| MLR | Mixed Liquor Recirculation |
| MLSS | Mixed Liquor Suspended Solids |
| MLVSS | Mixed Liquor Volatile Suspended Solids |
| $NH_4$-N | Ammonia nitrogen |
| $NO_3$-N | Nitrate nitrogen |
| OUR | Oxygen Utilization Rate |
| PFD | Process Flow Diagram |

| | |
|---|---|
| QA/QC | Quality assurance and Quality control |
| RSF | Relative Sensitive Function |
| RSM | Response surface method |
| $S_I$ | Non-biodegradable soluble COD |
| SCA | Scenario Analysis |
| SEE | Standard Error Estimate |
| SMU | Submerged Membrane Unit |
| SRT | Sludge Retention Time |
| $S_S$ | Readily biodegradable COD |
| STOWA | Dutch Foundation for Applied Water Research |
| TKN-N | Total Kjehldahl Nitrogen |
| TMP | Trans-Membrane Pressure |
| TN | Total Nitrogen |
| TP | Total Phosphorous |
| TSS | Total Suspended Solids |
| WWTP | Wastewater Treatment Plant |
| $X_I$ | Particulate inert COD |
| $X_S$ | Slowly biodegradable COD |
| $Y_{NH4-N}$ | Response Function of ammonia-nitrogen |
| $Y_{NO3-N}$ | Response Function of nitrate-nitrogen |

# 1 Introduction

## 1.1 Background

Addis Ababa, the capital city of Ethiopia, despite current development in infrastructure still lacks the basic sanitation services. Addis Ababa Water and Sewerage Authority (AAWSA) still find it hard to meet the demand of improved sanitation from the increasing population. The increasing rate of development of housing projects, public and commercial facilities make the problem even worse.

Obligated by the rapid development of the city housing projects Addis Ababa Water and Sewerage Authority (AAWSA) is deploying containerized Membrane Bioreactors as packages treatment plants. The project is being constructed under a project titled "Design, Supply and Build MBR Package Wastewater Treatment Plant including all the required civil works to treat domestic sewage with a Daily Treatment Capacity". These decentralized plants are meant to serve the immediate needs of Kilinto, BoleBulbula, Karakore I, Deginet and Mekenisa Kotari condominium sites as part of Lot1 and Tulu Dimtu, Bole Arrabsa, Genet Menafesha and Oromiya condominiums as part of Lot 2.The MBR units add 20,000 cubic meter of treatment capacity to the existing 7,500 cubic meter.

Membrane Bioreactor (MBR) is a containerized system that comprises biological and filtration units. It refines waste using membrane filtration rather than gravity separation. MBR process is an emerging advanced wastewater treatment technology that has been successfully applied at an ever increasing number of locations around the world (Judd, 2011).

Membrane Bioreactor (MBR) is a great technology; it has more removal efficiency and takes only small foot prints. However, it is a sophisticated system and requires trained operator (Hai et.al, 2014).Unlike conventional activated sludge treatment plant, MBR plant operator has to deal with both microbial parameters and operational parameters involved in membrane separation. As its efficiency highly depend on the membrane permeability, fouling and clogging problems should be monitored regularly. These setbacks have been resolved in other setups by focusing on operational strategies of the plant and regular evaluation after construction. For these reasons, this study focused on assessing the performance of Mekenisa Kotari Treatment Plant and evaluating optimum operational parameters by using mathematical models.

## 1.2 Statement of the problem

Even though installations of membrane bioreactor as decentralized treatment plants is a big step forward for a country like Ethiopia, two major things went unnoticed. The first is installing the MBR units without assessing the performance of the already existing treatment plants. The failure to regularly asses the performance restricts the authority from knowing the relevance of the expensive treatment plants for site specific problems. The second challenge is poor denitrification capacity of most of the installed plants due to absence of anoxic tank in the MBR Treatment Plant configuration.

MBR technology, despite having high effluent quality, its market is expensive. It is advised to be used where foot print limitation exists. With current commercial solutions, MBR systems are not cost-effective for most decentralized or semi-central applications (Lesjean et.al, 2011). AAWSA is spending millions on the MBR plants expecting them to double the wastewater treatment capacity of the city. According to Addis Ababa Sanitation Improvement Master Plan, further decentralized units are expected to be installed for area with topographic problems. Hence, much is expected from these treatment plants and evaluating the performance of the treatment units is very crucial to know whether the expectations are met. Besides all these expectations neither the city Environmental Protection Agency (EPA) nor AAWSA is doing regular monitoring of the installed plants. Performance monitoring of the plants help to analyze the sustainability of decentralized wastewater treatment system which is going to be adopted further at different location of the city.

All Addis Ababa MBR plants are not designed to remove nitrate nitrogen. All tanks available in the treatment plants are aerobic and due to this, the denitrification capacity of the plants is poor. The MBR treatment plants were not designed to fulfill denitrification process because at design stage it was planned to reuse the treated water for gardening. But what is being practiced currently at all of the MBR treatment plants is different from the plan. The reclaimed water is directly joining the river with no reuse. Since disposing nitrate nitrogen to water bodies causes eutrophication, process upgrade should be considered. The upgrade could be done either by adding anoxic tank or by working on operational strategies of the plant. For the already installed treatment plants the limited land availability and cost related with anoxic tank construction make this first option obsolete. The second alternative is more suitable for Addis Ababa MBR plants. Hence, the sophisticated biological process needs as much attention as the design and installation of the membrane bioreactor in order to improve nitrogen removal of the plant.

Membrane bioreactor is a biological treatment process and due to the existence of so many variables (including the quality of the wastewater entering, the type and mass of the microorganism and their growth and decay rates) operation of the treatment plant needs well understanding of the biological process. The novelty of the technology to Ethiopia enhances the challenges associated with it. Prior to the installation of MBR, Addis Ababa didn't even have the conventional activated sludge treatment plant and hence limited skilled manpower is available. Due to lack of skilled operators, effluent from the expensive technology has more than the permissible concentration level for some parameters. Most of the MBRs do not have operation strategy that could improve nitrogen removal efficiency without increasing operation costs. As operational costs is the main obstacles of Membrane bioreactor technology, taking cost into consideration is crucial (Comas et al., 2015). The operating cost of MBR is higher than those of conventional wastewater treatment plant. The investment of MBR system is about 1.5 times higher than that of the conventional process and the operating cost is almost 2 times higher (Yang et. al, 2015). Hence for the MBRs to operate effectively a plant strategy that reduces operational costs and reject disturbances is necessary (Busch, 2008). That is, in order to run a plant economically operation costs such as aeration energy should be minimized. At the same time, the discharge concentrations should be kept at the permissible levels. Minimizing the operational costs and still generating quality effluent may lead to conflicting goals. In other words, a proper operation in a membrane bioreactor plant can be translated into a constrained optimization problem.

## 1.3 Research Questions

- Are the membrane bioreactors installed as decentralized wastewater treatment plant by Addis Ababa Water and Sewerage Authority (AAWSA) performing up to the expectations? If not what are the recommendations?
- Is it possible to minimize energy utilization of the MBRs and still enhance denitrification capacity of the plant?

## 1.4 Objective

### 1.4.1 General objective

The aim of this research was to assess the current performance and predict the sustainability of the membrane bioreactor technology for Addis Ababa.

### 1.4.2 Specific objective

In line with the general objective, the specific objectives were to

- Determine the characteristics of untreated and treated wastewater
- Evaluate the performance and identify performance limiting factors of the treatment plant
- Recommend optimal operational strategy that minimize operation expenditure while meeting effluent quality requirement

## 1.5 Limitation of the study

Due to resource and time limitations, the study had focused on water quality parameters and membrane evaluation matrices for assessing the performance of the wastewater treatment plant. Performance evaluation of preliminary and sludge treatment were not included in the study.

## 1.6 Outline of the thesis

This thesis has been structured into five chapters and supporting appendices. The first chapter is intended to provide an introduction to MBR technology in Addis Ababa and help to give insight about the operation problem related with the technology. It also shows the aim of the research.

The second chapter is a review of literature about the MBR technology and wastewater modeling tools available. Since mathematical models were applied for optimization of the treatment process, an extensive literature is presented about activated sludge models (ASM) in this chapter.

Chapter 3 describes the methodology applied for sampling analysis, performance indicators, the model development, calibration, sensitivity analysis and investigation of the operational parameters. Chapter 4 presents the results and discusses the results from the three main output; the sampling process, the performance indicators parameter estimation step and the modeling and optimization step. Chapter 5 provides the conclusion and recommendation based on the result found.

# 2 Literature review

## 2.1 Decentralized wastewater treatment system

Decentralized wastewater treatment system are defined as the collection, treatment, and reuse of wastewater at or near the point of waste generation which are most commonly used where installation of a centralized sewer system is not technically, politically, environmentally, or economically feasible (Tchobanoglous et al. 2003). Apart from avoidance of the centralized sewer lines, there are several motivations for adapting decentralized treatment systems. Decentralized systems enhance nutrient recovery and reclamation of water. Decentralized wastewater treatment involves using a combination of treatment technology options both traditional and innovative. There are number of treatment modes that are presently used in developing countries for decentralized wastewater treatment. Membrane bioreactor is one of these technologies.

## 2.2 Membrane bioreactor

### 2.2.1 Principle

Biological treatment for municipal wastewater are designed to transform dissolved and particulate constituents into acceptable end products, capture suspended and colloidal solids into biofilm, remove nutrients and sometimes remove trace organic compounds. This is done by using microorganisms metabolize the organic matter into gases and cell tissues. (Tchobanoglous et al. 2003)

Conventional activated sludge process is one of the most used biological wastewater treatments. The working principle of activated sludge process is keeping the microorganisms, which are responsible for stabilization of organic content of the wastewater under aerobic condition, in suspension by mechanical means or thorough mixing by aeration. The activated sludge process is capable of removing organic electron donors collectively called the BOD and ammonia (Tchobanoglous et al., 2003). The removal is performed mainly by the degradation of soluble matter. Activated sludge process has five basic components to perform this activity. These are the primary sedimentation tank, the aerator reactor, aeration mechanism, the clarifier and the mechanisms to collect the solids either to recycle as returned activated sludge or remove them form the process as waste activated sludge. Figure 2-1shows a typical schematic diagram of conventional activated sludge process. The primary sedimentation tanks serves as a suspended solid remover. They are usually designed to achieve 50% to 70% removal of suspended solids and 25% to 40% removal of BOD. The BOD removed is associated with the organic fraction of the suspended solids. The aerator mechanism in the aerator reactor is a source of oxygen for the suspended microorganisms as the organic material is changed to $CO_2$, $H_2O$ and cell mass. The cell mass formed is separated by gravity settling process in a clarifier. Part of the separated sludge is recycled back to the aerator reactor in order to augment the microorganisms' growth and therefore the degradation of the organic substances.

A typical MBR has a conventional activated sludge process together with membrane separation to

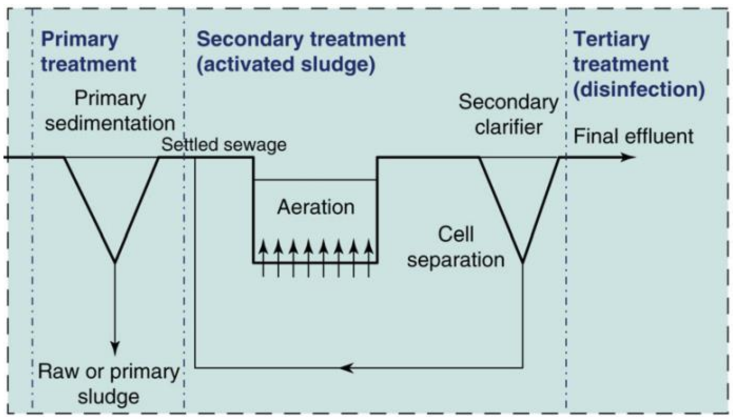


*Figure 2-1 Schematic diagram of conventional activated sludge process, (Judd, 2011)*

separate the biomass from the mixed liquor. The biological process in the reactor is similar to the conventional activated sludge process but the gravity separation process is substituted by the physical barrier separation mechanism, which is the membrane filtration process.

Membranes technology was used initially for the desalination of brackish water. It was in the 1970s that MBRs were used on wastewater experimentally. Before the 1990s, most of the installed MBRs were used for industrial water treatment. With the introduction of submerged MBR configuration, the application of MBRs for municipal wastewater treatment increased (Zaerpour et.al, 2014). The MBR market is currently experiencing accelerated growth. The global market has reached $838.2 million in 2011. The compound annual growth rate from 2011 to 2018 is also estimated to be 15.2%. Even though the market for MBR is not as high as the global market, in the last three years it witnessed rapid growth (Hai et.al, 2014).

Currently MBRs are being used where very high quality effluent is required for discharge, footprint area limitations exist and land area is very expensive and where operation and maintenance requirements can be reliably accomplished. The reclaimed effluent water form MBRs' is being used for toilet flushing, turf irrigation, cooling, or ornamental fountains.

The membrane units used in MBRs typically rely on ultrafiltration and microfiltration to separate constituents that are about 0.04 μm or larger. The membranes used are either hollow fiber, tubular or spiral wound. The hollow fiber membrane module is the most used type of all where a bundle of hollow fibers is placed inside a pressure vessel. In a tubular module, the membrane manufactured as flat sheet is cast on the inside of a support tube. Bundles of these tubes are placed in a pressure vessel. A spiral wound module consists of flat membrane sheets separated by flexible spacers, rolled into a circle, and placed in a pressure vessel. Membranes can also be pressure driven or vacuum driven (Siegrist, 2015). Early membranes used pressurized filtration with permeate produced by a flow path of inside-to-outside. These membranes had high-energy use and required frequent back washing and chemical cleaning. However, nowadays membranes use a low vacuum and permeate is produced by a flow path of outside-to-inside. These membranes have more membrane surface area, lower energy use, and cleaning is done more easily.

In addition to the biomass separation and retention, the membrane units also remove molecular and colloidal constituents and most pathogenic microorganisms. For these reasons, MBRs normally produce tertiary quality effluents with disinfection if they are correctly designed and properly operated and maintained.

Membrane bioreactors can have different configuration. Each configuration differentiated with its own distinctive features. The common configuration types are external pressure-driven (side stream) sMBR and integrated submerged (immersed) iMBR. Side stream membranes are installed external to the bioreactor and the mixed liquor from the bioreactor is pumped to the membranes. The separation of water from the sludge is driven by pressure. In order to balance the biomass concentration, the separated sludge is recycled back into the bioreactor. Side stream MBRs are manufactured commonly in a tubular configuration. To maintain permeability and improve performance, a pre-treatment device such as a fine screen or a cloth-media filter is installed ahead of the membrane unit.(Metcalf & Eddy, 2007)

Immersed membranes bioreactors are submerged either directly into the activated sludge reactor or in a separate membrane separation tank. Since they use vacuum to draw water (permeate) through the membrane while retaining solids in the membrane separation tank, the membranes are constantly subjected to a vacuum, which is usually less than 50 kPa (Metcalf & Eddy, 2007). In immersed membrane bioreactor, a manifold at the bottom of the reactor provides oxygen constantly. This ensures aerobic condition in the reactor and in addition, it helps to clean the exterior of the membranes. As the air bubbles rise to the surface, scouring of the membrane surface occurs and rejected material is returned to the mixed liquor. Internal cleaning of the membrane is done by an automation program by switching the MBR process from filtration mode to clean-in-place mode. Cleaning is done by using chemical solutions, such as citric acid and sodium hypochlorite (1%) solution, which can remove the organic and inorganic fouling.

From the above mentioned MBR configuration types, the submerged configuration is more commonly used than the recirculated configuration. This is due to less pumping energy requirement of the former configuration type.

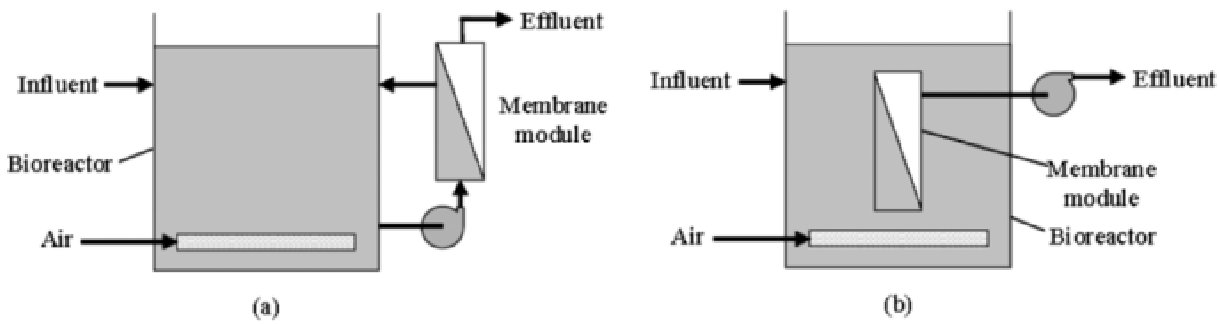


*Figure 2-2 Schematic diagrams of basic membrane bioreactor configurations (a) MBR with external membrane module and (b) MBR with immersed membrane module, (Hai et.al, 2014)*

### 2.2.2 Advantage and Disadvantage

MBR technology has many advantages over the conventional activated sludge process. High treated wastewater quality is one of the most important advantages. Since sludge water separation is done by membrane filtration rather than sedimentation, solids and colloids are eliminated. The membrane has an effective pore size of less than 0.1mm, which is significantly smaller than the pathogenic bacteria and viruses in the sludge.  For these reasons, the effluent is considerably disinfected. This makes the reuse application of the effluent from MBR for different purposes possible. (Visvanathan et al. 2000)

The membrane separation enhances the biomass concentration inside the bioreactor. Due to this independence of solid separation on the MLSS concentration and characteristics, MBRs usually operate at higher mixed liquor suspended solid (MLSS) concentration. This enables MBR to have small reactor size and increased bio treatment efficiency. The smaller footprint advantage is attributed also by the MBR ability to displace three or four secondary and tertiary individual process. The bio treatment efficiency increases as the biomass retained in the reactor for a longer time and nitrogen bacteria are developed. This enables the removal of dissolved solids such as ammonia by nitrification. (Judd, 2011)

The ability of MBR to work at higher mixed liquor suspended solid (MLSS) not only decreases the reactor size but also the sludge production rate. Since the biomass is retained in the bioreactor the sludge retention time (SRT) increases which assists low solid production (Judd, 2008).

The application of membrane separation instead of gravity separation in MBR technology has additional advantages. In the conventional activated sludge, process the sludge retention time (SRT) and hydraulic retention time (HRT) are hard to separate for one is dependent on the other. The solid particle in a mixed liquor should have sufficient size (>50μm) to be separated by sedimentation. This requires a minimum hydraulic retention time for growth. Due to this, independent control of sludge retention time (SRT) is hard in conventional activated sludge process. Nevertheless, in MBR the solid particles need only be larger than the membrane pore size. Thus in this process the sludge retention time (SRT) and hydraulic retention time (HRT) can be completely separated (Judd, 2008). This helps to have optimum control over the biomass and to propose different operational strategies. (Skouteris, 2010)

Since MBRs process equipment is tightly closed within the system, odor dispersion is seldom a problem in MBR (Visvanathan et al. 2000). This makes MBR an ideal decentralized wastewater treatment plant for a plant located close to the community.

Besides all the above-mentioned advantages, MBR technology has its own drawbacks. MBR technology is associated with high capital equipment and operation costs. The technology requires far more capital cost than the conventional activated sludge process(Judd, 2008). However, a conventional activated sludge process with tertiary treatment has reported to have a higher CAPEX than an MBR achieving comparable effluent quality (Figueras, 2014). In addition, the capital cost of membranes is significantly decreasing in the past years. These reasons make the operating costs, which help to ensure the cleanliness of membrane, the more significant contributor of the elevated cost. The aeration energy used to avoid membrane fouling equals 30-50% of the energy consumption of the conventional activated sludge process.(Krzeminski et al. 2012)

In order to avoid clogging of the membrane and flow channels, the raw sewage should be rigorously treated. The screens prior to the MBR should at least limit the entry of large particles (1-3mm) which is more conservative than the conventional activated sludge process. (Judd, 2008)

## 2.3 Mathematical models of biological wastewater treatment

Application of mathematical models for simulating biological process has become an effective tool for design, control and operation of wastewater treatment plants. It helps to understand the dynamics of treatment process, relation between them and to optimize operational parameters. The effects of changes in operational strategies on process performance can also be analyzed using these models. Modeling can help to minimize the gap between lab-scale experiments and full-scale applications, and in turn is considered as a time and cost-saving tool for the optimization of treatment strategies. (Soliman et al, 2015)Activated sludge models (ASM) are one of the most important models that are used for the modeling of the biological processes occurring under domestic wastewater treatment. Activated Sludge Models (ASM1, ASM2-ASM2, and ASM3) are proposed by a task group first of the IAWPRC (International Association on Water Pollution Research and Control), then called IAWQ (International Association on Water, and finally IWA (International Water Association). In the following sub-section, activated sludge models are described in detail.

### 2.3.1 Activated sludge models: state of the art

The ASM development starts no long before 1970, but it was very limited by computer capacities and by the complexity of the written model presentation. It was not until 1982 that this changed, when the IAWPRC (International Association on Water Pollution Research and Control) established a task group on mathematical modeling for design and operation of activated sludge processes (Henze et al, 2000). The first result of the task group was ASM1 also known as IAWPRC model or IAWQ model. In the subsequent years, the group developed other models as ASM2 (that includes phosphorus removal) and ASM3 (that include internal storage). Together with the proposed models, guidelines for wastewater characterization and calibration of the model were also presented. This set a strong platform for using the models even until these days. In addition, the rapid development of computers models helped ASM to be used extensively and the understanding of the activated sludge processes advanced well.

The simple matrix representation used when presenting ASM helped to have a common language between researchers. The matrix format, which is named the Gujer matrix or Petersen matrix, contains a stoichiometric matrix and a kinetic vector. This representation is very convenient, as it gathers complex models into a condensed form (Hauduc et al., 2013). A simple example used in literature to illustrate the matrix notation is considering heterotrophic bacteria in aerobic environment. Since the bacteria is utilizing a soluble organic matter two fundamental processes occur: the biomass increases by cell growth and decreases by decay. The Petersen matrix shown in Table 2-1 helps to follow up the interaction of components in these processes. The first step in setting up the matrix is identifying the components to be considered. As in Table 2-1 the components: biomass, substrate and oxygen are listed at the first column and the processes are at the left most column. The process rates of each components are at the last column; for this situation, the Monod equation is used as a process rate. The figures within the table are stoichiometric coefficients, which set out the mass relationship between the components. For the first process, growth of biomass, (+1) biomass is produced using (-1/Y) substrate and (-(1-Y)/Y) of oxygen. During decay substrate is not part of the process but biomass is depleted using oxygen.

*Table 2-1 Simplified representation of Petersen matrix*

| **Component** / **Process** | $X_b$ | $S_O$ | $S_S$ | Process rate (gCOD/(L.d)) |
|---|---|---|---|---|
| **Growth** | $1$ | $-\frac{1-Y}{Y}$ | $-\frac{1}{Y}$ | $\mu\left(\frac{S_S}{K_S+S_S}\right)X_b$ |
| **Decay** | $-1$ | $-1$ | | $bX_b$ |
| Stoichiometric Parameter:<br>Y: biomass yield | **Biomass gCOD/L** | **Oxygen gCOD/L** | **Substrate gCOD/L** | Kinetic Parameter:<br>μ: specific growth rate<br>$K_S$: substrate saturation constant<br>b: decay coefficient |

With the matrix notation, it is easy to get the reaction rate of each components. Summation of the product of stoichiometric coefficients and process rate of each process gives the net reaction rate of a component. For instance, the reaction rate of biomass which depend on both the growth and decay rate is

$$\frac{dX_b}{dt} = (+1) * \left(\mu\left(\frac{S_S}{K_S + S_S}\right) X_b\right) + (-1) * (bX_b)$$
$$= \left(\mu\left(\frac{S_S}{K_S + S_S}\right) X_b\right) - (bX_b)$$

Likewise, the oxygen depletion rate is

$$\frac{dS_o}{dt} = \left(-\frac{1 - Y}{Y}\right)\left(\mu\left(\frac{S_S}{K_S + S_S}\right) X_b\right) - (bX_b)$$

In addition, the substrate conversion rate is

$$\frac{dS_S}{dt} = \left(-\frac{1}{Y}\right)\left(\mu\left(\frac{S_S}{K_S + S_S}\right) X_b\right)$$

In order to describe the accumulation rate and concentration of particular components within a system it is common to use the mass balance expression (Equation 2.1)

$$Acuumulation = Input - Output + Reaction \tag{2.1}$$

The rate of the components from the Petersen matrix is used to identify the reaction rates. Solving the mass balance give the output concentration of interest.

### 2.3.2 Activated sludge model 1 (ASM1)

#### *2.3.2.1 Description of ASM1*

Activated sludge model 1 is a mathematical model that helps to describe carbon oxidation, nitrification and denitrification process realistically. Within this model, the IWA task group modelers managed to incorporate processes that are not only essential to mimic the major events occurring in reality, but also that could be solved with reasonable degree of efforts. According to the task group, the model besides predicting concentration of effluent, it helps to predict solid concentration of the activated sludge and electron acceptor requirements. Since these two things vary significantly from plant to plant, a good evaluation is important for plant operation and optimization (Henze et al., 2000).

In ASM1 thirteen components and eight processes are included, resulting in eight rate expressions. Seven components correspond to the organic matter measured as COD and four others correspond to nitrogen compounds. Chemical Oxygen demand was chosen as the proper measurement unit for describing those model components incorporated with organic matter. The total COD is partitioned according to biodegradability (readily biodegradable, slowly biodegradable, and non-biodegradable) and physical state (soluble and particulate). The nomenclature assigned by the task group for the components are S for soluble components, and X for particulate. Biodegradable components are subscribed s and non-biodegradable are subscribed i. The thirteen components are described below.

The carbon material in ASM1 is divided into biodegradable COD, non-biodegradable COD (inert material) and biomass. Biodegradable carbon is divided into two fractions: readily biodegradable, Ss, assumed as if it were all soluble and slowly biodegradable, Xs, assumed as if it were all particulate. All readily biodegradable carbon are assumed to be simple soluble molecules that can be readily absorbed by the organisms and metabolized for energy and synthesis, whereas the slowly biodegradable substrate is assumed to be made up of particulate/colloidal/complex organic molecules that require enzymatic breakdown prior to absorption and utilization. Even if the reality might deviate slightly from this, the assumptions help in predicting in the electron acceptor requirement.

The non-biodegradable COD is divided into soluble ($S_I$) and particulate ($X_I$) material. Both are considered to be unaffected by the biological action in the system. The inert soluble material leaves the system with the treated wastewater, whereas the inert particulate material accumulates as inert solids until removed with excess sludge removal.

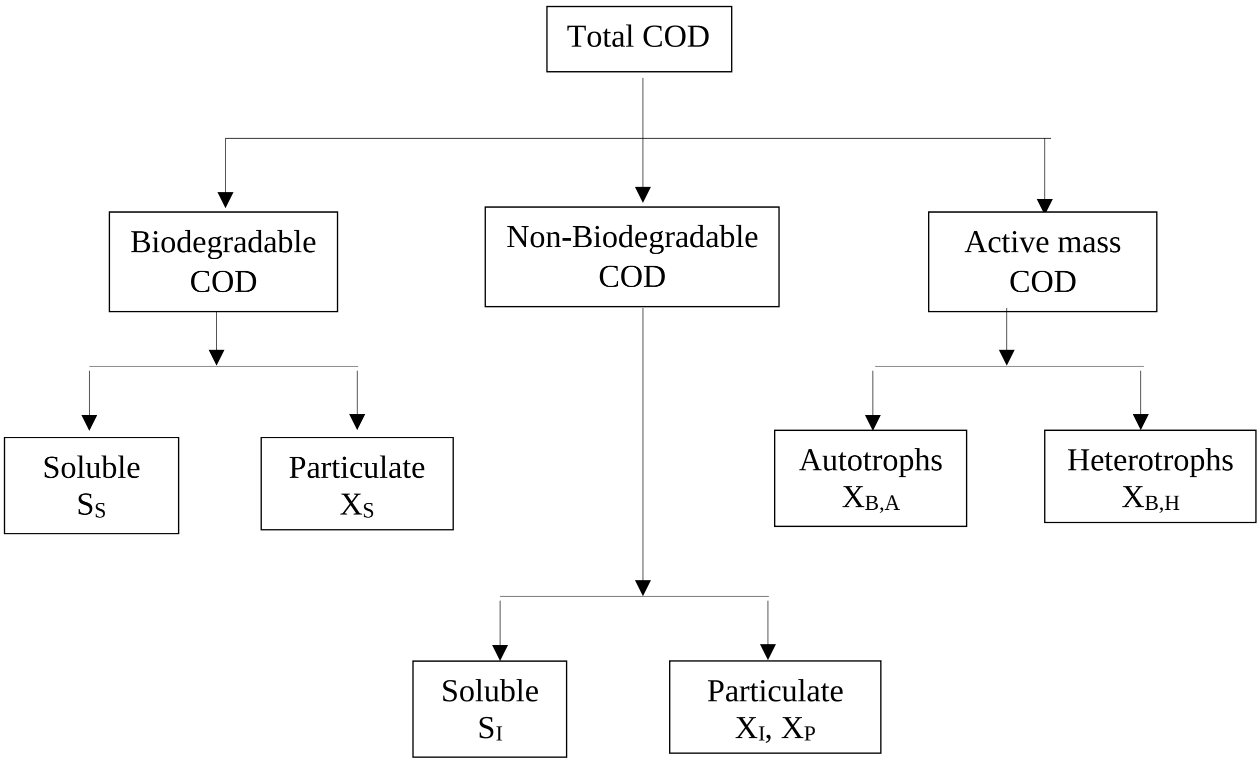

*Figure 2-3 COD component in ASM1*

Similar to the carbon material classification, nitrogen in the wastewater is divided as shown in Figure 2-4 into two total Kjeldahl nitrogen (TKN) and nitrate nitrogen ($S_{NO}$). The total Kjeldahl nitrogen (TKN) is further divided into ammonia nitrogen ($S_{NH}$), organically bound nitrogen and active mass nitrogen, that is, a fraction of the biomass, which is assumed as nitrogen. The organically bound nitrogen is divided into soluble and particulate fractions, which in turn may be biodegradable or non-biodegradable. Though it is classified as such, only particulate biodegradable organic nitrogen, ($X_{ND}$) and soluble biodegradable organic nitrogen ($S_{ND}$) are explicitly included in the model. To take production of particulate biodegradable nitrogen from decay of biomass into consideration the active mass nitrogen ($X_{NB}$) is included in the model.

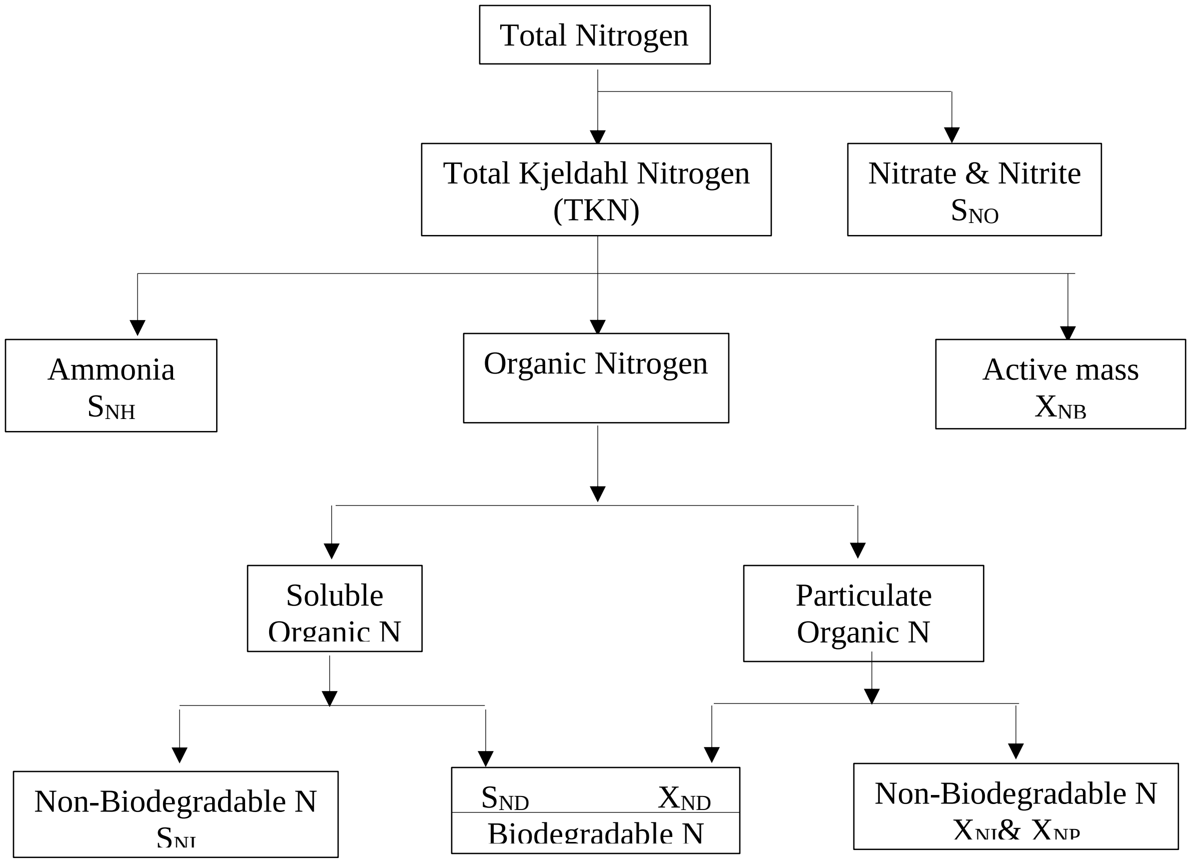


*Figure 2-4 Nitrogen Component in ASM1*

The other two components included in the ASM1 are the dissolved oxygen concentration ($S_O$), expressed as negative COD, and the alkalinity ($S_{ALK}$). The alkalinity does not affect any other processes in the model. As stated above there are eight dynamic process incorporated within ASM1which are described briefly in Annex A.

#### ***2.3.2.2 Influent characteristics***

Determining the initial concentration of the thirteen components of ASM1 that are coming with the influent wastewater is a crucial step and a dominant factor for the model success (Langergraber et al., 2004). Since the data collected at municipal WWTPs are lumped parameters that do not show the degree of biodegradability (readily, slowly and inert) and also the physical aggregation state (soluble, particulate) of the feed water, it cannot be a direct input for ASMs. Rather a detailed characterization of influent COD and nitrogen is needed. In general, two approaches have typically been used for COD fractionation while using ASMs: respirometric methods or physiochemical methods with BOD analysis. In this section, the first method is discussed briefly and focus is given for the second method.

Respirometric method is defined as the measurement and interpretation of the respiration rate of activated sludge under well-defined experimental conditions. The experimental condition could either be a batch or flow-through aerobic experiment. The respiration rate is estimated by measuring the oxygen utilization rate (OUR). Using the respirometric methods the total non-biodegradable COD fraction, the total biodegradable COD fraction, heterotrophic biomass, and readily biodegradable COD can be identified (Spanjers et.al, 1995).This method needs skilled laboratory staff, specific experimental appliances. Moreover, it has been found to be more expensive than the physiochemical method.(Sin et.al, 2005)

The second alternative is using physiochemical methods with BOD analysis. According to the Dutch Foundation for Applied Water Research (STOWA), a guideline for wastewater characterization, the physiochemical methods with BOD analysis can be used for estimating fraction of ASM components by assuming negligible biomass fraction in the influent. The characterization result can be used for process optimization, trouble shooting and design assistance (Roeleveld et.al, 2002). Physical-chemical method is based on filtration with 1.2, 0.45 or 0.1 µm filters, and sometimes flocculation with iron ($FeCl_3$) or zinc (Zn). Assuming the size of organic matter and their biodegradability is directly linked, the readily biodegradable COD or non-biodegradable organics are identified by the filter method. Since both biodegradable and inert COD pass through the filter, the inert fraction $S_I$ is determined independently. The Dutch Foundation for Applied Water Research (STOWA) suggest to take ninety percent of the effluent COD as the inert fraction $S_I$. The BOD measurement is used for characterizing the biodegradable fraction of the influent ($S_S$+$X_S$). Since it is easy to use and it has practical experimental methods with less cost, the Dutch Foundation for Applied Water Research (STOWA) is usually recommended for practitioners and new modelers (Sin et al., 2005).

#### *2.3.2.3 Calibration*

Since the activated sludge models (ASMs) are not considered universal for all biological treatment plants, the default parameters must be calibrated for every specific WWTP. However, according to (Ruano et al., 2007) calibration of ASMs should adjust only the values of a few parameters, which are believed to be identifiable from the data available, while the rest of the parameters can be assumed default from literature values. However, due to limited availability of data needed for the process calibration of activated sludge model is usually over parameterized (Ruano et al., 2007). So far, to minimize this problem parameter selection for calibration have been done by doing sensitivity analysis of model parameters and/or by following systematic calibration protocols suggested which make use of process knowledge and experiences reported from activated sludge systems. In this section, some of these calibration protocols are discussed in detail.

Biomath-Calibration protocol for ASMs offered by first(Petersen, 2000) and refined by (Vanrolleghem et al., 2003)included four steps: (1) definition of the target(s), (2) the collection of the detailed information on the activated sludge plant, (3) steady-state and dynamic calibration, and (4) decision-making. This protocol suggests that each of these steps is done in settling, hydraulic and biological sub-models of the WWTP. Each sub-model is first calibrated separately using the average influent data collected. After separate calibration, the three models are incorporated into the full-scale model. Sensitivity analysis is proposed as a tool to point out the parameters that influence the process behavior most. Based on the sensitivity result and the target of the study, it is proposed to do respirometric measurements for the sensitive parameters. Moreover, Optimal Experimental Design (OED) is proposed as a tool for design and comparison of (dynamic) measurement campaigns and batch experiments. The major challenge in adapting this protocol is the respirometric methods, which require dedicated software and trained users.(Sin et al., 2005) states the BIOMATH protocol as the most sophisticated of all protocols published these days. It is also mentioned that it may not be the most user friendly for new modelers entering the field. This may be due to the protocol is oriented at employing scientifically more exact methods rather than using practically applicable methods.

The STOWA calibration protocol is developed in the Netherlands based on experiences obtained from calibration of over 100 WWTPs. It includes nine steps. (1) Formulation of objectives, (2) Process description, (3) Data collection and data verification, (4) Model structure, (5) Characterization of flows, (6) Calibration, (7) Detailed characterization, (8) Validation, (9) study and evaluation (Hulsbeek et al., 2002). In addition to the aforementioned steps, mass balance is strongly advised in the protocol as errors in flows or control set points prove more sensitive to the simulation model(Meijer et al., 2002).It will also help to check data consistency. If the mass balance failed to be conserved, operational parameters are suggested to be corrected before doing the calibration. The protocol suggests the calibration to be done in a stepwise procedure that is based on order of importance. Based on the experiences of STOWA protocol users, different parameter subsets of the Activated sludge models that can be adapted while doing the procedure are indicated:

i. Sludge composition and production influent $X_s$and $X_i$, $i_{NX}$, $i_{NI}$
ii. Ammonium concentration in the effluent (nitrification): $K_{O2}$, $k_{NH4}$, $b_A$
iii. Nitrate concentration in the effluent (denitrification): $\eta_{NO3}$, $b_H$, $K_{O2,}$ $K_{NO3}$, $K_{OH}$

While calibrating the sludge composition it is advised by the guideline to first check the SRT of the system. (Hulsbeek et al., 2002) argued that once the SRT is well known the sludge production could be calibrated by adjusting the fraction of inert particulates listed above. The subsequent process suggested to be calibrated is the nitrification process that is done by ammonium effluent content. The guideline states that the affinity coefficients for oxygen or ammonium could be used for calibration of ammonium content only if it is checked that DO does not cause the discrepancy. The denitrification process is calibrated using measured nitrate concentration in the effluent. It is advised to adjust the anoxic reduction factor for anoxic growth and hydrolysis processes, or eventually the heterotrophic decay factor to calibrate the denitrification process. If further calibration of a model is needed, STOWA protocol suggests the use of sensitivity analysis. After successful calibration, it is advised to do validate of the model prior to the actual study using data from a completely different condition (when primary settlers are taking maintenance, different temperature period). (Sin et al., 2005) expressed STOWA protocol to be the most straightforward, practical, easy to follow and implement protocol. It is also stated that the protocol helps to understand the significant steps underlying a calibration study. For these reasons, STOWA protocol has become suitable for new modelers and practitioners.

(Langergraber et al., 2004)in Germany proposed another model calibration protocol called the Hochschulgruppe (HSG) protocol. This guideline includes seven steps:(1) definition of objectives, (2) data collection and model selection, (3) data quality control, (4) evaluation of model, (5) Data collection for simulation study, (6) Calibration/Validation, (7) Study and evaluation of success. Similar to BIOMATH protocol the WWTP model is divided hydraulics, settler, controllers and biological compartments sub-models. Even though there is no method suggestion for settling characterization, it is stated in this protocol to use computational fluid dynamics (CFD) method for the hydraulic sub model prior to calibration of the full model. This is to determine number of tanks in series for adequate modeling of the aeration tank mixing behavior. However (Sin et al.,

2005) suggested that the CFD might further complicate the model calibration study, as it is a computationally demanding and time-consuming task. Unlike to the other protocols, HSG protocol presents general guidelines to be followed for well documentation of the steps followed until the target was reached. HSG protocol provides only general guidelines. Neither detailed settling characterization nor particular methods for parameter estimation are stated in the guideline. The choice of the parameters to be calibrated is also left to the practitioners. The free choice of experimental methodologies might be challenging for non-experienced modelers or new practitioners.(Sin et al., 2005)

WERF protocol for ASM calibration, which is common in North America, share most of the methodologies listed in the preceding protocols (Melcer, 2003). Moreover, it manages to present a new approach called tiered approach for model calibration. In this approach, the model calibration starts from a simple level and advances to a more sophisticated level. Level 1 uses default values and assumptions for the full-scale model. Level 2 only uses historical data for calibration of the WWTP. It is suggested to use statistical methods for correcting, cleaning or conditioning biases and outliers from the raw historical data. In the third level dynamic measurements campaign are recommended. This level improves the results of the calibration. Stress tests are also suggested to determine the maximum capacity of the plant under extreme conditions. In calibration level 4, direct parameter measurements of kinetic/stoichiometric parameter are suggested. The protocol advises to use this calibration level when level 3 fails to be successful due to poor information of the dynamic measurement campaign data or complexity of the model. The modeler can go through the four level or not depending on the objective of the study. Even though this protocol lacks structured overview of the steps to be followed and gives no emphasis for kinetic parameters than nitrification, it was found to be good for new modelers(Sin et al., 2005).

#### *2.3.2.4 Software*

In the following paragraphs, computer packages that include the activated sludge models are discussed, with special emphasis on those models that are freeware such as AQUASIM, ASIM, and STOAT.

#### 2.3.2.4.1 Commercial simulator software

##### 2.3.2.4.1.1 GPS-X

GPS-X is a simulator developed by Hydromantis, Canada in 1992.GPS-X is capable of doing the mathematical modeling, simulation, optimization and management of wastewater treatment plants.ASM1, ASM2 and ASM3 are available in the modeling tool. Since GPS-X has a good user-friendly interface that enables users to drag and drop with comprehensive database in each unit process, assembling treatment plant model, entering characterization data and doing simulation is done quickly. GPS-X uses over 20 process objects in its models, including a conventional activated sludge, fixed film configurations and a membrane bioreactor. The MBR in GPS-X has submerged configuration. It can be modeled by ASM models as either completely stirred tank reactor (CSTR) or plug flow reactor. There are three different MBR model modes: simple, intermediate and advanced. Simple ignores the filter operation and the separation is only defined by a solids capture efficiency. Intermediate and advanced mode include filter operation such trans-membrane pressure (TMP), cake formation, fouling, backwashing and membrane resistance. (Hydromantis, 2013).

##### 2.3.2.4.1.2 BioWin

BioWin was developed by EnviroSimInc, which is located in Canada. BioWin contains most of the feature in GPS-x model, which makes it equally user friendly. One unique feature of this model is it utilizes BioWin's full General Activated Sludge Anaerobic Digestion Model (ASDM). The ASDM contain 50 components and over 70 processes, which makes it more complex than the conventional Activated Sludge Models (ASMs). BioWin models the MBR with a combination of activated sludge and submerged separation unit. The model uses a simple point separator, where it is possible to define the solids capture efficiency. It does not model TMP, fouling, cake formation, nor backwashing routine.(EnviroSim, 2012)

##### 2.3.2.4.1.3 WEST

WEST (Wastewater Treatment Plant Engine for Simulation and Training) was developed by Hemmis, Belgium. As this model includes most of the features and modules needed from a wastewater process simulator, it is used by many researchers (Vanhooren et al., 2003). Another reason for its wide application could be most of the models in this simulator are open source and open code; thus, the models can be modified if necessary. WEST has been mainly used in the context of wastewater treatment research. MBR reactors can be modeled as a submerged membrane bioreactor or with a side stream setup. The membrane can be modeled either as an ideal separation unit, where the solids capture efficiency is defined, or it can be modeled by considering fouling process. The model uses an empirical relationship to calculate build-up of cake resistance in the membrane.

##### 2.3.2.4.1.4 SIMBA#

SIMBA is developed by Institutfür Automation und Kommonikation (IFAK) in Germany. SIMBA allows for the complete consideration of sewer systems, wastewater treatment plants, sludge treatment and river quality. Hence, this makes SIMBA suitable for integrated simulation and control. Even if it is a commercial simulator, it does not provide as many wastewater treatment modules as its competitors. It does not have any modeling units for MBR processes.

#### 2.3.2.4.2 Free Simulator software

##### 2.3.2.4.2.1 STOAT

STOAT (Sewage Treatment Operation and Analysis over Time) was developed by WRC Water Research Centre in England. It is designed to dynamically simulate the performance of a wastewater treatment works including sludge treatment processes. STOAT includes the implementation of ASM1 and the Takács settler model. Until recent time STOAT was a commercial model but since it was not being updated actively, recently it is released as a free modeling tool. STOAT contains biofilm processes (trickling filter, biological aerated filter), sludge treatment (anaerobic digestion, dewatering), chemical phosphorus removal. But it does not contain the membrane bioreactor as an independent unit process. It provides capabilities for sensitivity analysis, calibration and optimization.

##### 2.3.2.4.2.2 ASIM

The ASIM (Activated Sludge SIMulation program) was developed at the Swiss Federal Institute of Aquatic Science and Technology in Switzerland. Although it is one of the first simulation model, flexibility is relatively limited in this simulator; with this simulator only activated sludge process, can be modeled using the standard ASM models. It does not come with a membrane bioreactor model. It does not include any tools for automatic calibration, sensitivity analysis or scenario analysis.

##### 2.3.2.4.2.3 AQUASIM

AQUASIM is also a simulator developed at Swiss Federal Institute of Aquatic Science and Technology (Eawag) in Switzerland. Even though this software does not come with predefined ASM models, it uses object-oriented approach. This approach enables the user to define variables and process that can be activated within a system of linked compartments. Due to this any kind of biokinetic model can be defined within this software. It is a flexible simulator where completely mixed reactors, biofilms and river sections can be modeled(Reichert, 1998)

As the commercial software, AQUASIM does not enable the user to drag reactors and connection. This makes the software less user-friendly with a higher learning curve (Urdalen, 2015). However, it includes powerful tools to perform sensitivity analysis and parameter estimation**.**

## 2.4 MBR and Activated Sludge Model No 1

The following section critically discusses how previous researches have applied activated sludge models for design, operation optimization and energy cost reduction of membrane bioreactors. Gaps in research and assumption in application of the models have also been discussed.

Although activated sludge models were initially developed to model conventional activated sludge process, it has been used since the late nineties to simulate biomass kinetics in MBRs systems as well (Fenu et al., 2010). Since MBRs encompass the Activated Sludge Process (ASP) as their fundamental process, biological modeling of MBRs by these models has been accepted and reported in various studies. Some researchers have been using the unmodified ASM i.e. as defined by (Henze et al., 2000) and other used ASM with additional state variable which take care the soluble microbial production (SMP) effect. The plain ASMs have been used when the model objectives were determination of either process design, effluent characterization, oxygen demand or sludge production. Modified ASM have been used to model MBR when the objective of the study was either linking biology with fouling, soluble chemical oxygen demand (COD) predictions in the bulk. Modified ASM has also been used while modeling reactors with high SRT. According to (Fenu et al., 2010) utilizing the modified ASM for reasons other than those is not recommended as the concept creates difficulty in calibration of newly introduced parameters.

Several efforts have been made towards application of unmodified ASMs for MBRs. Earlier research used default parameter values stated in (Henze et al., 2000). It is reported that some of these studies have been able to find successful result even from non-calibrated ASM1 models. A reasonable estimate of effluent COD and TKN was found from this model but the accuracy decreased with very low HRT and very high SRT systems (Fenu et al., 2010). ( Delrue et.al, 2010) has stated that ASM1 is suitable for modeling MBR plants if influent characterization and systematic calibration of aeration can be taken care of.(Spérandio et.al, 2008) has also found success while modeling submerged membrane bioreactor for municipal wastewater treatment using ASM1. The result drawn from this research was the same with the former study; ASM can provide satisfactory prediction of aerobic biological process as long as high SRT (above 100 days) does not prevail. This drawback-initiated studies towards investigation of appropriate parameter sets that take the nature of MBR biology into account. These days' models and researches with more systematic calibration specific for MBR operating conditions become frequent.

The literature available highlights the important role ASM application to MBR has in design and operation. Some of the studies, which used ASM for operation optimization and energy cost reduction, are briefly discussed in the following paragraphs.

(Baek et al., 2009) modeled aerobic membrane bioreactor treating municipal wastewater using Activated sludge model no. 1 (ASM1). The model for the aerobic MBR was calibrated using AQUASIM 2.0. This study tried to assess the performance of MBR process in terms of chemical oxygen demand (COD) removal and ammonia nitrogen ($S_{NH}$) nitrification at different operating conditions such as hydraulic retention time (HRT), solid retention time (SRT) and mixed liquor suspended solids (MLSS) concentrations. The results from the simulations provided a better understanding of the mechanisms and kinetics of the MBR process including sludge removal reduction. Operational energy cost reduction was not included in this study.

(Verrecht, 2010) in a PhD dissertation used ASM to aerobic MBR and verified the application of such models for community scale MBR. A calibrated model of ASM2 was able to predict effluent nutrient concentrations and MLSS concentrations accurately. A scenario analysis (SCA) was carried out using the calibrated model to simulate the effect of operational parameters variation on effluent quality, MLSS concentrations and aeration demand. The parameters included in the scenario analysis were SRT, recirculation ratio and DO set point on effluent quality. The analysis results showed that decreasing membrane aeration and SRT were the most beneficial towards total energy consumption, while increasing the recirculation flow led to TN removal improvement but at the same time also deterioration in TP removal. The optimum SCA also resulted in 23% energy reduction without compromising effluent quality. A conclusion that could be made from this study is that modeling approach can be used to determine operational parameters with the desired water quality and reduced energy cost.

(Comas et al., 2015) has done optimization on full-scale submerged membrane bioreactor. In this study a mechanistic model, which is based on ASM2, was developed to reproduce the operation of the full-scale MBR and predict the effects of the optimization actions. The optimization result suggested the reduction of the default DO value, which is set by the manufactures, by 0.4 mg/l. The DO modification increased the nitrogen removal efficiency by 27% and reduced the aeration energy cost by 7% without compromising sludge property.

All the above-mentioned studies show how optimization plays a key role in MBR system performance. Model based investigation were the commonest method used among scholars to perform optimization analysis. This method is often preferred by most due to its ability of doing “what if” studies. While solving the optimization problem rather than the classical mathematical methods statistical methods were widely used. Due to the existence of many variables in wastewater modeling process, the classical method becomes time consuming and inflexible to solve such problems. (Bashir et al, 2015) states response surface method (RSM) as the most reliable statistical and diagnostic tool to optimize wastewater treatment process. In MBR plant (Lim et.al, 2011) has applied RSM for solving dual optimization of COD and total nitrogen removal. ASM model were used to generate the experimental data for the optimization. Internal recirculation rate, DO and waste sludge rate were used as the most important optimization parameter of the plant. The research suggested that COD and nitrogen removal efficiency can be enhanced to 88% and 62.67% respectively if the MBR is operated under optimal conditions which are internal recycling at 250m$^3$/day, DO set point 1mg/l and waste sludge of 875m$^3$/d.

# 3 Material and Methods

## 3.1 Description of the treatment plant

The experimental work for this thesis was performed at the membrane bioreactor in Addis Ababa. Hence, the next paragraphs contain a detailed description of the decentralized wastewater treatment plant.

One of the package decentralized wastewater treatment facilities that are being constructed by Addis Ababa Water and Sewerage Authority is submerged membrane bioreactors. These plants are under construction at twelve locations in the city. The Mekenisa Kotari Treatment Plant is one of the three MBRs that is in operation currently. It is designed to serve around 15,000 persons with 1700m$^3$/d capacity. The plant mainly consists of preliminary treatment units, biological treatment units and sludge dewatering systems. The preliminary treatment stage of the plant involves the removal of solids in the influent as it passes through manual screens and automatic fine screens with Archimedes screw for grit separation. Two parallel equalization concrete tanks follow the preliminary treatment units. These tanks have aeration system inside them to avoid inconvenient odor problem. Since the package WWTP is very close to the residential area, avoiding such problem is crucial.

*Figure 3-1 Combined unit with automatic screening and grit removal*

From the equalization tank, the wastewater joins the secondary treatment units with the help of pumps coupled with flow meters. These units comprise aerobic biological treatment. The biological treatment is carried out in two distinct concrete tanks, 106.8 $m^3$pre-aeration and 110$m^3$ MBR tank. Both tanks are aerobic basins no anaerobic or anoxic tank is included in the treatment plant. Recirculation of mixed liquor is performed from MBR to pre-aeration tank at three times the design flow rate Figure 3-2. This keeps the biomass concentration in pre-aeration tank constant. Due to additional oxygen requirement for biological requirement the pre-aeration tank with fine bubble diffusers is provided. It adds the necessary volumetric requirement for biological degradation.

*Figure 3-2 Pre-aeration tank*

The MBR tank has Kubota submerged membrane unit (SMU) with coarse bubble diffuser for scouring effect. The submerged membrane contains multiple cartridges with an average size of 0.4µm. The extraction of water through the micro filtration membrane is held by use of suction pumps. Although the instruction manuals of Kubota SMU states the trans-membrane pressure (TMP), which force the passage of permeate through the cartilage, as 0.1-0.4 bar, at Mekenisa Kotari site it is restricted to 0.2bar. When clogging of membranes increases the pressure gage reading gets closer to negative 200mbar, the automaton shows an alarm for operators to schedule a chemical cleaning. The plant stops functioning if the TMP is greater than 300mbar. Setting working range of TMP is necessary in order to avoid higher degree of membrane fouling that could not be removed by conventional chemical cleaning.

Mekenisa Kotari Wastewater Treatment Plant is expected to produce about 1816.88kg/day of dried sludge. Waste sludge is dewatered before discharge. Removal of the water from the sludge is essential to reducing weight and the cost of further treatment or disposal. In Mekenisa Kotari Wastewater Treatment Plant, waste sludge is pumped from MBR to the flocculation tank. After mixing the sludge with polyelectrolyte conditioner in the flocculation tank the sludge is dewatered with a decanter centrifuge (Figure 3-3). Dewatered sludge is transmitted to solid waste storage area. The supernatant is discharged back to the equalization basin.

*Figure 3-3 Decanter centrifuges for dewatering sludge*

The MBR taken as a pilot plant for this study has a volumetric loading rate of 1.2kg/m$^3$.day. The MLSS concentration of the bioreactor is controlled as 9g/l, while the hydraulic retention time and sludge age are 3.5 hours and 12 days respectively. With the specified ideal condition, it is expected to remove organic matter to 50mg/l COD and 5mg/l of BOD. Although anoxic and anaerobic tanks are not part of the plant configuration the client expected this plant TKN and Phosphorus removal efficiency to be around 75%. Process flow diagram of the wastewater treatment plant is enclosed in Annex C.

## 3.2 Influent characterization

The wastewater characteristics of influent, effluent and supernatant samples were measured in terms of i) Chemical oxygen demand (COD), Biochemical oxygen demand (BOD) ii) total nitrogen (TN), ammonia-nitrogen, nitrate-nitrogen and iii) total phosphorus concentrations according to the Standard Methods (Apha et.al. 1999).

### 3.2.1 Sampling and analysis

The setting up of a proper sampling program was the first step into characterizing the constituents of the wastewater. To meet the need of representative sampling in time and space, several factors have been taken into consideration. These factors include total number of samples needed to achieve statistically representative output values from the analyses, sampling locations, type of samples (grab or composite samples), and time intervals between samples. The details of the sampling program adapted are discussed in the following paragraphs.

To include both dry and wet season in sampling frame two sampling programs were undertaken. The first sampling program was carried out from March to May 2017 which correspond to the dry season. The wet season sample was taken from July to August 2017. The sampling dates were set by considering weekend inclusion within the sampling periods. This helped to capture the temporal variation of the wastewater character. Each sample was implemented as a composite sample out of six grab samples. The samples were collected in plastic bottles that were rinsed with the wastewater before filling.

An important component of the sampling program is the quality assurance and quality control (QA/QC) plan, the objective of which is to ensure that there has been no external contamination of the samples from initial sampling through final analysis(Zhang, 2007). During the sampling and analysis process of this research, duplicate, trip blank, and field blank samples were prepared and subjected to laboratory analysis. One duplicate sample was prepared for every 10 samples taken. One filed sample was split into two aliquots for having duplicate sample; this helped to ensure the reproducibility of the sampling procedure. A bottle with distilled water was prepared and transported with the empty plastic bottles to the sampling site and returned to the laboratory with the samples analyzed. This was done to identify if there were problems of contamination in the preparation of sample container and shipping procedures. The third type of QA/QC prepared was field blanks. Empty sample bottles were filled with tap water at the treatment plant following the general sampling procedures used for collection of all waste samples. The field blanks were returned to the laboratory for analysis. Field blanks were necessary to address contamination issues with the field sampling procedures. All wastewater characterization was performed on the same day as sampling.

The samples for analysis were collected from five points of the treatment plant as follows:

- Sampling point 1- Biological unit influent
- Sampling point 2 The pre-aeration tank
- Sampling point 3The membrane bioreactor
- Sampling point 4 Outlet of membrane bioreactor(final effluent)

*Table 3-1 Overview of parameters sampled at different location of sampling points*

| No | Sampling point | Parameter | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | COD Total | COD Filtered | $BOD_5$ | $NH_4$-N | $NO_3$-N | TKN | TP | MLSS | DO | Temp | PH |
| **1** | Biological unit influent | ■ | ■ | ■ | ■ | ■ | ■ | ■ | | | | ■ |
| **2** | Pre-aeration tank | | | | | | | ■ | ■ | ■ | ■ | ■ |
| **3** | Membrane bioreactor | | | | | | | ■ | ■ | ■ | ■ | ■ |
| **4** | Outlet of membrane bioreactor | ■ | ■ | ■ | ■ | ■ | ■ | ■ | | | | ■ |

Analysis of parameters was carried out according to the Standard methods for the examination of water and wastewater (Apha et.al. 1999). COD was analyzed in accordance to section 5220D, the closed reflux colorimetric method. Hach glass digestion and block heater were used for COD analysis. High concentration digestion solution for influent and low concentration digestion solution for effluent were selected. BOD was analyzed by using the Lovibond® sensor system,OxiDirect®. The working principle of this device is in accordance to Standard Methods section 5210 D. The BOD was measured continuously to establish a BOD curve. The concentrations of ammonia nitrogen, oxidized nitrogen ($NO_3$) and TKN nitrogen were determined by using Hach DR 2000 instrument.

Collecting only the lumped parameters using the standard method that does not show degree of biodegradability and physical aggregation state was not enough to determine the state variables of Activated Sludge Model. Since the model would be applied for further scenario analysis the bulk values from influent characterization mentioned above were not enough for the study. Hence, as proposed in STOWA: (Roeleveld et.al, 2002) the physical chemical method with BOD analysis was used as additional sampling analysis. Non-biodegradable soluble COD ($S_I$) was assumed to be 90 % of filtered effluent COD. The difference between filtered influent COD and $S_I$ was assumed as readily biodegradable $S_S$. The BOD test was used to estimate the total biodegradable fraction of the influent. Once identifying that the slowly biodegradable COD ($X_S$) was determined from the difference of the ultimate BOD and readily biodegradable $S_S$. The remaining COD was taken to be particulate inert COD ($COD_{tot} - S_I - S_S - X_S$). Similar to the organic matter (COD) fraction the nitrogen should have been segregated to its components. But according to (Roeleveld et.al, 2002) since the major part of nitrogen is present in ammonia form there is no need of doing detailed analysis as the organic matter fraction. Hence, the standard composition factors for the ASM1 model was used for total nitrogen fraction, and the model ammonium influent was set to the measured ammonium value.

## 3.3 Performance Evaluation

Performance evaluation of the treatment plant was done by analyzing the effluent wastewater quality and filterability of the sludge. The effluent wastewater quality was analyzed by taking samples of the treated wastewater following the procedures discussed in subsection 3.2.1.

The filtration system performance was evaluated by analyzing three typical parameters of membrane filtration process; trans-membrane pressure (TMP), flux and permeability. According to (Hai et.al, 2014) these parameters are the most important process performance indicator matrices.

The trans-membrane pressure (TMP) is the pressure differential across the membrane which forces the water out. The TMP of the MBR plant was recorded from the online measuring tool at site. The average TMP of each day was plotted in order to analyze and discus the interpretation of the field data.

The flux, usually called the filterability velocity, is the quantity of wastewater passing through a unit area of membrane per unit time. The net flux was calculated for each day in the testing period by dividing the daily average permeate flow to the total membrane area provided. The result was compared to (Hai et.al, 2014) recommendation for well performing MBR which is between 10 to 150 liters per $m^2$ per hour (LMH).

The permeability of a plant is the amount of water that passes through the membrane per unit of time, per unit of area and per unit of TMP. This parameter was calculated by dividing the flux to the TMP. The permeability expresses the ability of the liquid to pass through the membrane or the productivity of the plant. (Judd, 2011) recommends permeability between 150 and 250 LMH/ bar for aerobic full-scale domestic MBRs.

The MBR module was evaluated using recorded data of TMP, flux & permeability of five-month operation. The average result of these parameter and the trend of change with time was used to discuss on the filterability capacity and sustainability of the treatment plant.

## 3.4 Process Optimization

The best possible operational strategy, in terms of wastewater treatment plant performance, can always be identified based on combination of environmental and process criteria such as effluent quality and energy consumption. In order to upgrade the performance of the Mekenisa Kotari MBR plant, enhancing the denitrification capacity of the plant while reducing energy use was taken as a main objective of the process optimization. As only organic matter removal and nitrification is part of the treatment process in Mekenisa Kotari MBR, the optimization will greatly decrease the total nitrogen concentration in the effluent.

To get insight into the Mekenisa Kotari MBR plant performance, model-based investigation was preferred over other methods. This preference was made because model-based analysis is relatively easy to investigate system response and behavior. If fitting the model to reality is performed well, the model can prove its value by being used for all kind of extrapolation, “what if” studies and optimization (Brdjanovic et.al 2015).

### 3.4.1 Model Development

Simulations were performed using ASM1 model (Henzeet.al, 2000) in AQUASIM 2.1. AQUASIM was used for implementing such model because it allows users define relevant variables and processes; an approach that makes it a lot flexible when compared to other software. It can also estimate parameter values and do the uncertainty of calculated results.

Before constructing and running the simulation, physical plant data and plant operational data, which are input of the model, were collected. The input parameters that needed to be determined were process flow diagram (PFD) of the WWTP, the aeration and MBR tank dimensions, the times of feeding, feed volumes, the mixed liquor recirculation (MLR) flow rate, DO concentration of aerated zones, temperature of the reactor and the properties of the feed. Both primary data and secondary data were used to construct the plant model. The influent characterization and design document of the treatment plant helped to develop the preliminary model. Two reactors (pre-aeration and MBR tank) were configured in AQUASIM as mixed reactor compartments. A link was defined to model the MLR of the plant. The wastage flow was taken directly from the MBR tank. The membrane separation was modelled as an ideal separation unit with complete retention of solids. Since studying the complex flirtation process was not the objective of the study, modelling of fouling or removal of soluble compounds was not included. This is taken to be acceptable as no consensus on its mechanisms of fouling has been reached yet (Maere, et.al, 2011)

### 3.4.2 Model Calibration and Validation

For a reliable simulation study the sludge age (SRT) should be known within 95% accuracy (Brdjanovic et al., 2000; Meijer et al., 2001). Therefore, a check on the SRT (or sludge production) is strongly recommended. For the evaluation of sludge production, the overall phosphorus balance was used as proposed by (Nowak et.al, 1999) . The phosphorus balance logic can be done because the total phosphorus does not leave the treatment unit by gas phase. It is either in the wastewater solubilized or leave the system with sludge as solid form. Total phosphorus was measured in all in- and outgoing flows and the sludge flow rate was determined by solving the total phosphorus balance. Rather than the design value of SRT, value from the mass balance was used for the modeling. The two mass balance equations formulated are shown in Equation 3.1 & 3.2

$$Q_{in} = Q_{eff} + Q_{ex} \tag{3.1}$$

$$Q_{in}P_{in} = Q_{eff}P_{eff} + Q_{ex}P_{mbr} \tag{3.2}$$

Where $Q_{in}$ is flow rate of the raw wastewater (m$^3$/d) $Q_{eff}$ is flow rate of effluent wastewater (m$^3$/d), $Q_{ex}$ is the excess sludge flow rate (m$^3$/d) $P_{in}$ is total phosphorous concentration in the raw wastewater (mg/l), $P_{eff}$is the total phosphorous concentration in the effluent wastewater (mg/l, $P_{mbr}$ is the total phosphorous concentration of the mixed liquor suspended solid in MBR tank (mg/l)

Once the operational data were checked and measurements were balanced the model was ready for calibration. The ASM1 parameters were calibrated under steady-state conditions by fitting the COD, MLSS, $NH_4$-N & $NO_3$-N model predictions to the data obtained from the sampling program. The model calibration followed the stepwise approach described by (Hulsbeek et al., 2002) that includes the calibration of: (1) the solids balance and mixed liquor suspended solids (MLSS) in the aeration tanks, (2) effluent COD, and (3) effluent $NH_4$and $NO_3$

Sensitivity analysis was done to identify the most important parameters, which needed to be adjusted during model calibration. The sensitivity method was used to reduce the number of model parameters that included in the calibration step. The sensitivity of sludge production and the concentration of ammonium and nitrate in the effluent were analyzed for all parameters in the activated sludge model. The linear sensitivity analysis used was relative-relative function, Equation (3.3). This sensitivity function was preferred over the absolute- absolute function, for its unit does not depend on the parameter unit. The sensitivity analysis was performed by AQUASIM.

$$\delta_{y,p}^{a,r} = \frac{p}{y}\frac{\partial_y}{\partial_p} \tag{3.3}$$

Where $\delta_{y,p}^{a,r}$is sensitivity function, y is an arbitrary variable value calculated by AQUASIM and p is a model parameter.

The relative-relative sensitivity function measures the relative change in y for a 100% change in p. Positive sign of the result of the sensitivity function indicates y will increase when p is increased and negative sign indicates y will decrease when p is increased. The sensitivity analysis provides a ranking of the significant parameters that influence outputs (Reichert, 1998). In this research the influence of parameter was interpreted as proposed by (Petersen, et. al, 2002). For relative sensitive function between -0.25 and 0.25, a parameter was considered to have no significant influence on a certain model output; if ±0.25 ≤RSF <±1, the parameter was considered to be influential; if ±1≤RSF<±2, the parameter was considered to be very influential; and if RSF>2, the parameter was considered to be extremely influential. The sensitivity of the sludge production, concentrations of ammonium and nitrate in the effluent were analyzed for all 22 stoichiometric and 42 kinetic parameters of the activated sludge model. Since not only the parameters of the activated sludge model influence the outputs, but also the influent composition, a wide range of sensitivity analyses was performed for the distribution of COD in the influent.

Once the sensitive parameters were identified the value of each parameter was estimated by fitting the model to the dataset. The identification of the best value for the parameters was based on Chi squared ($\chi^2$), the sum of squares of the weighted deviation between the measured and simulated, minimization (Equation (3.4)).

$$\chi^2(p) = \sum_{i=1}^{n} \left( \frac{y_{meas,i} - y_i(p)}{\sigma_{meas,i}} \right)^2 \tag{3.4}$$

Where $y_{meas,i}$ is the observed value; $y_{meas,i}$ is the simulated value for the parameter set p; $\sigma_{meas,i}$is the standard measurement error of the observation $y_{meas,i}$ and n represents the number of data points to which the model was fitted.

The optimizer module of AQUASIM was used to minimize the $\chi^2$formula. The maximum and minimum value of parameters was defined as constraint. The ASM1 parameters default range was adapted from (Henze et al., 2000) and included in Annex A.

To test the fit between the simulated data and the measured data, a Chi-Square test for goodness of fit was performed. The null hypothesis ($H_0$) was that observed and predicted values are unequal. The alternate hypothesis (Ha) was that observed and predicted values do not have the same distribution. The computed $\chi^2$from parameter estimation was compared to the tabulated value of the chi-squared distribution for n-1-$n_\theta$ degrees of freedom (n is number of measured data and $n_o$ is number of estimated parameter) and 95% confidence level to decide whether to reject or accept the null hypothesis.

In addition, standard error of estimate (SEE) was also observed to measure the accuracy of predictions (Equation (3.5))

$$SEE = \sqrt{\frac{\Sigma\left(y_{meas,i} - y_i(p)\right)^2}{N}} \tag{3.5}$$

Prior to the calibrated model usage for scenario analysis, validation of the model was performed. as (Hulsbeek et al., 2002) suggests. The calibrated model was tested for different conditions (different temperature and flow). The wet season sample taken from July to August 2017 was used to verify the model.

### 3.4.3 Model-based Optimization

#### *3.4.3.1 Intermittent Aeration*

In this section, it was tried to experience different simulations by the calibrated model in order to upgrade the performance of the treatment plant. Since Mekenisa Kotari MBR plant only has aerobic tanks, nitrate nitrogen is discharged untreated with effluent water. Wastewater treatment plants are required to treat nitrogen, in either ammonia or nitrate form, for its presence in permeate causes eutrophication and toxicity. Conventional biological nitrogen removal technologies include an aerobic stage for nitrification and an anoxic stage for denitrification. Typically, a biological nitrogen removal plant upgrade requires additional capacity in the form of an anoxic basin, which requires additional funding and space. One option to avoid the construction of additional anoxic basin is to convert the aeration tank to an intermittent aerobic/anoxic cycle structure (Grady et.al, 1999).

As proposing new plant layout for effluent quality enhancement might be hard to implement, the optimization method of this study focused in creating alternating aerobic anoxic condition in one tank by providing intermittent aeration. This is considered acceptable as for most small size plants, i.e. 20,000 population equivalent or less, often carry out the two process in a single basin (Fikar et. al, 2005). Since Mekenisa Kotari MBR plant is automated, it would likely only need to develop programs for implementing the proposed cyclic aeration.

The aim of the optimization was determining optimum duration of the aeration and non-aeration sequences, which satisfied nitrification and denitrification processes in the aeration tank and minimize the operation costs. The total nitrogen removal concentration, which was found from summation of nitrate and ammonia concentration in the effluent, was taken as the objective function of the optimization. Low aeration fraction results in high ammonia concentration whereas high aeration fraction causes high nitrate-N concentration. An optimum exists at which the total discharge of nitrogen is minimum. As the aeration fraction is in excess of the optimum, nitrification is complete but the anoxic period is insufficient to allow reduction of nitrate. On the contrary, at aeration less than the optimum the aerobic period becomes insufficient for growth of nitrifying bacteria and no nitrogen removal takes place(Grady et.al, 1999). The model based optimization for the MBR has taken this principle as the base for process optimization.

The aeration and non-aeration sequences were represented by two parameters; aerated fraction (AF) and cycle time ratio (CTR). The aerated fraction (AF) is the fraction of time within a cycle time in which the reactor is aerated. The cycle time ratio (CTR) was used to identify the ratio of the total cycle time to the hydraulic detention time. The cycle time ratio was always kept less than one in all cases to ensure the influent water coming meet both the aeration and non-aeration stage at least once. The sludge residence time (SRT) value of the MBR was also considered as optimization parameter as it is left to be controlled by site operators.

Since the determination of the optimal sequence of aeration/non-aeration times was done based on model simulation, all data necessary for the optimization were found by running the calibrated/validated model for different scenarios. The alternating aerobic and anoxic condition was defined in the model by modifying the aeration rate in ASM1 (Equation (3.6)).

$$u_b k_{La}(S_O^{max} - S_O) \tag{3.6}$$

Where $u_b$is coefficient for intermittent aeration, $k_{La}$is oxygen transfer coefficient, $S_O^{max}$ is the saturated dissolved oxygen concentration and $S_O$ is dissolved oxygen concentration

The coefficient added for modification $u_b$ was taken as a binary sequence switching between 1 and 0 that represents the state of blowers (on/off). For time equal to aeration fraction the value of $u_b$ was kept as 1 and time equal to the difference of cycle time and aeration time $u_b$ was set as 0. This brought the needed alternating aerobic/anoxic cycle. At the time t=0 the simulation was started with $u_b$ = 1, which was aerating phase.

### *3.4.3.2 Response Surface Method*

Optimizing the proposed intermittent process through classical mathematical method was found to be inflexible and time consuming. Hence an alternative method, Response surface method (RSM), which uses both mathematical and statistical techniques was used to solve the optimization problem. When variables to be optimized satisfy an important assumption that they are measurable, continuous and controllable RSM can be used with negligible errors (Myers, et.al, 2016). Hence, RSM can be employed to optimize operational parameters and enhance removal efficiency in wastewater treatment process to obtain the maximum output (Bashir et al, 2015).

RSM uses sequence of simulation or observed experimental data to illuminate a spot where high response is found. In RSM the data is used for developing regression equations usually of second order polynomial type. Once a good regression model is fitted to the data, the equation can be represented graphically either in 3D space or in contour plots. The graphs are used to visualize the shape of the response surface and to read the optimum value in single response optimization.

In order to apply RSM as an optimization tool there are five typical steps that should be carried out. These are 1) selection of the most important and independent variables and their levels through screening studies 2) choice of numerical experimental design and carrying out the selected experimental matrix to get data for the regression model3) fit of polynomial equation through mathematical and statistical method 4) evaluation of model fitness and verification of the accuracy of the regression model 5) obtaining the optimum values for each independent variable. The following paragraphs discus how each of these steps was performed in this study.

#### 3.4.3.2.1 Screening Experiment

In order to make sure that the optimization variables are influential and to approach the general vicinity of optimum it is recommended to do screening procedures in the early stages of the optimization process. (Myers, et.al, 2016). The screening procedure used had two stages to be followed; fitting linear regression model and evaluating the steepest descent direction. This two stages helped to approach the optimum region rapidly.

In the first stage, a linear model was fit to describe only the flat surface within a small range chosen using best prior knowledge. Not to be misled by non-linearity the range was not set too far. The variation range of the operational parameters was based on values from the literature for MBR configuration. Accordingly, the aeration fraction were varied from 0.6 to 0.8 as recommended by (Hanhan et.al, 2011). The range of sludge residence time (SRT) was taken to be 12 to 18 days (Hai et.al, 2014) and cyclic time ratio (CTR) was from 0.2-0.5. Fractional factorial design was used to determine sets of combination of different SRT, AF and CTR values within the specified range. After designing the experiment, the validated ASM1 model was simulated for the designed experimental matrix.

The effluent concentration data generated from the simulation in ASM1 helped to fit first order regression Equation (Equation (3.7)).

$$f = \beta_0 + \beta_i x_i + \beta_j x_j + +\beta x_i x_j + \varepsilon \tag{3.7}$$

The first order square fit approximation of (Equation 3.7) was not suitable for analyzing maximum or minimum points because it did not consider curvature. It was only used to lead the analysis rapidly and efficiently to the optimum vicinity. The steepest descent was the second analysis that was performed in the screening process. This procedure helped to get the direction of reduction and to explore new level for AF, SRT and CTR. The direction of steepest descent is the direction in which response decreases most rapidly. This direction was taken to be normal to the fitted first order response surface (Anderson et.al, 2016). The steps along the path were calculated by taking the proportions of the regression coefficients $\beta$ obtained from the first order regression equation. The simulations were conducted along the path of steepest descent until no further reduction in response was observed. This was taken as sign of arrival of the optimal region.

**3.4.3.2.2 Numerical experiment design**

Once the region of optimum has been set, a more elaborated second order model was employed. Prior to generating polynomial equations for effluent concentration, identifying the combination of variables to be simulated has to be selected. As these data would highly affect the nature of the polynomial equation to be generated. In this study, central composite design (CCD) was used for experimental design since a second order polynomial was the equation to be fitted. The operation variables were considered at three levels namely, low (−1), central (0) and high (1). Twenty simulations of the set of all combinations of these variables were performed using the validated model.

**3.4.3.2.3 Fitting of polynomial function**

The simulations results were then used to generate second order polynomial equation that shows the relationship between the parameters and the response of the plant. $NH_4$-N, $NO_3$-N and Total Nitrogen concentration was considered as major responses.

Each response was represented by second order polynomial model as expressed in the Equation (3.8).

$$y = \beta_0 + \beta_i x_i + \beta_j x_j + \beta_{ii} {x_i}^2 + \beta_{jj} {x_j}^2 + \beta x_i x_j \quad (3.8)$$

Where y is response variable, x is input variable, i and j are the linear and quadratic coefficients, respectively, $\beta$ is the regression coefficient. The Design Expert Software (version 7.0) was used for the statistical design of experiments and data analysis, quadratic model buildings,

#### 3.4.3.2.4 Evaluation of the model equations

The quality of the fitted polynomial model equations was assessed by the lack of fit test, the coefficient of determination ($R^2$) and the adjusted coefficient of determinations (Adj. $R^2$). The significance of regression coefficients was checked by F-test using P=0.05 as significance level. These parameter values were estimated by the Design Expert Software in the analysis of variance (ANOVA) report. In addition to these tests, verification was carried out to determine the accuracy of the regression equations in describing the experimental data. The verification was done by examination of residuals. Residuals, which are the difference between observed and predicted responses, were examined using the plots of normal probability. The regression equations were taken to be adequate when the points on the normal probability plots of the residuals form a straight line

#### 3.4.3.2.5 Determination of the optimal values

Three-dimensional response surface and contour of the fitted equations were plotted using the Design Expert software. Although the 3D plots from the RSM showed the optimum values and also its location the exact values were solved by numerical method. This helped to avoid approximation error while reading.

One of the most commonly used approaches to solve the multi-response optimization problems is desirability function approach (Plakhov et.al, 2014), (Myers, et.al, 2016). The general approach of this method is to first convert each response into individual desirability function d that varies between 0 and 1.

The function equals to 1 as it approaches its goal or target. If the response is outside an acceptable region D becomes 0. The optimum parameters were then chosen to maximize the overall desirability D that is shown in Equation (3.9)

$$D = (d_1 * d_2 * \ldots * d_m)^{1/m} \qquad (3.9)$$

The individual desirability function is defined as follows

$$d = \begin{cases} 1 & Y_i \leq A \\ \left[\frac{Y_i - B}{A - B}\right]^r & A \leq Y_i \leq B \\ 0 & Y_i \geq B \end{cases}$$

Where m is response number, $Y_i$ is response variable, A is the minimum allowable value of $Y_i$ and B is the limit value of $Y_{i.,}$ r is the weighted value.

If a response value ($NH_4$-N or $NO_3$-N concentration) was more than B for a set of parameter combination, then d=1, and if $Y_i$ was less than A, then d=0. The optimum parameters were the ones that have an overall desirability, D closer to 1.

In this study reducing the effluent concentration of nitrate without excessive increment of ammonia concentration was the goal of the optimization. Hence, response functions of nitrate and nitrogen of Equation 3.8 were converted to desirability equations. To define the desired region both the Ethiopian Environmental Protection Agency limit and the more conservative USEPA National Pollutant Discharge Elimination System (NPDES) discharge limit was adopted. The first was used as upper limit whereas the second was set as lower limit. The objective function of the multi-response optimization problem is formulated as shown in Equation (3.10)

$$\text{Maximize } D = (d_1 * d_2)^{\frac{1}{2}} \qquad (3.10)$$

Subjected To=

$$d_1 = \begin{cases} 1 & Y_{NH4\text{-}N} \leq 2 \\ \frac{Y_{NH4-N} - 30}{20 - 30} & 2 \leq Y_{NH4\text{-}N} \leq 5 \\ 0 & Y_{NH4\text{-}N} \geq 5 \end{cases}$$

$$d_2 = \begin{cases} 1 & Y_{NO3\text{-}N} \leq 10 \\ \frac{Y_{NO3-N} - 20}{10 - 20} & 10 \leq Y_{NO3\text{-}N} \leq 20 \\ 0 & Y_{NO3\text{-}N} \geq 20 \end{cases}$$

# 4 Results and Discussions

## 4.1 Wastewater characteristics

Characteristics of feed water that passed through grit removal of Mekenisa Kotari MBR plant during 18 March–21 June 2014 are summarized in Table 4-1 whereas the COD and Nitrogen fraction are shown in Figure 4-1. This categorization is based on municipal wastewater characterization of STOWA protocol, as already explained in sub-chapter 3.2.1. The average values presented were calculated as an arithmetic mean of the data collected at the different sampling dates. Even though the loading rate is the same with design value the influent characterization results of the wastewater are higher than the typical wastewater characteristics values reported in (Henze et.al, 2000). This deviation can be explained by the variation of the strength of sewage with per capita water consumption (Agarwal, 2009). As the water supplied for Mekenisa Kotari condominium is intermittent by its nature, the increased concentration during the dry weather is related with low per capita water consumption. Table 4.1 shows the dilution effect in the wet weather. This result manifested that even though the sewer line in Mekenisa MBR plant is a separate system there is an intrusion of rainwater.

*Table 4-1 Influent and Effluent characteristics*

| Parameter | Dry Weather | | | | Wet Weather | | | |
|---|---|---|---|---|---|---|---|---|
| | Influent (mg/l) | | Effluent (mg/l) | | Influent (mg/l) | | Effluent (mg/l) | |
| | *Average* | *St.Dev.* | *Average* | *St.Dev.* | *Average* | *St.Dev.* | *Average* | *St.Dev.* |
| **Total COD** | 1444.0 | 296.8 | 37.5 | 10.2 | 593.7 | 79.2 | 17.1 | 3.8 |
| **COD Soluble** | 505.4 | 103.9 | | | 137.9 | 19.9 | | |
| **$BOD_5$** | 558.8 | 146.9 | 12.2 | 5.1 | 248.5 | 49.6 | 5.4 | 0.8 |
| **TKN-N** | 114.8 | 54.1 | | | 87.3 | 11.8 | | |
| **$NH_4$-N** | 84.2 | 37.4 | 0.4 | 0.12 | 61.13 | 8.3 | 0.57 | 0.3 |
| **NO3-N** | less than one | | 82.7 | 30.1 | less than one | | 59.7 | 6.6 |
| **TP** | 18.04 | 3.95 | 7.81 | 1.72 | 9.82 | 2.1 | 4.419 | 3.64 |
| **TSS** | 992 | 202.62 | 5.63 | 2.90 | 324.78 | 11.00 | 2.67 | 1.70 |
| **Temperature** | | | 25.95 | 0.45 | | | 22.83 | 0.62 |
| **PH** | 6.84 | 0.01 | | | 7.16 | 0.30 | | |

The COD and nitrogen fractions for influent wastewater are represented in Figure 4-1. The soluble COD fraction ($S_S$+$S_I$) was 35%of total COD. 94% the soluble COD was the slowly biodegradable matter. The particulate COD fraction was 65% of the total COD. The biodegradable particulate COD fraction ($X_I$) was 44% of the total COD. The non-biodegradable inert COD is closer to the border of the acceptable range. From this, it can be speculated that the inert matter may affect the filtration performance of the MBR.

The dominant nitrogen species was ammonia ($S_{NH}$) (72.6% of total nitrogen) Figure 4-1. Its acceptable range is between 60–75% of total nitrogen (Roeleveld et. al, 2002). The fractions of organic particulate ($X_{ND}$) and soluble nitrogen ($S_{ND}$) were 21.7% and 9.52%, respectively. Other nitrogen species, such as nitrite and nitrate ($S_{NO}$), inorganic nitrogen factions were small, and likely to be measured with the fraction of inorganic COD.

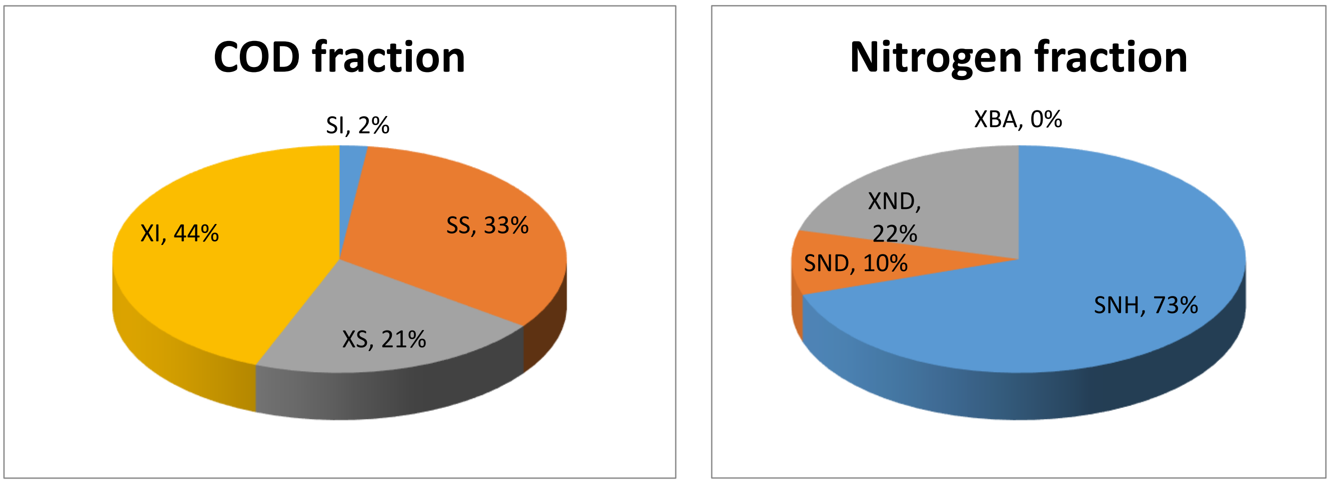


*Figure 4-1 COD and Nitrogen fractions in the influent wastewater*

In general, the COD fractions from the Mekenisa Kotari wastewater were similar to those from previous reports, as shown in Table 4-3, except for the inert particulate matter ($X_I$). The difference can be attributed to the nature of the methods: on the one hand, the respirometric based Hochschulgruppe (HSG) characterization protocol and, on the other hand, the more practically-oriented STOWA protocol. The difference was taken to be acceptable as inert particulate matter was recommended to be a calibration parameter because of the high uncertainties it has during measurements (Abusam, 2014), (Henze et.al, 2000).

## 4.2 Membrane Biological Reactor Performance

In order to assess the existing performance of the MBR both the effluent quality and filterability performance were taken into consideration. To assess the effluent quality in addition to the influent characterization effluent sampling was carried out and the result is summarized in Table 4-1.

Generally, an efficient preliminary treatment unit that replace primary sedimentation tank at small WWTP removes from 25% to 40% of suspended solids and 25% to 50% of BOD (Metcalf & Eddy, 2007). In Mekenisa Kotari Treatment Plant the removal efficiency achieved by the bar screens, fine screens and sand separation unit, based on the average wastewater characterization, was only 21% of solid removal and 13% of BOD removal. Compared to the recommended value the efficiency of the preliminary treatment unit was found to be less. Consequently, this might cause membrane fouling problem in the membrane operation.

Focusing on the MBR process, the removal efficiency was calculated by considering organic matter, nitrogen, and phosphorous removal. The organic matter average removal efficiency achieved for the system was 97%. The nitrogen removal efficiency was estimated for nitrification and denitrification process independently. Nitrification was almost always complete, with an average $NH_4^+$–N removal efficiency of 99%. On the other hand, the denitrification process was less efficient, with $NO_3$–N concentration values varying in the permeate flow from 71 to 123mg/l. This denitrification inefficiency was correlated to the absence of anoxic tank in the treatment plant. For the reduction of nitrate to nitrogen gas to occur, the DO level must be at or near zero. Therefore, in that case, the bacteria could use the available excess nitrate as electron acceptor and together with the carbon source available conversions to nitrogen occur. Since Mekenisa Kotari Treatment Plant is not designed to have an anoxic area no denitrification has taken place and consequently the nitrate concentration in the effluent is way above the standard value.

Nitrate has serious health effects when it enters drinking water wells and consumed. Excess nitrate in water is related to several diseases such as “blue baby disease” occurring in infants. Moreover, it may induce mutations of DNA, causing gastric cancer. High nitrate concentration can also have poisonous effects on the environment (Radjenović et al., 2008). Due to these serious health problems even if the ammonia nitrogen removal was efficient, the enhancement of denitrification capacity of the plant needs much attention.

Even though the treatment plant has neither anaerobic tank nor any chemical was applied for phosphorous removal, the plant’s total phosphorous removal efficiency was approximately 58% in both testing period. This removal is attributed to particulate phosphorous removal during membrane filtration. Wastewater treatment plants with no specific phosphorous removal mechanism are expected to have 60-70% removal efficiency (Dueña et.al, 2003). Hence, the performance of the MBR is satisfactory. Nevertheless, the effluent concentration did not comply with Ethiopian Environmental Protection Agency regulation to inland water disposal. The authority permits disposal of treated wastewater with 5mg/l of dissolved phosphorous. The effluent from Mekenisa Kotari Treatment Plant had more than 5mg/l total phosphors concentration (Table 4-1).The problem was worse during the dry period. As the effluent is used neither for gardening nor irrigation but discharged to a river it causes eutrophication to the water body. This suggests that either a chemical or biological treatment process for phosphorous removal is required in order to satisfy the disposal limit.

Membranes can remove biological and non-biological particles by sieving and adsorption. Hence effluent from MBR is expected to have bacterial reduction. The quality performance monitoring of the treated wastewater has shown the wastewater to be completely clear of fecal coliform and 16 per 100ml of total coliforms. These results agree with what was expected from membrane rejection ability (Judd, 2008). Hence, the high quality provides options of reuse of the treated wastewater. In addition to the effluent quality, the filtration condition of the MBR system has as much, if not more than, relevance for the wellbeing of the treatment unit. Hence, the membrane system characteristic has been assessed by analyzing the operating values of trans-membrane pressure (TMP), flux and permeability.

The average daily TMP was recorded and plotted as shown in Figure 4-2. From the graph it can be concluded that the average operating pressure was 0.1bar. The maximum TMP was also strictly maintained below 0.2 bar as recommended by (Judd, 2011).

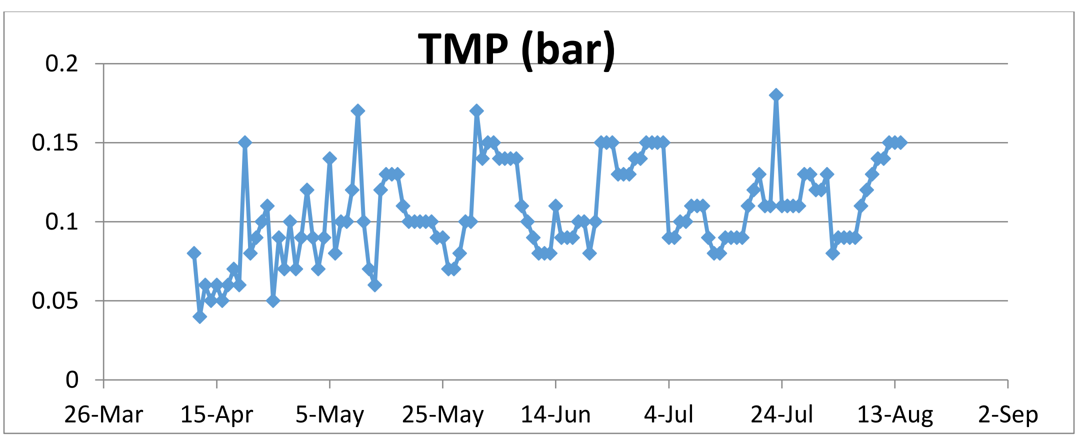


*Figure 4-2 Daily average trans-membrane pressure recorded during the testing period*

The second filterability performance indicator considered was the net flux which is the total permeated flow over 24hr divided by the total installed membrane surface area. Figure 4-3 shows the volume of treated wastewater produced by the process. The maximum treated volume throughout the testing period was only 450m$^3$. This volume represents only half of the total capacity of the plant. Even though the plant was operating with acceptable TMP, this was achieved by highly compromising the design flow rate. In dry period this might be due to the low water consumption of the community but in the wet season the reduced permeated volume can only be explained by membrane fouling.

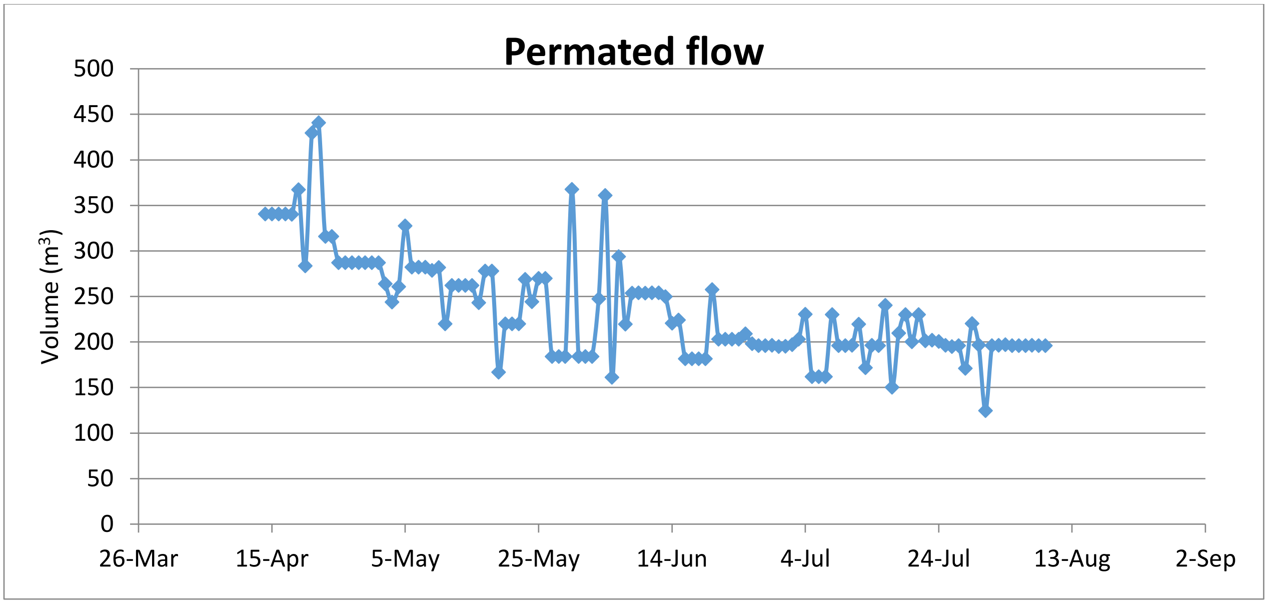


*Figure 4-3 Volume of water produced per day*

The low permeated volume was also justified by comparing the flux to standard values. (Hai et.al, 2014) states that, MBRs should generally operate at fluxes between 10 and 150 LMH. It can be shown from Figure 4-4 that Mekenisa Kotari MBR was operating below 10 LMH for most of the testing period. The decreased flux and permeated volume show the existence of membrane fouling.

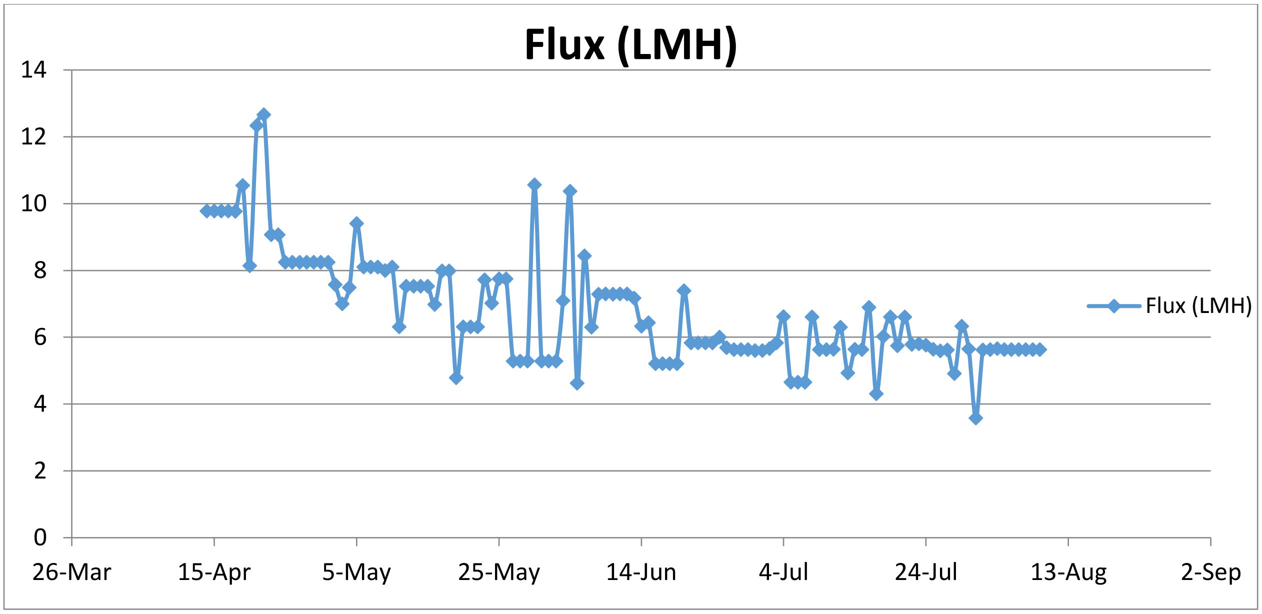


*Figure 4-4Net flux recorded for the testing period*

The third performance indicator assed was permeability that is the flux rate divided by TMP. Since permeability relates both flux and TMP, it helped to get the general effect of both. Figure 4-5 show the daily average permeability achieved by the filtration system. Comparing the graph to what is recommended for well operating MBR by (Judd, 2011) the plant has much lower permeability capacity. For well-functioning domestic MBRs, it is recommended to have permeability between 150 and 250 LMH/ bar (Judd, 2011). The MBR plant had permeability values within this range until mid of May.

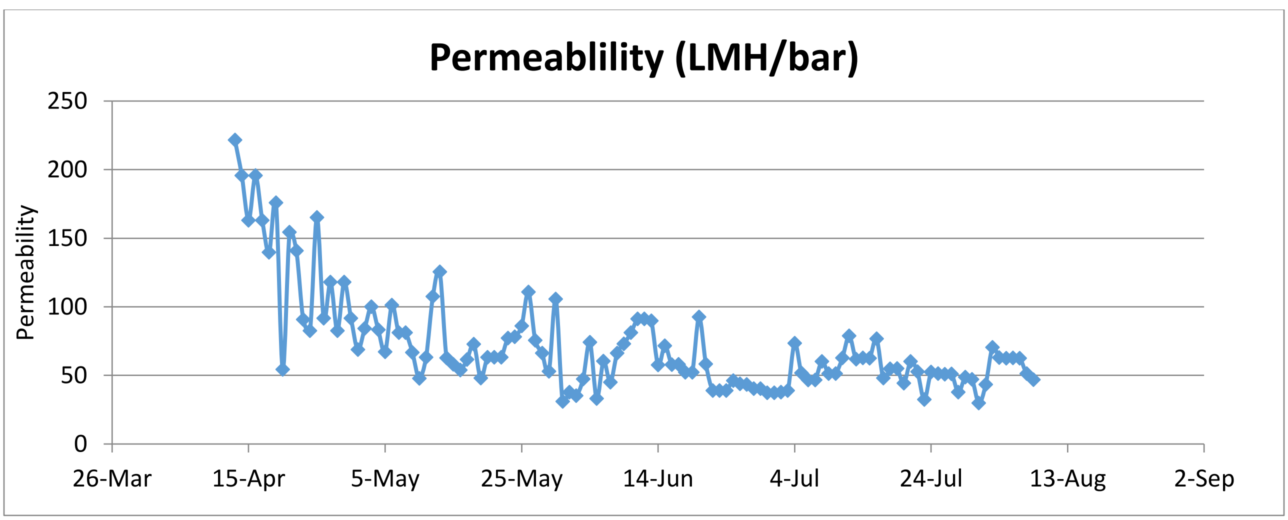


*Figure 4-5 Permeability achieved by the filtration system*

One of the reasons for the low permeability and consequently for the fouling problem is the characteristics of the solute in the wastewater. As stated in the wastewater characteristics result the influent water to the biological unit had high inert COD concentration. This might be due to the inefficiency of the preliminary treatment or absence of the primary sedimentation tank. Fouling of membrane for MBR plants without primary sedimentation tank has also been reported in other studies. (Jimenez et.al, 2010) operated two MBR plants side by side under the same operating conditions, one fed by screened (1 mm screen) raw municipal wastewater and the other by primary settled wastewater. The screened wastewater was found to contain 30% more solid. Membrane wear was also found to be worse for the MBR with screen only.

The low permeability capacity of the MBR has already forced the treatment plant to use its emergency line for the excess wastewater. Since the emergency line is directly connected to the water body untreated wastewater was joining the river during sometimes of the wet season. If no measure is taken, over a period of time the flux might reduce to the point where membrane filtration cannot be used for sludge separation and replacement of membrane might be need.

## 4.3 Model -based process optimization

In order to suggest upgrade options that could enhance the performance of the Mekenisa Kotari MBR plant, model-based optimization was carried out. Since the performance of the plant was not satisfactory regarding total nitrogen removal the optimization was focused on improving efficiency of the total nitrogen removal.

### 4.3.1 Sensitivity analysis

Even though the model contains large parameter set, it was necessary to consider only the most influential parameters for calibration. In order to simplify the calibration procedure the sensitivity of calibrated parameters was analyzed using Equation (3.3) (Reichert, 1998). The sensitivity of the sludge production, concentrations of ammonium and nitrate in the effluent for all parameters was analyzed. The sludge production was mainly influenced by distribution of COD over slowly biodegradable and inert organic matter. This result coincides with (Hulsbeek et al., 2002) study and it also confirms the suggestion of STOWA protocol for adjusting sludge production in steady state calibration. As mentioned in section 2.3.2.3 the protocol suggests modifying the influent fraction for calibration of sludge composition and production. Besides the influent composition, the value of $Y_H$ also influenced the sludge production. The effluent ammonia concentration appeared to have a relative sensitivity function of more than one towards only $\mu_A$ , $b_A$, $K_{NH4}$. The sensitivity of $K_{O_2H}$, $b_h$ and $\eta_g$ is found to be high toward nitrate concentration in the effluent.

*Table 4-2 Sensitivity analysis ranking result*

| | Ranking | Parameter | RSF |
|---|---|---|---|
| **MLSS** | 1 | COD fraction | -0.8639 |
| | 2 | $Y_H$ | 0.6729 |
| | 3 | $b_H$ | -0.08319 |
| | 4 | $X_I/COD_{tot}$ | -0.06135 |
| **$NH_4$-N** | 1 | $\mu_A$ | -1.955 |
| | 2 | $b_A$ | 1.559 |
| | 3 | $K_{NH4}$ | 1.267 |
| **$NO_3$-N** | 1 | $\eta_g$ | -0.3835 |
| | 2 | $K_{O_2H}$ | 0.38 |
| | 3 | $b_h$ | -0.09 |
| | 4 | $K_{NO}$ | 0.02 |

### 4.3.2 Model calibration and verification

Following the evaluation of the most sensitive parameter, the model of the WWTP was calibrated. First, the solid was fitted based on the average measurement taken on dry season for mixed liquor suspended solid. Next, the nitrification and denitrification were calibrated again based on the average measurement of ammonia and nitrate concentration respectively.

The simulated MLVSS in pre-aeration and MBR tanks using default value of ASM1 and the influent fraction from wastewater characterization were slightly underestimated. In order to calibrate this value with the measured value the influent composition was adjusted as shown in Table 4-3. Table 4-3 shows the evaluated influent COD fractions using the STOWA protocol and the adjusted fraction after calibration. The values found are comparable values with previous studies. Comparison to both default municipal influent COD fractions and to other MBR influent COD fractions is included in Table 4-3. The fraction found from the calibration of solid lay between the default range of municipal wastewater fraction that is suggested by (Hulsbeek et al 2002). The inert particulate matter fraction value is closer to the upper boundary of the range of municipal wastewater, this is likely due to the fact that the adsorbtion of soluble colloidal material to the biomass, and since it is not biodegraded, it remains as particulate inert material (Pereira, 2014). Since the heterotrophic biomass yield ($Y_H$) was the second most sensitive parameter, next to the influent fraction adjustment $Y_H$ was also used for calibration. It was increased from its default value 0.67 to 0.68causing an increase of the MLSS due to the higher biomass yield.

*Table 4-3 Summary of the evaluated COD influent wastewater fraction and comparison against other municipal wastewater MBRs*

| Influent fraction | WW characteristics (This study) | From Fitting sludge production (This study) | (Delrue et.al, 2010) | (Abusam, 2014) | Default range of muncipal wastwater (Hulsbeek et al 2002) |
|---|---|---|---|---|---|
| $X_S$ | 21 | 23.32 | 31.4 | 20.9 | 10-48 |
| $X_I$ | 44 | 47.3 | 28.9 | 45 | 23-50 |
| $S_S$ | 33 | 27.76 | 37.6 | 32.39 | 9-42 |
| $S_I$ | 2 | 2.49 | 2.1 | 1.7 | 2-10 |

Three parameters were used to fit effluent ammonia concentration to measured values. These parameters included$\mu_A$, $b_a$ and $K_{NH4}$ (Table 4-4). Maximum specific growth rate for autotrophic biomass ($\mu_A$) was set to 0.82$d^{-1}$ to obtain a reasonable model fit. This value is slightly higher than the default value of ASM1. This was done to promote the growth of autotrophic bacteria and consequently decrease the concentration of ammonia. The decay coefficient of autotrophs ($b_a$) was changed to 0.17. Even if this value lay within the default range it is higher than the typical value of ASM1. The observed high $b_h$ is probably due to the high turbulence existing in the MBR since nitrifiers are generally regarded to be more sensitive to critical environmental conditions. Another study which was done on side stream MBR (Jiang, et. al, 2005) has also found a higher value of $b_H$ than the default values in ASM1. The second parameter in calibration of ammonia was the half saturation coefficient for autotrophs ($K_{NH4}$). $K_{NH4}$ was decreased from 1 to 0.78. The adjustment of this parameter also decreased ammonia concentration of the effluent. Since the simulated result using default parameter values was over estimated, the adjustment helped to fit the measured value better.

In order to meet the real nitrate effluent characteristics with sufficient accuracy, the following ASM1 kinetic parameters were adjusted:$\eta_g$, $b_h$, and $K_{OH}$. $\eta_g$. $b_h$ and $K_{OH}$ were decreased to 0.53 0.42 and 0.04 respectively. Most of these values are related with growth of heterotrophs under anoxic condition. Since the absence of anoxic tank hinders the growth, the decrement of these factors was expected. As observed in Table 4-4 the other parameters changed are in good agreement with literature done on MBR plant.

*Table 4-4 Set of calibrated and default ASM parameters for MBR*

| | **Yh** | $\mu_A$ | **$b_a$** | **$K_{NH4}$** | **$b_h$** | $\eta_g$ | **$K_{OH}$** | **Koa** | $\mu_H$ |
|---|---|---|---|---|---|---|---|---|---|
| **Default for CAS** | 0.67 | 0.8 | 0.15 | 1 | 0.62 | 0.8 | 0.2 | 0.4 | 6 |
| **Delure for MBR** | 0.67 | 0.8 | 0.1 | 0.25 | 0.62 | 0.8 | 0.03 | 0.3 | 6 |
| **This study** | 0.68 | 0.82 | 0.17 | 0.78 | 0.42 | 0.53 | 0.04 | 0.4 | 6 |

To test model fit to the measured data, a Chi-Square test for goodness of fit was done. The null hypothesis ($H_0$) was that observed and predicted values are unequal. The alternate hypothesis ($H_a$) was observed and predicted values have the same distribution. The null hypothesis ($H_0$) is rejected in favor of the alternate hypothesis ($H_a$). This is done with a 95% confidence level. The standard error estimate (SEE) measures the overall error made by the model using the units of the parameters. The value of the standard error obtained was small compared to the average concentrations of each parameters (Table 4-5).

*Table 4-5 Summary of statistical analysis of Chi-Square test for goodness of fit at a 95% confidence level in relation to SEE*

| **Parameter** | **Chi-Square Test** | | **SEE** |
|---|---|---|---|
| | **p-value** | **p- value <$\alpha$** | |
| **Effluent COD** | <0.00001 | ReH | 0.28 |
| **MLSS in pre-aeration** | 0.004 | ReH | 130 |
| **MLSS in MBR** | <0.00001 | ReH | 5.62 |
| **Effluent $NH_4$-N** | <0.00001 | ReH | 0.102 |
| **Effluent $NO_3$-N** | 0.00063 | ReH | 1.56 |

Note ReH = Reject $H_o$ Hypothesis; $\alpha$ to all expression = 0.05

However, even though the calibration was successful with only nine parameters (Figure 4-6), most likely other parameter sets can also provide reasonable simulation results. This is mentioned as the challenge for calibrating ASM models, which are over parameterized in literature (Urdalen, 2015). For a more accurate calibration, it would have been desirable to perform off-line laboratory batch experiments to estimate the values of parameters such as $Y_H$ and $\mu_H$. However, measurement of each parameter was laborious due to complexity of experiments and large number of total parameters.

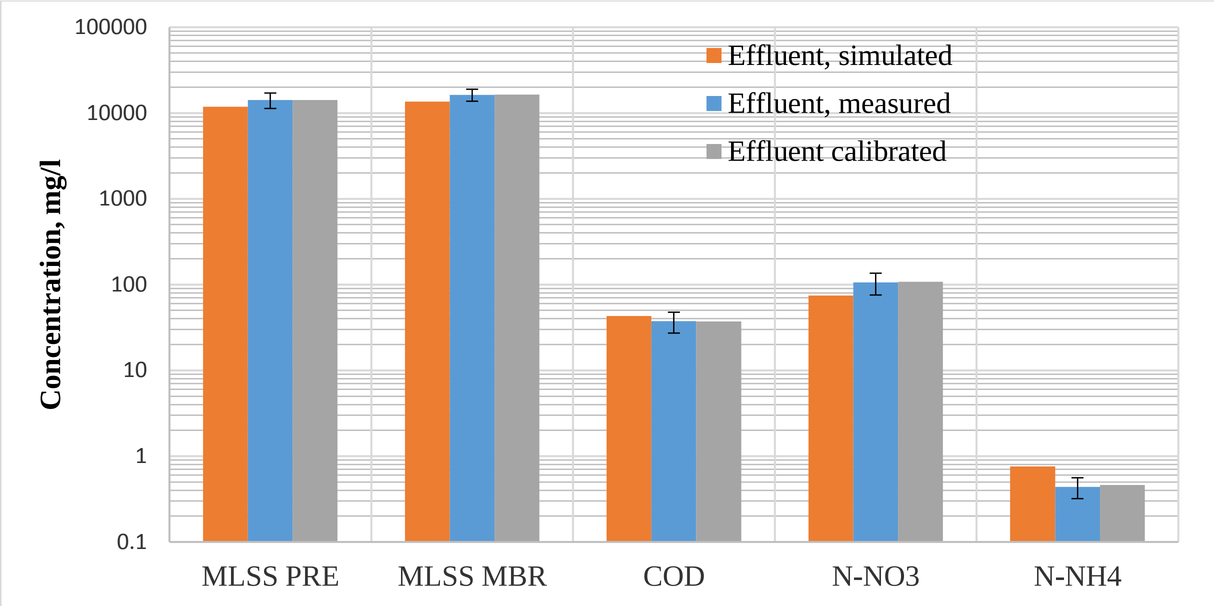


*Figure 4-6 Measured and Calibrated concentrations*

Samples taken in the wet season were used for model verification process. Since the wet season samples had different conditions (flow and temperature) the data was assumed to be independent of the one used for calibration (Brdjanovic et al., 2000). Predicted values of COD, MLSS, $NH_4$-N, and $NO_3$-N concentration from the calibrated model were compared to the observed values as shown in Table 4-6. Overall, the model was able to provide a satisfactory description of the plant performance as observed by the relatively good agreement between the model prediction and the average measured values. Hence, it was concluded that the model can achieve the performance requirement needed for further analysis.

*Table 4-6 Result from model validation*

| Parameter | Unit | Influent | Effluent (model prediction) | Effluent (measured value) | Model Deviation |
|---|---|---|---|---|---|
| **Total COD** | mg/l | 593.7±79.2 | 16.28 | 17.1±3.8 | 4.79% |
| **NH4-N** | mg/l | 61.13±8.3 | 0.52 | 0.57±0.3 | 8.7% |
| **$NO_3$-N** | mg/l | 0.03[a] | 66.1 | 59.7±6.6 | 10.7% |
| **MLSS (pre-aeration)** | g/l | | 14.67 | 15.82±2.6 | 7.2% |
| **MLSS in MBR** | g/l | | 17.20 | 17.79±2.8 | 3.3% |

[a] In the influent wastewater the nitrate concentration has low values and due to this the standard deviation is not presented in the table.

### 4.3.3 Optimization

Modeling of the wastewater treatment plant was implemented in order to analyze the performance of the treatment plant under different scenarios and also to better optimize and control operational parameters. Optimization of operational parameters was performed using response surface method.

#### *4.3.3.1 The steepest descent*

Prior to applying response surface method for optimization, the steepest descent method was used to locate the optimum region of the variables. Based on the coefficients in the first order model obtained from fractional factorial design experiments the path of steepest descent was applied to find the proper direction of reduction (Equation (4.1)).

$$Total\ nitrogen\ (mg/l) = 59.21 - 22.15AF - 14.76CTR + 12.11SRT \tag{4.1}$$

The steepest descent was calculated by starting at the center of the factorial design and changing factors in proportion to the coefficients of the coded fitted equation. In this case, the path was moved along a vector that went down 22.53 units in AF, down 14.76 units in CTR, and up 12.11units in SRT. Data used to fit the first order polynomial equation and detailed calculation of the steepest descent are included in Annex B. Experiment design based on the steepest descent and the corresponding responses are represented in Table 4-7. After run number 4 no further reduction in response was observed hence aeration fraction interval between 0.41 and 0.54, cycle time ratio interval 0.5-0.6 and SRT was 16-18 was taken for central composite design.

*Table 4-7 Experimental design of steepest descent and corresponding responses*

| Run No | AF | CTR | SRT (days) | Total Nitrogen (mg/l) |
|---|---|---|---|---|
| **1** | 0.15 | 0.46 | 19.2 | 82.78 |
| **2** | 0.28 | 0.50 | 18.6 | 77.27 |
| **3** | 0.41 | 0.53 | 17.5 | 73.46 |
| **4** | 0.47 | 0.55 | 17.0 | 23.67 |
| **5** | 0.54 | 0.57 | 16.4 | 36.50 |
| **6** | 0.67 | 0.59 | 15.6 | 46.38 |
| **7** | 0.80 | 0.66 | 14.8 | 63.13 |

Table 4-7 shows that maximum total nitrogen concentration was exhibited under two scenarios; in low and high aeration time fraction. The first case was caused due to short time aeration. This makes the aerobic period insufficient for growth of nitrifiying bacteria which leads to the release of untreated ammonia with effluent.

The second cause of high total nitrogen effluent concentration is too much aeration. Although high aeration fraction results in efficient nitrification process, because the non-aeration fraction is short it also leads to accumulation of the generated nitrate. Accordingly, the incomplete denitrification process leads to high nitrogen concentration.

Since the aeration strategy at Mekenisa Kotari Treatment Plant is continuous (aeration fraction=1), the last two rows of Table 4-7 better manifest the operation strategies adopted at the treatment plant. This result showed that in order to have high nitrogen removal efficiency the aeration should be minimized. But it should also be high enough in order to carry out nitrification process.

#### *4.3.3.2 The second order polynomial*

The vicinity of the optimum points was determined by steepest descent combined with factorial design. Thereafter the optimization of total nitrogen removal for the three parameters was carried out. The first step in using the method of the optimization was to design an experiment that can model the quadratic effect of each factor. Designing the experiment mainly dealt with generating data to be fitted by regression model. The central composite design method was used to select the combination set of candidate points. Table 4-8 shows the 20 runs made in the validated ASM1 model and its result for COD, $NH_4$-N, $NO_3$-N and Total Nitrogen concentrations.

*Table 4-8 Central Composite Design in actual units of variable*

| Run No. | Aeration Fraction (AF) | Cycle Time Ratio (CTR) | SRT (days) | COD (mg/l) | $NH_4$-N (mg/l) | $NO_3$-N (mg/l) | Total Nitrogen (mg/l) |
|---|---|---|---|---|---|---|---|
| 1 | 0.45 | 0.5 | 16 | 38.06 | 13.61 | 10.76 | 24.37 |
| 2 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 3 | 0.5 | 0.55 | 18.68 | 37.89 | 4.95 | 17.81 | 22.76 |
| 4 | 0.54 | 0.6 | 16 | 37.39 | 4.37 | 22.24 | 26.61 |
| 5 | 0.45 | 0.6 | 18 | 38.06 | 10.65 | 12.18 | 22.83 |
| 6 | 0.5 | 0.63 | 17 | 37.89 | 5.76 | 17.05 | 22.81 |
| 7 | 0.42 | 0.55 | 17 | 39.41 | 49.05 | 4 | 53.05 |
| 8 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 9 | 0.54 | 0.5 | 18 | 37.72 | 3.64 | 23.89 | 27.53 |
| 10 | 0.45 | 0.5 | 18 | 38.19 | 18.16 | 15.06 | 33.22 |
| 11 | 0.5 | 0.47 | 17 | 37.81 | 4.8 | 18.9 | 23.7 |
| 12 | 0.45 | 0.6 | 16 | 39.03 | 28.95 | 13.21 | 42.16 |
| 13 | 0.5 | 0.55 | 15.32 | 37.97 | 5.69 | 17.68 | 23.37 |
| 14 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 15 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 16 | 0.54 | 0.5 | 16 | 37.72 | 3.74 | 23.89 | 27.63 |
| 17 | 0.57 | 0.55 | 17 | 37.79 | 3.39 | 27.29 | 30.68 |
| 18 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 19 | 0.5 | 0.55 | 17 | 37.89 | 5.17 | 17.85 | 23.02 |
| 20 | 0.54 | 0.6 | 18 | 37.79 | 4.14 | 22.11 | 26.25 |

The intermittent aeration did not have significant impact on COD removal yielding on an average efficiency of 97.33% (Table 4-8). Since carbon removal is carried out in both aerobic and anoxic condition, the unaffected result can be justified by that. In aerobic condition heterotrophic bacteria consume the carbon while in anoxic condition the COD is used as carbon source in the denitrification process However, the average TN removal was highly affected, varying between 67% and 82 %. This finding is also supported by (Adohinzin et.al, 2014) who conducted experiment on nutrient removal by intermittent aeration for lab scale MBR. COD removal were not affected by the aeration pattern, only nitrate concentration was varying in the experimental study.

Since the effect of the intermittent aeration was significant only on nitrogen removal the regression model was not fitted for carbon removal. The quadratic model equations from least square analysis of the data in (Table 4.8) is presented in Equation (4.2) and (4.3).

$$y_{NH4-N} = 440.56 - 3418.02AF + 1295.1CTR + 13.7SRT - 335AF * CTR + 33.5AF * SRT - 57.5CTR * SRT + 2838.2AF^2 - 123.2CTR^2 \quad (4.2)$$

$$y_{NO3-N} = -316.9 + 590.8AF + 205.5CTR + 12SRT - 150AF * CTR - 8.5AF * SRT - 13.65CTR * SRT - 246.5AF^2 + 83.02CTR^2 \quad (4.3)$$

#### *4.3.3.3 Statistical evaluation of the polynomial equations*

The statistical significance of the quadratic models was evaluated by the analysis of variance (ANOVA). The ANOVA result for all the models is summarized in Table 4-9. The high $R^2$ (0.95) indicates a good explanation of the variability by the selected models. Moreover, the models high F-value and prob >F close to 0 imply the models are significant on a confidence level of 95% (Table 4-9). Therefore, from this analysis the regression functions appear to be reliable models for predicting ammonia nitrogen and nitrate nitrogen concentration for any combination of the three variables.

*Table 4-9 ANOVA result for response parameter*

| Source | DF | | SS | | Mean SS | | F-value | | p-value | |
|---|---|---|---|---|---|---|---|---|---|---|
| | $Y_{NH4-N}$ | $Y_{NO3-N}$ | $Y_{NH4-N}$ | $Y_{NO3-N}$ | $Y_{NH4-N}$ | $Y_{NO3-N}$ | $Y_{NH4-N}$ | $Y_{NO3-N}$ | $Y_{NH4-N}$ | $Y_{NO3-N}$ |
| **Regression** | 9 | 9 | 13992.6 | 1734.3 | 1554.8 | 192.7 | 1713.2 | 156.2 | <0.001 | <0.001 |
| **Residual Error** | 10 | 10 | 9.1 | 12.3 | 0.9 | 1.2 | | | | |
| **Lack of fit** | 5 | 5 | 9.1 | 12.3 | 1.8 | 2.5 | | | | |
| **Total** | 19 | 19 | 14001.9 | 1746.7 | | | | | | |

Note: DF; degree of freedom, SS; the sum of the square, F-value; Fisher's test, p-value probability value

The purpose of the predicted equations was to visualize the removal efficiency surface and to get optimal values. However, before doing so the adequacy should be checked. The adequacy of the developed regression functions was evaluated by using diagnostic plot of normal probability plot of residuals. These plots helped to find out the relationship between the data predicted from the regression equation and the experimental response values. The data points on this plot lie very close to the diagonal line, which indicates the good agreement between the two data. Figure 4-7 shows the normal plot of residuals for ammonia-N, plot for nitrate nitrogen model is included in Annex B.

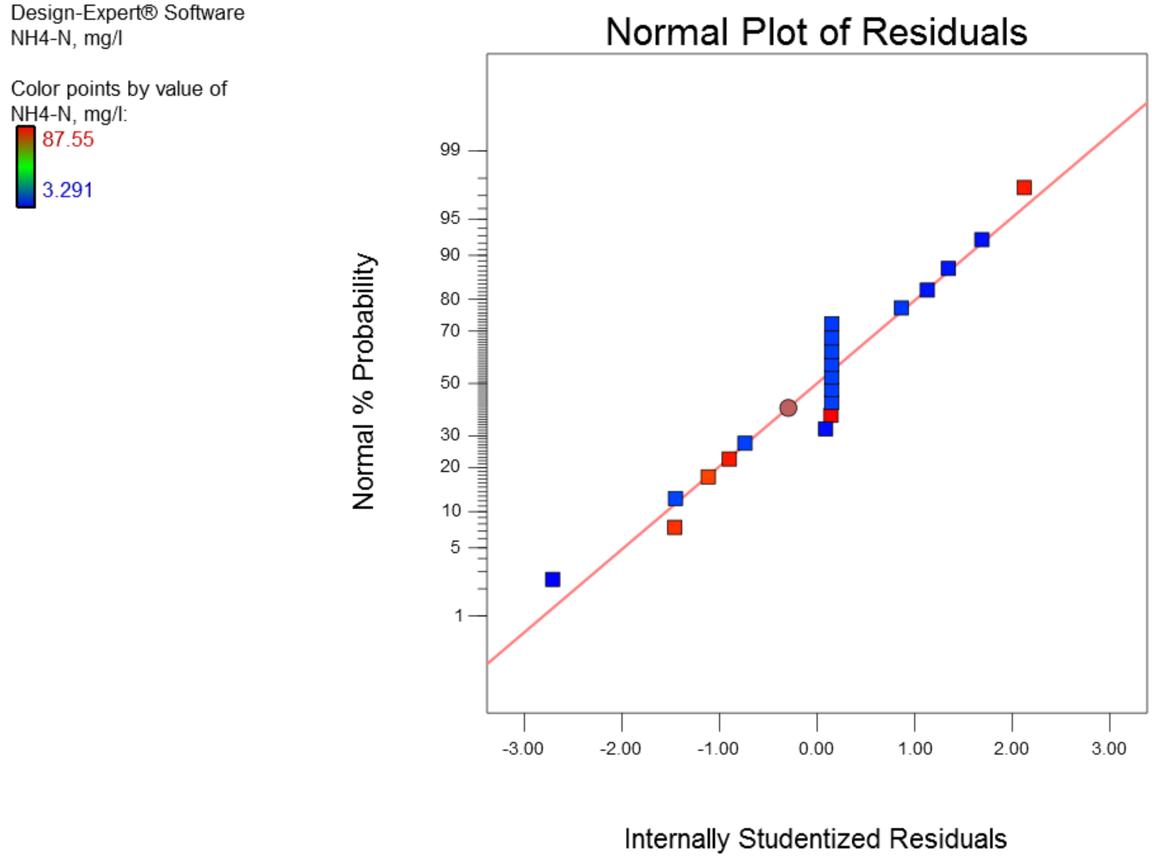


*Figure 4-7 Normal probability plot of residual*

#### *4.3.3.4 Optimal values*

Three-dimensional (3D) responses surface plots were constructed from the developed models in order to study the individual and interactive effect of the three variables (AF, CTR and SRT) on the response. These plots are graphical representation of the regression equations. All response plots show clear peaks implying that the optimum conditions of the response are attributed to AF, SRT, and CTR

Figure 4-8 shows the simultaneous effects of aeration factor and cycle time ratio on total $NH_4$-N concentration at a fixed sludge retention time of 17 days. As it can be seen in the Figure 4- rising the aeration fraction from 0.4 to 0.45 sharply decrease effluent ammonia nitrogen concentration. But after an inflection point between 0.47 and 0.53 the concentration start to increase with aeration fraction increment. In the range studied, lower cycle time ratio and higher sludge age values resulted in low effluent nitrogen concentration.

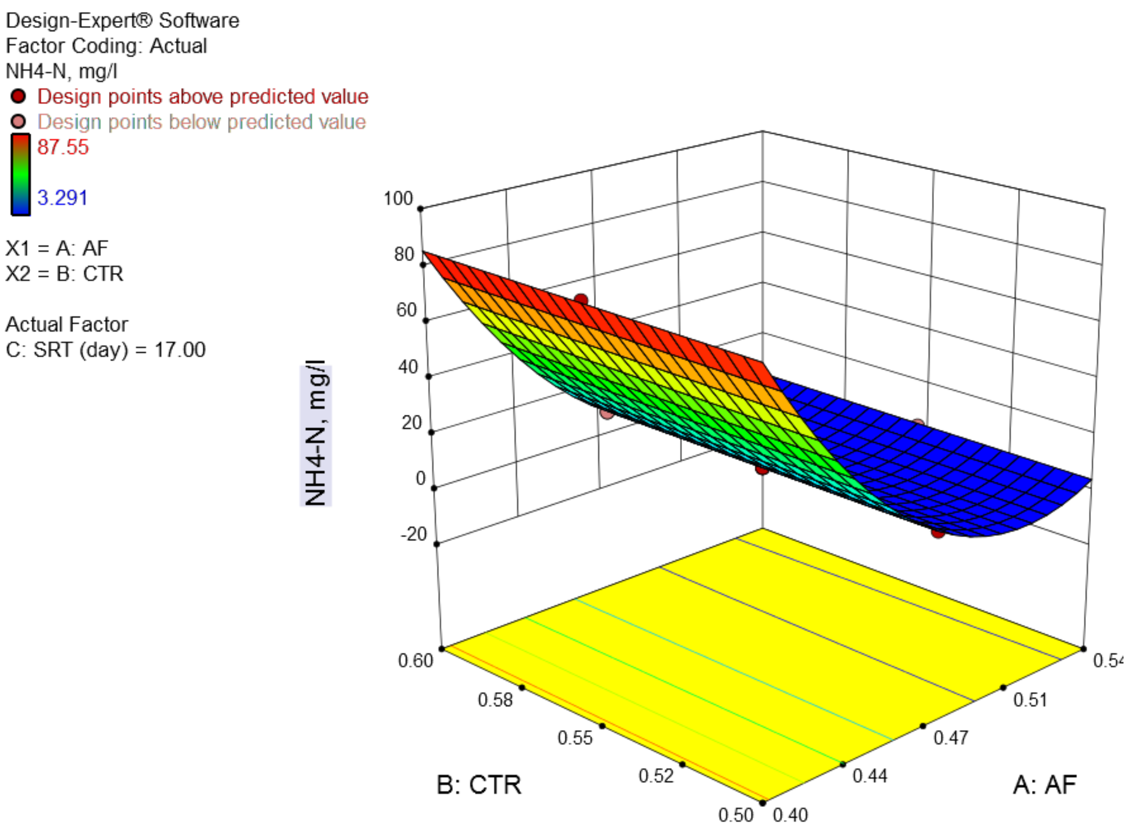


*Figure 4-8 Response surface curve showing the effects of CTR and AF on $NH_4$-N concentration in the effluent wastewater*

Figure 4-9 shows the combined effects of cycle time ratio and aeration factor on nitrate nitrogen concentration at a fixed SRT of 17 days. The plot shows direct proportion of nitrogen concentration and aeration factor. This could be justified by absence of anoxic period as aeration fraction gets closer to one. As aeration fraction increases the anoxic period within a cycle time decreases which leads to nitrate accumulation. Consequently, the nitrate nitrogen concentration increases in the effluent water. Similar finding has been found in pervious study (Habermeyer et.al, 2005).

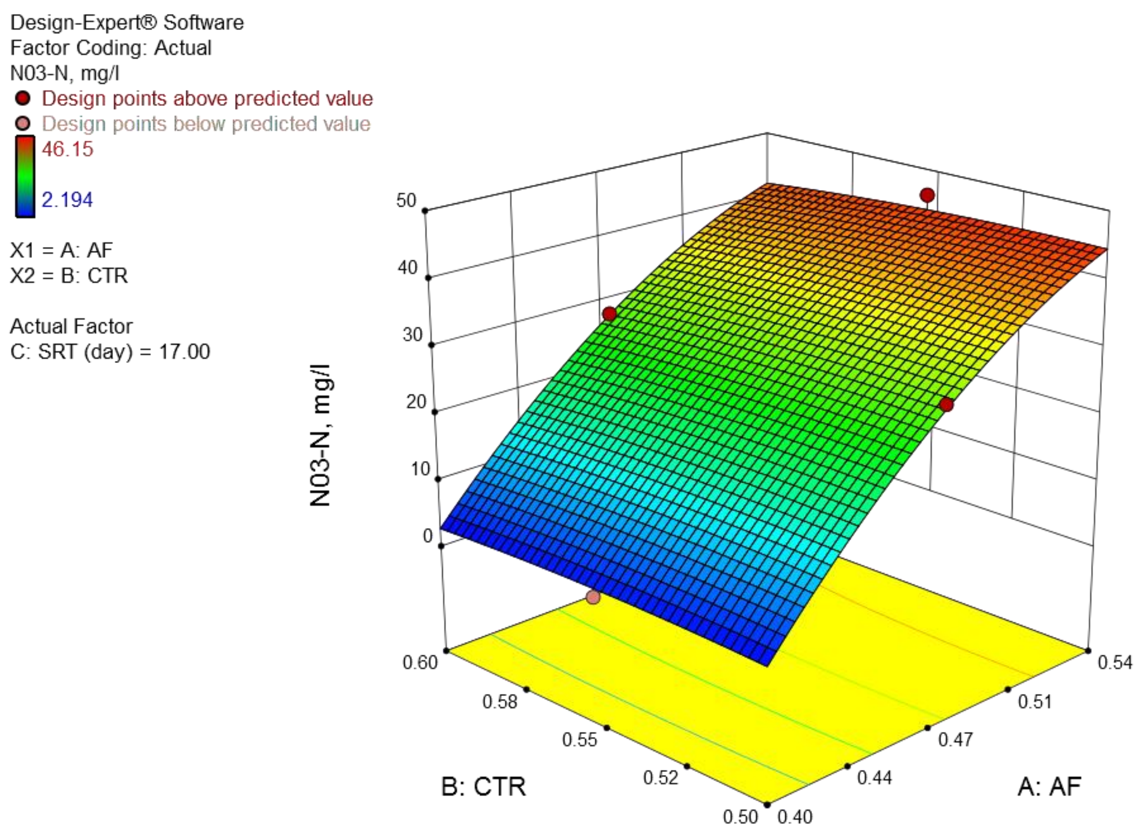


*Figure 4-9 Response surface curve showing the effects of CTR and AF on $NO_3$-N concentration in the effluent wastewater*

Figure 4-8 and Figure 4-9 show that at low aeration fractions, the average ammonia-N concentration rises whereas the average nitrate-N concentration decreases. This clearly calls for a need of an optimization problem to obtain the point at which the summation of ammonia-N and nitrate-N is minmum. At aeration fractions in excess of the optimum, nitrificaiton is complete. Howerever, the anoxic period is insufficient to allow much reduction of $NO_3$-N. Figure 4-8 shows aeration factor more than 0.5 is excessive aeration and hence waste of energy without siginificant improvement of treatment process. Contrary to this at aeration fraction less than the optimum, the aerobic period becomes insufficient for growth of the nitrifiying bacteria. For the MBR plant studied total process failure occur around 0.42 aertion fraction (Figure 4-8)

Besides analyzing the 3D and contour plots of the responses, numerical optimization has also been performed to determine the exact optimal values of aeration fraction, SRT and cycle time ratio that result the $NH_4$-N less than 5 and $NO_3$-N less than 20. Both limits were taken from Ethiopian EPA standards for disposal in inland water bodies. The numerical optimization finds a point that maximizes the desirability function. Table 4-10 presents the specific optimum condition from the numerical analysis.

*Table 4-10 Optimum values, prediction, and desirability of model*

| SRT | AF | CTR | COD (mg/l) | $NH_4$-N (mg/l) | $NO_3$-N (mg/l) | Total Nitrogen (mg/l) | Desirability |
|---|---|---|---|---|---|---|---|
| 18 | 0.49 | 0.55 | 37.9 | 4.76 | 17.02 | 26.28 | 0.68 |

The optimal results of the intermittent aeration were obtained when longer non-aeration time, 43minutes aerobic and 54minutes anoxic phase, was chosen. This implies that denitrification is the rate limiting step. This result is in line with other published experimental studies done on lab scale municipal MBR that has intermittent aeration pattern. Laboratory analysis MBR of 12hr HRT has given the best nitrogen and organic matter removal when the non-aerating time was two third of the cycle time (He et.al 2010).

The high cycle time ratio (CTR=0.55) which results in longer cycle time is an important feature of the optimization output as it gives enough time for biomass to shift from aeration to anoxic condition and vice versa (Grady et al., 1999).

Under the evaluated optimal conditions, both ammonia-N and nitrate-N concentration are within the desired range. In addition, the total nitrogen is 26.2mg/l which is below the EPA standard for total nitrogen concentration (80mg/l).

The optimization result from numerical analysis of the desirability function was verified with simulation of the calibrated ASM1 model. By taking the optimal values as input, the ASM1 model was simulated and the result closely matched the optimization output. Figure 4-10 shows the effluent concentration fluctuation when the proposed optimal operation conditions were used in ASM1. As mentioned above the COD removal is not affected by the aeration pattern. Regardless of operating conditions, more than 95% of COD were removed.

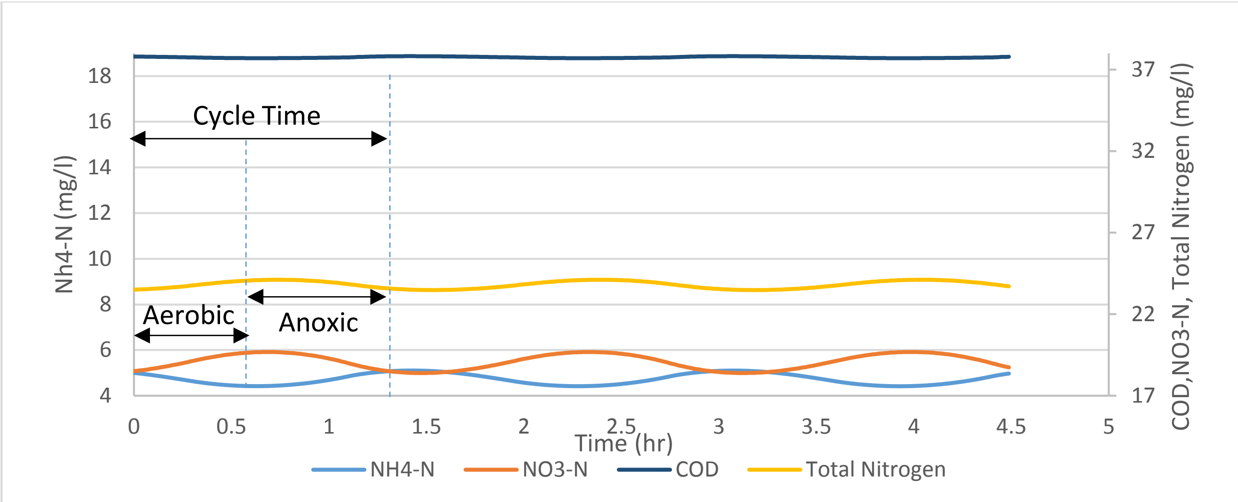


*Figure 4-10 Performance of the intermittent aeration*

Figure 4-11 shows the comparison of the current operation result at the treatment plant and the proposed intermittent aeration result. The COD concentration didn't show significant change. But the total nitrogen concentration was managed to be less than 80 mg/l which is the limit set by EPA for discharge of treated wastewater to inland water body. Ammonia nitrogen and nitrate nitrogen are also less than the respective limits. The intermittent operation not only improves the quality of the effluent water but also reduces energy consumption. The result obtained from this study shows that there is a 51% reduction in the total aeration time this reduce the run time of the blowers and accordingly the energy consumption of the treatment plant.

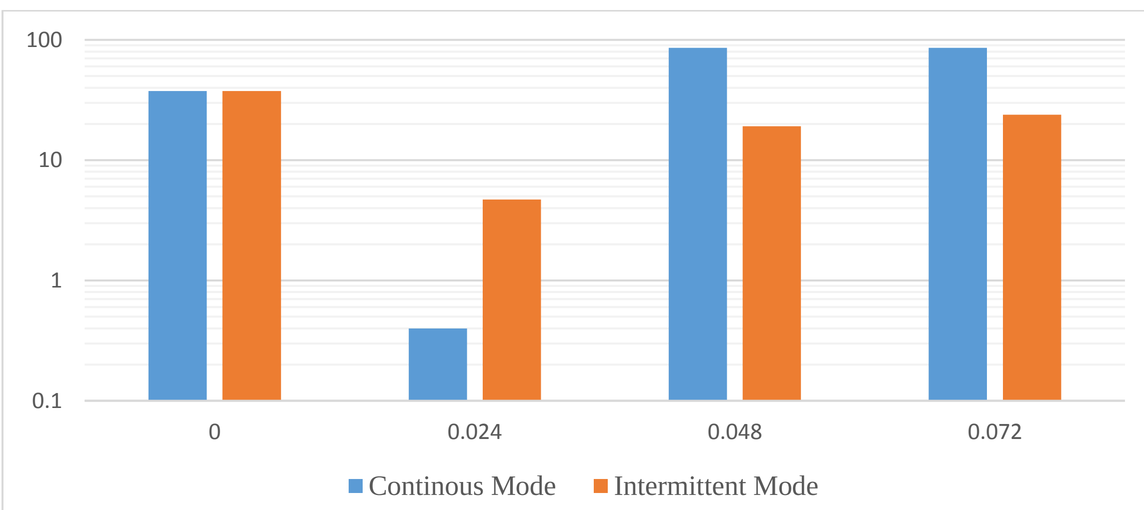


*Figure 4-11 A comparison of mean effluent concentrations during continuous and intermittent operation mode*

The result from this simulation proved that intermittent aeration is a competitive alternative for nitrogen removal. The proposed optimal operational parameters can increase the total nitrogen removal efficiency of the plant to 71%. Instead of intermittent aeration if an anoxic tank was proposed to be constructed, around 150 $m^3$ tank would have been required to get the same removal level (Hai et.al, 2014).

The modeling steps of this research did not include membrane filtration and fouling process because modeling fouling or removal of soluble compounds mechanisms is not a well-developed science yet (Maere, et.al, 2011). For this reason, assessing the effect of the intermittent aeration on filtration process was not one of the main objectives. However, cyclic filtration and intermittent aeration mode have been mentioned as bio fouling control methods in literatures (Hai et.al, 2014), (Verrecht et al, 2011). The use of intermittent aeration has resulted in lower fouling rates when compared to continuous aeration. The use of 10sec on/30sec off intermittent aeration result in lower fouling rates when compared to continuous aeration.

The cycle time suggested above for improving fouling are in seconds and seems very small compared to what is proposed in this research. But other studies has managed to control fouling even for higher cycle time. (Taylor et al., 2010) has used an experimental lab scale membrane bioreactor to assess nitrogen removal by intermittent aeration. This study was done with 180 minutes cycle time. While running the experiment for 150 days no significant sign of reduction in flux was exhibited. Air scoring and circulation of sludge were the two methods applied for mitigating fouling and sustaining constant filtration rate. In the aeration phase air bubbles from under the membrane through the diffusers were used, while in the non-aerated phase mixed liquor was pumped from the bottom on the membrane surface. Through this mechanism a stable membrane filtration was achieved even for long non-aerated phase in the membrane tank. Hence, even if further study is required to determine the performance of membrane filtration under cyclic aeration, from previous studies there is a good chance that the interment aeration could prevent fouling problem.

# 5 Conclusions and Recommendations

## 5.1 Conclusions

The purpose of this research was to investigate the performance of membrane bioreactor installed by AAWSA at condominiums. As the technology is new to the country and much is expected from it, the performance assessment and suggestion of operational strategies that could enhance treatment performance while reducing energy cost is crucial. Hence, both biological treatment and filtration system performance have been assessed in this thesis. In addition, optimal values of operational parameters have been evaluated. ASM1 biological model has been used for the optimization. It was calibrated using the STOWA protocol. In accordance with this study, the following conclusions were obtained.

- The raw sewage from the condominium site has generally medium to high range strength. This is related with the poor and intermittent water supply the site has. More than 60% of the influent water was found to be biodegradable which indicates that the wastewater is easily biologically treatable. The nitrogen component of the raw sewage is in ammonium nitrogen form.
- The quality of the MBR treatment unit effluent was generally very good and in alignment with expectations for effluents from municipal wastewater except for total nitrogen concentration. On average, treatment about 10 mg/l TSS and 30 mg/l COD concentrations (i.e., 85% removals of the total TSS and COD) were achieved during the testing period. Complete nitrification was always accomplished regardless the conditions in the system. Although the ammonium removal was 95%, nitrate concentration was well above the accepted value. The high nitrate concentrations of the effluent were expected, since the system is designed to remove neither nitrate nor phosphorus. Due to this both total nitrogen and phosphorus effluent criteria were not fulfilled.
- Filtration performance of the treatment plant was evaluated using permeate flux, trans-membrane pressure (TMP) and permeability as performance indicators. Even though the TMP was maintained below 20mbar in all operation period this was done by compromising the design flow rate. The permeate flow or the flux was declining below 10LMH during the last period of operation. This affected the permeability capacity of the plant; the

permeability was only 60LMH/bar. This value is very low compared to the standard for average operation of MBRs (150LMH/bar-250LMH/bar).

- To enhance the denitrification capacity of the plant the possibility of intermittent aeration was investigated using ASM1. The alternating aerobic/anoxic cycle process was able to improve the nitrogen removal efficiency. Adopting 43min aerated and 54min non aerated period was found to be the most optimal operational strategy. And it also resulted in 40% increment in total nitrogen removal efficiency and almost 55% of the energy consumed due to bioprocess aeration was saved.

## 5.2 Recommendations

The adsorption of solids on membranes results in either a reduction of permeate flux or an increase of TMP. In Mekenisa Kotari Treatment Plant the permeate flux has already shown a decreasing trend. Long period of operation under this condition might lead to irreversible membrane fouling and in due course replacement of membrane might be needed. This problem would likely be reduced by adopting the following two solutions.

- Pre-treatment of wastewater plays an important role in MBR systems (Yang et al., 2015) but installing primary clarifier is not feasible for the Mekenisa Kotari site as it is located close to the residential area and available land is limited. Hence, installing more advance and efficient fine screens before the MBR system could be a solution and it can also remove fibers that cause problem in the membrane inside out filtration process (Metcalf & Eddy, 2007).
- As for the already happened fouling problem the above mentioned preventative solution does not help, instead the chemical membrane cleaning process should be improved. Performing intensive cleaning such as Cleaning-out-of-place (COP) is not recommended to be done, besides the significant reduction of flux. This is because the MBR plant served for not more than a year and if COP is done in regular basis the membrane become damaged and require replacement (Kalkavoura, 2014). Hence, what is suggested instead is applying cleaning-in-place (CIP). Even though at Mekenisa Kotari Treatment Plant CIP has been done at most once in a month, (Hai et.al, 2014) recommends the cleaning interval to be 3 to 7 days for municipal MBRs. Accordingly, applying chemical cleaning within this period might relief the exhibited fouling problem.

As it has been mentioned the quality of the effluent water has problem only in two of the measured parameters, total phosphorus and total nitrogen concentrations. In order to improve the removal efficiency of the plant for these parameters the following operational conditions has been recommended.

- To improve phosphorous removal efficiency chemical phosphorus removal (CPR) and Enhanced Biological Phosphorous Removal (EBPR) are the two alternatives available. Since the later mechanism requires additional tank volume and sludge transfer pumps the first method is recommended for Mekenisa Kotari Treatment Plant. It is reported that the cost of implementation of chemical precipitation is approximately one-half of the cost of implementation phosphorous removal by biological means (Russel, 2006). Addition of metal salt, alum or ferric chloride, directly to the pre-aeration tank, precipitate soluble phosphorous and the solid can ultimately be removed from the process with the dewatered sludge through the membrane filtration.
- The low nitrogen removal efficiency is related with poor denitrification capacity of the plant. In order to enhance this, alternate anoxic/oxic process is proposed. As the process require only minimal, if any, additional configuration reform it is the most cost-effective solution to meet the effluent standard limit. Since Mekenisa Kotari MBR plant is automated, it would likely only need to develop programs for implementing the proposed cyclic aeration. In addition installing online $NH_4$-N and $NO_3$-N probes, similar to the already installed DO probe, facilitate the operation and control process.

### 5.2.1 Future Work

Results from this thesis showed that alternative anoxic/oxic cyclic process can be possibly adopted to maintain effluent concentration of total nitrogen below Ethiopian Environmental Protection Agency standard. As the evaluation was performed using validated mathematical model, future studies should include experimental evaluation of the proposed process on a laboratory scale pilot plant system preferably operating with real municipal wastewater. This will help to reassure model outputs.

Since the modelling work of this study did not incorporate fouling into consideration future researchers can work on assessing effects of intermittent aeration in on filtration process and fouling. This could preferably done using laboratory sale system because modeling fouling or removal of soluble compounds mechanisms is not a well-developed science yet (Maere, et.al, 2011). The studies could also work on determining optimal membrane cleaning intervals. As cleaning interval is dependent on the type of membrane used and wastewater characteristics, those studies result in site specific better result.

Future works can also be done in broadening the suggested model-based optimization. More than half of the energy consumption due to bioprocess aeration was saved by adopting the intermittent aeration process; however, in order to further reduce energy consumption in membrane bioreactors, an integrated approach to aeration optimization can be developed considering both membrane aeration and nitrification/denitrification and mixed liquor recirculation.

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# Annex A Processes of Activated Sludge Model 1

*Table A- 1 Peterson Process Matrix for ASM1*

| | Component / Process | $S_I$ | $S_S$ | $X_I$ | $X_S$ | $X_{B,H}$ | $X_{B,A}$ | $X_P$ | $S_O$ | $S_{NO}$ | $S_{NH}$ | $S_{ND}$ | $X_{ND}$ | $S_{ALK}$ | Process Rate |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Aerobic growth of heterotrophs | | $\frac{-1}{Y_H}$ | | | 1 | | | $\frac{-(1-Y_H)}{Y_H}$ | | $-i_{XB}$ | | | $\frac{-i_{XB}}{14}$ | $\mu_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{S_O}{K_{O,H}+S_O}\right)X_{B,H}$ |
| 2 | Anoxic growth of heterotrophs | | $\frac{-1}{Y_H}$ | | | 1 | | | $\frac{-(4.57-Y_A)}{Y_A}$ | $\frac{-(1-Y_H)}{2.86Y_H}$ | $-i_{XB}$ | | | $\frac{1-Y_H}{14*2.86Y_H}-\frac{i_{XB}}{14}$ | $\mu_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right) * \left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\eta_g X_{B,H}$ |
| 3 | Aerobic growth of autotrophs | | | | | | 1 | | | $\frac{1}{Y_A}$ | $-i_{XB}-\frac{1}{Y_A}$ | | | $\frac{-i_{XB}}{14}-\frac{1}{7Y_A}$ | $\mu_A\left(\frac{S_{NH}}{K_{NH}+S_{NH}}\right)\left(\frac{S_O}{K_{O,A}+S_O}\right)X_{B,A}$ |
| 4 | Decay of heterotrophs | | | | $1-f_P$ | -1 | | $f_P$ | | | | | $-i_{XB}-f_P i_{XP}$ | | $b_H X_{B,H}$ |
| 5 | Decay of autotrophs | | | | $1-f_P$ | | -1 | $f_P$ | | | | | $-i_{XB}-f_P i_{XP}$ | | $b_A X_{B,A}$ |
| 6 | Ammonification of nitrogen | | | | | | | | | | 1 | -1 | | | $k_a S_{ND} X_{B,H}$ |
| 7 | Hydrolysis of entrapped organics | | 1 | | -1 | | | | | | | | | | $k_h\frac{X_S/X_{B,H}}{K_X+(X_S/X_{B,H})}\left[\left(\frac{S_O}{K_{O,H}+S_O}\right)+\eta_h\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\right]X_B$ |
| 8 | Hydrolysis of entrapped organic nitrogen | | | | | | | | | | | 1 | -1 | | $\rho_7(X_{ND}/X_S)$ |

- *Aerobic growth of heterotrophic biomass $X_{B,\ H}$:* This process is the main contributor to the production of new biomass and removal of COD. A fraction of the readily biodegradable substrate (SS) is used for growth of heterotrophic biomass with oxygen being the electron acceptor. Some ammonia nitrogen is incorporated into the biomass during cell synthesis. The growth rate modelled using Monod kinetics is shown in Equation A-1

$$\mu_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{S_O}{K_{O,H}+S_O}\right)X_{B,H} \tag{A-1}$$

Where $\mu_H$=maximum specific growth rate for heterotrophic biomass (1/d)

$K_S$=Substrate half-saturation coefficient for heterotrophic biomass (gCOD/l)

$K_{O,H}$=Oxygen half-saturation coefficient for heterotrophic biomass ($gO_2$/l)

- *Anoxic growth of heterotrophs biomass $X_{B,\ H}$*: Heterotrophic biomass is cable of consuming the readily biodegradable substrate (SS) using nitrate as electron acceptor in the absence of oxygen. The process will lead to a production of heterotrophic biomass and nitrogen gas (denitrification). The process rate is very similar to the aerobic growth rate except that in this case it is nitrate instead of oxygen used in the double nutrient limitation Monod expression. There are two additional factors added in anoxic growth rate expression. The first factor, ηg (<1), reduces $\mu_H$ since only few heterotrophic biomass can function with nitrate as electron acceptor. The second factor is a switch-off function used to change from an aerobic to an anoxic situation. The process rate considered is shown in Equation

$$\mu_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)*\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\eta_g X_{B,H} \tag{A-2}$$

Where $\eta_g$=Correction factor for $\mu_H$ under anoxic conditions

$K_{NO}$=Nitrate half saturation coefficient for heterotrophic biomass (gN/l)

- *Aerobic growth of autotrophic biomass $X_{B,\ A}$ (nitrification)*: while this process takes place, autotrophic biomass starts growing by consuming ammonia nitrogen. Oxygen serves as electron acceptor. This process plays a major role for oxidation of ammonia to nitrate (nitrification). Monod kinetics is used for the rate expression oxygen and ammonia being the two limiting factors of the reaction (Equation A-3).

$$\mu_A\left(\frac{S_{NH}}{K_{NH}+S_{NH}}\right)\left(\frac{S_O}{K_{O,A}+S_O}\right)X_{B,A} \quad \text{(A-3)}$$

Where $\mu_A$=maximum specific growth rate for autotrophic biomass (1/d)

$K_{NH}$=Ammonia half-saturation coefficient for autotrophic biomass (gN/l)

$K_{O,A}$=Oxygen half-saturation coefficient for autotrophic biomass ($gO_2$/l)

- *Decay of heterotrophs*: During this process, the biomass dies off and a portion of it contributes to $X_P$ fraction, being non-biodegradable and the other add up to the slowly biodegradable substrate ($X_S$). Particulate organic nitrogen is also released as some are related with $X_S$. The rate of this process is shown in Equation (A-4).

$$b_H X_{B,H} \quad \text{(A-4)}$$

Where $b_H$=Decay coefficient for heterotrophic biomass (1/d)

- *Decay of autotrophs*: The process is modelled in the same way as usedto describe decay of heterotrophs. Particulate substrate, inert organics and organic nitrogen are also released but the decay coefficent is different. The procee rate is Equation (A-5).

$$b_A X_{B,A} \quad \text{(A-5)}$$

Where $b_A$= Decay coefficient for autotrophic biomass (1/d)

- *Ammonification of soluble organic nitrogen*: Ammonification is a process that amino acids and other bitrogen containg organic compounds undergo biodegradation and release ammonia. While ammonification of soluble organic nitrogen takes place soluble biodegradable organic nitrogen ($S_{ND}$) is transformed into ammonia nitrogen. The reaction has a first order equation rate and it is done under the precencess of hetetrophic biomass Equation (A-6).

$$k_a S_{ND} X_{B,H} \quad \text{(A-6)}$$

Where $k_a$= Ammonification rate (1/(gCOD.d))

- *Hydrolysis of entrapped organics*: Slowly biodegradable substrate (Xs) enmeshed in the sludge mass is broken down to readily biodegradable substrate available ($S_S$). This reaction occur under both anoxic and aerobic conditions. The rate of the reaction is first-order with respect to the heterotrophic biomass present but saturates as the amount of entrapped slowly biodeegradable substrate becomes large in proportion to the biomass. The reaction rate is shown in Equation (A-7)

$$k_h \frac{X_S/X_{B,H}}{K_X + \left(X_S/X_{B,H}\right)} \left[\left(\frac{S_O}{K_{O,H} + S_O}\right) + \eta_h \left(\frac{K_{O,H}}{K_{O,H} + S_O}\right)\left(\frac{S_{NO}}{K_{NO} + S_{NO}}\right)\right] X_{B,H} \qquad \text{(A-7)}$$

Where kh=maximum specific hydrolysis rate (gCOD/(gCOD_biomass.d))

$\eta_b$= Correction factor for hydrolysis under anoxic conditions

$K_X$= hydrolysis half-saturation coefficient (gCOD/(gCOD_biomass.d))

- *Hydrolysis of entrapped organic nitrogen*:similar to the enmeshed organic hydrolysis this is the biodegradadatuib ofthe particulateorganic nitrogen ($X_{ND}$) is broken down to soluble organic nitrogen at a ratedefined by the hydrolysis reaction for entrapped organics describedabove.The process rate is shown in Equation (A-8)

$$k_h \frac{X_S/X_{B,H}}{K_X + \left(X_S/X_{B,H}\right)} \left[\left(\frac{S_O}{K_{O,H} + S_O}\right) + \eta_h \left(\frac{K_{O,H}}{K_{O,H} + S_O}\right)\left(\frac{S_{NO}}{K_{NO} + S_{NO}}\right)\right] X_{B,H} * X_{ND}/X_S \qquad \text{(A-8)}$$

All the components and process mentioned are presentented in the Petersen matrix representation Table A-1. For more details about the model, the reader is referred to the original publication.(Henze, et.al, 2000a)

The range of various values and the deafualt values of different model parametrs for ASM1

*Table A- 2 Range and default values of model parameters for ASM1*

| Symbol | Description | Default | Range | Unit |
|---|---|---|---|---|
| $Y_H$ | Heterotrophi yield | 0.67 | 0.46-0.69 | gCOD/gCOD |
| $Y_A$ | Autotrophic yield | 0.24 | 0.07-0.28 | gCOD/gN |
| $i_{NX}$ | Mass of N per mass of COD in biomass | 0.086 | | gN/gCOD |
| $i_{NI}$ | Mass of N per mass of COD in inerts | 0.06 | 0.02-0.1 | gN/gCOD |
| $\boldsymbol{\mu}_H$ | Maximum specific growth rate for heterotrophic biomass | 6 | 3.0-13.2 | l/d |
| $K_S$ | Half saturation coefficient for heterotrophic biomass | 20 | 10-180 | $gCOD/m^3$ |
| $K_{OH}$ | Saturation/ inhibition coefficeint for oxygen, for heterotrophic biomass | 0.2 | 0.01-0.20 | $gO_2/m^3$ |
| $K_{NO}$ | Saturation/ inhibition coefficeint for nitrate | 0.5 | | $gNO_3\text{-}N/m^3$ |
| $b_H$ | Decay rate for heterotrophic biomass | 0.62 | 0.05-1.6 | l/d |
| $b_A$ | Decay rate for heterotrophic biomass | 0.15 | | l/d |
| $\eta_g$ | Reduction factor for anoxic growth | 0.8 | | — |
| $\eta_h\, \eta_{NO3}$ | Reduction factor for anoxic hydrolyses | 0.4 | 0.6-1.0 | — |
| $k_H$ | Maximum specific hydrolysis rate | 3.0 | 1.0-3.0 | l/d |
| $K_X$ | Saturation coefficient for particulate COD | 0.03 | 0.01-0.03 | gCOD/gCOD |
| $\boldsymbol{\mu}_A$ | Maximum specific growth rate for autotrophic biomass | 0.8 | 0.34-0.8 | l/d |
| $K_{NH4}$ | Saturation coefficient for ammonium | 1.0 | | $gNH_4\text{-}N/m^3$ |
| $K_{OA}$ | Saturation coefficient for oxygen for autotrophs | 0.4 | | $gO_2/m^3$ |

# Annex B Data and Calculation

*Table B- 1 Fractional Factorial Design and Response in actual units of variable*

| Run No | AF | CTR | SRT | Total Nitrogen (mg/l) |
|---|---|---|---|---|
| 1 | 0.35 | 0.5 | 16 | 101.59 |
| 2 | 0.60 | 0.5 | 16 | 56.37 |
| 3 | 0.35 | 0.6 | 16 | 99.02 |
| 4 | 0.60 | 0.6 | 16 | 54.92 |
| 5 | 0.35 | 0.5 | 18 | 101.09 |
| 6 | 0.60 | 0.5 | 18 | 56.31 |
| 7 | 0.35 | 0.6 | 18 | 100.76 |
| 8 | 0.60 | 0.6 | 18 | 54.6 |
| 9 | 0.47 | 0.55 | 17 | 37.28 |
| 10 | 0.47 | 0.55 | 17 | 37.28 |
| 11 | 0.47 | 0.55 | 17 | 37.28 |

*Table B- 2 Anallysis of Variance for factorial design*

| Source | Sum of Squares | df | Mean Square | F Value | p-value Prob > F | Remark |
|---|---|---|---|---|---|---|
| **Model** | 1261.00 | 3.00 | 420.34 | 923.46 | < 0.0001 | significant |
| **A-AF** | 1222.40 | 1.00 | 1222.40 | 2685.57 | < 0.0001 | |
| **B-CTR** | 37.45 | 1.00 | 37.45 | 82.29 | 0.0001 | |
| **C-SRT** | 1.15 | 1.00 | 1.15 | 2.52 | 0.1634 | |

| | |
|---|---|
| **R-Squared** | 0.998 |
| **Adj R-Squared** | 0.997 |
| **Pred R-Squared** | 0.991 |
| **Adeq Precision** | 65.531 |

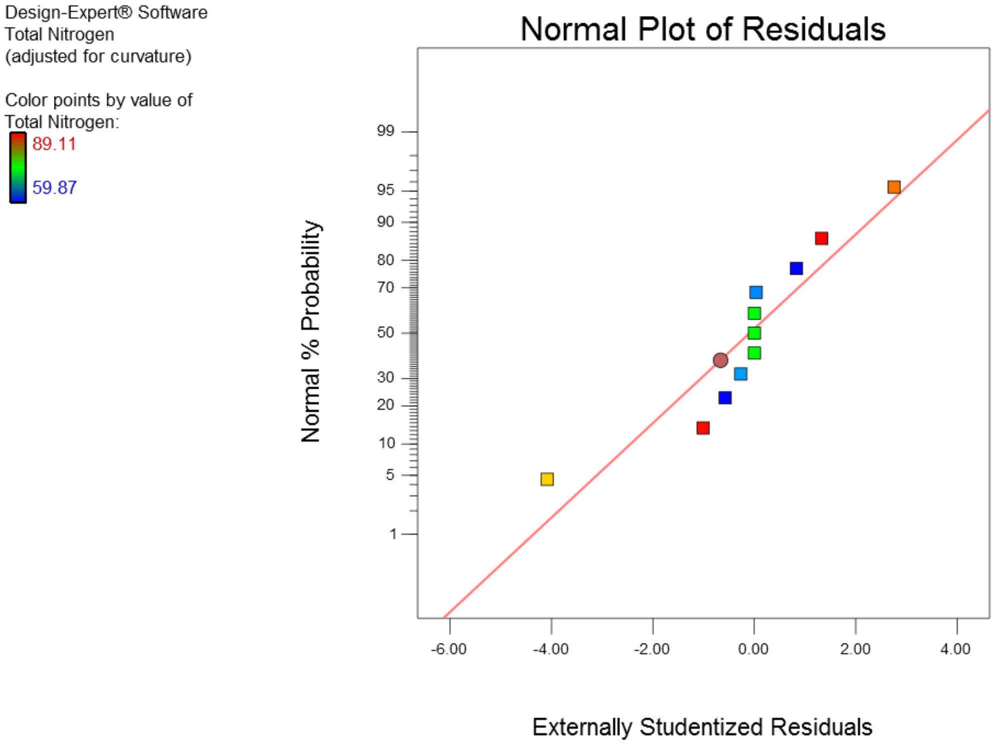


*Figure B- 1 Normal plot of Residuals for factorial regression equation of TN*

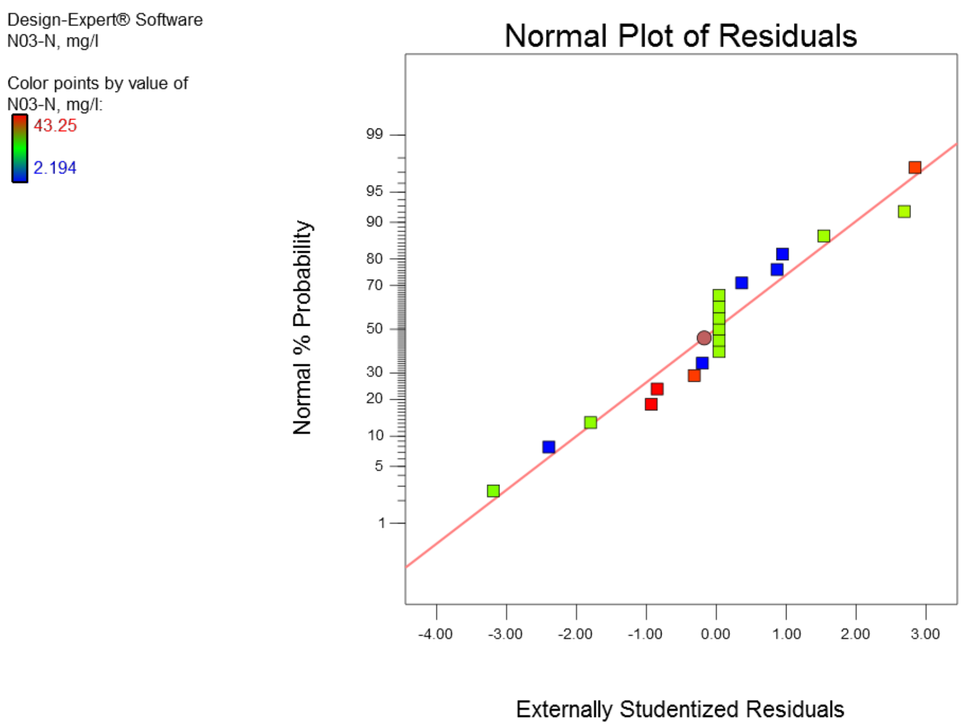


*Figure B- 2 Normal Plot of Residuals of CCD regression model of NO3-N*

## Annex C Process Flow Diagram of Mekenisa Kotari