DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

A CASE STUDY ON ENERGY OPTIMIZATION FOR MEMBRANE BIOREACTOR SYSTEM VIA A COMPUTER SIMULATION: KONACIK MUNICIPALITY

by
Servet EROL

March, 2012
İZMİR

A CASE STUDY ON ENERGY OPTIMIZATION FOR MEMBRANE BIOREACTOR SYSTEM VIA A COMPUTER SIMULATION: KONACIK MUNICIPALITY

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M.Sc THESIS EXAMINATION RESULT FORM

We have read the thesis entitled **"A CASE STUDY ON ENERGY OPTIMIZATION FOR MEMBRANE BIOREACTOR SYSTEM VIA A COMPUTER SIMULATION: KONACIK MUNICIPALITY"** completed by **SERVET EROL** under supervision of **PROF. DR. NURDAN BÜYÜKKAMACI** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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ABSTRACT

In this thesis, designing and operating conditions of existing treatment plant units compared to the results with ATV-DVWK-A 131 E standards for Konacık Municipality in Bodrum Domestic Wastewater Treatment. After then decreasing of energy consumption of the treatment plant was aimed and a computer simulation program (BioWin) was used for this purpose. In order to minimize the operational costs depending on energy consumption, two alternative operating strategies were generated by using BioWin program. At the end of the study, existing treatment plant operating data, results of calculations made considering ATV-DVWK-A 131 E standards, and results of the simulation program were compared. Existing design and operational conditions were confirmed according to ATV-DVWK-A 131 E standards and the most cost effective and appropriate operational condition considering effluent quality was determined.

Keywords: Wastewater, membrane, design, simulation, optimization

BİLGİSAYAR SİMÜLASYONU İLE MEMBRAN BİYOREAKTÖR SİSTEMİNDE ENERJİ OPTİMİZASYONU İÇİN ÖRNEK ÇALIŞMA: KONACIK BELEDİYESİ

ÖZ

Bu çalışma kapsamında, Konacık Belediyesi Atıksu Arıtma Tesisinde öncelikle mevcut 1.etap projelendirmesinde kullanılan biyolojik giderim verim hesabına göre yapılan tasarım hesaplamaları ATV-DVWK-A 131 E tasarım kriterine göre tahkikinin yapılması planlanmıştır. İkinci olarak bilgisayar simülasyon programı (BioWin) kullanılarak mevcut işletme maliyetlerinin minimizasyonu için işletme stratejileri belirlenmiştir. Bu amaçlarla farklı işletme süreçlerinde mevcut tesis teknik verileri, ATV-DVWK-A 131 E hesaplama sonuçları ve simülasyon programı sonuçları karşılaştırılmıştır. Mevcut tasarım hesabının ATV-DVWK-A 131 E tasarım hesabına göre kontrolü teyit edilmiş, çıkış suyu kalitesi göz önünde bulundurularak maliyet ve teknik açıdan en uygun işletme koşulu belirlenmiştir.

Anahtar Kelimeler: Atıksu, membran, tasarım, simülasyon, optimizasyon

CONTENTS

CHAPTER ONE
INTRODUCTION

1.1 Introduction

Membrane Bioreactor (MBR) systems, which are essentially consists of combination of membrane and biological reactor systems, are the emerging technologies. The membranes used in MBR systems have porosities smaller than 0.4 microns. This level of filtration provides high quality effluent without sedimentation units. Since MBR systems have several advantages, they are widely used for both municipal and industrial wastewater treatment.

Design modeling programs have been commonly used for wastewater treatment unit designing and analyzing. Advantages of modeling programs can be arranged as:

1- Controlling and examining the available design data
2- Controlling the system performance and reducing the operation cost
3- Operation optimization with process evaluation
4- Obtaining variable data via design based on modeling
5- Possibility of choosing the most appropriate facility

The first activated sludge process software based on ASM1 (Activated Sludge Model No:1) modelwas developed in 1988 (Yang and Pan, 2011). Simulation studies which started from 1980's are firstly applied on activated sludge processes and it has been developed as involves MBR and Ultrafiltration technologies. For the modeling of wastewater treatment plants, a wide choice of models, such as GPS-X, STOAT, EFOR, etc., is available (Zhong, 2011; Pal,2010; Pons, 1999).

In the scope of this study, BioWin program, which Envirosim Associates Ltd. offer, was applied in simulation for the treatment units.

This program was developed by Barker and Dold (2003) and it is a Microsoft Windows-based simulator used world-wide in the analysis and design of wastewater treatment plants.

In this study, Konacık Municipality in Bodrum Domestic Wastewater Treatment plant was selected as pilot plant. There is a Membrane Bioreactor with Ultrafiltration system and it is the first flat sheet membrane treatment plant for a municipality in Turkey. In addition to the existing treatment plant facility (1^{st} stage), constructions of 2^{nd} and 3^{rd} stages have been continuing. Existing process diagram(1^{st} stage), equipment data and wastewater input values of the plant are entered to BioWin program. With these modeling studies following items are aimed:

1- Comparing biological effluent relevant results with ATV-DVWK-A 131 E standards in 1^{st} stage and to control the accuracy of calculation results.
2- Evaluating alternative operational conditions by using a simulation program to decrease energy consuming.

1.2 Scope of the Thesis

This thesis is mainly composed of 5 Chapters, including general conclusions and perspectives. General introduction is given in Chapter1 and MBR systems and some modeling programs have been proposed to explain the biological reactions in MBR processes are discussed briefly. In Chapter 2, the detailed literature review is performed. Conventional wastewater treatment systems are introduced. In addition, membrane bioreactors are described. Conventional treatment and MBR process are also compared. Materials and methods used in this study are given in Chapter 3. Pilot plant and BioWin program details are introduced. The details of the results are given results and discussion, and conclusion and recommendationssections of the thesisin Chapter 4 and 5, respectively.

CHAPTER TWO
LITERATURE REVIEW

2.1 Conventional Wastewater Treatment Systems

Wastewater can be defined as the spent water from homes, commercial establishments, industries, public institutions etc. Wastewater may contain organic and inorganic substances, industrial wastes, and agricultural wastes. It influences groundwater, storm runoff, and other similar liquids. The keyword in the definition of wastewater is “used” or “spent”. Moreover, it is synonymous with sewage and to become sewage, it is enough that water becomes polluted whether or not it had been used (Sincero&Sincero, 2003).

The normal constituents of domestic wastewater are shown in Table 2.1. The parameters shown in the table are typical composition of untreated municipal wastewaters. As indicated, untreated domestic wastewater is categorized as weak, medium, and strong based on constituent concentration (Sincero&Sincero , 2003).

Wastewater must be treated before discharging. It can be treated by physical, chemical, and biological methods. Physical treatment of water and wastewater defines as a process applied to water and wastewater in which no chemical changes occur. Chemical treatment of water and wastewater defines as a process applied to water and wastewater in which chemical changes occur. In the overall aspect, physical–chemical treatment of water and wastewater is a process applied to water and wastewater in which chemical changes may or may not occur.

Table 2.1 Typical composition of untreated domestic wastewater (Sincero&Sincero 2003, p.166).

Constituent	**Concentration (mg/L)**			
	Strong	**Medium**	**Weak**	**KonacıkWWTP Medium**
BOD_5 at 20°C	420	200	100	222
Total organic carbon (TOC)	280	150	80	193
Chemical oxygen demand (COD)	1000	500	250	430
Total solids	1250	700	300	1960
Dissolved	800	500	230	1750
Fixed	500	300	140	1093
Volatile	300	200	90	656
Suspended	450	200	70	209
Fixed	75	55	20	34
Volatile	375	145	50	175
Settleable solids, mL/L	20	10	5	12
Total nitrogen	90	50	20	64
Organic	35	15	10	24
Free ammonia	55	35	10	40
Nitrites	0	0	0	0
Nitrates	0	0	0	0
Total phosphorus as P	18	10	5	3.9
Organic	5	3	1	1
Inorganic	13	7	4	2.9
Chlorides	110	45	30	-
Alkalinity as $CaCO_3$	220	110	50	250
Grease	160	100	50	60

Figure 2.1 shows the schematic of a conventional wastewater treatment plant using preliminary, primary, secondary, and advanced treatment (http://www.fao.org).

As can be seen from the Figure 2.1, raw wastewater is introduced either to the screen or to the comminutor. The grit channel removes the larger particles from the screened sewage, and the primary clarifier removes the larger particles of organic matter as well as inorganic matter that escape removal by the grit channel. Primary treated sewage is then introduced to a secondary treatment process train downstream where the colloidal and dissolved organic matters are degraded by microorganisms. Low rate (such as stabilization ponds, aerated lagoons etc.) or high rate (activated sludge, trickling filters, rotating biocontactors, etc.) processes may be used in secondary treatment. Advanced treatment includes disinfection, nitrogen removal, phosphorus removal, suspended solids removal, organics and metals removal, dissolved solids removal.

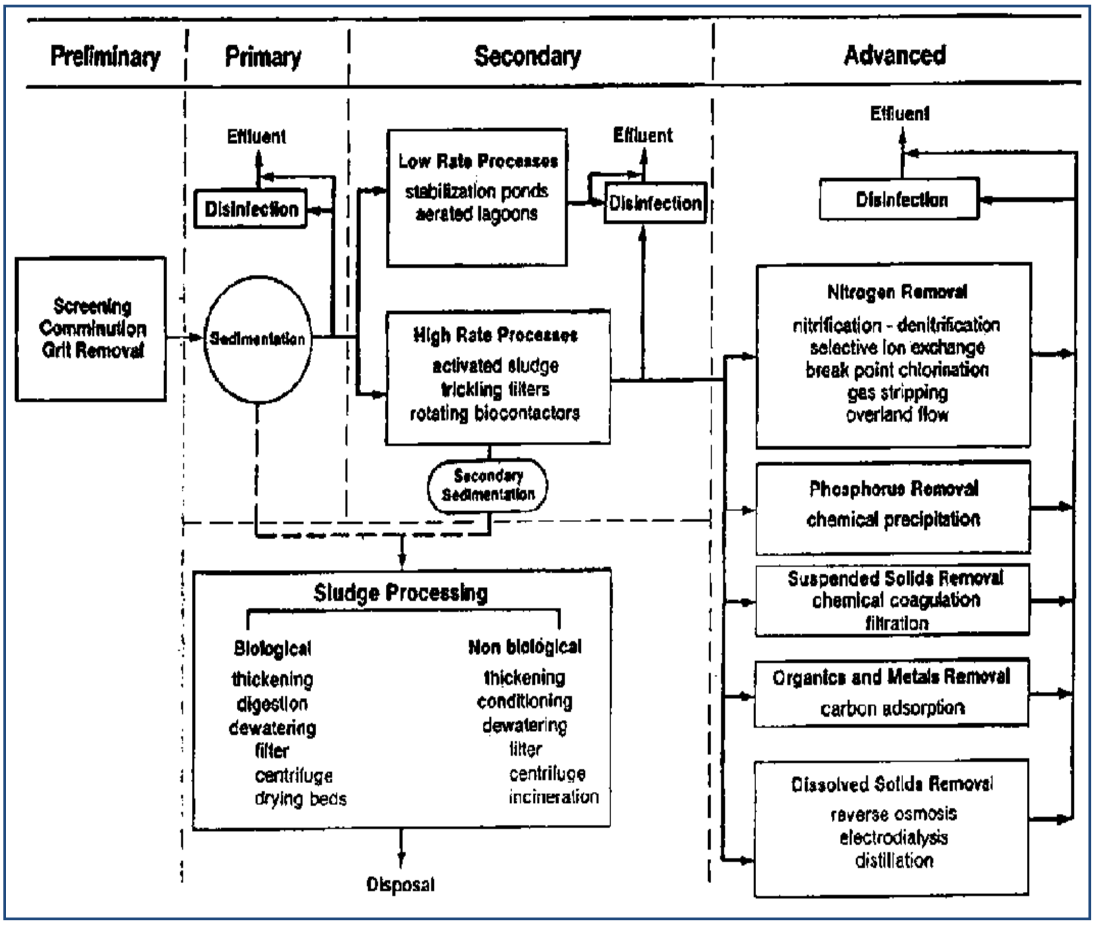

Figure 2.1 Schematic of a conventional wastewater treatment plant (http://www.fao.org)

Physical treatments are called unit operations. Unit operations of water and wastewater treatment include pumping; screening, settling, sedimentation–flotation; mixing–flocculation; conventional filtration; advanced filtration–carbon adsorption; aeration–stripping. However, in the biological or chemical scene where materials are changed, unit operations have counterparts called unit processes.Unit processes of water and wastewater treatment include coagulation; water softening; chemical stabilization; removal of iron–manganese; removal of phosphorus; removal of nitrogen by nitrification–denitrification; ion exchange; and disinfection. Removal of nitrogen by nitrification–denitrification is a biological process (Sincero&Sincero, 2003).

The activated sludge process was developed in the early 1900s in England (Metcalf & Eddy, 1991). Initially fill-and-draw systems were used into operation but they were quickly converted into continuous flow systems. Despite more frequent occurrence of settling problems, continuous flow systems became popular and spread world-wide. As a known, the activated sludge process is the most commonly used technology for biological wastewater treatment. It consists of two stages, a biochemical stage (aeration tank) and a physical stage (secondary clarifier). In the aeration tank, organic carbon, ammonium, and phosphate are removed from the wastewater by the microorganisms of activated sludge. Biomass retention is an important subject in order to increase the biomass concentration in the biochemical stage. A good separation (settling) and compaction (thickening) of activated sludge in the secondary clarifier is a necessary condition to guarantee a good effluent quality from the activated sludge process. Therefore, this separation is based on the formation of compact flocs. In the aeration tank, contact time is provided for mixing and aerating influent wastewater. During this time, bacteria use oxygen as an electron acceptor and organic matter as an electron donor to obtain energy and to synthesize new cells. Secondary clarifier is used to settle out the biomass and separate biomass from the effluents. Some of the settled sludge in the secondary clarifier is recycled back to the aeration tank to increase the biomass concentration there, and the rest is disposed off as waste sludge to obtain desired sludge age(Loosdrecht, 2008).

In the activated sludge system, the mixing regime in the reactor and the sludge return are part of the system constraints. There are two extremes of mixing; completely mixed and plug flow (Figure 2.2). In the completely mixed regime, the influent wastewater is instantaneously and homogenously mixed with the reactor contents. Hence the effluent flow from the aeration tank (reactor) has the same constituent concentrations as the reactor contents. The reactor effluent flow passes to a secondary clarifier and the underflow is concentrated sludge and is recycled back to the reactor. The shape of the reactor is square or circular in plan, and mixing is usually by mechanical aerators or diffused air bubble aeration (Ekama, 2008).

In a plug flow regime, the reactor usually is designed as a long channel type basin. The influent is introduced at one end of the channel, flows along the channel axis and is mixed by air spargers set along one side of the channel or horizontal shaft surface aerators. Discharge to the settling tank takes place atthe end of the channel. The underflow from the settling tank is returned to the influent end of the channel in order to inoculate the influent waste flow with organisms (Ekama, 2008).

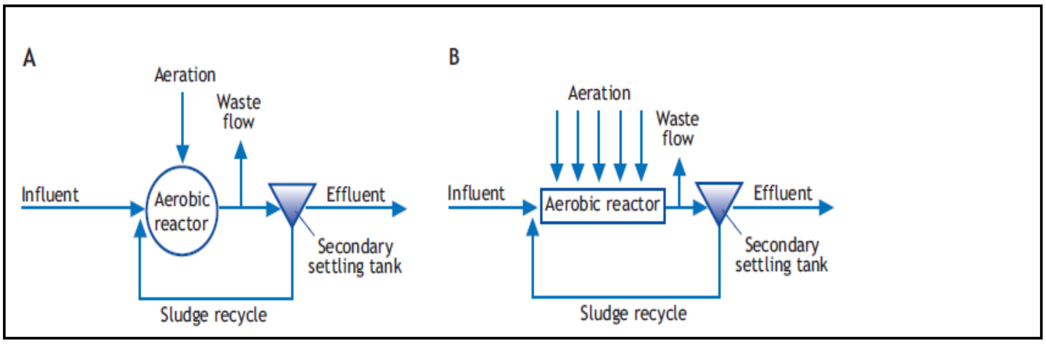

Figure 2.2 Activated sludge systems with (a) a single reactor completely mixed reactor mixing regime, and (b) a flow/intermediate reactor mixing regime (Ekama, 2008, p. 55).

Conventional activated sludge system and extended aeration activated sludge system flow scheme is shown in Figure 2.3 and 2.4.

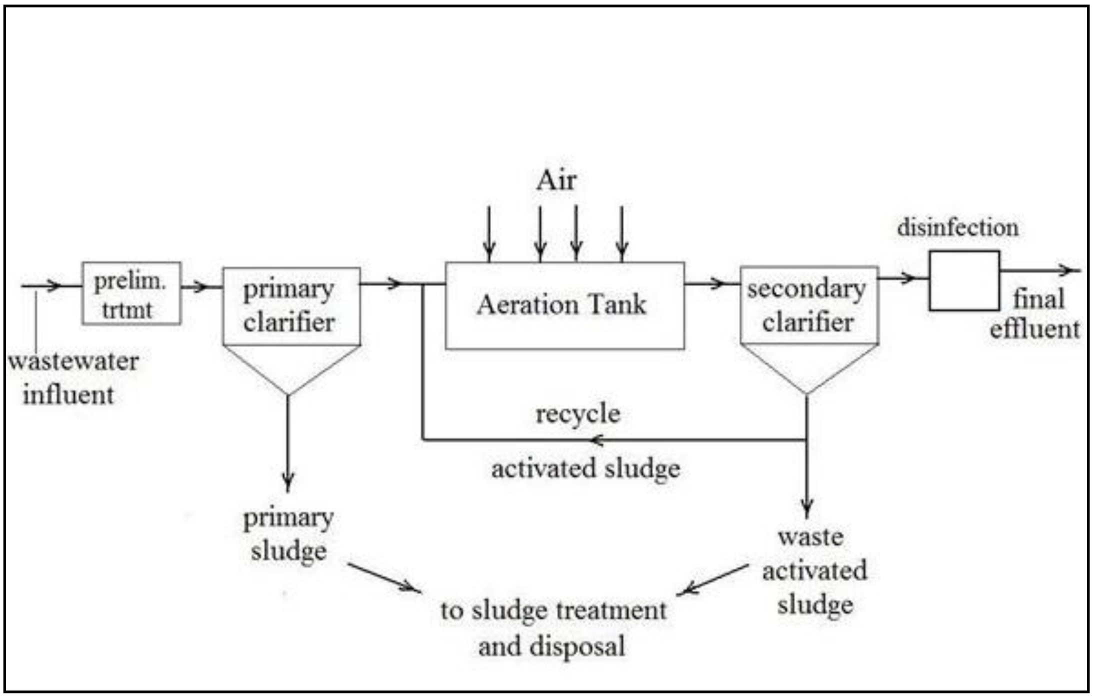

Figure 2.3 Conventional activated sludge process

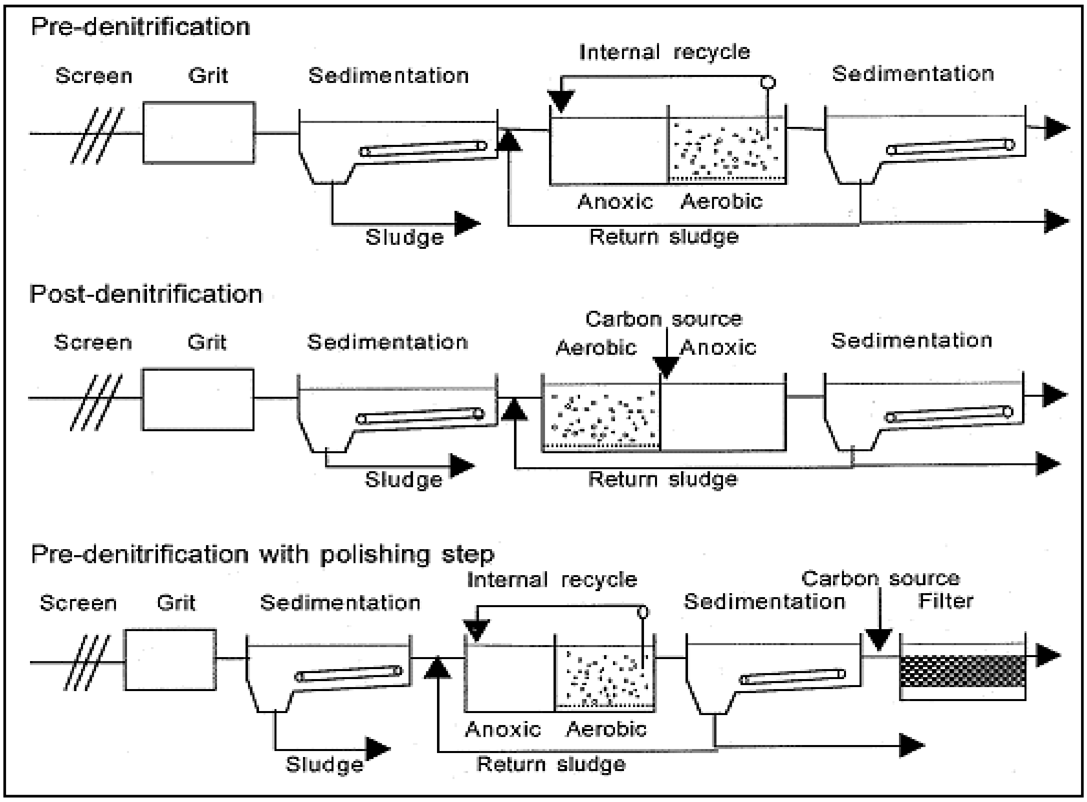

Figure 2.4 Extended activated sludge process

2.2 Membrane Bioreactors (MBR)

Microfiltration (MF) is a membrane process where ideally only suspended solids are rejected, while even proteins pass the membrane freely. Ultrafiltration (UF) is a membrane process where the high molecular weight component (HMWC), such as protein, and suspended solids are rejected, while all low molecular weight component (LMWC) pass through the membrane freely. There is consequently no rejection of mono- and di-saccharides, salts, amino acids, organics, inorganic acids or sodium hydroxide.Nanofiltration (NF) rejects only double charged ions, such as sulfate or phosphate, while passing single charged ions. NF also rejects uncharged, dissolved materials and positively charged ions according to the size and shape of the molecule in question. Reverse Osmosis (RO) is the tightest membrane process in liquid/liquid separation. Water is in principle the only material passing through the membrane; essentially all dissolved, non-dissolved, and suspended material is rejected. Table 2.2 compares four membrane processes.

Table 2.2 Comparing four membrane process (Baker, 2004).

	Reverse Osmosis	Nanofiltration	Ultrafiltration	Micro filtration
Membrane	Asymmetrical	Asymmetrical	Asymmetrical	Symmetrical Asymmetrical
Thickness Thin film	150 μm 1 μm	150 μm 1 μm	150 - 250 μm 1 μm	10-150 μm
Pore size	<0.002 μm	<0.002 μm	0.2 - 0.02 μm	4 - 0.02 μm
Rejection of	HMWC, LMWC sodium chloride glucose amino acids	HMWC mono-, di- and oligosaccharides polyvalent neg. ions,	Macro molecules, proteins, polysaccharides vira	Particles, clay bacteria
Membrane material(s)	CA Thin film	CA Thin film	Ceramic PSO, PVDF, CA Thin film	Ceramic PP, PSO, PVDF
Membrane Module	Tubular, spiral wound, plate-and-frame	Tubular, spiral wound, plate-and-frame	Tubular, hollow fiber, spiral wound, plate-and-frame	Tubular, hollow fiber
Operating pressure	15-150 bar	5-35 bar	1-10 bar	<2 bar

Membrane bioreactor (MBR) is defined as a combination of a biological process and membrane separation. General flow scheme of MBR systems is given in Figure 2.5. The applicability of MBR technology in municipal and industrial wastewater treatment has sharply increased because of its several advantages compared with the traditional activated sludge processes, such as limited space requirement, low energy requirement, and very high effluent quality. The MBR technology for full-scale municipal wastewater treatment was developed some ten years ago, most notably in Japan and Canada. In an MBR process, the separation of activated sludge and treated wastewater is not done by sedimentation in a secondary clarification tank, but by membrane filtration. However, the MBR process and the conventional system show great differences both technologically and biologically. A schematic presentation of the traditional wastewater treatment concept and three different configurations of the MBR concept are shown in Figure 2.6 (Roest, 2002).

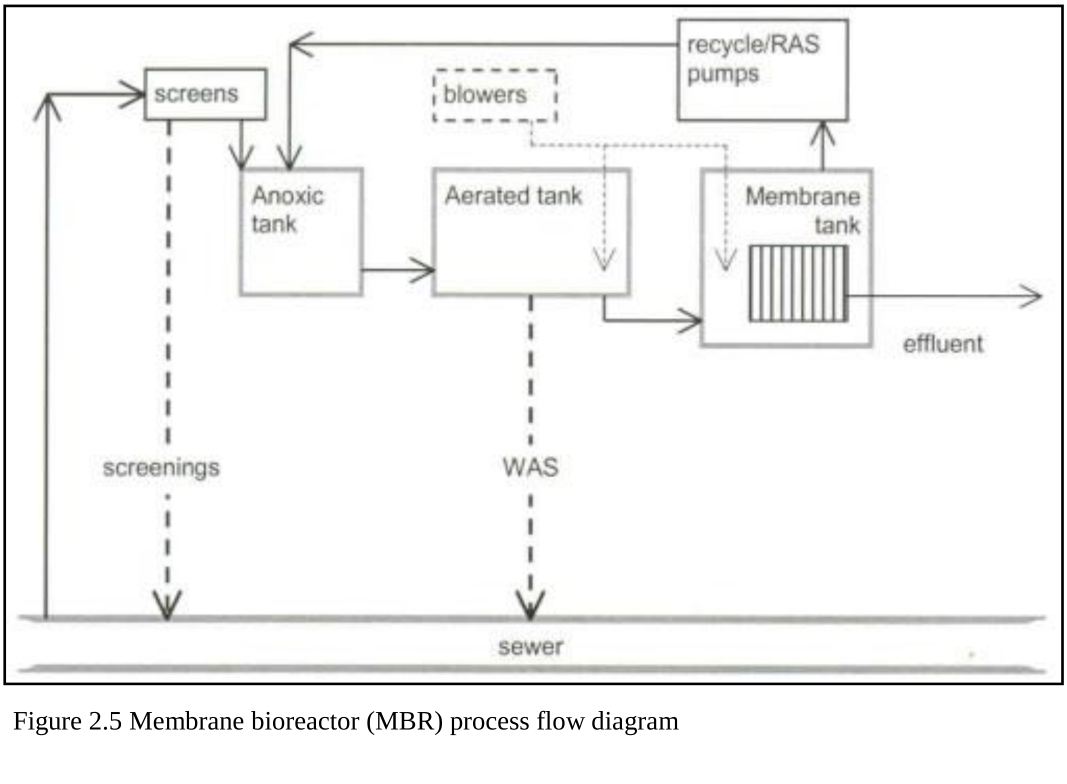

Figure 2.5 Membrane bioreactor (MBR) process flow diagram

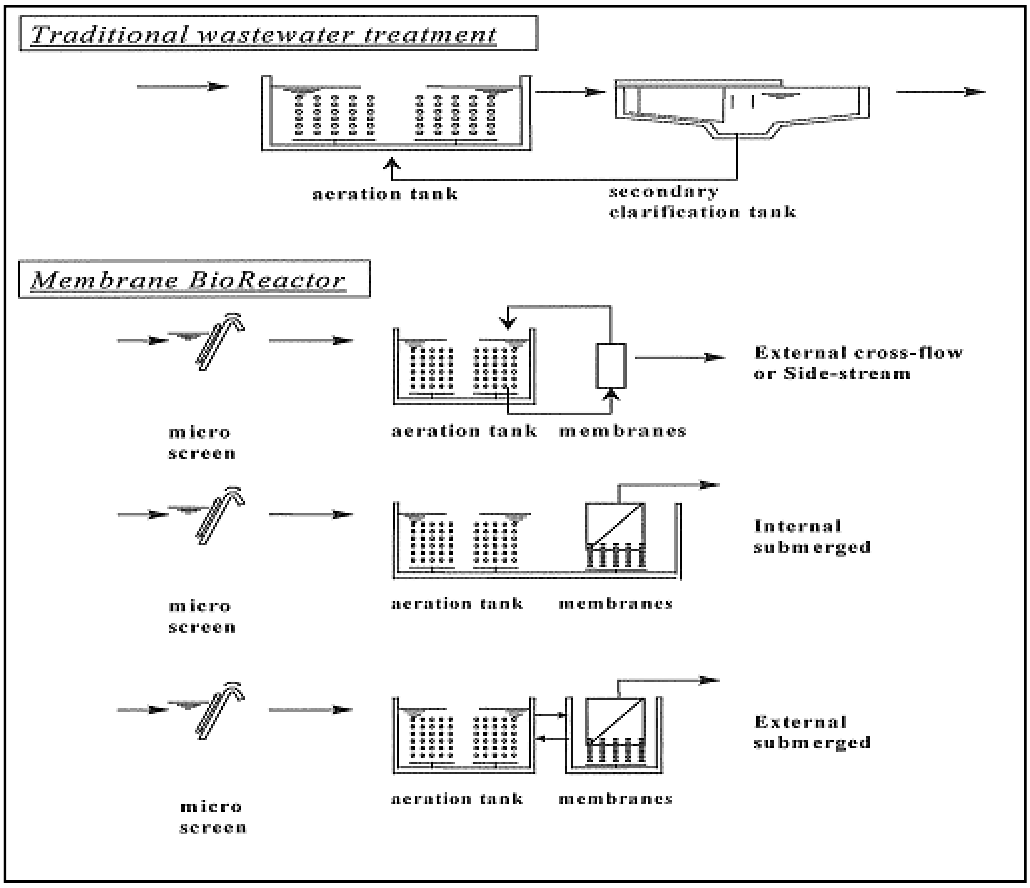

Figure 2.6 Conventional activated sludge treatment system and different MBR types (Roest, 2002,p.5)

MBR can operate two different principles as external or internal operation. The first generation MBRs consisted of cross-flow operated membranes (external operation), which were installed outside the activated sludge tank. High flow velocity is used in the cross-flow principle to prevent the build-up of solids on the membrane surface, also known cake-layer formation. This principle required large amounts of energy to generate the sludge velocity across the membrane surface for membrane cleaning. The main disadvantage of cross-flow operation is excessive shear stress causes smaller floc size. The second generation MBRs was proposed to submerge the membrane in the aeration tank (internal operation). This type of submerged membrane filtration in a biological system was referred to as submerged MBR (SMBR). Energy consumption was significantly reduces when compared with cross-flow MBR operation. Furthermore the re-circulation pump was not used in the SMBR configuration.

The mechanism used to create the cross-flow stream over the membrane surface was low pressure air diffusion. The air diffusion facilitates two processes: the supply of oxygen to the biomass and the cleaning of the membrane surface. Moreover, the shear stress in the mixed liquor of the SMBR was much lower compared with cross-flow system, and as a result, sludge characteristics were much better (Roest, 2002; Judd, 2006).

Microfiltration (MF) and ultrafiltration (UF) are the basis processes applied in MBR concepts. MF is typically used to separate or remove relatively large particles, such as suspended solids, emulsified oils, and macromolecules. Pore sizes of MF membranes range from approximately 0.05 µm to 2 µm. UF membranes are able to achieve higher levels of separation including bacteria and viruses. UF displays a pore size ranging from approximately 0.005 µm to 0.1 µm. The filtration ranges in MBR processes are presented in Figure 2.7.

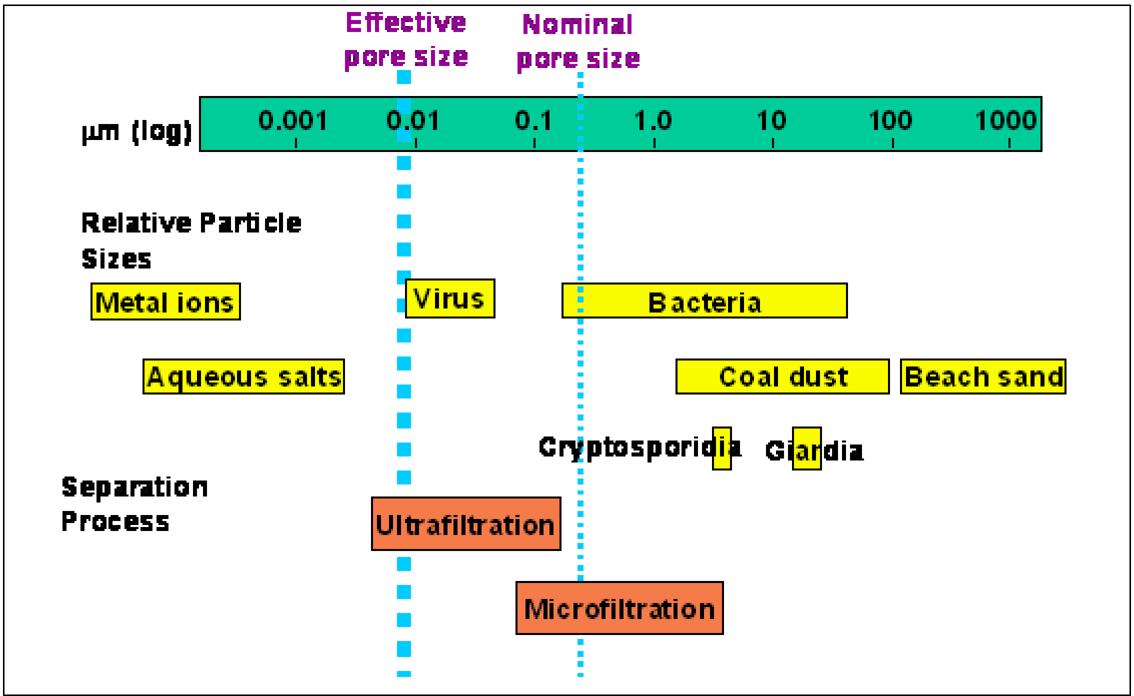

Figure 2.7 Filtration ranges in MBR process

2.3 Comparison of Conventional Treatment and MBR Systems

2.3.1 Advantages of MBR

There are many advantages of MBR when compared to the conventional treatment systems. First of all, the retention of all suspended solid and most of the soluble compounds lead to excellent effluent water quality and discharge limits (Liu, 2003; Chiemchaisri, 1993; Yamamoto, 1989).

One of the most important advantages is the size of MBR. Because of the small footprint, the area needed is very small when compared to conventional activated sludge process (Visvanathan, 2000).

In MBR, MLSS can be increased up to desired concentration, which causes long θ_c and low food-to-microorganism (F/M) ratio. However, the long θ_c also causes less sludge production. Consequently, it has been reported that less sludge production can be achieved while short hydraulic retention time is applied in MBR process (Yoona, 2004).

Additionally, exact control of θ_c provides optimum control over the microbial population and flexibility in operation. The potential for operating the MBR at a very high sludge ages allows high biomass concentration in the bioreactor (Muller, 1995).

Chlorine usage for disinfection of the effluent is needed in the conventional treatment systems to kill the pathogenic microorganisms. But, the usage of chlorine may lead to accumulation of certain disinfection by-products, such as trihalomethanes (THM) trihaloacetic acid (THA), which may pose significant cancer risk for human. In the MBR, retaining of all bacteria and viruses result in a sterile effluent, this reduces the need for extensive disinfection (Winnen, 1996).

The other advantage of MBR is nitrification process. In conventional biological nitrogen removal systems, maintenance of adequate levels of nitrifiers in the aeration tank is very difficult. However, because of long θ_c in the MBR, these bacteria persist in the system and make it possible to have adequate nitrification in the biological treatment system (Gao, 2004).

MBR also eliminate settling problems, which is usually one of the most troublesome parts of conventional wastewater treatment (Muller, 1995).

2.3.2 Disadvantages of MBR

The most important disadvantages of the MBR are mainly high investment and high maintenance costs (Trouve, 1994). Also, membrane fouling which can lead to frequent regeneration of the membrane is a very important problem for the membrane bioreactors. Membrane fouling causes to stop operation and requires cleaning cycles and chemicals usage.

Another limitation of the MBR, when operated at high θ_c, is the possibility of non-filterable inorganic compounds accumulation in the bioreactor. This can reach high concentration levels that can be toxic to the microbial population (Manem, 1996).

Complexity of the MBR is another drawback for the operation of the system (Komesli, 2006).

2.4 Design of Conventional Activated Sludge Systems and MBR Systems

2.4.1 General Design Parameters of Conventional Treatment Systems

Sludge Retention Time (SRT): The most difficult and crucial point in the designing is the accurate determination of the actual total SRT. The total SRT –also known as the most sensitive operating parameter- directly affects the solid balance on the system as well as the active biomass concentration (Werf, 1999). The use of solid balance calculations over the system facilitates the estimation of the actual sludge age of the system. In reality, the activity of biomass is lowered because of elevated protozoan activity, the formation of inert materials *etc.* at higher sludge ages (Loosdrecht&Henze, 1999).

Hydraulics: The hydraulics and type of reactor configurations are important factors and should be well-defined in order to mimic the real layout of the system. As an example, the plug-flow regime of a reactor influences the oxygen distribution along the channel and may trigger the denitrification by creating spatial anoxic zones along the reactor. In addition to that, the internal flow rate determination is of great importance since the nitrified recycle will carry oxygen and nitrate over the anaerobic and/or anoxic reactors which directly affect the total nitrogen and phosphate removal capacity of the system. Theoretically, if the internal recycle rate is changed, the travel time of mixed liquor from one aerator to another will be changed accordingly. So, the mixed liquor will be subjected more frequently to air on/off period if the internal recycle rate is increased (Clercq, 1999).

Air (Oxygen) Transfer: In activated sludge designs, the oxygen transfer is expressed by the volumetric oxygen transfer coefficient K_La under process conditions (Werf, 1996). The oxygen concentration in a CSTR tank can be formulized as follows:

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

Where;

K_La_f: the volumetric oxygen transfer coefficient under process condition

$S_{O\infty f}$: the oxygen saturation concentration under process condition

S_O: the actual oxygen concentration in the tank

OURs: the oxygen uptake rates for nitrification and heterotrophic biomass

The determination of oxygen (air) input into the system is an important issue since it switches on and off the nitrification, carbon oxidation and denitrification processes depending upon oxygen half saturation constants (K_O).

Based on Monod expressions, the lower the K_O values, the higher the oxygen consumption rates (Naidoo, 2002; Ritmann & Langeland , 1985; Insel, 2004; Munch, 1996).

Sludge Settling: The performance and operation of final settling tanks is of great importance since (1) it can maintain a mass fraction of mixed liquor depending upon operating conditions (i.e. RAS, surface loading, retention time) (2) the effluent quality can be deteriorated by resuspension, bulking and rising of the sludge blanket and (3) additional nitrate removal due to denitrification can take place (Henze, 1993) together with possible secondary phosphate release (Wouters&Wasiak et al., 1996). The dynamics and contribution of final settling tanks to overall system efficiency should be well-defined and incorporated in the modeling task.

Process Control: Automatic control technologies have become fashionable tools to provide process stability for nutrient removal (i.e. Supervisory Control and Data Acqusition-SCADA systems). The aeration, return and internal cycles together with RAS, sludge wastages, chemical dosing etc. in continuous and batchwise systems became important controlled variables (actuators) for system optimization (Copp, 2002; Olsson & Jeppsson, 1994). In that way, the control algorithms may alter the performance of the system by influencing the composition of the mixed liquor.

2.4.2 General Design Parameters of MBR systems

There are essentially three main elements of an MBR contributing to its design and operation, and specifically operating costs, ignoring membrane replacement (which can only be estimated). These are (i) aeration, (ii) liquid pumping, (iii) membrane maintenance (Henze, 2008; Judd, 2006).

2.4.2.1 Aeration

Aeration for MBR system is an important factor. It is used for both demand of the mixed liquor and membrane cleaning.

The first component of aeration concerns the bioreactor and, specifically, the demand of the mixed liquor for air required for agitation of the solids and dissolved oxygen (DO) for maintaining a viable micro-organism population for bio-treatment. In bio-treatment DO is normally the key design parameter. The oxygen requirement for a biological system relates to the feed flow rate, substrate degradation, sludge production and concentration of TKN that is oxidized to form nitrate. The oxygen is most commonly transferred to the biomass by bubbling air, or in some cases pure oxygen, into the system through diffusers. Only a portion of the air, or oxygen, which is fed to the system, is transferred to the biomass. The transfer efficiency is dependent on the type of diffuser used and the specific system design. Key differences between bio-treatment using an MBR as compared with a conventional bio-treatment process relate to the biomass concentration, which tends to be significantly higher for an MBR and leads to generally lower ratios of food to micro-organisms (F:M ratios), and the floc size, which tends to be smaller (Henze, 2008; Judd, 2006).

2.4.2.2 Liquid Pumping

In-process liquid pumping relates to transfer of sludge between tanks and to permeate withdrawal. For an immersed MBR the TMP is very low and thus the energy demand associated with permeate withdrawal is correspondingly low. Sludge transfer between tanks generally exerts a greater energy demand (Henze, 2008; Judd, 2006).

2.4.2.3 Membrane Maintenance:Cleaning

For an immersed configuration the membrane is maintained both by membrane aeration and cleaning. Physical and chemical membrane cleaning incur process downtime, loss of permeate product (in the case of backflushing) and membrane replacement. The latter can be accounted for simply by amortization, although actual data on membrane life is scarce since for most plants the start-up date is recent enough for the plants still to be operating with their original membranes.

Physical and chemical backwashing requirements are dependent primarily on the membrane and process configurations and the feedwater quality. Fundamental relationships between cleaning requirements and operating conditions usually flux and aeration for submerged systems, have been generated from scientific studies of fouling (Henze, 2008; Judd, 2006).Key design parameters relating to membrane cleaning are:

- period between physical cleans (tp), where the physical clean may be either backflushing or relaxation,
- duration of the physical clean,
- period between chemical cleans,
- duration of the chemical clean,
- backflush flux,
- cleaning reagent concentration and volume normalised to membrane area (Judd, 2006).

2.5 Modeling of Conventional Treatment and MBR Systems

A model can be defined as a purposeful representation or simplified description of a system of interest (Wentzel &Ekama, 1997). This consequently means that the model will never be exactly reflecting the reality (Biological wastewater treatment). Typical traditional wastewater treatment design methods are based on the so-called black box approach focusing on plant influent and effluent characteristics, while nothing or very little is known about what is happening inside the wastewater treatment plant. However, the black-box model can work out well in practice. A simplified representative of black-box model is shown in Figure 2.8.

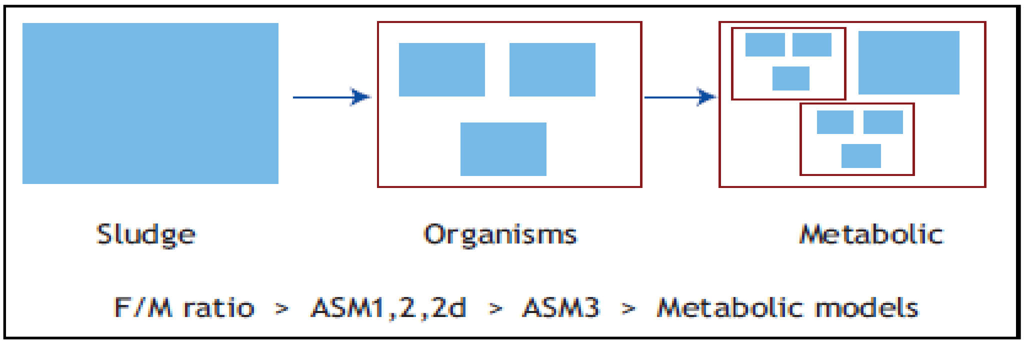

Figure 2.8 Schematic representation of the step-wise refinement of a model (Smolders, 1995)

The metabolism of the organisms and the metabolic routes inside the organisms can also be described with some models. By such an increase in information, the approach becomes close to glass-box modeling (such as the Activated Sludge Model No.3: ASM3 (Gujer, 1999) and the TU Delft EBPR model: TUDP model (Veldhuizen, 1999). This results in a bigger and more complex model.

In the literature, two extremes in type of mathematical models can be identified: empirical and mechanistic models. An empirical model is based on recognition of the parameters that seem to be essential to describe the behavioral pattern of interest, and linking these by empirical relationships established by observation. The mechanisms and/or processes operating in the system are not known or are ignored: a classical black-box approach. In contrast, a mechanistic model is based on some conceptualization of the biological/physical mechanisms operating in the system, i.e. is based on a conceptual idea (or model). Because mechanistic models have some conceptual basis, they are often more reliable than the empirically based models. Because of their black-box approach, the empirical models have an application strictly limited by the boundaries (e.g. wastewater characteristics, system parameters) within which the model was developed; only interpolation is possible.

For mathematical modeling of wastewater treatment systems two different kinds of mathematical models are generally developed: steady state and dynamic models. Steady state models have constant flows and loads and tend to be relatively simple which makes these models useful for design. In these models complete descriptions of system parameters are not required. The dynamic models have varying flows and loads and accordingly include time as a parameter. Dynamic models are more complex than the steady state ones. The dynamic models are useful in predicting time dependent system response of an existing or proposed system. Their complexity means that for application the system parameters have to be completely defined. For this reason the use of dynamic models for design is restricted. For activated sludge systems, selecting the level of organization at the surrogate organism or mass behavior of populations, until recently the dynamic models have been structured to consider only the net effects as present in the bulk liquid. For example, in using monod's equation the kinetic rate has been determined by the bulk liquid soluble COD and surrogate organism concentrations. The components of a full wastewater treatment model are schematically given in Figure 2.9.

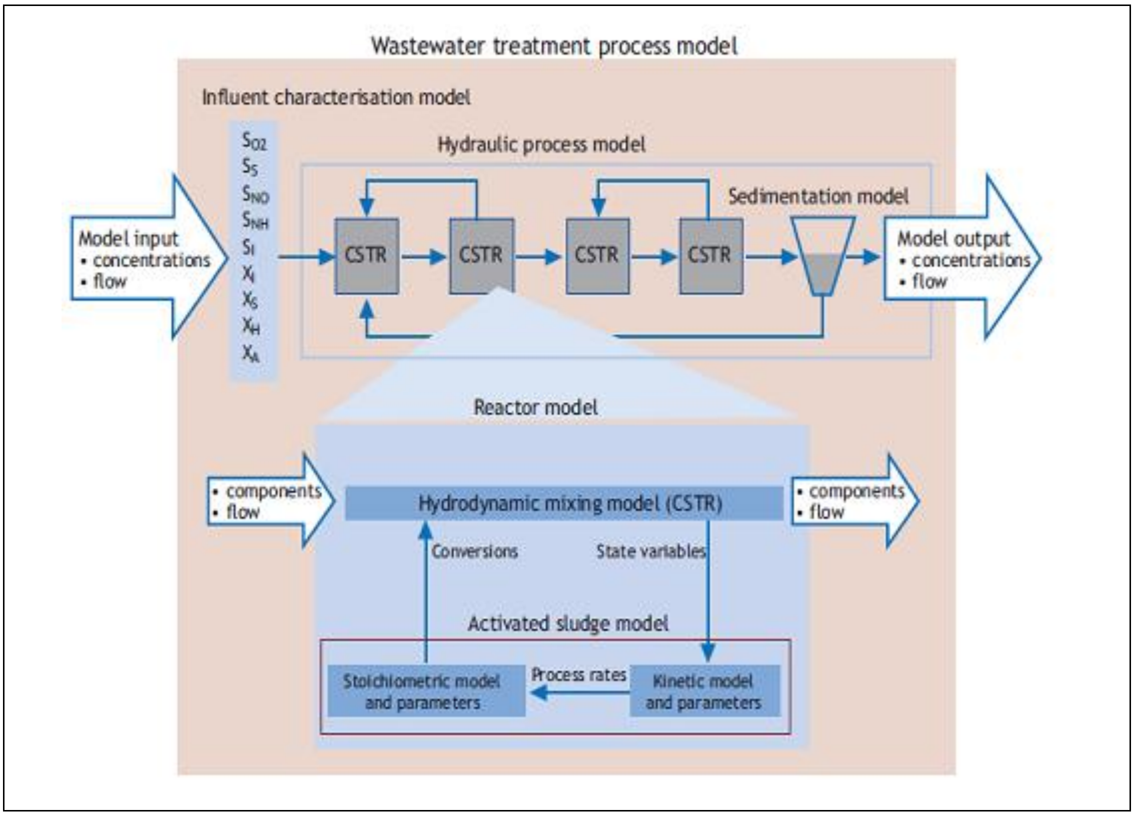

Figure 2.9 **S**chematic representation of a complete wastewater treatment plant model (Meijer, 2004).

The wastewater treatment plant is modeled hydraulically describing the different zones/reactor compartments of the plant, including the settler. Each reactor compartment is modeled individually for its mixing and mass transfer (e.g. aeration) characteristics. Usually a completely mixed tank reactor is used. So effectively there are four models: the process model, the hydraulic model, the reactor/compartment model and finally the activated sludge model.

The focus is on the recent developments of activated sludge models, mainly the family of activated sludge models developed by the International Water Association (IWA) and the metabolic model developed at the Delft University of Technology (TUDP model). Table 2.3 summarizes essential features of these and several other activated sludge models (Biological wastewater treatment).

Table 2.3 Overview of selected activated sludge models (Gernaey, 2004).

Model	Nitrification	Denitrification	Heterotrophic / autotrophic decay	Hydrolysis	EBPR	Denitrifying PAO	Lysis of PAO / PHA	Fermentation	Chemical P removal	Reactions	State variables	Reference
UCTOLD	●	●	DR, Cst	EA						8	13	Dold *et al.*, 1980, 1991
ASM1	●	●	DR, Cst	EA						8	13	Henze *et al.*, 1987
ASM3	●	●	ER, EA	Cst						12	13	Gujer *et al.*, 1999
UCTPHO	●	●	DR, Cst	EA	●		Cst	●		19	19	Wentzel, 1988, 1989a,b
ASM2	●	●	DR, Cst	EA	●		Cst	●	●	19	19	Henze *et al.*, 1995
ASM2d	●	●	DR, Cst	EA	●	●	Cst	●	●	21	19	Henze *et al.*, 1999
B&D	●	●	DR, Cst	EA	●	●	EA	●		36	19	Barker and Dold, 1997
TUDP	●	●	DR, Cst	EA	●	●	EA	●		21	17	Meijer, 2004
ASM3-bioP	●	●	ER, EA	Cst	●	●	EA			23	17	Rieger *et al.*, 2001

Den. PAO, Denitrifying PAO activity included in the model; DR, death regeneration concept; EA, electron acceptor depending; ER, endogenous respiration concept; Cst= not electron acceptor depending

In 1983, the International Association on Water Pollution Research and Control, later known as the International Association on Water Quality and now the International Water Association (IWA), formed a task group to develop a practical model for the design and operation of the biological wastewater treatment process. The product of the group's efforts is Activated Sludge Model No. 1 Henze (1987), introduced in 1987. Versions that expanded and improved upon the first model were introduced by the association in later years. They include Activated Sludge Model No. 2 Henze (1995), which incorporates phosphorus removal from wastewaters; Activated Sludge Model No. 2d Henze (1999), which accounts for the ability of phosphorus-accumulating organisms to use cell internal substrates for denitrification; and Activated Sludge Model No. 3 Gujer (1999), which does not include phosphorus removal but addresses problems found in the first model.

Membrane bioreactor (MBR) technology is a promising method for water and wastewater treatment because of its ability to produce high-quality effluent that meets water quality regulations. Due to the intrinsic complexity and uncertainty of MBR processes, basic models that can provide a holistic understanding of the technology at a fundamental level are of great necessity.

Models that can accurately describe the MBR process are valuable for the design, prediction, and control of MBR systems. Complex models that are also practical for real applications can greatly assist in capitalizing on the benefits of MBR technology.

2.5.1 MBR Design (ATV-DVWK-A 131 E)

2.5.1.1 Plants with Nitrification

The aerobic dimensioning sludge age to be maintained for nitrification ($t_{ss,aerob}$);

$t_{ss,aerob} = SF*3.4*1.103^{(15-T)}$ ($t_{ss,aerob}$: Aerobic sludge age referred to V_N)

The value of 3.4 is made up from the reciprocal of the maximum growth rate of the ammonia oxidants at 15°C (2.13 d) and a factor of 1.6. Through the latter it is ensured that, with sufficient oxygen that and no other negative influence factors, enough nitrificants can be developed or held in the activated sludge.

Using the safety factor (SF) the following are taken into account;

- Variations of the maximum growth rate caused by certain substances in the wastewater, short-term temperature variations or/and pH shifts.
- The mean effluent concentration of the ammonium.
- The effect of variations of the influent nitrogen loads on the variations of the effluent ammonia concentration.

Based on all experiences it is recommended, for municipal plants with a dimensioning capacity up to $B_{d,BOD}$=1200 kg/d (20000 PT), to reckon with SF=1.8 due to the more pronounced influent load fluctuation and for $B_{d,BOD}$>6000 kg/d (100000 PT) with SF=1.45 (SF: Safety factor for nitrification,$B_{d,BOD}$:The organic loading).

2.5.1.2 Plants with Nitrification and Denitrification

For nitrification and denitrification the total dimensioning sludge age ($t_{ss,aerob,dim}$);

$$(t_{ss,aerob,dim}) = t_{ss,aerob} * (\underline{1}/1-(V_D/V_T))$$

($t_{ss,aerob,dim}$: Aerobic sludge age upon which dimensioning for nitrification is based)

(V_D/V_T:Volume of the biological reactor used for denitrification)

2.5.1.3 Determination of The Proportion of The Reactor Volume for Denitrification

For designing of nitrogen removal systems, denitrified nitrate is, $S_{NO3,D}$ (mg/l);

$$S_{NO3,D} = C_{N,IAT} - S_{orgN,EST} - S_{NH4,EST} - S_{NO3,EST} - X_{orgN,BM}$$

The influent nitrate concentration ($S_{NO3,IAT}$) is in general, negligibly small.The concentration of organic nitrogen in the effluent can be set as $S_{orgN,EST}$=2 mg/L. To be on the safe side the ammonium content in the effluent for dimensioning is, as a rule, assumed as $S_{NH4,EST} = 0$.

Nitrogen requirement for biomass, $X_{orgN,BM}$ (mg/L);

$$X_{orgN,BM}= 0.04 \text{ to } 0.05 * C_{BOD,0}$$

$S_{NO3,D}/C_{BOD,IAT}$ gives the necessary denitrification capacity;

$$S_{NO3,D}/C_{BOD,IAT} = (0.75 * OU_{C,BOD} / 2.9) * V_D / V_{AT}$$

With the relevant BOD_5 of the inflow to the biological reactor one obtains the ratio $S_{NO3,D}/C_{BOD,IAT}$ which gives the necessary denitrification capacity.

Where:

$S_{NO3,D}$: The daily average nitrate concentration

$C_{N,IAT}$: Influent nitrogen concentration

$Sorg_{N,EST}$: Organic nitrogen in the effluent

$S_{NH4,EST}$: The ammonium content in the effluent

$S_{NO3,EST}$: The relevant effluent concentration of the nitrate

$X_{orgN,BM}$: The nitrogen incorporated in the biomass

Table 2.4 Standard values for the dimensioning of denitrification for dry weather at temperatures from 10°C to 12°C and common conditions (ATV-DVWK 131-E, 2000).

V_D/V_{AT}	$S_{NO3,D}/C_{BOD,IAT}$	
	Pre-anoxic zone denitrification	Simultaneous denitrication
0.2	0.11	0.06
0.3	0.13	0.09
0.4	0.14	0.12
0,5	0.15	0.15

Standard values for the dimensioning of denitrification for dry weather at temperatures from 10°C to 12°C and common conditions are given in Table 2.4. Denitrification volumes smaller than V_D / V_{AT}= 0.2 and greater than V_D / V_{AT}= 0.5 are not recommended. For temperatures above 12°C the denitrification capacity can be increased by capacity 1% per 1 °C.If the dimensioning or re-calculation takes place on the basis of COD, one can reckon with $S_{NO3,D}/C_{BOD,IAT}$ = 0.5 * ($S_{NO3,D}$ /$C_{BOD,IAT}$).

If the required denitrification capacity is larger than, $S_{NO3,D}/C_{BOD,IAT}$ = 0.15 , then a further increase of V_D / V_{AT} is not recommended. It is to be investigated whether a volume reduction or partial by-passing or primary settling tank and/or, if applicable, a separate sludge treatment are conducive to meeting the target. An alternative is to carry out the planning for the addition of external carbon.

2.5.1.4 Phosphorus Removal

Phosphorus removal can take place alone through simultaneous precipitation, through excess biological phosphorus removal, as a rule combined with simultaneous precipitation and through pre- or post precipitation.

Anaerobic mixing tanks for biological phosphorus removal are to be dimensioned for a minimum contact time of 0.5 to 0.75 hours, referred to the maximum dry weather inflow and the return sludge flow ($Q_{DW,h}$ + Q_{RS}). The degree of the biological phosphorus removal depends, other than on the contact time, to a large extent on the ratio of the concentration of the readily biodegradable organic matter to the concentration of phosphorus. If, in winter, the anaerobic volume is used for denitrification, then during this period a lower biological excess phosphorus removal will establish.

For thedetermination of the phosphate to be precipitated a phosphorus balance, if necessary for different types of load, is to be drawn up:

$$X_{P,Prec} = C_{P,IAT} - C_{P,EST} - X_{P,BM} - X_{P,BioP}$$

$X_{P,BioP}$ = 0.01 to 0.015 * $C_{BOD,IAT}$ or 0.005 to 0.007 $C_{COD,IAT}$ respectively with upstream anaerobic tanks;

- if, with lower temperatures,$S_{NO3,EST}$ increases to ≥15 mg/l, it can be assumed: $X_{P,BioP}$ = 0.005 to 0.01 $C_{BOD,IAT}$ or0.0025 to 0.005 * $C_{COD,IAT}$ respectively with upstream anaerobic tanks
- in plants with pre-anoxic zone denitrification or step-feed denitrification,but without anaerobic tanks,an excess biological phosphorus removal of $X_{P,BioP} \leq 0.005 * C_{BOD,IAT}$ or0.002 * $C_{BOD,IAT}$ respectively can be assumed.
- if, at low temperatures, the internal recirculation of pre-anoxic zone denitrification is discharged into the anaerobic tank, one can reckon with $X_{P,BioP} \leq 0.005 * C_{BOD,IAT}$ or0.002 * $C_{BOD,IAT}$ respectively. Where:

$X_{P,BioP}$: The excess biological phosphorus removal

$X_{P,Prec}$:Concentration of phosphorus removed by simultaneous precipitation

$X_{P,BM}$: Concentration of phosphorus embedded in the biomass

2.5.1.5 Determination of the Sludge Production

The sludge produced in an activated sludge plant is made up of organic matter resulting from degradation and stored solid matter as well as sludge resulting from phosphorus removal:

Sludge production for biological phosphorus removal is (SPd,P);

$$XP,BioP = 0.01 * CBOD,0$$

$$SPd,P = Q * 3 * XP,BioP$$

Sludge production for carbon removal is, (SPd,C);

SPd,C (COD based) = Q * ((XWAS,COD/0.8) * 1.45 + Xf) / 1000

SPd,C (COD based) = Bd,BOD*(0.75+0.6*(XSS,IAT/CBOD,IAT) – ((1-0.2)*0.2*0.17*0.75tss*FT) / 1 + 0.17 * tss * FT

Total sludge production is, (SPd);SPd = SPd,C + SPd,P

2.5.1.6 Assumption of the Sludge Volume Index and the Mixed Liquid Suspended Solids

The sludge volume index depends on the composition of the wastewater and the mixing characteristics of the aeration tank (Table 2.5). A high fraction of readily biodegradable organic matter, as are contained in some commercial and industrial wastewater, can lead to higher sludge volume indices.

Table 2.5 Standard values for the sludge volume index (ATV-DVWK 131-E, 2000)

Treatment target	SVI (l/kg)	
	Favourable	Unfavourable
Without nitrification	100-150	120-180
Nitrification and denitrification	100-150	120-180
Sludge stabilization	75-120	100-150

If no usable data are available, the values listed in table are recommended for dimensioning taking into account critical operating conditions.

The respectively lower values for the SVI can be applied, if

- primary settling is dispended with
- a selector or/an anaerobic mixing tank is placed upstream
- the biological reactor is designed as a cascade (plug flow)

The concentration of mixed liquid suspended solids (SS_{AT}) is determined in the process of dimensioning the secondary settling tank.

2.5.1.7 Volume of the Biological Reactor

The required mass of suspended solids in the biological reactor is, (MSS,AT);

$$MSS,AT = tss,Dim * SP,d$$

The volume of the biological reactor is, VAT;

$$VAT = MSS,AT / SSAT$$

2.5.*1.8 Required Recirculation and Cycle Time*

The necessary total recirculation flow ratio (RC) for pre-anoxic zone denitrification results using $S_{NH4,N}$, the ammonium nitrogen concentration to be nitrified, as follows:

$$RC = SNH_{4,N}/SNO_{3},EST - 1$$

2.5.*1.9 Oxygen Transfer*

The oxygen uptake is made up of the consumption for carbon removal (including the endogenous respiration)and, if necessary, the requirement for nitrification as well as the saving of oxygen from denitrification.For carbon removal the following approach, using the Hartwig coefficients, oxygen transfer for carbon removal is, (OUd,C):

$OU_{d,C}$(BOD based) = Bd,BOD *(0.56 + (0.15 * tss * FT) / (1+0.17*tss*FT)

$OU_{d,C}$ (BOD based) = Q * (CCOD - (SI - XWAS,COD)) / 1000

For nitrification the oxygen consumption is assumed to be 4.3 kg O_2 per kg oxidized nitrogen taking into account the metabolism of the nitrificants.

Oxygen transfer for nitrification is, $OU_{d,N}$;

$$OU_{d,N} = Qd * 4.3 * (S_{NO3,D} - S_{NO3,IAT} + S_{NO3,EST}) / 1000$$

For denitrification one reckons for carbon removal with 2.9 kg O_2 per kg denitrified nitrate nitrogen.For denitrification oxygen credit is, $OU_{d,D}$;

$$OU_{d,D}= Qd * 2.9 * S_{NO3,D} / 1000$$

The oxygen uptake rate for the daily peak is, (OU_h);

$$OU_h= (fc*(OU_{d,C}- OU_{d,N})+f_N* OU_{d,N})/ 24$$

The peak factor f_C and f_N represents the ratio of the oxygen uptake rate for carbon and nitrogen removal in the peak hour to the avarage daily oxygen uptake rate (Table 2.6).

Table 2.6 Peak factors of the oxygen uptake rate (ATV-DVWK 131-E, 2000)

	Sludge age (d)					
	4	6	8	10	15	25
f_C	1.3	1.25	1.2	1.2	1.15	1.1
f_N for $B_{d,BOD}$≤1,200 kg/d				2,5	2	1.5
f_N for $B_{d,BOD}$≤6,000 kg/d			2	1.8	1.5	

2.5.1.10 Suspended Solids Concentration in the Return Sludge

The achievable suspended solids concentration in the bottom sludge SS_{BS} can be estimated empirically in dependence on the sludge volume index SVI and thickening time.The suspended solids concentration in the bottom sludge, SS_{BS};

$$SS_{BS} = 1000/SVI * (t_{Th})^{3/2}$$

The suspended solids concentration of the return sludge (SS_{RS}), as a result of the dilution with the short-circuit sludge flow; can be assumed in simplified form to be:

with scraper facilities - $SS_{RS} \equiv 0.7 *SS_{BS}$

with scraper facilities - $SS_{RS} \equiv 0.5$ to $0.8 * SS_{BS}$

The suspended solids concentration of the return sludge, (SS_{RS});

$SS_{RS} \equiv 0.7 * SS_{BS}$

f_{obs} = (dilution factor of return sludge) = 0.7

2.5.1.11 Surface Overflow Rate and Sludge Volume Surface Loading Rate

The surface overflow rate q_A is calculated from the permitted sludge volume loading rate q_{SV} and the diluted sludge volume DSVas:

$$q_A = q_{SV} / D_{SV} = q_{SV} / SS_{EAT} * SVI$$

In order to keep the concentration of suspended solids $X_{SS,EAT}$ and the resulting COD and phosphorus concentration in the effluent of horizontal flow secondary settling tanks low, the following sludge volume loading rate q_{SV} shall not be exceeded:

$q_{SV} \leq 500$ L/m^2 h for $X_{SS,EST} \leq 20$ mg/L

For mainly vertical flow secondary settling tanks, the following applies with the formation of a close sludge blanket or with an easily flocculating activated sludge:

$q_{SV} \leq 650$ L/m^2 h for $X_{SS,EST} \leq 20$ mg/L

The surface overflow rate q_A shall not exceed 1.6 m/h with predominantly horizontal flow secondary settling tanks, and with predominantly vertical flow secondary settling tanks it shall not exceed 2.0 m/h.

2.6 Energy Consumption in Conventional and MBR Systems

A large amount of energy depends on both treatment process and used equipment is consumed during operation of wastewater treatment plants. There are a lot of different treatment plant equipments that consume either too little or too much energy. Saving of energy in wastewater treatment plants has become an important topic due to the ever increasing energy costs in Turkey. Energy management in wastewater treatment plants means “meeting the desired standards of discharge limits of treated wastewater with minimum cost and provide optimum and continuous energy for sustainable development” (Demir,2010).

Pumps are used to transfer liquid from a lower level to a higher level. Most of the energy in treatment plants is consumed by these pumps. Areas where there is little or no need for such pumps are ideal for treatment plant constructions. If wastewater can be transferred to the plant with gravity or little pumping, a great deal of energy shall be saved during the operation of the plant(IWSA,1999).

The energy to be consumed in wastewater treatment plants also depends on the used equipments. Significant amount of energy can be saved by choosing treatment equipments with high efficiency and low energy demand. Especially, choosing the energy efficient both pumps and blowers that intensively consume energy shall enable energy saving as well (Demir,2010).

Process based energy consumption distributions in classical systems and purification capacity- energy need in classic systems are shown in Figure 2.10 and 2.11, respectively (Insel, 2008).

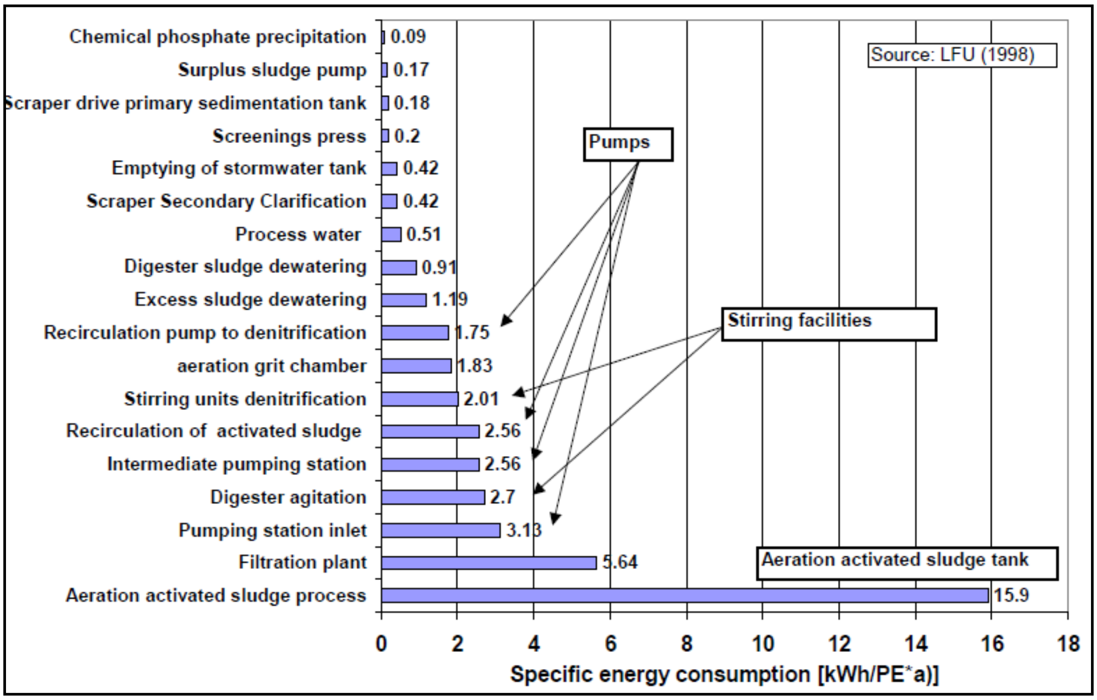

Figure 2.10 Process based energy consumption distributions in classical systems (Insel, 2008)

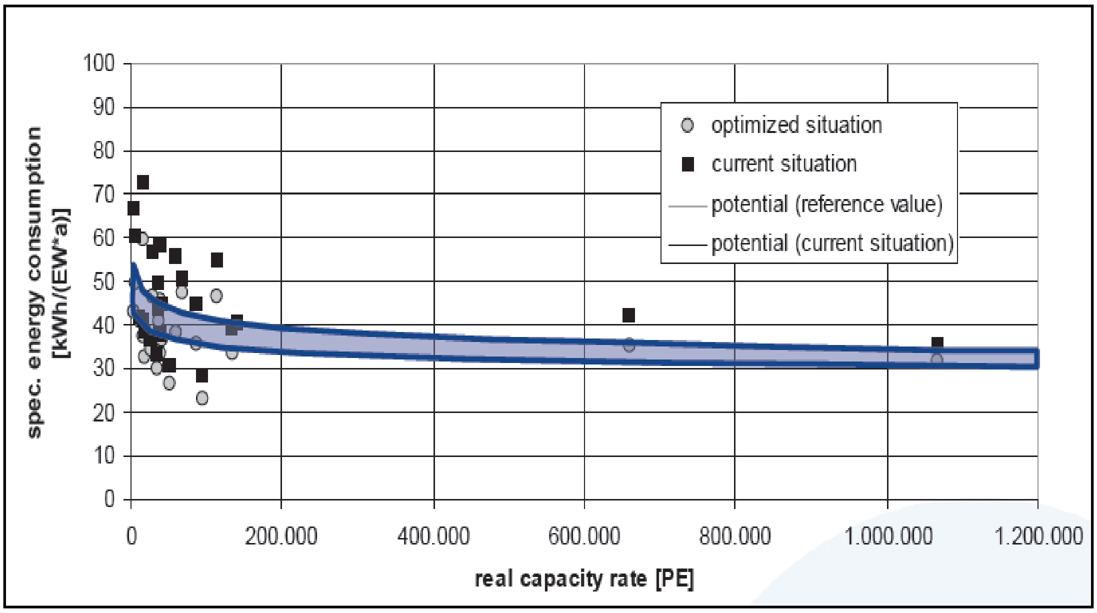

Figure 2.11 Purification capacity- energy need in classic systems (Insel,2008)

Historically, the energy requirement for an MBR typically exceeded that of a conventional activated sludge plant by a factor of 1.5 to 3 (Wallis, 2009). A summary of the energy requirement for various operating MBRs is provided in Table 2.7.

Table 2.7 Energy requirements for various operating MBR (Wallis, 2009)

Plant	Capacity MLD	MBR Type	Start-up Year	kWh/m^3
Brescia, Italy	42	Zenon	2003	0.85
Schilde,Belgium	8.5	Zenon	2004	0.62
Seelscheid, Denmark	11	Kubota	2004	0.9 – 1.7
Nordkanal, Germany	17	Zenon	2004	0.9
Varsseveld, NL	18	Zenon	2005	0.9
UluPandan, Singapore	23	Zenon	2006	0.55
METU-Ankara,Turkey	0.8	Huber	2007	1.8
Konacık,Turkey*	1.5	Kubota	2009	1.72

*For reuse energy consumption is % 25of total consumption.

In METU-MBR wastewater purification center, consumed energy for per m^3 waste water changes between 1-2.25kWh and mean is about 1.8 kWh/m^3.

In Ankara, the electricity price is nearly $0.12/kWh, so treatment cost for per m^3 changes between $ 0.12 -0.25.

In Konacik Wastewater Treatment Plant, consumed electric value for per m^3 is between 1.4 and 2.10 kWh and mean 1.72 kWh/m^3 (Erol, 2011). Energy consumption distribution depending on the treatment units is shown in Figure 2.12.

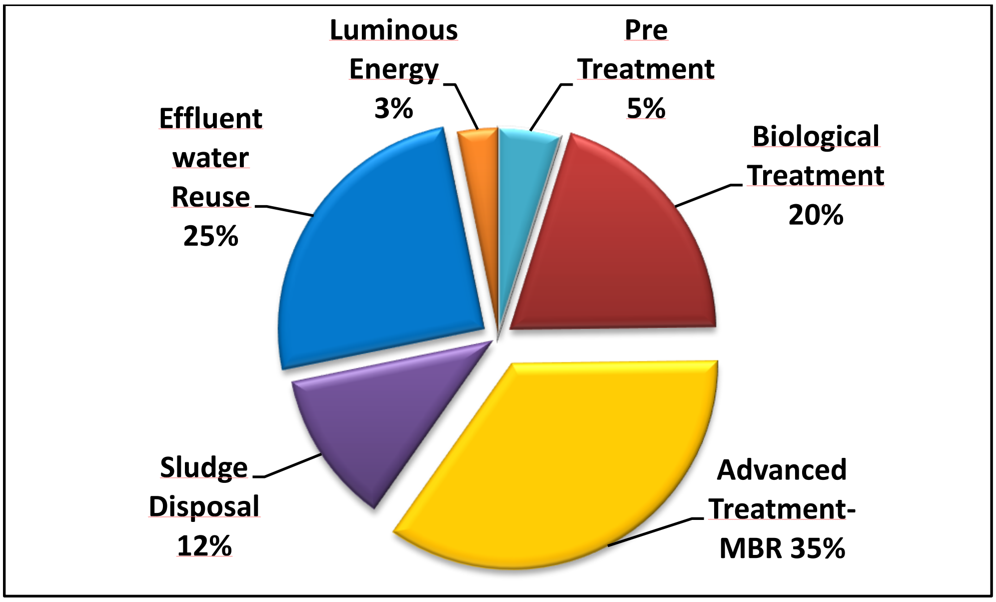

Figure 2.12 Energy consumption rates of Konacik Municipality wastewater treatment plant (Erol, 2011)

CHAPTER THREE
MATERIAL AND METHODS

3.1 Pilot Plant

In this study, Konacık Municipality in Bodrum Domestic Wastewater Treatment plant was selected as pilot plant (Figure 3.1). There is a Membrane Bioreactor with Ultrafiltration system and it is the first flat sheet membrane treatment plant for a municipality in Turkey. The treatment plant serves 10000 people and treats 1500 m^3/day and it has been operating by KonacıkMunicipality since 2008.

Figure 3.1 General view of Konacık wastewater treatment plant

3.1.1 Units of Konacık Wastewater Treatment Plant

KonacıkMunicipality wastewater treatment plant mainly consists of pre-treatment units (coarse and fine screen, grit removal, and equalization), biological treatment units (MBR system) and sludge dewatering systems. Flow diagram of Konacık wastewater treatment plant is given in Figure 3.2.

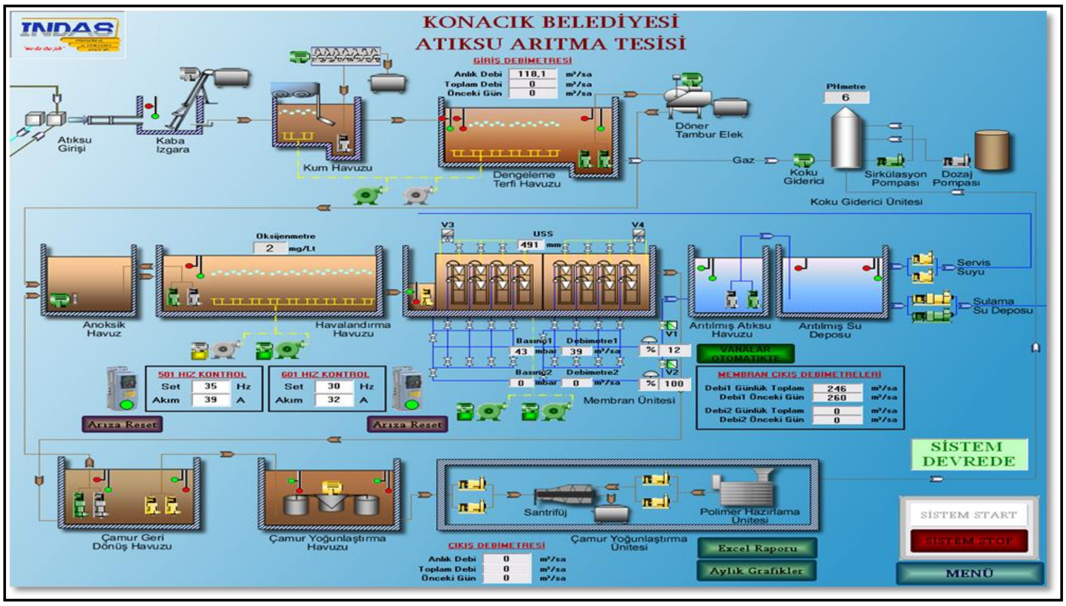

Figure 3.2 Flow diagram of Konacık wastewater treatment plant

3.1.1.1 Pre-treatment Units

It contains filtration and precipitation processes that used for separating sinkable and swimmer solid materials in wastewater. Wastewater input manhole is formed of coarse grid, grid collector unit, balancing and promotion pool and fine grid parts (Figure 3.3).

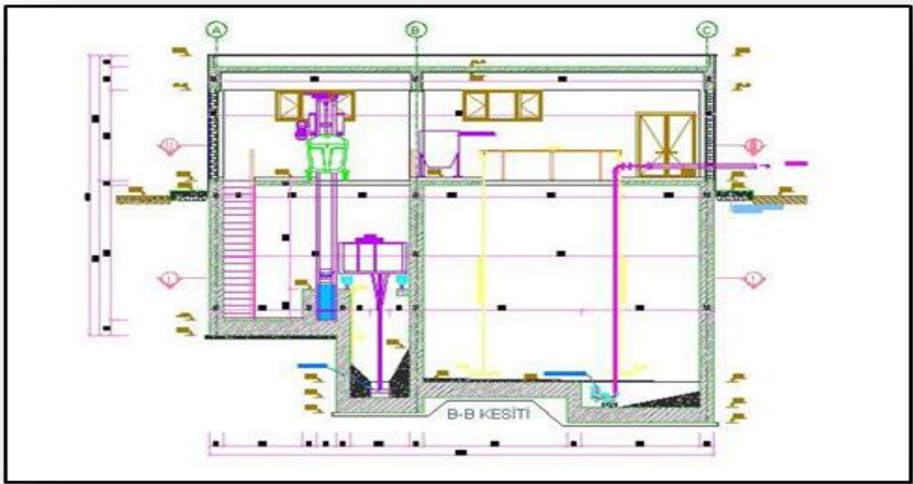

Figure 3.3 Pretreatment units

Coarse Screen (Screening Process):

Grids are used to protect the pumps, valves, pipe lines and other lines from damages by rags and other big objects. Gridding part is formed of parallel stick structure. Space between sticks is 20 mm and grid stick wideness is 10 mm. Scope of grid is 70 º angles (Figure 3.4).

Figure 3.4 Coarse screen

Grid collectors are used to protect dynamic mechanic equipments from rubbing and to reduce the accumulation of solid materials that leads to collapsing in pipe lines and ducts. Removing process is worked out as sand, stone and ash settles in waste water (Figure 3.5).

Figure 3.5 Grid collector mobile bridge

Equalization Tank:

Equalization basin is used to protect biological systems from shock hydraulic and organic loads. In Figure 3.6, photo of the equalization tank in shown.

Figure 3.6 Equalization tank

Rotary Screen (Fine Screen) :

Fine grid is used for removing slimmer solid materials that cannot hold on to the coarse screen. Clear openings are 2 mm (Figure 3.7). Basket screenwith 2 mm openings is used for safety after rotary screen equipment (Figure 3.8).

Figure 3.7 Rotary Screen View

Figure 3.8 Basket screen

3.1.1.2 Biological Treatment Units

The aim of biological treatment of wastewater is to disrupt organic and partly inorganic polluting materials in wastewater by microorganisms. In Konacık Wastewater Treatment Plant, MBR system is used followed by anoxic tank.

Anoxic Tank :

Feed wastewater entered the anoxic tank where nitrate (NO_3) is converted to nitrogen gas (N_2). This process is known as denitrification. The anoxic basin is mixed, but not aerated. A submersible propeller is used to prevent sedimentation. Volume of the anoxic tank is 270 m^3 (Figure 3.9).

Figure 3.9 Anoxic tank

MBR System:

The MBR is an activated sludge process with ultrafiltration membrane filtration on low pressure to separate solid/liquid material. It is possible to operate MBR processes at higher mixed liquor suspended solids (12.000 – 18.000 mgMLSS/L) concentrations compared to conventional settlement separation systems, thus reducing hydraulic retention time and the reactor volume to achieve the same loading rate. So, investment costs of MBR systems are lower than for conventional wastewater treatment plants.High quality effluent can be obtained with MBR systems because of the low porosities of membranes.

MBR process is formed of membrane plates that settled to units and air under units are ventilated by bubble system. Membrane plates are submerged in activated sludge. Air bubbles provide membrane scouring. Air is used to control membrane fouling, provide sufficient amount of oxygen, and enhance filtration efficiency.

In MBR system flat plate submerged membrane is used and it was purchased from a Japan's company called KUBOTA (Figure 3.10). These flat plate membranes that are set into cassette are installed to the effluent of aeration tank.

The membrane specifications which are produced by KUBOTA are listed as follows:
Type 203: 226 mm x 316 mm x 6 mm (Active surface area 0.10 m^2/each)
Type 510: 490 mm x 1000 mm x 6 mm (Active surface area 0.80 m^2/each)

Because of there is any effluent hole, the water in the tank that membrane cassettes are set in tries to flow between membrane cassettes inside of membrane panels every which have effluent tubes. By the way the water which has been filtered flows in the tube pipes to effluent manifold after leaving solid material outside. The water pressure over the membranes (about 1.00-1.20 m) produces the energy which pushes the water into the effluent manifold. With the help of the water pressure, system does not need extra energy to push the water into the effluent manifold.

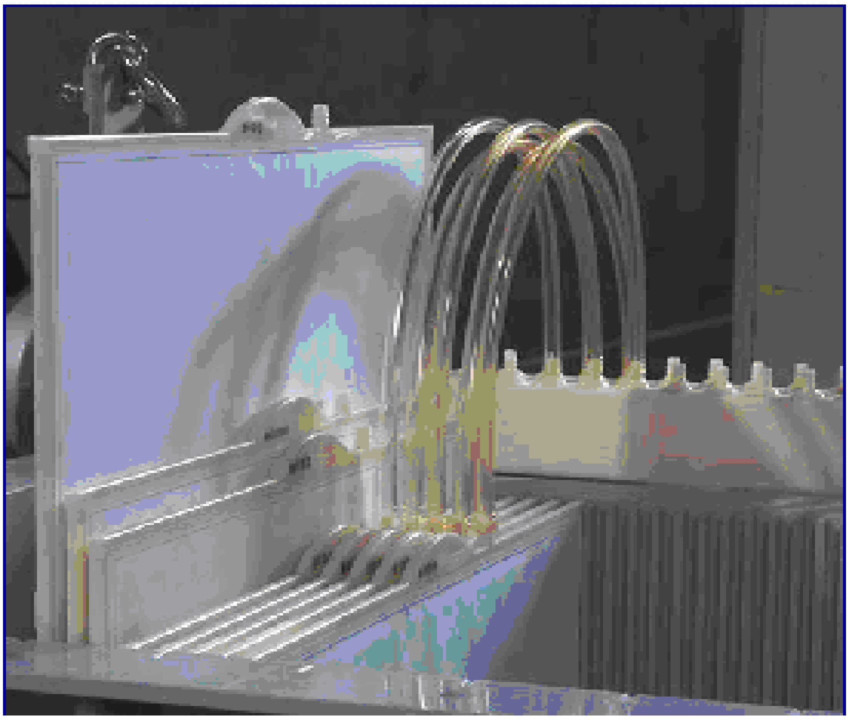

Figure 3.10 Flat plate membranes

Activating of The Membranes:

Because membrane plates were hydrophobic at the beginning, they were not contacted with water during installation (Figure 3.11). After installation period, membranes were waited in the clean water for 2–3 days (Figure 3.12). During this period, they sucked enough water; the chemical protection material dissolved and they became hydrophilic. When the MLSS concentration in aeration tank reached to the required levels (minimum 4000 mg/L), clean water was emptied from the tank and wastewater was pumped into the tank and aeration was started.

Figure 3.11 MBR installation

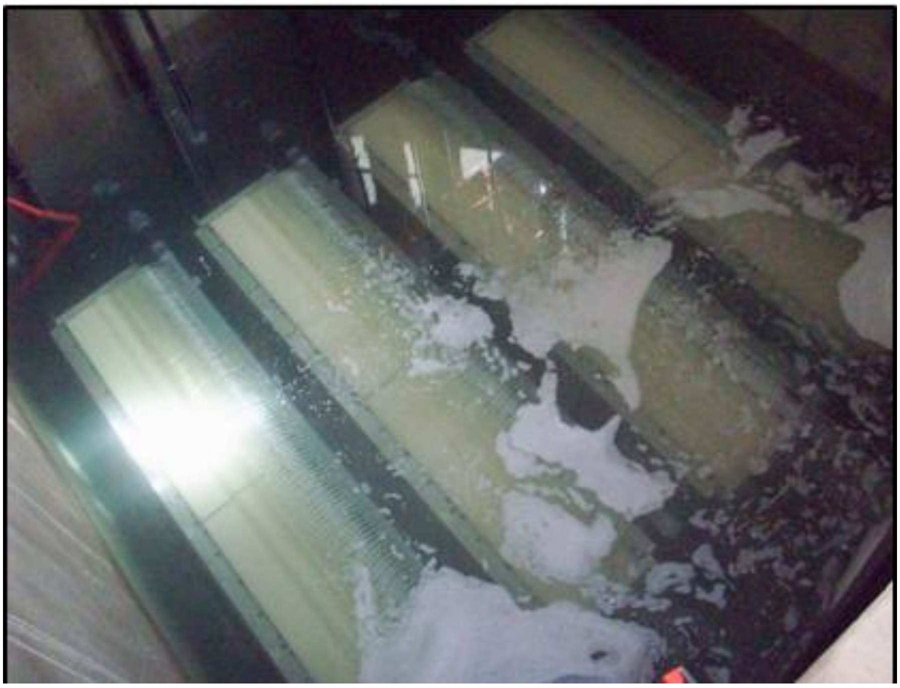

Figure 3.12 Clean water trials

Aeration:

Air diffusers under the membrane panels produce bubbles which set water flow through membrane panels with the velocity of 0.5 m/s upwards. This is necessary to create flow for filtration and to keep membrane surface's clean.

Cleaning The Air Line And Diffusers :

It is not enough to keep air flow instantaneously and affective to clean up MLSS membrane tanks. The air flow velocity must be equal in every membrane unit. Diffuser must be purified from sludge to keep them clean automation system deactivate pneumatic vanes (3 times in every 8 hours). This reduces water level. Ultrasonic sensors sense this situation and controls pneumatic vanes automatically. The materials inside the aeration pipes and pneumatic vanes are sprayed back into tank (Figure 3.13).

Figure 3.13 MBR automation system

Chemical Cleaning :

Cleaning with air is not enough to clean up organic materials, oil, and detergent. If these materials are not cleaned, capacity of membrane process will decrease. Decrease of capacity (advised minimum values are 5–50 mL/5 min) can be noticed daily filtering test and with the help of pressure sensors which are related to automation system. Automation system sends signal to notice chemical cleaning time. On the other way cleaning the system every 6 months is a routine. To clean up organic material and inorganic materials, diluted Sodium Hypochlorite (%0.5) and oxalic acid (%1) is used, respectively.

Chemical substances are kept with 3 L cartridges. At the end of membrane cleaning, chemical solution is pumped to pumping station and diluted with the incoming flow. It is unnecessary to shut down the whole process, empty the membrane tanks or take out membrane units when acting the cleaning process.

3.1.1.3 Sludge Dewatering Process

In Konacık Wastewater Treatment Plant, about 12,275 tons of sludge produced in 2010. 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 Konacık Wastewater Treatment Plant, waste sludge is pumped to the thickener. However since there is an aeration equipment (Figure 3.14), this tank serves as an aerated stabilization unit. After then sludge is dewatered with a decanter centrifuge(Figure 3.15) with adding polymer conditioner. Dewatered sludge is transmitted to solid waste storage area (Figure 3.16). Supernatant is pumped to the equalization basin.

Figure 3.14 Thickening tank

Figure 3.15 Decanter centrifuge

Figure 3.16 Sludge cake

3.1.1.4 Reuse Applications

Because there is a limited fresh water sources in Bodrum, treated water should be reused. 95% of Konacık WWTP effluent is reused especially for irrigation purposes (Figure 3.17) and fire protection (Figure 3.18). So, almost zero effluent discharge occurs in Konacık.

Figure 3.17 Irrigation systems

Figure 3.18 Fire protection

3.1.1.5 Environmental Analyses Laboratory

Most of the required operating parameters can be analyzed in an Environmental Analyses Laboratory placed in the wastewater treatment plant (Figure 3.19). Besides, this accredited laboratory serves for some water, wastewater, and microbiological analyses to Bodrum and around for this region.

Figure 3.19 Environmental analyses laboratory

3.1.2 Operational Conditions of Konacık Wastewater Treatment Plant

Influent wastewater properties of Konacık Wastewater Treatment Plant are given in Table 3.1 and acceptable wastewater pollution loads are shown in Table 3.2. General effluent characteristics are given in Table 3.3.

There is no industrial discharge to the wastewater treatment plant. Industrial wastewaters containing heavy metals and oil and grease are not accepted to the plant.

Table 3.1 Influent wastewater properties of Konacık WWTP

Parameter	Unit	Winter	Summer	Mean
Flowrate	m^3/d	1050	1090	1070
Temperature	^oC	17	24	20
COD	mg/L	479	380	430
BOD	mg/L	234	210	222
TKN	mg/L	66	62	64
NH_4-N	mg N/L	58	54	56
TP	mg P/L	4.30	3.50	3.90
SS	mg/L	165	253	209
VSS	mg/L	77	132	104.5
pH	-	8.10	7.68	7.89
TDS	mg/L	1700	1800	1750
Oil-Grease	mg/L	60	60	60

Table 3.2 Acceptable wastewater pollution loads for Konacık WWTP

Parameter	Unit	First Stage	Second Stage	Total
Flowrate	m^3/day	1500	3000	4500
BOD	kg/day	600	1200	1800
COD	kg/day	1000	2000	3000
SS	kg/day	900	1800	2700
Total N	kg/day	100	260	360
Total P	kg/day	30	60	90

Table 3.3 Discharge properties of Konacık WWTP

Parameter	Unit	Value
BOD_5	mg/L	<10
COD	mg/L	<25
SS	mg/L	<10
TN	mg/L	<15
TP	mg/L	<2
Turbidity	NTU	<1
pH	----	6 - 9

3.1.3 Operational Costs of Konacık Wastewater Treatment Plant

Daily and monthly operational cost is given in Table 3.4. Cost distribution and monthly costs are shown in Figure 3.20 and 3.21, respectively.

Table 3.4 Operational costs of Konacık WWTP in 2010

Time	**Facility Capacity - 2010**	**Operation Cost**	**Unit Cost**	**Cost Per Person**
	m^3	$	**$/m^3$**	$/**person/year**
Daily	1070	675.05	0.62	24.12
Monthly	32100	20101.65		

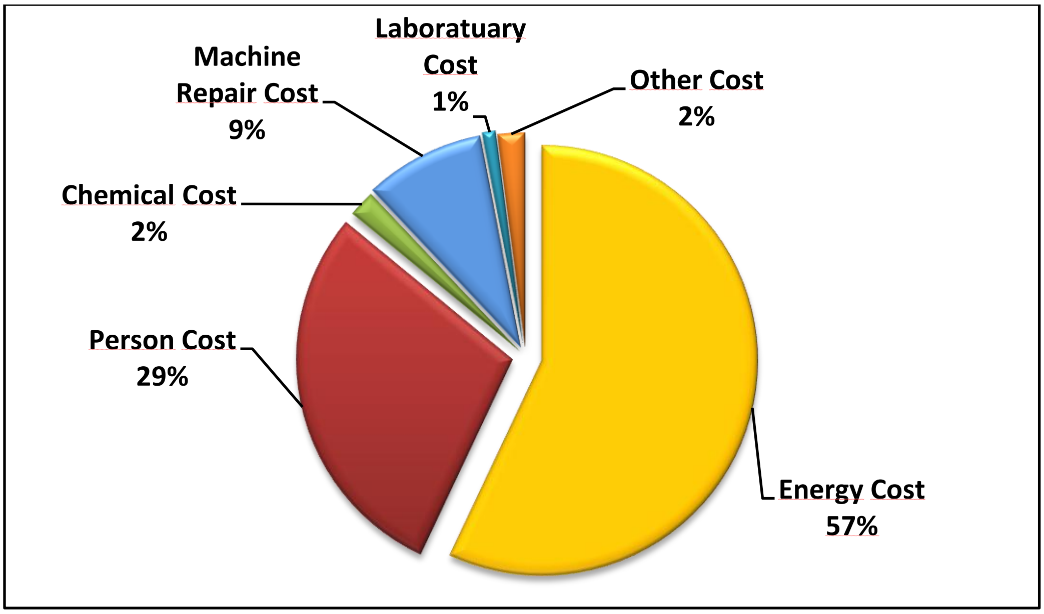

Figure 3.20 Cost distribution in 2010

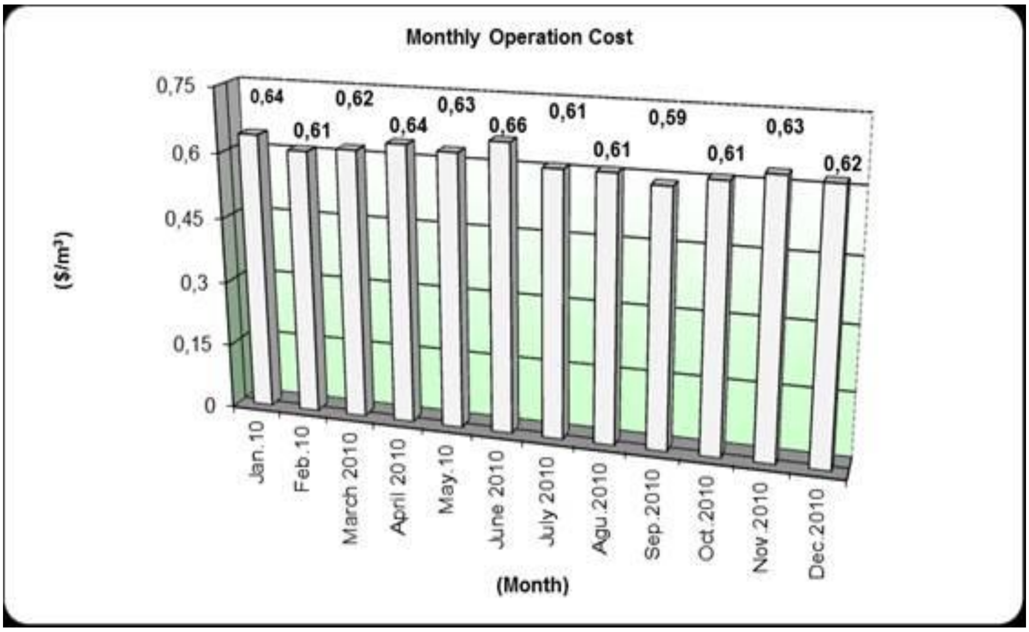

Figure 3.21 Monthly operational cost in 2010

3.1.4 Design of Treatment Plant Units

(Koza Engineering, 2008)

Flowrate:

First Level;

Q = 10000 x 150 = 1500000 L/day = 1500 m^3/day = 62.5 m^3/hour

Second Level;

Q = 20000 x 150 = 3000000 L/day = 3000 m^3/day = 125 m^3/hr

Influent Structure :

Sewer lines that collect wastewater around the region are separated with Ø600 andØ400 mm pipes and combines in the chimney of the wastewater treatment center. Connection will be formed with Ø600 mm pipe and 0.005 slope. Treatment plant influent structure will be designed to supply total discharge.

$Q_{project}$ = 62.5 + 125 = 187.5 m^3/hr= 0.052 m^3/s

Course Screen :

Mechanical cleaning type is used for course screen.Total course waste that accumulates on screenis transmitted to grid container and is removed from the system.

Channel amount = 1

Opening = 20 mm

Screen bar width = 10 mm

Channel width= 0.50 m

Equalization Basin:

$Q_{project}$ = 62.5 + 125 = 187.5 m^3/hr = 0.052 m^3/s

Volume of the tank; V = 187.5 m^3/hr x 0.50 hr = 93.75 m^3

Dimensions:

H_{water}= 2.00 m

L = 7.00 m

W = 7.55 m

V_{real}= 7.00 x 7.55 x 2.00 = 105 m^3

Diffusers are used to prevent precipitation and anoxic environment. It also provides homogenous mixing.

In first level:

Q = 1500 m^3/day = 62.5 m^3/hr= 0.017 m^3/s

t = 105 m^3 / 62.5 m^3/hr = 1.68 hr

Totally:

Q = 4500 m^3/day = 187.50 m^3/hr = 0.052 m^3/s

t = 105 m^3 / 187.50 m^3/hr = 0.56 hr

1+1 (reserve) pump is selected for first level and 1+1(reserve) pump is selected for second level in equalization basin.

Mechanical Fine Screen:

Rotary screen is used for fine screen. Total course waste that cumulates on screen is transmitted to grid container and is removed from the system.

For first level;Q = 75 m^3/hr

Screen interval = 2 mm

Drum Size = Ø 600 x 600 mm

Power = 0.55 kW

For second level;

Q = 187,5 m^3/hr

Screen interval = 2 mm

Drum Size = Ø 600 x 600 mm

Power = 0.55 kW

Aerobic Zone:

It contains two cells. Dimensions:

W = 10.00 m

L = 12,00 m

H_{water}= 5 m

Tank volume for selected size = 600 m^3

Volume required for denitrificationVd/Vt = 0.2 (accepted)

0. 2= Vd/Vd+Va (Vd=180 m^3)

Anoxic Zone:

Dimensions:

W = 3.00 m

L= 12.00 m

H_{water}= 5 m

MBR System:

First level;

Q = 1500 m^3/day = 62.5 m^3/hr= 0.017 m^3/s

Bioreactor includes anoxic, aerobic and membrane zones. It includes recirculation line to homogeny distribute all solid between process trains. Operation depth of aerobic zone is 4.5 m.Membrane Bioreactor Process design is suitable for conditions given in Table 3.5.

Table 3.5 Process conditions for membrane bioreactor design

Parameter	**Value**
Minimum Temperature, °C	15
MLSS in Bioreactor, mg/L	>8000(10000)
Minimum Sludge Age, day	10
Minimum F/M ratio, kgBOD/kgMLVSS/d	0.1
Minimum water depth, m	4.5

3.2 Simulation Program

3.2.1 Basic Principles

BioWin, is a versatile modeling area that designed for simulation on urban and industrial wastewater plants.BioWin use advanced interface for a dynamic and easier modeling. Process modeling, simulation technology, last innovations in graphic and performance tools helps simulation and evaluation of results. So, connection between different processes in plant can be observed in a dynamic and interactive way. To comprehend these connections are important for productive designing of wastewater treatment plant, controlling the design and operation optimizing.

BioWin is a Microsoft Windows-based simulator used world-wide in the analysis and design of wastewater treatment plants. Figure 3.22 shows an example of a nutrient removal system configuration set up in BioWin.Many different process units can be included to ‘build’ a specific treatment plant configuration.

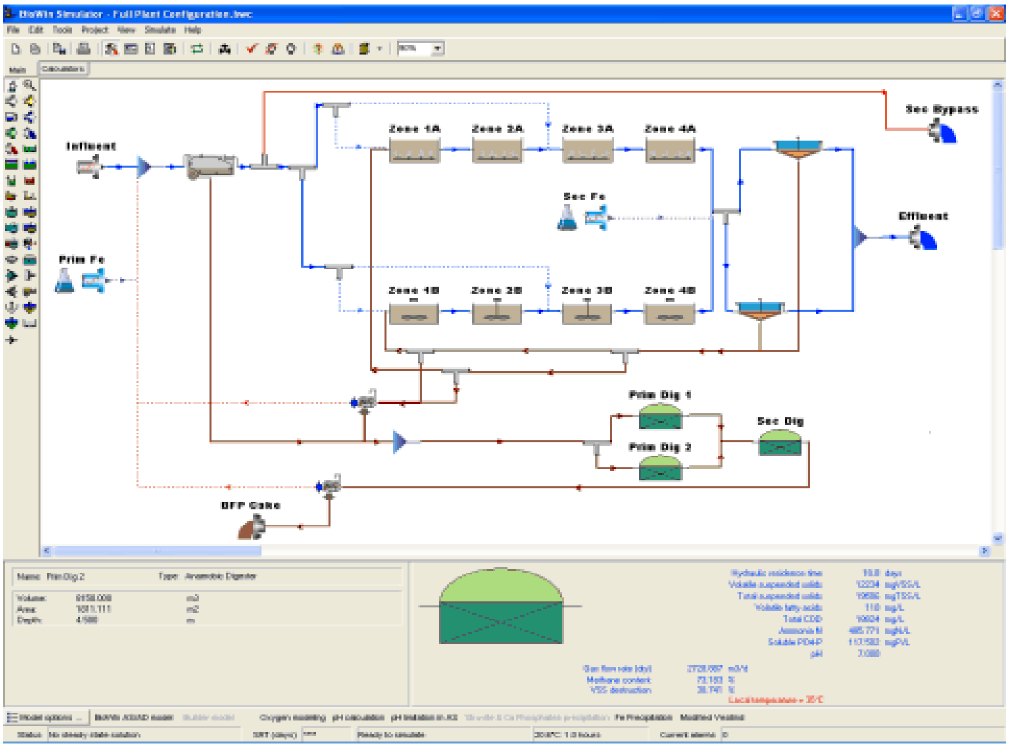

Figure 3.22 Example of a process configuration set up in BioWin (http://www.envirosim.com/products/bw32/bw32intro.php)

The facility to view simulation results rapidly, and in details of paramount importance in the design and analysis of systems.BioWin incorporates an Album for this purpose.The Album consists of a series of tabbed pages (somewhat like recent spreadsheet programs) showing simulation results in tubular and/or graphical format.

BioWin offers a number of features to aid in creating attractive, Professional reports, and includes its own internal.Notes editor help keep track of Project details.It is very easy to get results from BioWin into a word processor or spreadsheet. Charts, tables, system configuration layouts, etc. can be copied and pasted from BioWin to reports.Tables can be exported as tabbed text and then quickly converted to tables(http://www.envirosim.com/products/bw32/bw32intro.php).

3.2.2 Application to Membrane Bioreactor (MBR)

BioWin includes a new MBR module that simplifies system setup, and offers comprehensive operating and performance information.Membrane modules are characterized by the surface area and displaced volume per module (cartridge or cassette). The user specifies either the number of installed modules or the packing density (membrane area per unit volume). BioWin accounts for displaced liquid volume. The module provides many useful features; for example, independent membrane solids and colloidal retention settings. Output information on physical data (e.g. number of modules, surface area, displaced volume) and operating data (e.g. membrane flux) is accessed easily, in addition to standard bioreactor information (e.g.MLSS,MLVSS) (http://www.envirosim.com/products/bw32/BioWin3Flyer.pdf).

CHAPTER FOUR
RESULTS AND DISCUSSION

4.1 Operational Data of MBR System

The Membrane Bioreactor system was designed according to pre-denitrification principle and the removal of organic carbon and biological nitrogen is aimed. The total sludge age of the present system is at 48 day level and its design was realized according to extended aeration activated sludge principle. The average daily treated wastewater amount is 1070 m^3/day for the 1^{st}Stage. The flow scheme of the treatment plant is given in Figure 4.1. The present system contains 1 anoxic pool, 1 aerobic pool and 1 membrane pool. The present membrane pools contain a total of 8 membrane modules. The total surface area of the membranes is 2560 m^2. Air is given at 1800 Nm^3/hour by blowers to the membrane pool to avoid membrane clogging. The air used for backwashing of membranes is 0.75 Nm^3/m^2 for each membrane area and this value is approximately 40 Nm^3/m^3 per unit of treated wastewater.

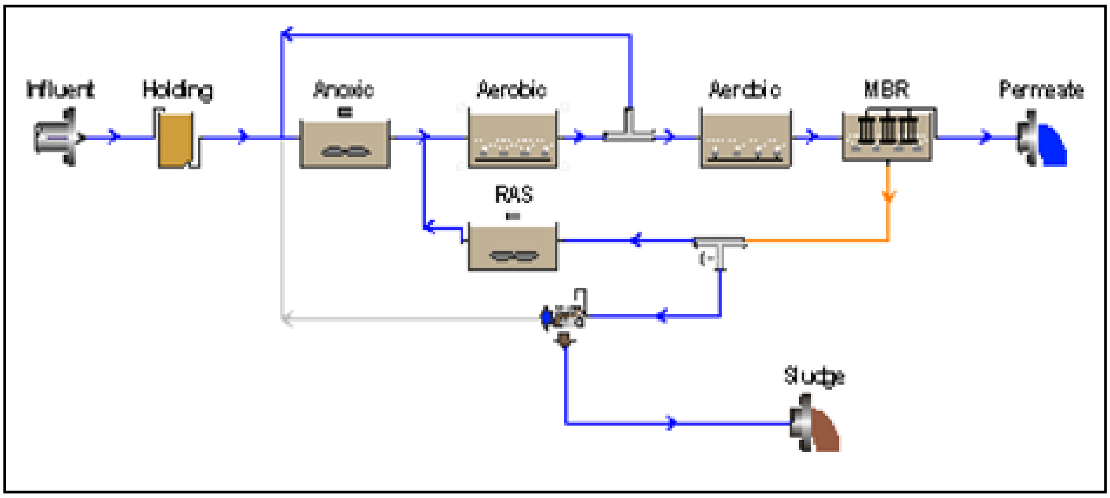

Figure 4.1 The flow scheme of Konacık WWTP

Anoxic/Total volume ratio (V_D/V) is 23%. The reactor volume,hydraulic retention time (HRT) and water depth of each unit and information for equipment used in the facility is given in Table 4.1and Table 4.2, respectively.Briefly, 2 different blower groups with equal capacity are used for aeration pool (1+1) and membranes (2+1). All blowers are connected to frequency converter and their air flow rates are adjustable.

Table 4.1 Reactor volume and dimensions

Reactor	**Wet Volume (m^3)**	**Water Depth (m)**	**HRT (hour)**
Equalization Tank	105.70	2.0	1.69
Anoxic Tank	270	5.0	4.32
Aerobic Tank	600	5.0	9.6
Membrane Tank	180	5.0	2.88
Sludge Recycle Chamber	30.80	5.0	0.49
Total	1186,5	-	18.98

Table 4.2 Membrane bioreactor system equipment and properties.

Equipment	**Reactor**	**Item**	**Properties**	**Power (KW)**
Submersible Mixer	Anoxic Tank	1	380V; 3ph; 50Hz	1.5kW
Blower-1	Aeration Tank	1+1	900 m^3/hour; 600mbar ROOTS; 380V; 3ph; 50Hz; IP55	30kW
Blower-2	Membrane Tank	1+2	2 items of 900 m^3/hour; 600mbar ROOTS; + 1 item of 1800 m3/hr 600 mbar ROOTS 380V; 3ph; 50Hz; IP55	45 kW+30 kW*2
Internal Recycle (IR) Pump	Aeration Tank	2	24,3L/s; 5.8mss Submersible; 380V; 3ph; 50Hz	2.0 kW
RAS Pump	Sludge RecycleChamber	2	175l/s; 6.1mss Submersible 380V; 3ph; 50Hz	15 kW

Accordingly, the total sludge age is adjusted in accordance with actual operation conditions in ATV-DVWK-A 131 E design and sludge amount, oxygen requirement, effluent quality and operational parameters are calculated. As derived from the Table 4.3, the anoxic/total volume ratio (VD/V) is higher in ATV-DVWK-A 131 E standard than the actual operation condition. However, it is possible to obtain denitrification with lower VD/V ratios for the same effluentquality. Raw wastewater properties are given in Table 4.4.

Table 4.3 Comparison of design method ATV-DVWK-A 131 E and operational conditions

Parameter	**Unit**	**Design ATV-131**	**Operational data**
Process			
Total Sludge Age, SRT	days	48	48
Anoxic Volume Ratio, V_D/V_T	%	43	23
Internal Return, IR	[-]	3.0	3.7
MLSS concentration*, X_{MLSS}	kgSS/m^3	10,200	10,000
Total Oxygen Requirement, OR_T	kgO_2/hour	20.3	-
Air Requirement, Q_{air}**	Nm^3/hour	-	460
Sludge Production, P_{XT}	kgDS/day	189	190
Effluent Quality			
COD	mgCOD/L		20
Ammonium, NH_4-N	mgN/L	0.5	0.2
NO_x-N	mgN/L	13.0	12.3
Total Nitrogen, TN	mgN/L	14.0	13.0
Total Phosphorus, TP	mgP/L	0.25	0.30
*MLSS concentration in MBR tank, **Also, 1800 Nm^3/hour in membrane pool.			

Table 4.4 Raw wastewater properties

Parameter	Unit	Winter	Summer	Average
Average Flow Rate	m^3/day	1050	1090	1070
Temperature	ºC	17	24	20
Total COD	mg/L	479	380	430
BOD	mg/L	234	210	222
TKN	mg/L	66	62	64
NH_4-N	mg N/L	58	54	56
TP	mg P/L	4.30	3.50	3.90
SS	mg/L	165	253	209
VSS	mg/L	77	132	104.5
pH	-	8.10	7.68	7.89
TDS	mg/L	1700	1800	1750
Oil-Grease	mg/L	60	60	60

Simulations were executed by using the configuration in the facility. The simulation flow diagram of this configuration is given in Figure 4.2. Primarily, a treated wastewater characterization, which has almost actual output quality, was obtained by using active sludge parameters suggested for MBR as indicated above. In treated wastewater (filtrate), parameters of nitrate, total nitrogen and phosphorus are very close to actual operational data.

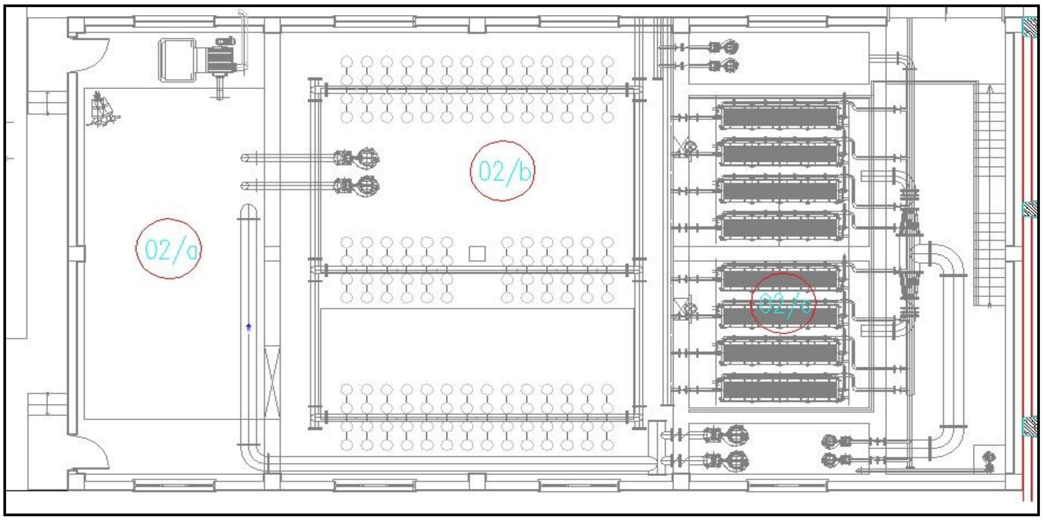

Figure 4.2 View of membrane bioreactor pool plan in Konacık WWTP

4.2 Modeling and Simulation Results of MBR System

In the first stage of the study, the design parameters of Konacık Membrane Bioreactor system were compared with the results of ATV-DVWK-A 131 Emethod, which is widely used in Turkey. A BOD_5 based design was considered for the chosen design method and calculated excess sludge (P_{XT}), total oxygen requirement (ORT) and output quality values (N,P) were compared with actual treatment facility data. In the design, a value of 1070 m^3/day given in Table 4.4was used for average wastewater characteristics and average flow rate of the treatment facility.

In the second stage of the study, the dimensions and capacities of units (Table 4.2and Table 4.3) were entered to the modeling program (BioWin) and these were used to determine performance for MBR system. The annual average wastewater characteristics given in Table 4.4were considered for wastewater characterization. For KOİ fractions, which will provide a base for model simulation, the input KOİ suggested fractions were used. The model simulation results were compared with operational data and results obtained from ATV-DVWK-A 131 Esolution. For calibration of the model, active sludge parameters suggested for MBR system were used (Insel, 2011a, 2011b; Sarioglu, 2011a; Sarioglu, 2011b).

In the final stage, the available capacity was used to consider two different operation alternatives (Alternative 1 and Alternative 2) and process simulations were realized for equal states of operational cost optimization. As a result, the profit to be made from the operational costs of the system was calculated regarding annual operation to provide discharge standards for output quality. The process modification will be realized by applying one of these alternatives. Process simulations were realized according to Barker and Dold (1997) model.

Table 4.5 Comparison of design method, operation and simulation results.

Parameter	**Unit**	**Design ATV-131**	**Actual Facility**	**Simulation**	
				Alternative 1	**Alternatie 2**
Process					
Total Sludge Age, SRT	days	48	48	21	12
Anoxic Volume Ratio, V_D/V_T	%	43	23	23	-
Internal Return, IR*	[-]	3.0	3.7	3.7	-
MLSS concentration*, X_{MLSS}	$kgSS/m^3$	10,200	10,000	10,000	13,400
Total Oxygen Requirement, OR_T	kgO_2/hour	20.3	-	15.37	17.20
Air Requirement,Q_{air}**	Nm^3/hour	-	460	1800	1450
Sludge Production, P_{XT}	kgDS/day	189	190	197	217
Effluent Quality					
Ammonium, NH_4-N	mgN/L	0.5	0.2	1.85	0.54
NO_x-N	mgN/L	13.0	12.3	6.05	11.90
Total Nitrogen, TN	mgN/L	14.0	13.0	9.32	13.90
Total Phosphorus, TP	mgP/L	1.95	1.95	1.22	1.13
*MLSS concentration in MBR tank, **Also, 1800 Nm^3/hour in membrane pool					

4.3 Process Optimization Using Activated Sludge Model

Following process modeling and simulation, two different actually applicable configurations were chosen (Alternative 1 and Alternative 2) and simulation based optimization studies were executed. These configurations were compared with the actually operated reference simulation (Figure 4.3) for evaluation regarding energy efficiency. In the first alternative, the sludge age of the system was reduced by disabling the aerobic reactor and the system was operated in pre-denitrification mode. As for the second alternative, the system was simulated according to simultaneous nitrification denitrification (SNdN) process (Insel et al., 2011a) and the anoxic reactor in the beginning was disabled. During SNdN process, the dissolved oxygen concentration in MBR pool and MLSS concentration again in MBR pool was increased and adjusted to a level of 0.50 mg/L (Insel, 2007).

For this procedure, the air flow rate in MBR pool was reduced and directed to the wastewater return pump station for obtaining total mixture. The purpose here is to obtain both nitrification and denitrification processes in a single reactor without internal recycle (IR) (Figure 4.4) for both alternatives, operation with effluent standards below TN<15 mgN/L and TP<2 mgP/L was aimed.

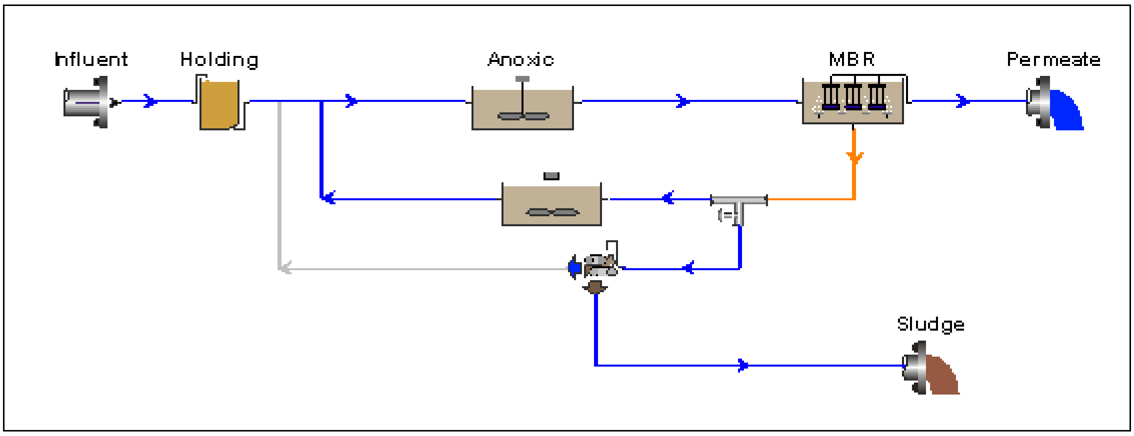

Figure 4.3 The simulation flow diagram of Konacık– MBR system (Alternative 1)

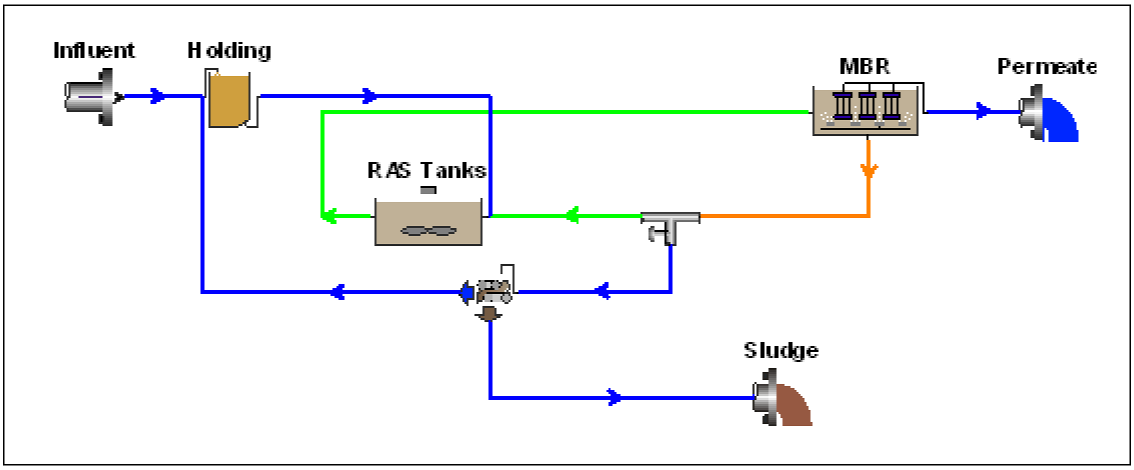

Figure 4.4 The simulation flow diagram of Konacık– MBR system (Alternative 2)

4.4 Calculation of Energy Consumption after Process Optimization

The energy and operation costs for the present situation of Konacık MBR system are given in Table 4.6 and Table 4.7. According to data from facility operation, the biological and post treatment forms 55% of total energy consumed. The daily electricity consumption is 3873.22 kWh and this value corresponds to an operation cost of 363 $/day. On the other hand, the unit wastewater flow rate and annual wastewater treatment costs per person are 0.62 $/m^3 and 24.12 $/person.year respectively. Monthly operation costs are summarized in Figure 4.5. However, the simulations performed for optimization do not reflect annual average.

Table 4.6 Unit based energy consumption ranges

Unit	**Energy Used (kWh)**	**%**	**Cost ($/day)**
Pre-Treatment	190.2	4.8	18.23
Biological Treatment	768	20	71.97
Advanced Treatment	1350	35	126.36
Sludge Disposal	456	12	42.73
Effluent Water Recycling	986	25	92.40
Lightening	123.02	3.2	11.52
TOTAL COSTS	**3873.22**	**100**	**363.21**

Table 4.7 Energy consumption and costs in wastewater treatment facility (year 2010)

Time	**Facility Capacity - 2010**	**Operation Cost**	**Unit Cost**	**Cost Per Person**
	m^3	**$/[time]**	**$/m^3**	**$/person.year**
Daily	1070	675.05	0.62	24.12
Monthly	32100	20,101.65		

Table 4.8 shows consumed energy and annual operation costs of present (reference) and alternatives. In the present situation, the amount of energy spent for each person is 141 kWh/person.year, which corresponds to an energy cost of 132,557 $. In Alternative 1, the aeration pool is disabled and sludge age is decreased to 21

days and the annual energy consumption is calculated as 114,074 \$/year, corresponding to an annual profit of 18,482 \$. As for Alternative 2, the operation cost is reduced to 100,540 \$/year by using SNdN process. In SNdN process, the sludge age is decreased to 12 days and the internal return along with anoxic reactor was disabled. As a result, a possible profit of 32,016 \$/year was observed. Accordingly, an energy requirement of 141 kWh/person.year in present situation will be reduced to 121 and 108 kWh/person.year in case Alternative 1 and 2 are applied, respectively.

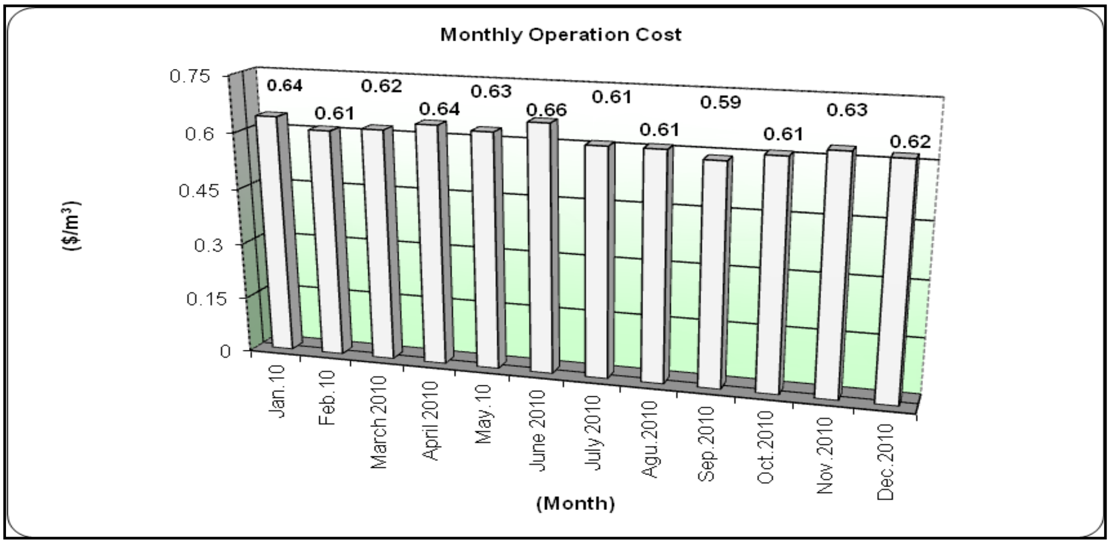

Figure 4.5 Monthly changes of unit operation costs

Table 4.8 Comparison of energy requirements with existing situation

Parameter	**Unit**	**Existing Situation**	**Alternative-1 Pre-denitrification**	**Alternative-2 SNdN**
Total Energy Consumed	kWh/day	3873	3335	2940
Annual Energy	kWh/person.year	141	121	108
Energy Saving	%	-	14	24
Annual Energy Cost	\$/year	132.54	114.07	100.54
Average Saving	\$/year	-	18.48	32.02
Unit Treatment Cost	\$/m^3 wastewater	**0.62**	**0.50**	**0.47**

CHAPTER FIVE
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions and Recommendations

In the last decade there have been significant increases in the environmental sector investments in Turkey. Especially, wastewater treatment plant constructions invested that very important for public health and protecting environment.

In our country, operating cost projections distributed to years are not taken into consideration while planning of first investments. However, the operation of wastewater treatment plants is important as constructions of them. Problems during operating are solved with trial and error method but when permanent technical problems occur, plant is not able to operate efficiently.

The aim of wastewater treatment plant is to remove wastes without harming the environment. In contrast to this purpose, these plants can consume huge amount of energy incase of they are not designed and operated properly. In our country, most of the energy is derived from fossil and alternative environmental friendly energy sources are not used widely. So, redundant energy consumption must be prevented in treatment plants.

For the reasons outlined above, it is important to use some simulation programs for decreasing energy consumption and obtaining other technical advantages in wastewater treatment plants during designing. The advantages of simulation program usage during plant planning can be summarized as follows:

1- Control of plant effluent quality
2- Control of permanent technical error
3- Decreasing of operating cost via optimization

In this thesis, Konacık municipality wastewater treatment plant was chosen as pilot plant. First aim of the study is to compare biological effluent relevant results with ATV-DVWK-A 131 E standards in 1^{st} stage and to control the accuracy of calculation results. The calculations show that anoxic/total volume ratio in ATV-DVWK-A 131 E standard is higher than current conditions. However, to obtain the same effluent quality, it is possible to use less ratio ($V_D/V = 0.23$). With regarded to obtained data, it is concluded that current biological effluent calculation has more advantageous based on first investment costs.

Second purpose of the study was to evaluate alternative operational conditions by using a simulation program to decrease energy consuming. For this aim, plant technical data is entered to the simulation program (BioWin), then energy optimization was carried out and two alternative plant operating conditions were offered.

In the present situation, the amount of energy spent per person is 141 kWh/person/year, which corresponds to an energy cost of 132,557 $. In Alternative 1, the aeration tank was disabled and sludge age was decreased to 21 days and the annual energy consumption was calculated as 114,074 $/year, corresponding to an annual profit of 18,482 $. In Alternative 2, the operation cost was reduced to 100540 $/year by using SNdN process. In SNdN process, the sludge age was decreased to 12 days and the internal return along with anoxic reactor was disabled. As a result, a possible profit of 32016 $/year was observed. Accordingly, an energy requirement of 141 kWh/person/year for present situation will be reduced to 121 and 108 kWh/person/year in case Alternative 1 and 2 are applied, respectively.

In existing conditions, unit wastewater treatment cost is 0.62 $\$/m^3$ and in this value decreases to 0.50 $\$/m^3$ and 0.47 $\$/m^3$ for Alternative 1 and Alternative 2, respectively. As a conclusion, although Alternative 2 has more advantageous than Alternative 1 depending on the unit treatment cost, Alternative 1 is selected as the most appropriate operational conditions considering some operational risks of Alternatives 2.

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