DECENTRALIZED WASTEWATER TREATMENT AND RECLAMATION USING MEMBRANE BIOREACTOR

by

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Abstract

The aim of decentralized approach is help to strictly treatment of wastewater before discharge into environment. Hence, onsite system performance is a function of decentralized wastewater treatment. The most popular of onsite treatment system is septic tank. At present, in user point of view, it is perfectly treatment system. That means wastes are treated appropriate when discharge into septic tank. However, the several studies reported that effluent septic tank is poor quality. It contains high concentration of SS, COD, TKN and Ammonia. In order to get effective of onsite treatment system, the appropriate performances concerned with high quality effluent are required.

Two modes of monitoring septic tanks were carried out which are normal operation and sludge withdrawal operation. Total 10 septic tanks were chosen to monitor for 6 months duration. Among them, monitoring of 8 septic tanks were carried out at normal operation mode and 2 remaining were withdrawn all sludge to monitor. The result of normal operation mode showed that SS, COD, TKN and Ammonia at high concentration were contaminated wastewater in these septic tanks. The results of sludge withdrawal mode indicated that it took 8 weeks for septic tanks recovery to its normal operation condition and SS was significant reduced after withdrawn sludge.

The measuring results of MFI, particle size distribution and specific resistance indicated that effluent septic tank has biggest mean diameter of particle with high potential of membrane fouling and high specific resistance compare to the mixed liquor of aerobic, anaerobic process and raw wastewater from AIT.

Two lab scale MBR processes, aerobic and anaerobic process, were carried out to treat effluent septic tank. The experiment was investigated of HRT 8 and 16 hours for both processes. It was noted that there was no removal efficiency for anaerobic process. The removal efficiencies in aerobic process were 80% of COD and 60% of TKN and Ammonia. The treated wastewater by aerobic process is acceptable to reuse for agriculture, gardening or unrestricted areas

The main issue of this experiment is rapid clogging of membrane. At HRT 16 and 8 hours in anaerobic process, membrane was clogged after 16 and 6 hours. However, it was longer in aerobic process, membrane was clogged after 9 and 23 days of operation, respectively.

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List of Abbreviations

AIT	Asian Institute of Technology
APHA	American Public Health Association
ATU	Aerobic Treatment Unit
BOD_5	Biological Oxygen Demand
Cap	Capita
CEETIA	Center for Environmental Engineering of Town and Industrial Areas
CERWASS	Centre for Rural Water Supply and Environmental Sanitation
CFU	Colony Forming Unit
CFV	Cross Flow Velocity
COD	Chemical Oxygen Demand
CST	Capillary Suction Time
DO	Dissolved Oxygen
EPA	Environmental Protection Association
EPS	Extracellular Polymeric Substances
HRT	Hydraulic Retention Time
MBR	Membrane Bioreactor
MF	Micro-filtration
MFI	Membrane Fouling Index
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
MOST	Ministry of Science and Technology
ND	Not Defined
NF	Nano-filtration
SADCO	Sewerage and Drainage Company
SMBR	Submerged Membrane Bioreactor
SRT	Solid Retention Time
SS	Suspended Solid
TKN	Total Kjeldahl Nitrogen
TMP	Trans-Membrane Pressure
TSS	Total Suspended Solid
UASB	Upflow Anaerobic Sludge Blanket
UF	Ultra Filtration
UNICEF	United Nations International Children’s Fund
URENCO	Urban Environmental Company
WHO	World Health Organization

Chapter 1

Introduction

1.1 Background

Centralized wastewater treatments have been used for long time ago. The aim of this system is improvement of sanitation condition in urban areas. General, advantage of this system is easier for management in term of operation, maintenance and effluent quality. However, its disadvantages are not only high cost in operation and maintenance but also large space requirement. Although, these systems have been successfully applied in developed countries. However, in developing countries, it is very difficult to performing these requests due to many reasons and one of the main reasons is their scanty financial state.

Decentralized wastewater treatment is system that use to treating of wastewater from individual or group of households, which are located near other. It is not costly much in term of construction, maintenance and operation. It can be applied not only for low-income countries but also in areas where communities with sparse populations. Decentralized system seems good approach to wastewater treatment in term of sanitation and environmental protection. Nowadays, developing countries always lack of sanitary system due to rapid urbanization. This approach seems good to improve sanitary condition for these countries. The simplest and most popular of onsite treatment system is septic tank. It provides the first and very important pre-treatment in the typical small-scale on-site wastewater treatment system.

Septic tank is the mostly common system that used for primary treatment wastewater generated by residence or small commercial and institutional. The quiescent condition inside tank allows portions of suspended solid to settle, floatable rise up and provides storage space for biological process to occur. In some developed countries, septic tank becomes a required section in their sanitation system. In order to manage effluent septic tank quality, there is an appropriate author monitor its operation and maintenance. Its effluent must be met regional standard if discharge to the land or sewer system (National Small Flows Clearinghouse, 2000). However, in developing and low-income countries, its effluent quality is not considering as pollutant. Wastewater after treatment can discharge in to sewer or leaching in to ground. For this reason, seriously environmental pollution occurred in these countries.

Economic development and global industrialization have conflict between water demand and water supply. In the 3rd World Water Forum held in Kyoto in 2003 emphasized that Africa and Asia are places that will be a shortage of fresh water in the world. Fresh water will be valuable resource in the 21st century. Thus, reuse of water and wastewater should be further promoted to save fresh water in the earth. Nowadays, in order to reuse wastewater as a water resource in the city for landscapes or recreational purposes, Japan has been built the systems to treating wastewater (Oota et al, 2005). In Singapore, They have been using membrane technology to produce high grade water from secondary treated sewage (Tao et al, 2005).

Membrane technology has been applied for water treatment for along time. The combination of membrane separation and biological treatment in to one process is called membrane bioreactor. It can be used to remove organic matters, nutrients, pathogens, and potentially micro pollutants in water. Membrane bioreactor is complex and small footprint than conventional treatment technologies. It not only can be operated under aerobic and anaerobic conditions but it is also operated in source with low or high organic loading. Another advantage of membrane bioreactor is the excellent quality of its effluent. Those are reasons why membrane bioreactor is widely used for water and wastewater treatment. At present, membrane bioreactor is considered as auspicious technology that can apply for decentralized treatment system.

1.2 Objectives of study

The main objective of the study is to investigate the feasibility of a decentralized wastewater treatment using membrane bioreactor and its possibility for reclamation and reuse. The details of the study can be described below:

1. To survey the existing sanitary system in Hanoi, Vietnam and suburban area of Bangkok, Thailand.
2. To performance aerobic and anaerobic membrane bioreactor and determine the suitable treatment process to improve quality of effluent septic tank.
3. To investiggate the optimum operating condition in obtaining quality effluent that could be reclaimed and reused.

1.3 Scope of study

This study is focused on two aspects. The first is evaluation of the current onsite sanitation in some areas in Asian developing countries such as Vietnam and Thailand. The second is determining the feasibility of integrating membrane bioreactor technology into septic tank, in order to improve its effluent quality and encourage reuse and reclamation of wastewater.

In this study, the existing sewerage and sanitary system in Hanoi - Vietnam and septic tanks in Klong 4 - Pathumthani - Bangkok in Thailand were chosen to review. Wastewater quality, design and operation in septic systems were mentioned. The duration of monitor septic tank was 6 months, from September 2005 to March 2006.

Two membrane bioreactor processes were installed to treat effluent from septic tank. The actual wastewater was used for experiment. Two HRT that are 16 hours and 8 hours were observed. Water quality in influent and effluent was analysed to evaluate efficiency of treatment.

Chapter 2

Literature review

2.1 Centralized and decentralized treatment system

2.1.1 Centralized wastewater treatment system

Centralized treatment system is also called off-site system. This type of system used to treat wastewater for large residential area as a city. The centralized treatment has been applied very successfully in industrialized countries. It has been installed in developed industrial countries long time ago (Willderer and Schereff, 2000). That idea seemed very good for sanitation and wastewater management.

This approach is only suitable for rich countries because of their financial ability for high cost investment for construction of sewer systems. Centralized system not only requires so much money for operation, maintenance, and collection wastewater from generate point to treatment place. This system also needs very good infrastructure support for its operation such as pipeline system, pump stations and electricity system. In the developing and lower incoming countries, it is very difficult to build this system because lack of money and they have to save financial investment for other thinks.

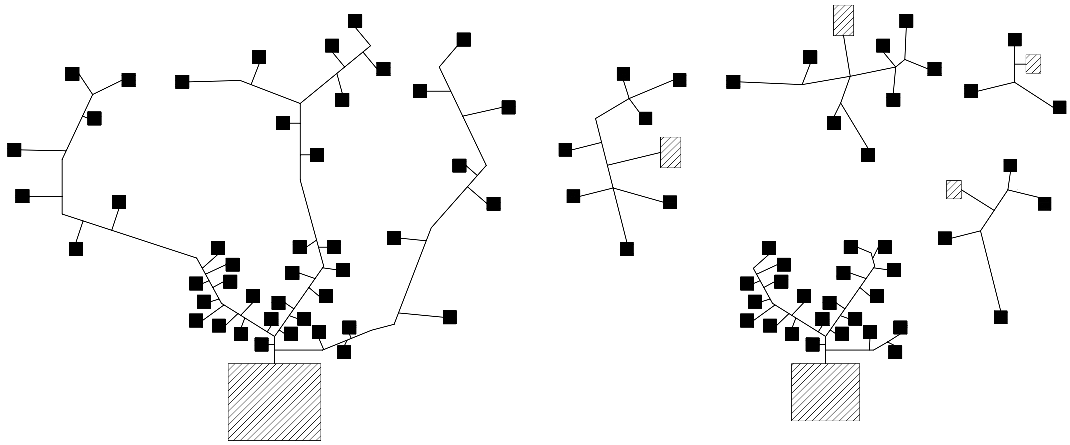
Lier and Lettinga (1999) showed that a large-scale sewer network that use in centralized has some problem as below:

- The requirement of relatively high tap water consumption in order to prevent sewer clogging that lead to contamination of large amount of water.
- The high risk of contaminants spreads into the environment (e.g. storm water, leaking of sewers); even in case off site treatment system has been installed.
- Residents and industries discharge the high risk of hazardous compounds into sewer. This frequently leads to a situation where excess sludge becomes unsuitable for reuse in agriculture.
- Need employ very expensive methods to treat of wastewater due to lager amount of wastewater is diluted.
- Exportation of rainwater from the residential areas may occur that lead to an undesirable drop of the groundwater level in city and regional dryness.
- The construction, maintenance of sewer and pulping station require high cost investment. Normally, the complete renovation is needed for every 50-60 years operation. The sub optimal maintenance leads to high losses of reclaimable water to the underground.
- Centralized urban sanitation systems depend highly on central services like electricity supply; consequently, they are insufficiently robust in periods of economic and political instability.

2.1.2 Decentralized wastewater treatment system

The term “decentralized wastewater treatment” is defined as “An onsite or cluster wastewater system that is used to treat and dispose of relatively small volumes of wastewater, generally originating from individual or groups of dwellings and businesses that are located relatively close together”. Decentralized treatment involves using a combination of treatment technology options, both traditional and innovative. (National Small Flows Clearinghouse, 2000). It consists of wastewater collection, wastewater treatment, reuse and disposal of municipal wastewater. Not every Decentralized wastewater treatment system have all of component as above, but it can be applied difference technology in order to get effective treatment same as centralized system.

Decentralized system is used in rural and urban for long time in both developed and developing countries. In urban areas, it seemed as pretreatment of wastewater and in rural areas this system used as the best solution for treating of wastewater. Decentralized wastewater system allow for flexibility in wastewater treatment and management. According to the National Small Flows Clearinghouse, management of decentralized wastewater systems is viable, long-term alternatives to centralized wastewater treatment facilities. Especially, they are often most cost-effective for rural and small communities in developing countries. Because population too spread out to make centralized wastewater treatment and some traditional existing onsite systems may get effective treatment.



Note: Treatment plant
Source

Figure 2.1 Centralized and decentralized approaches (Rocky Mountain Institute, 2004)

According to the National Small Flows Clearinghouse (2000), decentralized approach to wastewater treatment is beneficial for a number of reasons.

- It help to save money by deciding on a preventive strategy such as assessing needs and conditions of community to manage waste before a crisis occurs, thereby avoiding unnecessary cost for treat and re-create environment.
- Allow homeowners to continue use their septic systems with properly functioning.
- Eliminating the large transfers of water from one watershed to another that happens with centralized treatment
- Strategy may be the most cost-effective for treatment in rural communities with sparse populations
- It is appropriate for varying site conditions including ecologically sensitive areas. The treatment methods can be tailored to suit different site conditions.

In order to meet publish health and environmental protection goals using decentralized systems, a combination of process to treat and disposal of wastewater is the best way to achieve treatment goals. The combination consists of selection of technology, management, monitoring, operation, and maintenance.

The selection of technology is first part and very importance. At present, there are many options existing for wastewater treatment that can be applied for onsite process such as septic tank, constructed wetland…etc. Each of options has advantages and disadvantages. In order to get the best effect on treatment objective the selection must be careful carried out by technician. The second part is management and it is a key that keeps decentralized treatment system operating effectively. The management consist of installation, operation, maintenance, monitoring.

Lier and Lettinga (1999) summarized treatment options for domestic wastewater streams onsite in decentralized sanitation concepts by applying separation of low and high strength (toxics and nutrients contain) of wastewater streams. Anaerobic treatment technology is recommended as the good way to use in order to treat of domestic wastewater in decentralized concept. The options are showed in the table below:

Table 2.1 Treatment options for domestic wastewater in decentralized sanitation concepts (Lier and Lettinga, 1999)

Treatment of separate concentrated wastewater streams	
Anaerobic pre - treatment (for sludge mineralization, storage, and bio-gas production)	Post treatment of anaerobic effluent (for effluent polishing)
a. Accumulation type digester systems (for concentrated slurries) - Conventional systems - Improved modules systems b. Compartmentalized systems (for less concentrated slurries) - Accumulation type digesters - Sludge bed modules - Anaerobic filter modules - Hybrid modules	- Removal and recovery nutrients - Removal of remaining COD - Removal of pathogens
Totally mixed domestic wastewater	
- Sludge modules (treatment, mineralization, storage) - Anaerobic filter modules - Hybrid modules	- Wetland systems - Pond: fishponds - Slow sand filtration - Soil infiltration - (micro-) aerobic method

Note: a. Treatment and post – treatment of slurries (black water)
b. Toilet wastewater + kitchen wastewater

2.2 Sanitation properties and facilities

2.2.1 Characteristics of human waste

The various type of human waste in household is generated and discharged as below:

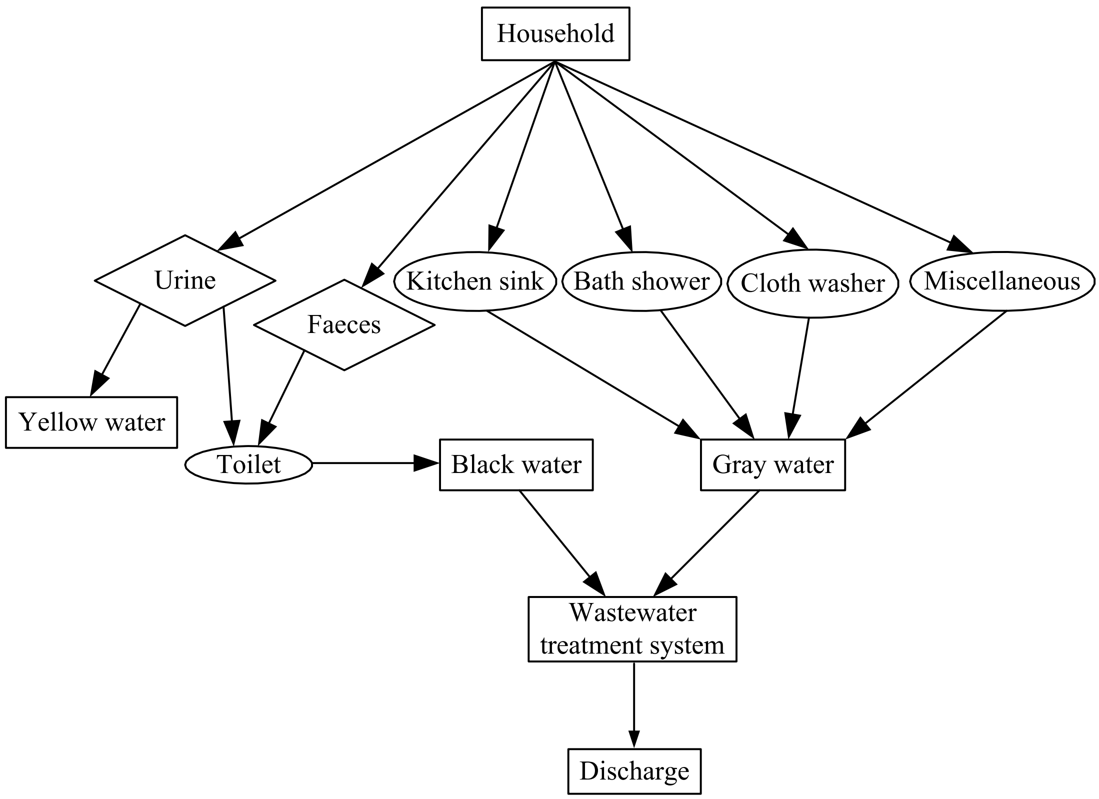


Figure 2.2 Various types of human waste

The wastewater from toilet is called blackwater. Amount of this water is very small but contain high in solid, COD and significant nutrients (as nitrogen and phosphorous). Other wastewater that generated and discharged from living activities of human such as cooking, bath, washing are called greywater. The greywater is high volume and contain high amount of organic matter but low in nutrients. The black water can be separated in to two types that are faeces and urine before it is mixed in the toilet. Faeces are known as brown water and urine is called yellow water.

Table 2.2 Characteristics of different components of human waste (Morel, 2002; Langergraber and Muellegger, 2005)

	Total	Greywater	Urine	Faeces
Volume[L/cap/year]	25,000-100,000	25,000-100,000	500	50
Nutrients (Nitrogen)	4.5 kg/cap/year	3%	87%	10%
Phosphorous	0.75 kg/cap/year	10%	50%	40%
Potassium	1.8 kg/cap/year	34%	54%	12%
COD	30 kg/cap/year	41%	12%	47%
Faecal Coliform	-	10^4-10^6/100ml	0	10^7-10^9/100ml

The characteristics of human waste are difference by the composition and concentration. If the components of human waste are separated, it will be useful resources for the crops. The sanitation that called ecology sanitation is seemed a good method of recovering nutrients from human waste, and recycling them back into the environment and productive systems.

2.2.2 Sanitation problems

According to the WHO, one in five persons does not have access to safe and affordable drinking water and so many people do not have access to safe and sufficient sanitation. Around 2.6 billion peoples do not have access to any type of improved sanitation facility. (WHO/UNICEF, 2003; Langergraber and Muellegger, 2005). The sanitation is not only problem in developing countries it also is problem in developed countries.

In the world, the options for sanitation problems have been applied as “drop and store” and “Flush and forget”. These forms of conventional wastewater management and sanitation systems are based on the perception of faecal material and excreta are waste that waste is only suitable for disposal. The conventional sanitation system is based on the collection and transport of wastewater through sewer system. This system mixes small quantities of potential harmful substances with large amounts of water.

In addition, the construction, operation and maintenance of hardware that use for “Flush and discharge” options are a heavy financial burden. These sanitation systems have fundamental shortcomings such as over-exploitation of limited renewable water sources, pollution of soil and groundwater, waste of valuable components in wastewater and the difficulty for an effective removal of pollutants. The systems that applying the “drop and store” principles are pit latrine. Various forms of this latrine are still dominantly use in developing countries. The disadvantages are obvious such as soil and groundwater contamination with pathogens, bad odors, fly/mosquito breading. In densely populated areas, the limits are more clear such as odors are not accepted, digging a new pit is not realize when the old one is full (Langergraber and Muellegger, 2005).

Otterpohl et al. (1997) have summarized the disadvantages of traditional concept in industrial countries as below:

- Nutrients are lost because wastewater is discharged after treatment.
- High energy demand use for destruction of organic mater and nitrification in wastewater.

- High pollution loads in the sewage sludge.
- A high amount of water is necessary for flushing human wastes to the treatment plants.
- Hygiene problems in receiving water after combined sewer overflows and water treatment plant effluent.
- The join of heavy metals can lead to a mobilization of the metals.
- High operation and maintenance cost for the drainage system and sewage treatment plant.

2.2.3 Ecological sanitation

The ecological sanitation that called “ecosan” is not a technology but a strategically sanitary approach. Comprehensive approach is trying to integrate all aspects of sanitation that is human waste, solid waste, greywater, drainage (Morel, 2002). The key objective of this approach is not to promote a certain technology, but rather a new philosophy of dealing with what has been regarded as waste in the past. The systems of this approach are based on the implementation of a material-flow-oriented recycling process as a holistic alternative to conventional solutions. Ideally, ecological sanitation systems enable the complete recovery of all nutrients from faeces, urine and greywater to the benefit of agriculture, and the minimization of water pollution. (Earle, 2001)

The ecological sanitation principles are not novel. In some countries such as China, Vietnam, ecological sanitation has been used for hundreds years. Today it still widely used in parts of East and Southeast Asia countries. In Western countries, this option has been a revival of interest (Langergraber and Muellegger, 2005). The working properties of ecosan are recognized waste as a resource, the nutrients in the waste is recycling, water used for sanitation is minimized and healthy living condition is promoted. Note that main principal characteristics of ecosan are containment, sanitization and reuse (Cann, 2005). The specialty of the new approach is to view urine, faeces, and greywater separately as components with different characteristics in term of pathogens, nutrient content, benefits to soil, and plants (Morel, 2002).

Table 2.3 Advantages and disadvantages of ecosanitation (Earle, 2001)

Advantages	Disadvantages
▪ Removal of pathogens from the domestic environment. ▪ Elimination of foul odours if properly constructed. ▪ Diseases are destroyed, not just contained. ▪ No potential to contaminate other water sources. ▪ Use very little, or no water. ▪ Nutrients and organic matter is recovered. ▪ Operation is very simple and easy. ▪ Low cost in term of construction and maintenance.	▪ Users do need to be taught how to use them properly. ▪ Faeces need to be treated in the correct manner, otherwise they pose a health risk. ▪ High density urban areas don't always have the capacity to use the byproducts produced.

2.2.4 Methodologies used of ecological sanitation

Separate treatment of greywater

Generally, greywater is divided in four greywater categories based on its origin: bathroom, laundry, kitchen, and mixed origin (Morel, 2005). Greywater contains minor amounts of nitrogen and phosphorus, but large amounts of organic matter. Experience shows that separate treatment of blackwater and greywater is requirement. Organic household waste can be separated and treated in the same process as the blackwater and yield a fertilizer and energy. The water consumption for sanitation can be reduced up to 50% (Jenssen, 2002). The separate treatment of greywater is a type of ecosan. In ecological sanitation, greywater is source separated from toilet systems, allows simpler treatment systems than conventional sewage treatment plants (Ecosanres, 2005).

Urine separation

Human urine contains the largest amount of nutrients. If no phosphorus detergent is used, it about 60% of phosphorous and 80% of nitrogen in household wastewater comes from urine. Source separating human urine that lead significant increased the amounts of nutrients recycled to arable land and decreased nutrients load of wastewater. Source separation of urine is based on toilet equipment. Source separation is possible with recent developed toilet that has the bowl divided into two parts. The urine and a small amount of flush water are collected in a tank and transport to farm for later use on arable land (Schonning et al., 2002). The flush water use for dilute urine is decrease. The concentration of nutrients and volume of urine is increase. The treatment of faecal material can be done by suitable ways. Thus, source separation of urine is a complementary function and can be added to sewage system (Jonsson et al., 1997).

(Jonsson et al., 1997) also concluded that toilet construction with a separate urine bowl saved 50% of flush water usage compare to conventional toilet. Source separated urine can us as fertilizer and compares with mineral fertilizers and with other organic fertilizers of urban origin such as sewage sludge and compost from solid waste. The source separating sewage systems may minimize risk for disease transmission due to microbiological die off in the urine (Hoglund et al., 1998).

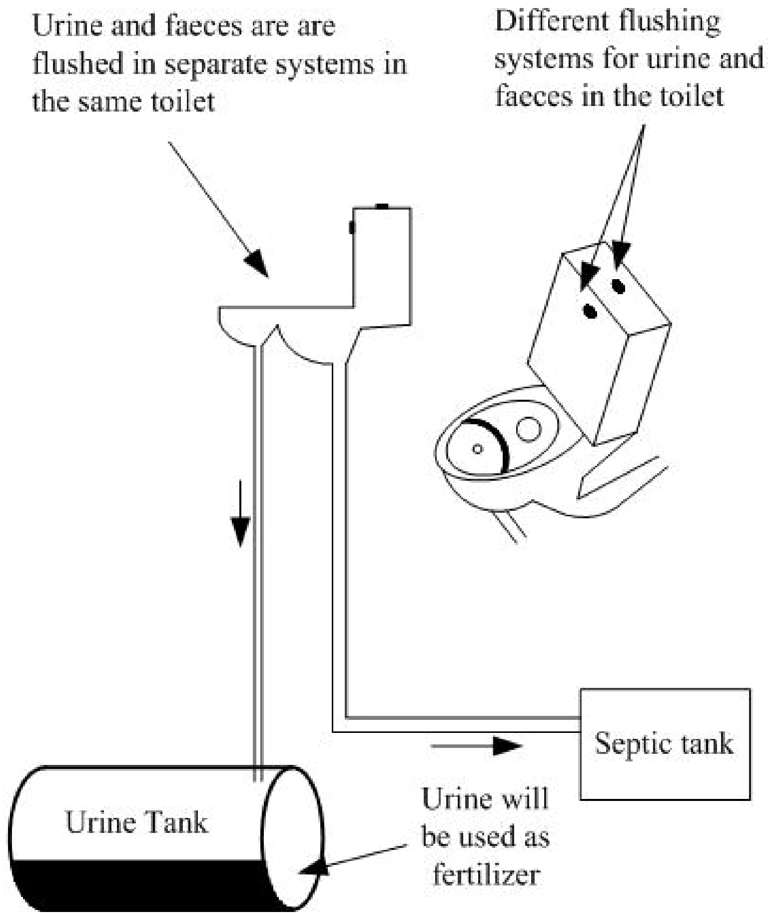


Figure 2.3 Schematic diagram of urine source separating (Hoglund et al., 1998)

2.3 Types of onsite treatment processes

2.3.1 Septic tank

Septic tank is the most well know and common method for onsite treatment of sewage. They are most common for small scale and decentralized treatment plants in the worldwide. The purpose of a septic tank is to provide a receiving vessel for all wastewater generated from domestic dwelling and to afford primary treatment that wastewater. The septic tank is a single or multi-chambered watertight vault. Septic tank provides the first and very important pretreatment in the typical small-scale on-site wastewater treatment system and accomplishes approximately 50% of the ultimate treatment within the tank (Sebloom et al., 2003). The process that occurs inside of septic tank is same as anaerobic process are settling of solids, the anaerobic conversion of organic matters and accumulation or digestion of sludge. (Lier and Lettinga, 1999).

Normally, septic tank is buried underground. The position of the septic tank in the household is based on region. In some region with spare space like rural area, septic tank is buried outside of house. In regions with narrow space like cities, septic is buried under bathroom. It can be explain in the figure below.

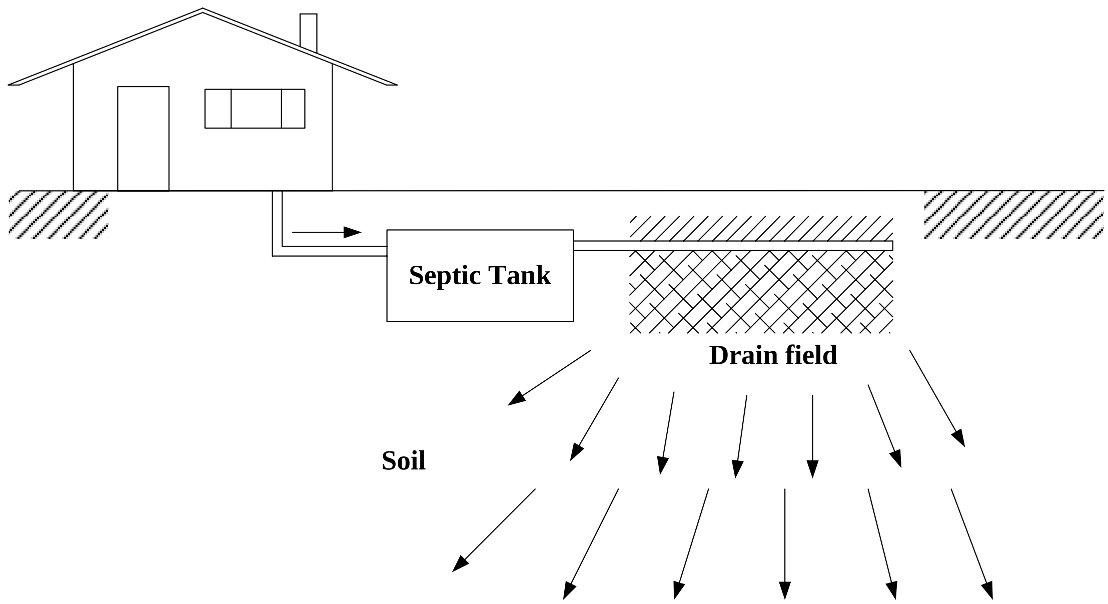


Figure 2.4 Position of septic tank (EPA, 2001)

Description of septic tank

Septic tank consists of an underground sedimentation tank having one or multiple compartments (Figure 2.5 and 2.6). These compartments can be separated in to different tanks but its function is not difference. The wastewater from the toilet, bath, kitchen, etc., enters the tank. Velocity of flow is reduced providing relatively quiescent conditions. That allows portions of the heavy solids to settle to the bottom. The lighter substances such as grease, oil and other floatable materials rise to the top and form a scum layer. Anaerobic bacteria break down wastes inside septic tank.

Normally, water from septic that has soluble substances is discharge in to ground through drain field and it is absorbed into the soil. Settled sludge will be stabilized by anaerobic digestion. The solid that are not decomposed remain in the tank called sludge. The sludge is accumulated in the tank. The settled sludge must be pumped out periodically. The period for pumping sludge depend on tank size, type of solid enter the tank, etc. Normally, this period can be 3 – 5 years.

There are many different types of septic tanks. The septic tank may be rectangular or cylindrical container made of concrete or polyethylene. Patterson (2003) described the older style of septic tank like as a tank has single chamber and capacity about 1,800 – 2,000 L. The clear water zone inside the tank provides suitable residence time (about 24 h) in order to allow lumps to disintegrate, settable materials to sink and floatable materials to rise. Short-circuiting of the wastewater from the inflow to the outflow is a possibility because the tank is unbaffled and particularly when the clear volumes is crowded with sludge and scum that is accumulated over time.

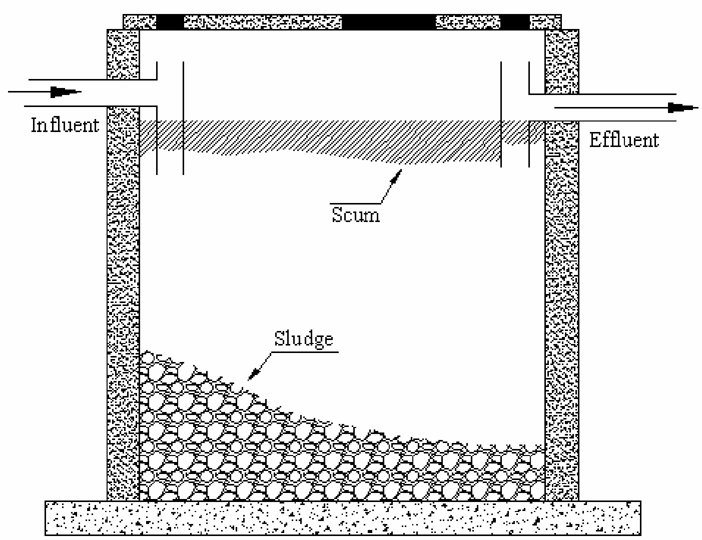


Figure 2.5 Schematic of one compartment septic tank

The two or more compartments tank is created in order to overcome the short circuit of one compartment tank type. The schematic of two compartments septic tank is described in the Figure 2.6 below:

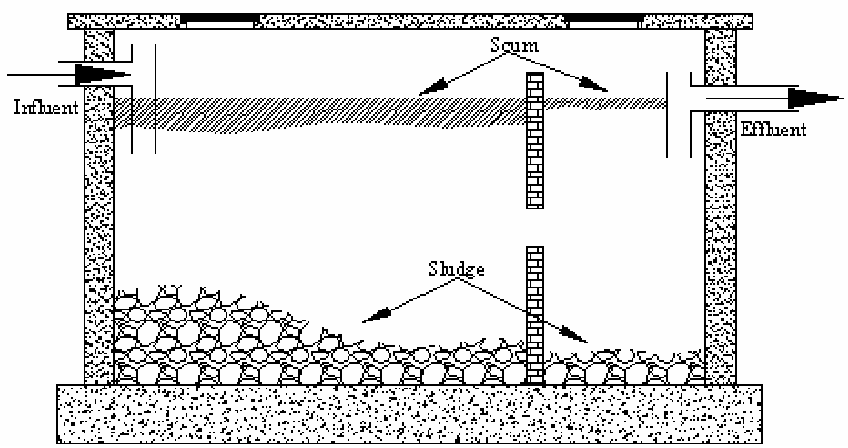


Figure 2.6 Schematic of two compartments septic tank (Crites and Tchobanoglous, 1998)

Table 2.4 Advantages and disadvantages of septic tank

Advantages	Disadvantages
▪ Simple operation ▪ Little space requirements (under ground) ▪ Low maintenance requirement ▪ Nutrients are returned to the soil ▪ Cost-efficiency regarding treatment ▪ Long-lasting	▪ Low treatment efficiency ▪ Enrichment of nutrients and disease caused microorganisms in effluent. ▪ Foul-smelling emissions created by anaerobic digestion

Septic tank effluent characteristics

Although, wastewater is treated by septic tank, its effluent still contains disease microorganisms and other pollutants. According to the EPA (2002), in general, quality of septic tank effluent is not good. This type of water contains high BOD, SS, and nitrogen. Especially, it has very high amount of harmful bacteria.

Table 2.5 Septic Tank Effluent Characteristics

Component	Brandes, 1977	Crites and Tchobanoglous, 1998	EPA, 2002
pH	7.2 - 8.5	-	6.4 – 7.8
SS	37 - 261	40 -140	40 - 350
COD	175 - 490	250 - 500	-
BOD_5	38 - 160	150 - 250	46 - 156
NH_4-N	120 - 160	30 - 50	-
NO_3-N	0.1 - 0.3	-	0.01 – 0.16
Total Nitrogen	140 - 170	50 - 90	19 - 53
Total Phosphorus	16 - 22	12 -20	7.2 - 17
Fecal Coliform Bacteria	$0.03*10^6$ - $0.09*10^6$	-	10^6-10^8

All units are mg/L except Fecal Coliform Bacteria in CFU/100mL and pH.

2.3.2 Baffled septic tank

The baffled septic tank (Figure 2.7), also known as “baffled reactor”, prefers use for wastewater with a high percentage of non-settleable suspended solids and low COD/BOD ratio. According to the Sasse (1998), Baffled septic tank is large and shallow tank. It is an improvement of septic tank and using the advantages of the UASB for treatment of wastewaters. It consists of 2 to 5 serial chambers with eventually a filter in the last part. The first compartment always is a settling chamber and a series of up – flow chambers are followed. There is an intensive contact occurring between fresh influent and resident sludge. The process-taking place in the chambers is the anaerobic degradation of suspended and dissolved solids. This process leads to a COD removal of 65 – 90 %. The importance parameter for design is low in up-flow velocity. This value should not excess 2 m/h.

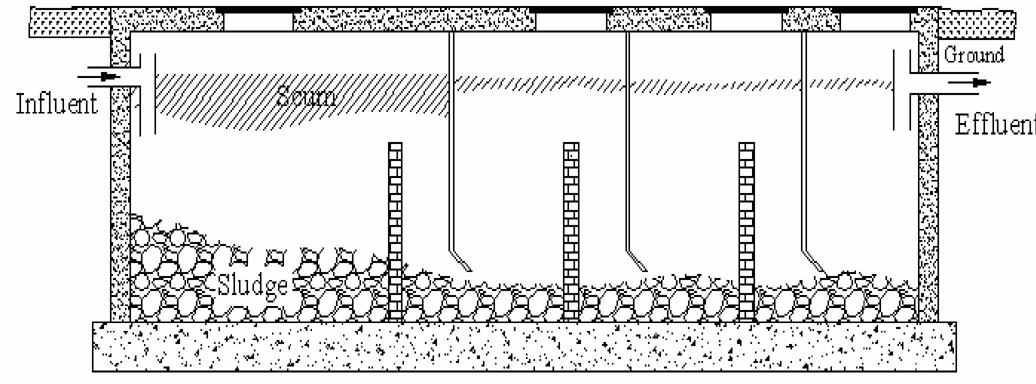


Figure 2.7 Schematic of a baffled septic tank having four chambers (Sasse, 1998)

Table 2.6 Advantage and Disadvantages of baffled septic tank (Sasse, 1998)

Advantages	Disadvantages
▪ High treatment efficiency ▪ Simple to built and operate ▪ Hardly any blockage ▪ Durable system ▪ Relatively cheap ▪ Low affect due to shock load and shock hydraulic.	▪ Less efficient with weakly polluted wastewater ▪ Long start-up phase ▪ Large volume requirement

2.3.3 Anaerobic/fixed bed filters

Anaerobic filters, also known as fixed bed or fixed film reactor, can be used for pre-settled domestic and industrial wastewater of narrow COD/BOD ratio and low SS concentrations. Therefore, they not only are used in combination with primary treatment (for example a septic tank or baffled septic tank), but also treat non – settleable and dissolved solids by bringing them in close contact with active bacteria mass on a filter media.

The filter should be rough. The rough surface of media is target for bacteria growth. Surface of filter material should be from 90 to 300 $m^2/m^3$of reactor volume. The materials such as gravel, rocks, cinder or specially formed plastic pieces provide additional surface area for bacteria to settle, the larger the surface for bacterial growth, the quicker digestion. The requirement tank volume should be 0.5 to 1 m^3/capita. The COD removal efficiency can achieve up to 70 – 90 %. Biogas utilization should be considered in the case of BOD concentration is higher than 1.000 mg/L. The hydraulic retention time should be higher than 24 hours.

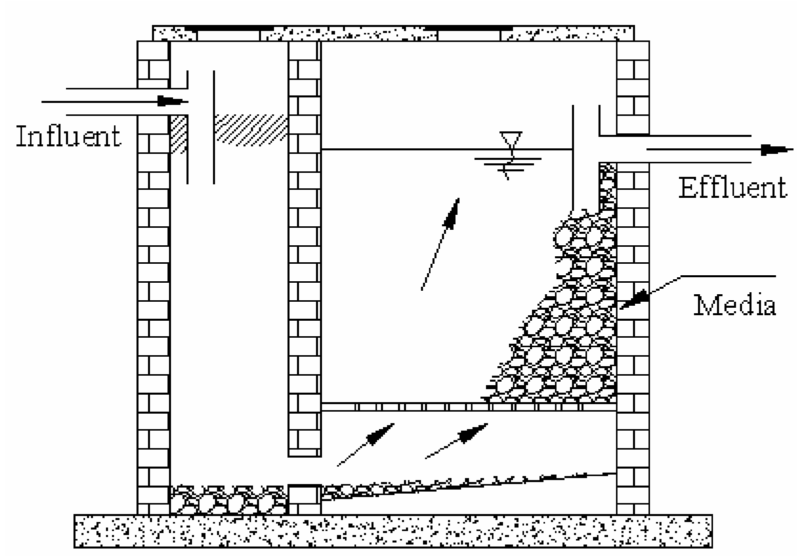


Figure 2.8 Schematic of Anaerobic/fixed bed filters (Sasse, 1998)

Table 2.7 Advantage and Disadvantages of Anaerobic/fixed bed filters (Sasse, 1998)

Advantages	Disadvantages
▪ Simple and durable system	▪ High construction costs (filter media)
▪ High treatment efficiency	▪ Blockage of filter possible
▪ Little space requirements	▪ Effluent can smell

2.3.4 Imhoff Tank

Inhoff tank (Figure 2.9) has some more advances than septic system, but its effluent fails to meet discharge criteria requirements, therefore, use is limited. Normally, Imhoff tanks are typically used for domestic or mixed wastewater flows above 3 m^3/d. The tank consists of a settling compartment above the digestion chamber. The sedimented solids flow from the upper chamber through a slot in the bottom into the lower one, where solids are accumulated and digested in anaerobic condition. The influent is separated firmly from the bottom sludge: funnel-like baffle walls prevent up-flowing foul sludge particles from being mixed with the effluent and from causing turbulence.

The effluent is fresh and odourless due to the suspended and dissolved solids do not get into contact with the active sludge. Sludge removal should be done right from the bottom of reactor to ensure that only fully digested substrate is discharged. Only a part of the sludge is removed regularly in order to keep some active sludge in the reactor. The removed sludge should receive further treatment immediately by in drying beds or compost pits for pathogen control.

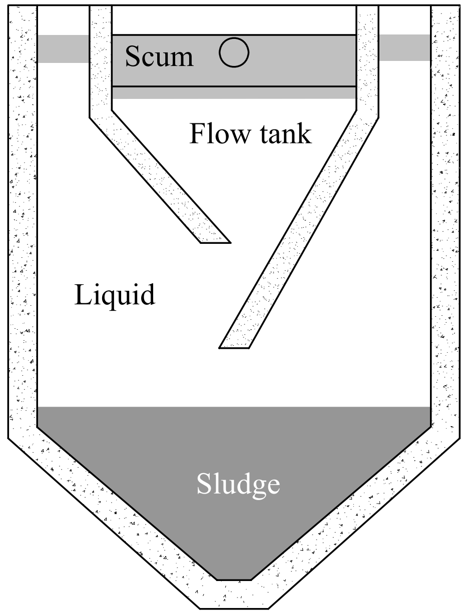


Figure 2.9 Cross section of Imhoff tank (Sasse, 1998)

According to the Sasse (1998), the advantages and disadvantages of the imhoff tanks are listed in the table below.

Table 2.8 Advantage and Disadvantages of imhoff tank (Sasse, 1998)

Advantages	Disadvantages
▪ Durable ▪ Little space due to underground construction ▪ odourless effluent	▪ Less simple than septic tank ▪ Need very regular desludging

2.3.5 Aerobic Treatment Unit

An Aerobic Treatment Unit pre-treats wastewater by adding air into water to break down organic matters, reduce pathogens, and transform nutrients (David et al., 2001). This process break down organic matter more efficiently comparing to conventional septic tanks, achieve quicker decomposition of organic solids, and reduce the concentration of pathogens in the wastewater. Due to supply air into system, aerobic systems are costly to operate and need maintenance compare to septic systems. The costs to operate an ATU are based on the run-time of the compressor, pumping, repairs, maintenance, and electricity. A properly operating system can produce high-quality effluent with less than 30 mg/L BOD, 25 mg/L TSS, and 10,000 cfu/mL fecal coliform bacteria. The advantages and disadvantages of aerobic treatment unit was mentioned by USEPA (2002) and David et al. (2001). The main points can be list as below:

Advantages

- Provide higher level of treatment than a septic tank
- Helps to protect valuable water resources where septic system is failed.

- Provides an alternative for sites where septic system are not suited.
- Help to extend the life of a drain field.

Disadvantages

- More expensive to operate than a septic system.
- Additional cost due to power consumption.
- Require more frequent maintenance components.
- Release more nitrates to groundwater than a septic system

2.4 Membrane bioreactor

Membrane bioreactor is combination of membrane separation and biological treatment in order to remove organic matters, nutrients, pathogens, and potentially micro pollutants in water. It not only can be operated under aerobic and anaerobic conditions but it is also operated in low and high organic loading (Kraume et al., 2005; Patel et al., 2005). That lead membrane bioreactor is widely used for water and wastewater treatments.

2.4.1 Membrane bioreactor properties

Micro-filtration (MF) and Ultra-filtration (UF) membrane are process that filter material based on particle size. Membranes are made of polyethylene or ceramic. MF has pore size from 0.1 to 0.4 µm UF has pore size from 2 to 50 nm. Both types of membrane are applied in bioreactor. A membrane must achieve some properties as much as possible such as mechanical strength, high degree of selectivity, high throughput of desired permeate. Membrane bioreactor (MBR) is devices that combine of activated sludge process with membrane separation. Both membranes are very popular in term of membrane bioreactor. MBR have two configuration based on the location of membrane in the module, that are submerged membrane bioreactor system and external cross flow membrane bioreactor system.

According to the Roest et al. (2002), some basic concepts that use in the membrane process are flux, trans-membrane pressure and permeability. The flow of liquid through a specific membrane surface area is called flux. Flux can be express as:

$$\text{Flux} = \frac{\text{permeate flow}}{\text{membrane surface used}} \qquad \text{Equation 2.1}$$

In this equation

Flux in $L/m^2.h$
Permeate flow in L/h
Membrane surface used in m^2

A flow through the membrane has associated with driving force and pressure drop. From these pressures, trans-membrane pressure (TMP) can be determined as:

$$\text{Tran} - \text{membrane pressure} = \text{Static pressure} - \text{Dynamic pressure} \qquad \text{Equation 2.2}$$

- Static pressure: pressure at zero permeate flow(bar)
- Dynamic pressure: pressure at permeate flow(bar)

- Tran-membrane pressure or (bar)

The permeability of membrane is determined by equation below:

$$\text{Permeability} = \frac{\text{Flux}}{\text{TMP}} \text{ (at temperature } T^{0}\text{C)} \qquad \text{Equation 2.3}$$

The unit express as: Permeability in $L/m^2.h.bar$
Flux in $L/m^2.h$
TMP in bar

2.4.2 Various membrane bioreactor processes

Cross flow membrane bioreactor system, the membrane is installed outside of the active sludge tank. The principle of cross flow is high flow velocity in order to prevent the building up of solid cake on the membrane surface. This method requires maintain the sludge velocity across the membrane surface for membrane cleaning and required pressure drop for permeation. This method is easy for operation and maintenance but require large amount of energy that lead to high operation cost. Because high velocity and excess shear break micro floc and system operate in unstable (Roest et al., 2002)

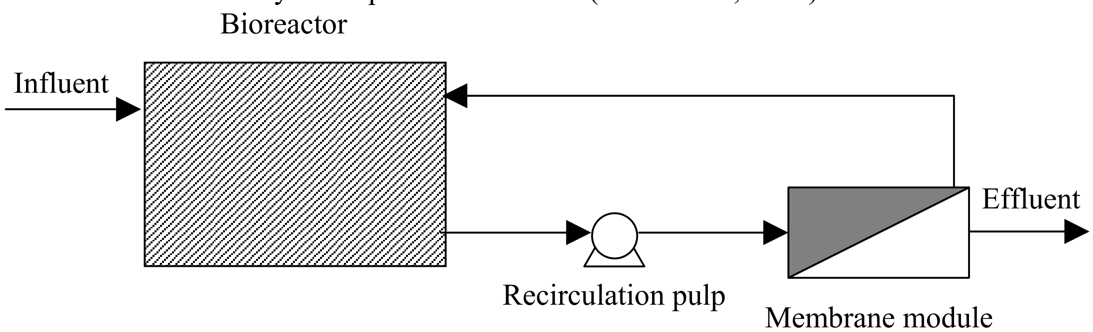


Figure 2.10 Cross flow membrane bioreactor (Ueda et al., 1997)

Submerged membrane bioreactor system (SMBR) is a membrane module that is immersed in a bioreactor. The permeation is extracted by suction or pressure on bioreactor. The pressure applied in permeate extraction is lower than that required for cross-flow permeation. In SMBR is absent of recirculation pump which is requirement in cross flow. The mechanical used to create the cross flow stream on the surface of membrane is low-pressure air diffusion and it can be considered part of activated sludge process. The air diffusion facilitates two processes that are cleaning of membrane surface and supply oxygen to the biomass.

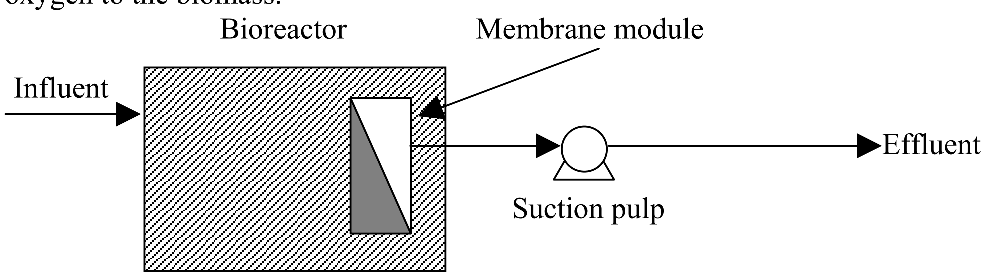


Figure 2.11 Submerged membrane bioreactor (Ueda et al., 1997)

2.4.3 Nutrients removal in the membrane bioreactor process

Membrane bioreactor that use membrane unit to separate treated water and mixed liquor can be replace for the sedimentation/ clarification techniques. Because no suspended solid are lost in the clarification step, the SRT and HRT are entirely separated, the population of microorganisms is easy control in operation (Cicek et al.,1999; Trouve et al., 1994). The MBR can operating at high sludge ages and high biomass concentration so higher strength wastewater can be treated and lower biomass yields are realized (Cicek et al.,1999; Muller et al., 1995). The process can achieve high degree of organic oxidation while produce a free solid effluent. Because it operated under long SRT, it can maintain a higher content of slow growing nitrifying bacteria (Chiemchaisri and Yamamoto, 2005).

In MBR process, nitrification has been to be greater than conventional activated sludge process (Kraume et al., 2005; Zhang et al., 1997) due to longer SRT and lower food/ microorganism ratio and mixed liquor contains large fraction of small particles (Chiemchaisri and Yamamoto, 2005). In wastewater, when aerate step is performed nitrogen present in the form of ammonia will be transformed to nitrite and nitrate. MBR can achieve total nitrification with effluent ammonia concentration is below than 1 mg NH_4-N/L. Otherwise, some microorganisms need nitrogen for growth, N is also removed with the excess sludge. In order to get high efficiency of nitrogen removal, a MBR should get SRT higher than 5 days (Kraume et al., 2005).

Traditionally, phosphorus removal is chemically obtained with coagulants and phosphorus is removed under co-precipitation. This leads high sludge production, more chemical consumption and high concentration of salts in the effluent. The recent observations tend to prove that phosphate removal is can or not in MBR. (Adam et al., 2002). Because, MBR operates in the starvation conditions, microorganisms of high concentrated sludge have to survive under these conditions. The bacteria that contain poly phosphate survive longer as a consequence of their accumulated energy source (hydrolysis of poly phosphate) and thus have an importance advantage in the competition of species (Adam et al., 2002; Ubakata and Takii, 1998). According to the Adam et al. (2002), the phosphorus removal is possible in the membrane bioreactor. The concentration of phosphorus in the effluent was always lower than 0.2 mg P/L. The other study was achieved phosphate removal to 99% without dosing of precipitants (Kraume et al., 2005; Gniss et al., 2003). Some research shown that phosphorus removal is possible in membrane bioreactors that operating at sludge ages up to 26 days (Adam et al., 2002).

2.4.4 Membrane fouling

In the bioreactor, when perform separate process use membrane, many factors effect on the process. It is well know that some characteristics of membrane such as pore size, porosity, and roughness are direct impact on membrane fouling (Chang et al., 2002). Membrane fouling is attributed to the physicochemical interactions between the biofluit and membrane. Biomass is separated on the surface of membrane. The cake layer increase on the membrane surface that lead resistance of membrane increase and flux decrease. The parameters that relate to the fouling of membrane are solid concentration, hydraulic retention time, size of sludge and temperature. Membrane material is also effect on the process. Flux and TMP are depending on membrane pores size. The schematic of the membrane fouling can express as figure below:

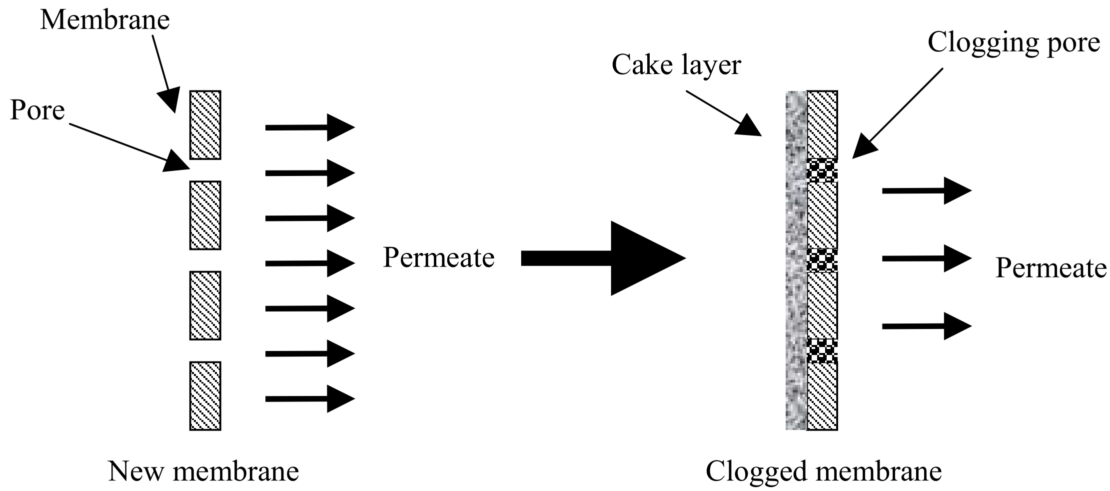


Figure 2.12 Membrane fouling

Many empirical and theoretical models have been proposed in order to describe the fouling phenomena. According to the Chang et al. (2002), total resistance of membrane and permeate flux can be expressed by equations below:

$$R_t = R_m + R_c + R_f \qquad \text{Equation 2.4}$$

- R_t : Total resistance
- R_m: Intrinsic membrane resistance
- R_c: External fouling resistance caused by cake layer
- R_f: Internal fouling resistance, due to fouling in to the membrane pores

$$J = \frac{TMP}{\eta . R_t} \qquad \text{Equation 2.5}$$

- J: Permeate flux
- TMP:Ttrans-membrane pressure
- η: Dynamic viscosity of the permeate
- R_t: Total resistance

Chang et al. (2002) also show that the factors which effect on to the membrane fouling are membrane properties, Biomass and operation conditions. That can be express as below:

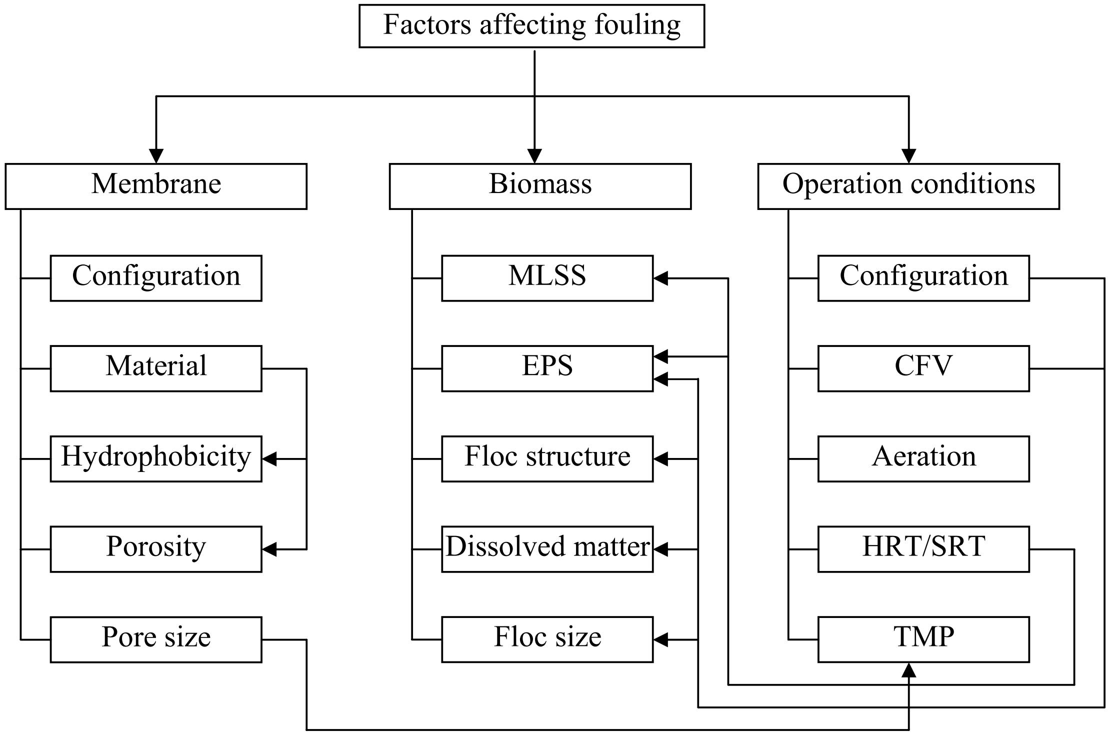


Figure 2.13 Factors affecting on membrane fouling in MBR process (Chang et al., 2002)

2.4.5 Development of MBR for wastewater treatment

The idea combines activated sludge with membrane separation from 1960s. The concept of immersed membranes is starting in the late 1980s / early 1990s by independent teams in Japan and Canada. Today, there are many researches on membrane bioreactor in order to improve the application of membrane process in the wastewater treatment. Many researchers have reported that MBR are much more effective in treating wastewaters than conventional suspended growth systems such as activated sludge system. Membrane bioreactors (MBR) have many advantages for wastewater treatment such as high quality of effluent, long contact time between activated sludge and organic pollutants, free from bacteria. Membrane bioreactors have potential for municipal and industrial reuse. Submerged membrane bioreactor technology has been applied to wastewater treatment and reclamation. In Europe, America and Japan, SMBRs are used to rebuild sewage treatment plants and to reclaim wastewater, although there are shortcomings of high-cost and high-energy consumption (Ren et al., 2005).

Trends in membrane technology and its applications in wastewater treatment is stated in many researches. The development can be explained like figure below.

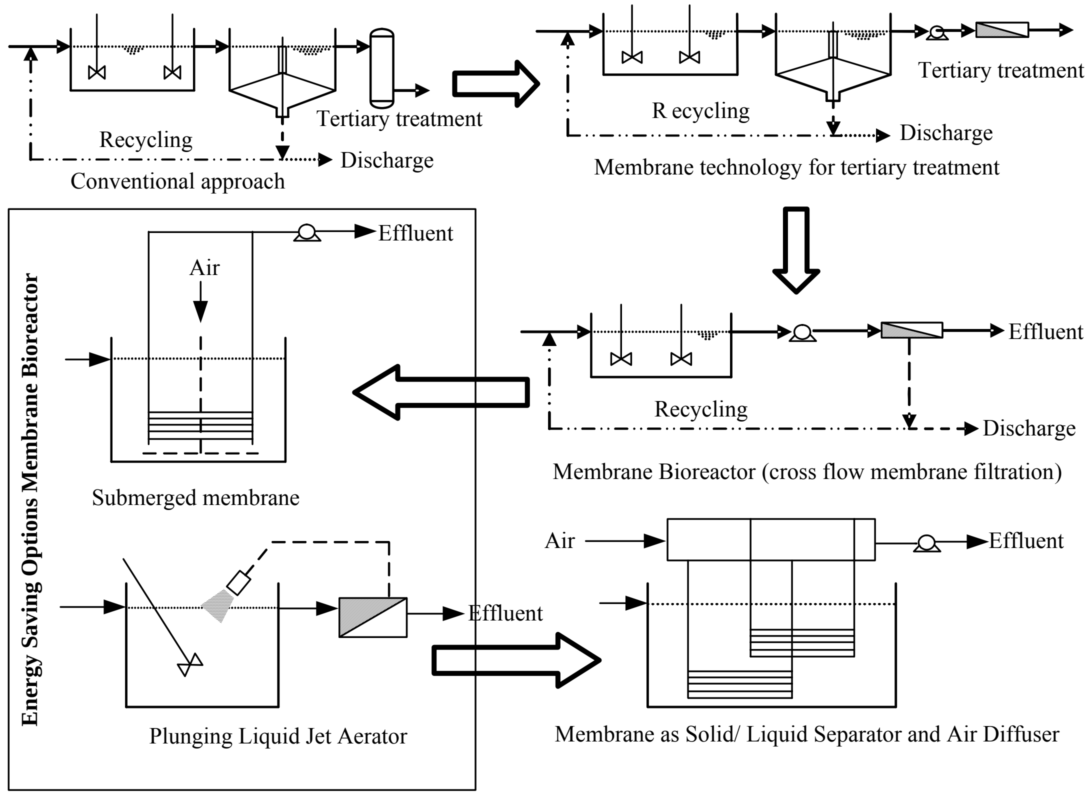


Figure 2.14 Trends in membrane bioreactor development (Visvanathan et al., 2000)

At present, the new process is to be experiment. In order to increase lifetime of membrane between two clean, You at al (2005) found that couple of anaerobic process and aerobic process and membrane separate could reduce the membrane scaling and fouling caused $CaCl_2$. This method will draw the new methods to solve fouling and scaling problems in membrane bioreactor separation.

Chapter 3

Methodology

3.1 Introduction

Based on the objectives listed in the chapter 1, this study included three main parts: survey, monitoring and experiment carried out in laboratory scale. The overall methodology is described in Figure 3.1.

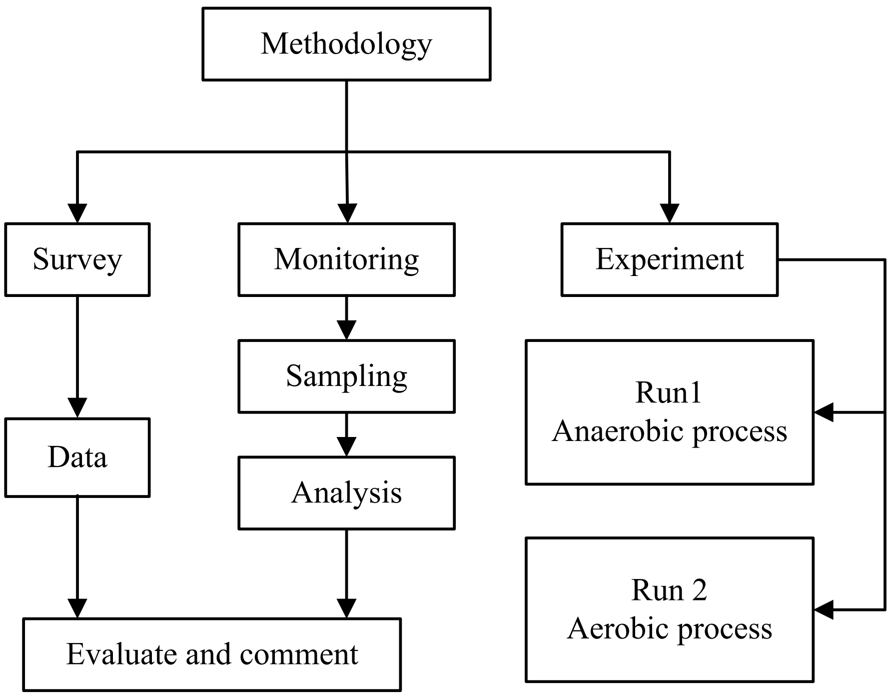


Figure 3.1 Overall of methodology

3.2 Survey and monitoring

3.2.1 Survey

The first part of the study was to collect data that related to existing sanitary condition in Hanoi Vietnam. The data was collected as follow:

- General information of Hanoi (population, economy, land use, geographically...)
- The design, operation and maintenance of septic tank
- Quality and quantity of municipal wastewater
- Properties of sewage system and its problems related to the environment
- Types of existing latrine
- Wastewater treatment system and its capacity

All most data required for the study was collected from the following sources:

- Hanoi Environmental Department – Ministry of Natural Resources and Environment
- Hanoi University of Civil Engineering
- Hanoi Sewerage and Drainage Company
- Hanoi Sewerage and Drainage project
- Photos from field trip around Hanoi

3.2.2 Monitoring

The septic tanks in Klong 4 - Pathumthani province were chosen to monitor from October 2005 to March 2006. The purpose of monitor is to evaluate the effluent quality, treatment ability and operational condition of these septic tanks. The grab samples were taken and measured parameters are pH, COD, BOD, SS, NH_4^+, NO_3^-, TKN and total phosphorus. Total 10 septic tanks were chosen to monitor at two modes of operation. Among 10, 8 septic tanks were monitored at normal operation and the other 2 septic tanks were monitor at sludge withdrawal condition. Samples in the withdrawn septic tanks were taken once per 2 week after sludge was withdrawal. The other 8 septic tanks were monitored monthly.

The other information related to these septic tanks such as designs, operate condition, frequent sludge withdrawal were also collected.

3.3 Laboratory scale experimental study

Experimental study was divided in to two main parts.

- Preliminary study: design reactor process, and acclimatization of sludge.
- Experimental runs.

3.3.1 Preparation of wastewater

Due to properties of septic tank effluent, actual wastewater was used for experiment. The wastewater was taken from septic tanks in Vietnamese food restaurant in AIT campus. A centrifuge pump was used to pump wastewater from second compartment of this septic tank to 30 litters containers which were transported to Ambient Laboratory and preserved in 5^0C room before use.

3.3.2 Activated sludge

The activated sludge was taken from aerated tank in municipal wastewater treatment plant of Thammasat University in Pathumthani province, Thailand. Then, it was acclimatized with synthetic wastewater and effluent septic tanks as feeding wastewater for two weeks before reactor run with membrane.

3.3.3 Membrane bioreactor

The laboratory scale membrane bioreactor was used for this study. The total reactor volume is 18.5 liters, made of transparent acrylic sheet and dimensions are shown in the appendix A. The used membrane is hollow fiber polyethylene membrane and manufactured by Misubishi Rayon company, Japan. Its dimensions are described in Table

3.1. This membrane was submerged in to the reactors and it connected to a pump which used to suck wastewater out.

Table 3.1 Dimensions of membrane

Items	Characteristic
Type	hollow fiber polyethylene
Pore size	0.4 µm
Surface area	0.2 m^2
Length of a fiber	0.14 m
Filtration rate	7.5 $L/m^2.h$
Manufacturer	Misubishi Rayon company

3.3.4 Experiment runs

The experiment was carried out in the Ambient Laboratory in AIT. The two processes of membrane bioreactor are anaerobic and aerobic. Feed water for both processes was effluent of septic tank. Target of this study is to improve effluent quality of septic tank before discharge or reuse for other purposes. The sketches of experiments are show in the Figure 3.2.

The wastewater is stored in holding tank and flowed gravity to the level control tank, which control water level in reactors. The wastewater was fed everyday in to holding tank by actual wastewater. These reactors were operated under two different modes that are aerobic and anaerobic condition. Membrane was submerged in reactor and suction pump was connected directly to the membrane to withdraw water that passed through membrane (effluent of treatment process). Water quality of influent and effluent was measured COD, TKN, total phosphorous and some other parameters in order to find out removal efficiency. The parameters and analysis frequency are showed in Table 3.4.

In both processes, U sharp Hg manometer was used to measure trans-membrane pressure. The reactor was stop for membrane cleaning when pressure equal to 60 kPa (equivalent 450 mm Hg). The clean process is described in the section 3.3.6

Type 1: Anaerobic condition

A hollow fiber membrane was submerged in the reactor. A centrifuge pump was used for circulating water in the reactor to maintaining suspended state of activated sludge inside reactor. The circulating flow was withdrawn at high level of water and distributed through perforating pipes at the bottom of the reactor. This system includes 2 pipes at two sides and 1 pipe at the central reactor and under membrane. The velocity of circulating water was maintained at 4 L/minute. The other conditions for this process are shown in the Table 3.2.

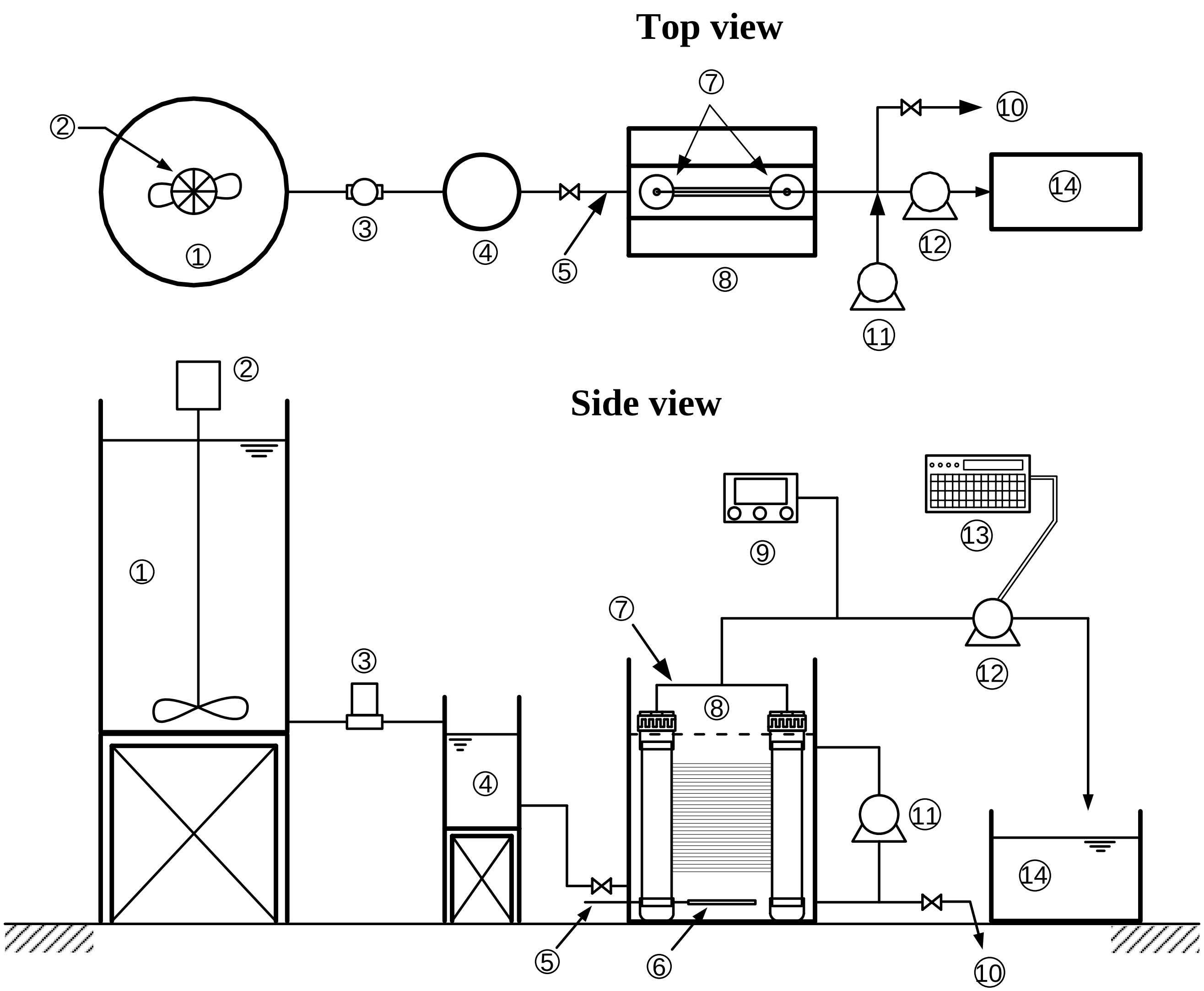
Type 2: Aerobic condition

The aerobic reactor has small difference to anaerobic reactor. It was divided in to 3 compartments (as Figures in Appendix A), in which two compartment outside used to put media, the middle compartment used to submerge membrane. The feed water from level control was run to reactor at the bottom of middle compartment. In this process, the air was supply. The aim is not only provide oxygen for microorganisms but also maintain the suspended condition of sludge. Air was come through stone diffusers located at the bottom of reactor. 2 ball diffusers were installed in each side compartment. The rectangular diffuser was put in the central compartment and under the membrane that created air flow through membrane for reduction of cake layer formed by activated sludge on the surface of membrane. The airflow rate was kept at 1 L/min per 1 liter of reactor. The other conditions for this run are also shown in the Table 3.2

Table 3.2 The operating condition of MBR systems

Parameter	**Unit**	**Condition**	
		Anaerobic process	Aerobic process
Filtration rate	L/h	0.6	1.3
HRT	h	16	8
SRT	d	30	30
MLSS	mg/L	3000 – 5000	3000 – 8000
Recirculation flow	L/min	4	-
Air diffusion*	L/min per 1 liter of reactor	-	1
Clogging protection		Water recirculation	Air diffuse
Operation mode	On: Idle	8: 2	8: 2

* Source Ueda et al (1997), Chang et al (2002)



Note:

① Feed tank
② Stirrer
③ Solenoid valve
④ Level control tank
⑤ Air supply
⑥ Air diffuse
⑦ Membrane module
⑧ MBR
⑨ Digital manometer
⑩ Sludge discharge
⑪ Recirculation pump
⑫ Suction pump
⑬ Timer
⑭ Treated wastewater tank

Figure 3.2 The sketch of experimental process

3.3.5 Membrane cleaning

The membrane cleaning is required, when trans-membrane pressure (TMP) increased up to 60kPa. Chemical cleaning was help to reduce the trans-membrane pressure back to the level that closes to the initial level. This cleaning also enables operation of membrane stable in a period. The membrane cleaning procedure as:

- Membrane was taken out of reactor and flushed with tap water to remove the cake layer attaching on the membrane surface.
- Membrane was immerged into chemical tank containing mixture of NaOH 4 % and chlorine 3000 mg/L (Sodium hypochlorite having effective when chlorine concentration around 3000 mg/L), leave for 6-24 h. Because chlorine in aqueous solution was not stable it was escaped and reduced concentration, so it was checked concentration of chlorine in sodium hypochlorite solution before using.
- After 24h immerged in chemical solution, membrane was taken out and rinsed with tap water to remove chemicals. It was required to make sure that there were no residual chlorine retain in the membrane before it was installed back to the reactor.
- Membrane was measured membrane resistance (Rm) before used in the next run by distilled water to find the percentage recovery. More than 80% of recovery was achieved before membrane was put back to reactor for next run

3.3.6 Membrane resistance

Membrane resistance is an indicator for efficiency of membrane. It was measured by using pure water at different filtration fluxes and recording the corresponding trans-membrane pressures. Membrane resistance was derived from the slope of the linear curve of relationship of ΔP and J as described by the equation:

$$J = \frac{\Delta P}{\mu * R_t} \Rightarrow \Delta P = J * \mu * R_t \qquad \text{Equation 3.1}$$

Where:

J: permeate flux ($L/m^2.h$)
ΔP: trans-membrane pressure (kPa)
μ: Viscosity of the permeate ($N.s/m^2$)
Rt: total resistance (m^{-1})

$$R_t = R_m + R_c + R_f \qquad \text{Equation 3.2}$$

R_m: intrinsic membrane resistance
R_c: Cake resistance from by the cake layer
R_f: fouling resistance caused by solute adsorption into the membrane pore

3.4 Filtration test

In order to compares characteristics of suspended solid in the effluent of the septic tank to other types of wastewater, two selected wastewaters were chosen, that are raw wastewater and effluent of sedimentation tank in municipal wastewater treatment plant.

The parameters and methods were used to compare characteristics of suspended solid in these wastewater are described as below

3.4.1 Particle size distribution

Particle counting and size distribution analysis can help to determine the makeup of natural waters, treatment plant influent, process water, and finished water. Similarly, it can aid in designing treatment processes, making decisions about changes in operations, and/or determining process efficiency.

Three types of instrument can be used to measure particle size distribution: electrical sensing zone instruments, light-blockage instruments, and light-scattering instruments.

The details of method selection, sampling and handling, data reporting are described in the standard methods for the examination of water and wastewater (APHA et al., 1998).

3.4.2 Capillary Suction Time

The capillary suction time test determines rate of water release from sludge. It provides a quantitative measure, reported in seconds, of how sludge releases its water. The results can be used to assist in filtration processes or sludge dewatering processes. The lower the CST the better is the filterability of the sludge (Baskerville and Gale, 1968). The measuring process as:

- Turn on and reset CST meter.
- Dry CST test block and reservoir.
- Put a new CST paper on lower test block.
- Add upper test block.
- Insert reservoir into test block seat it using light pressure and a quarter turn to prevent surface leaks
- Pipet 6.4 mL sample into test cell reservoir.
- Record CST shown on digital display.

The CST device begins time measurement as liquid being drawn into the paper reaches the inner pair of electrical contacts. Timing ends when the outer contact is reached.

3.4.3 Membrane fouling index

Membrane fouling index was measured volume of filtrate versus with time. Hence, the unit of MFI was T/L^6 (often use s/L^2). MFI was measured by stirred cell, pressure filter holder made by Germany. The procedure used to measure membrane fouling index as follow:

- Open valve on the top of the nitrogen gas container.
- Adjust air flow rate to 1 bar by close V-2 and V-4 & Adjust V-1 and V3 to get constant pressure of 1 bar maintaining in system.
- Prepare distilled water and samples with volume of 200 mL each.
- Insert membrane(*) and other membrane support layers as the figure 3.4
- Fill sample into Filter Holder.
- Prepare a beaker 250 mL on a balance connected with computer for weighing filtrate
- Activate weighing software to start data recording
- Open V- 4 to start measurement of filtrate volume versus time

- Stop measurement at the time of constant weight recorded
- Close valve on the top of gas cylinder
- Open V-2 for air released.
- Close V-1 and V-4 before reinstalling Filter Holder
- Reinstall and clean equipment.

(*) Membrane that used for distilled water can be used for real sample.

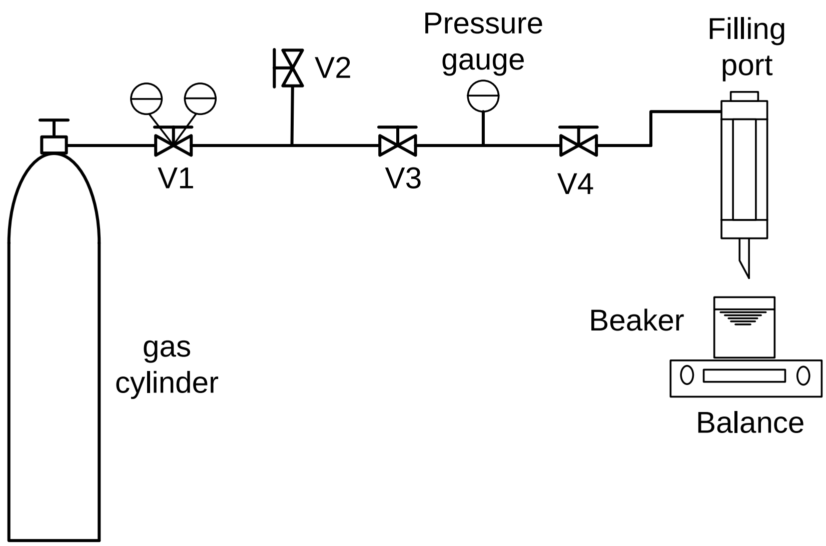


Figure 3.3 Sketch of MFI measurement

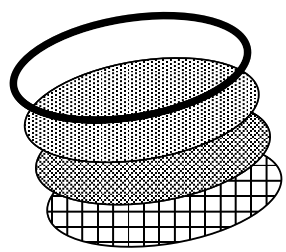


Figure 3.4 Membrane configuration inside of filter holder

3.5 Analysis

Almost analytical analysis use in this study is mentioned in the standard methods. List of these parameter are listed in the table 3.3. Those parameters use to monitor septic tank and check effective removal of experiment processes.

Table 3.3 Various parameters for analysis

Parameter	Analytical methods	Analytical equipment	Range	Sampling point	Frequency for MBR	References
pH		pH meter	0 - 14	MBR, septic tank*	Daily	
DO		DO meter	0 - 8	MBR, septic tank*	Daily	
SS	Dry at $103^0 - 105^0$C	Oven	-	Septic tank*	-	APHA et al 1998
COD	Dichromate reflux	Titration	20 - 300	MBR, septic tank*	1/days	APHA et al 1998
BOD_5	5-Days BOD test	Titration	-	Septic tank*	-	APHA et al 1998
NH_4^+	Distillation	Titration	5 - 100	MBR, septic tank*	1/three days	APHA et al 1998
TKN	Digestion/distillation	Titration	4 - 400	MBR, septic tank*	1/three days	APHA et al 1998
Total Phosphorous	Stannous Chloride	Spectrophotometer	0.3 - 2	MBR, septic tank*	1/three days	APHA et al 1998
MLSS	Dry at $103^0 - 105^0$C	Oven	0 – 10,000	MBR	1/week	APHA et al 1998
MLVSS	Dry at 550^0C	Oven	-	MBR	1/week	APHA et al 1998
Particle size distribution	Light scattering technique	Matersizer S	0.05 - 900	Septic tank, raw wastewater, mixed liquor in reactors	-	APHA et al 1998
CST	Capillary suction time	CST apparatus	0 - 9999	Septic tank, raw wastewater, mixed liquor in reactors	-	APHA et al 1998
MFI		Pressure filter holder	-	Septic tank, raw wastewater, mixed liquor in reactors	-	Choo and Lee, 1996

* The frequent of the analysis for septic tank is monthly

Chapter 4

Results and Discussion

This chapter includes three parts. The first part is summary of the survey about sanitary condition in Hanoi, Vietnam. Second presents analysis results of the wastewater from septic tank in suburban area of Bangkok, Thailand. The third part is experimental results and discussion.

4.1 Hanoi sewerage and sanitation system

4.1.1 Introduction

Hanoi is capital of Vietnam. The total land area is about 927.380 km^2 and land use for agriculture is 53.38%, land use for resident is about 14% in which land for urban area only one fifth. The population is 2.8 millions persons in which urban population is 1.5 million and rural (peri-urban) is 1.3 millions peoples. Population density is 2,900 people/km^2. There are many buildings for living quarters. Most of them were constructed long time before. Recently, newly buildings have been constructing in new urbanization areas. On other hand, many peoples have their own house.

Total daily water production is 600,000 – 650,000 m^3/d. there are 9 large water treatment plants, each with water production of 30,000 - 80,000 m^3/d and smaller water treatment plants with production of 1,000 - 2,000 m^3/d for each. Besides, there are numbers of household scale well (more than 100,000 wells). At present, there is about 70% population served with clean water. Totally, it about 1.5 million people is served with average consumption of 120 liters/capital/day. (CERWASS, 2004)

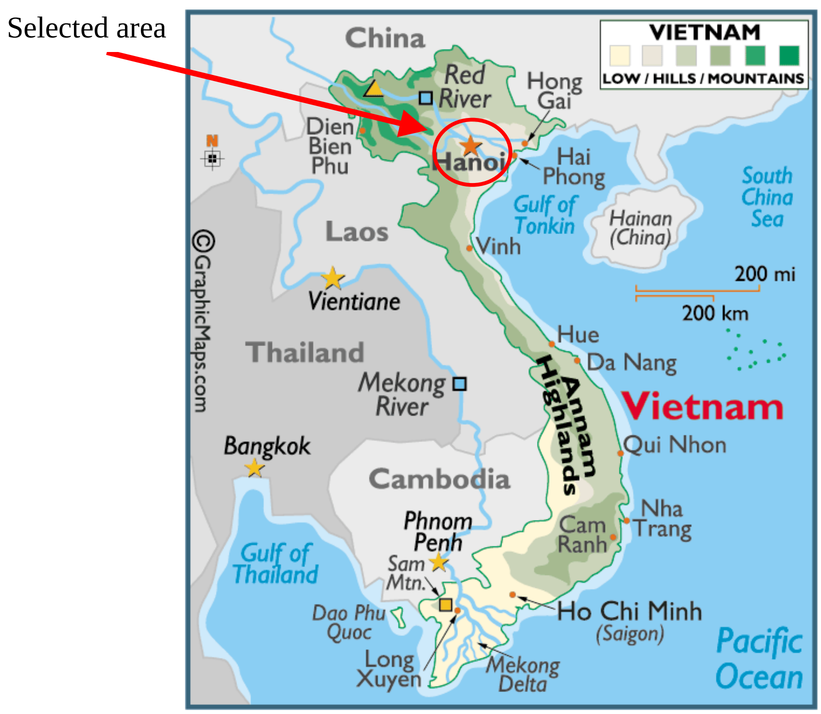


Figure 4.1 Map of selected area in Vietnam

4.1.2 Sewerage and drainage system

At present, in Hanoi, single drainage system is used to collect both storm water and wastewater. This system comprises rivers, canals, regulating ponds, ditches, and pipes. Wastewater from domestic and other use is discharge directly into open lakes or canals by ditches and pipes that connect from the wastewater system in the houses. For example, some canals as Thuy Khe, Hao Nam, Thinh Hao, Luong Su, Lang Trung, Khuong Thuong. The canals inside the city are narrow and these canals are seriously polluted. Water in the canals is black, with bad odour problems. Flooding often occurred after raining. The Figure 4.2 presents about sewerage system in Hanoi.

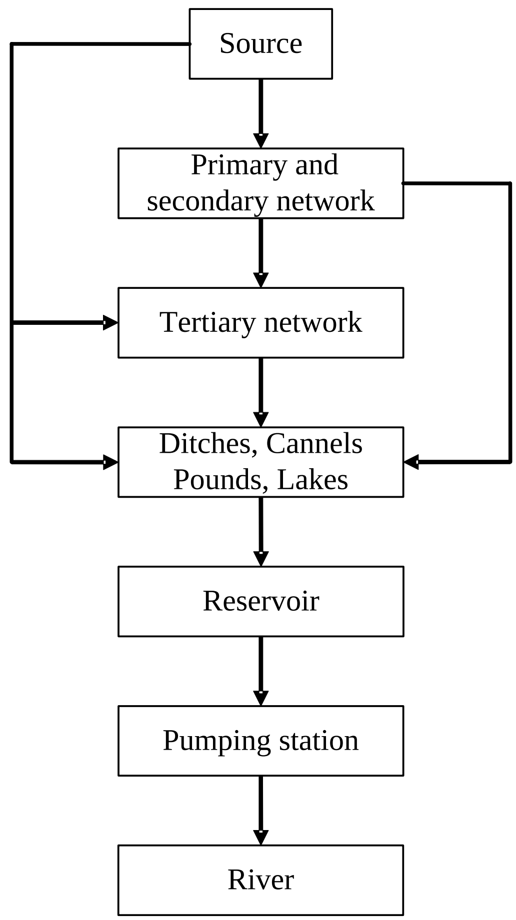


Figure 4.2 Scheme of sewerage and drainage organization of Hanoi city (Vietnam Water and Environment Company, 2002)

According to the Hanoi SADCO (2003), the coverage of the sewerage and drainage system is only about 40% the areas of the city. Most of the system is very old, some parts were constructed long ago in the French time (more than 100 years), and now its quality is very bad. Besides, some parts of the system have been reconstructed or newly built by the Hanoi master plan for sewerage and drainage project.

Hanoi has four main rivers use for drainage of wastewater that are To lich, Set, Lu and Kim nguu. These rivers use to drain wastewater in the city to the reservoir called Yen so. In recent years, many new drains and components are built, especially a large system of regulating reservoirs and a pumping station with capacity of 45 m^3/s has built (designed capacity is 90 m^3/s) to pump wastewater directly to the Red River.

The total volume of wastewater generated in Hanoi city is about 460,000 m^3/day. The ratio of sewer length per capita in Hanoi city is only about 0.3 m/person, and it is less than average ratio in other developing countries, normally at least 2 m/person. (Vietnam Water and Environment Company, 2002).

The monitor data from Hanoi Science Technology and Environment Department indicated that water quality in open cannels and lakes is very terrible, due to directly discharge of domestic and industrial wastewater. The characteristics of sewer in some discharge gates and canals in Hanoi are shown in Tables 4.1 and 4.2 below

Table 4.1 Wastewater characteristics in selected sewer gates in Hanoi

Parameters	**Tran Binh Trong into Bay Mau lake**	**Lo Duc into Kim Nguu river**	**Trung Tu into Lu river**	**Kim Lien**
pH	7.15	7.2	7.4	7.7
SS, mg/L	285	240	125	270
DO, mg/L	1.5	0.5	1.2	0.4
BOD_5, mg/L	85	180	46	250
COD, mg/L	182	329	72	315
NH_4^+, mg/L	20.2	30	12	45
PO_4^{-3}, mg/L	4.2	7.1	0.6	12.5
Cl^-, mg/L	71	125	105	105
Coliform, MPN/100mL	$5.1 * 10^3$	$1.1 * 10^4$	$6.1 * 10^3$	$1.4 * 10^5$

Wastewater in some discharge points in the Table 4.1 above with pH from 7.1 to 7.7 is acceptable to discharge into surface water source. However, this wastewater is low in DO, high in SS. Even DO is very low, lower than 2 mg/L whereas SS is higher than 100 mg/L. Especially, SS is about 300 mg/L in point Tran Binh Trong discharge to Bay Mau Lake. For BOD, it is quite high in some discharge points. However, in other point, it is very high such as in Kim lien. The value is 250 mg/L and it around fivefold higher than standard of Vietnam for domestic wastewater. Other parameters such as ammonia, COD and total phosphate are not high, because this is domestic wastewater.

In the canals, quality of water is also terrible. Water is low in DO but high in SS, BOD and coliform. Some points in the canals are high in COD such as Khuong Thuong 1 and 2, it could be explained by effect of industrial wastewater due to some factories located in these areas. However, water high in BOD and coliform is affected by domestic wastewater. COD is up to 2000 mg/L in the Khuong Thuong 1 and DO is reduced to 0.2 mg/L in the Thuy Khe indicated the strong pollution of wastewater in the canals. The details of water quality in the canals in Hanoi are shown in the Table 4.2 below.

Table 4.2 Water quality of some canals in Hanoi

Parameters	Hao Nam		Khuong Thuong		Thuy Khe	
	1	2	1	2	1	2
pH	7.4	7.4	7.3	7.1	7.4	7.8
SS, mg/L	189	98	104	193	200	120
DO, mg/L	0.8	2.9	0.5	1.5	0.2	4.5
BOD_5, mg/L	180	60	135	135	156	54
COD, mg/L	256	89	1536	2080	358	102
NH_4^+, mg/L	20	8.6	29	26	18	6.8
PO_4^{-3}, mg/L	4.0	2.4	3.8	3.6	3.9	2.0
Coliform, MPN/100ml	$5.6 * 10^6$	$3.2 * 10^5$	$1.7 * 10^6$	$1.2 * 10^5$	$1.5 * 10^6$	$8.5 * 10^4$

According to the Vietnam Water and Environment Company (2002), two pilot scale wastewater treatment plants have been building in Kim Lien and Truc Bach section of Hanoi. These plants will be served for domestic wastewater treatment. These plants will be put on service in June 2006. They were designed and built by Japanese constructor.

4.1.3 Sanitation in Hanoi

According to the Hanoi URENCO, Hanoi still has about 10,000 double vault latrines and 2000 bucket latrines in operation. Sanitation types in Hanoi comprise pit latrines, ventilated improved pit latrines (VIPs), double - vault urine-diverting toilets, single - vault urine diverting toilets, pour-flush toilets with filtration pit, pour-flush toilets with septic tanks; wherein, septic tank is the most popular. Biogas digesters are only found in sub-urban districts of the city. The distribution of sanitation means in Hanoi urban areas is shown in Table 4.3.

Table 4.3 Sanitation facilities in urban areas (CERWASS, 2004)

Types of on-site sanitation	Ratio (%)			
	1995	1998	1999	2000
Septic tank	54	56	63	68.4
Pour- latrine	2	12	10.4	9.2
Double - vault toilet	20	12	4.7	4.9
Single - vault toilet	16	9	3.8	1.8
Public Toilet				
Single-vault toilet	4	2.4	0.5	-
Double - vault toilet	2	0.7	7.3	-
Septic tank	2	7.9	10.3	15.7

Based on data in the Table 4.4, the results indicate that quantity of septic tank is increasing with time and other types of latrine are decreasing. The reason can be the urbanization of peri-urban areas in Hanoi due to economical development and expansion of city. At present, many newly buildings have been constructing in peri-urban areas that make quantity of septic tank increasing very fast and it is policy of community. Number of public toilet system will be increased year by year. This also is other reason of increasing quantity of septic tank.

Table 4.4 Sanitation facilities in Sub-urban Areas (CERWASS, 2004)

District	Single-vault toilet	Double-vault toilet	Septic tank	Others	total household
Tu Liem	4161	6533	29945	372	50,100
Soc Son	26663	8020	6773	4634	50,000
Dong Anh	15194	18540	25918	2029	63,000
Gia Lam	5429	22839	13504	590	43,000
Thanh Tri	5956	3818	18009	3579	32,000
Total	57403	59750	94149	11204	238,000

Due to pollution of domestic wastewater occurred in almost all resident areas, the reason could be low efficient of septic tank. Center for Environmental Engineering of Towns and Industrial Areas was survey the treatment efficiency from 3 septic tanks in Hanoi for 6 months duration. Generally, results are shown in Table 4.5.

Table 4.5 Results from survey on septic tanks in Hanoi

Parameter	Inlet	Outlet	Standard of Vietnam*
Temperature, ^oC	20.1 – 29.4	21.1 – 29.5	ND
pH	7.32 – 8.1	6.17 – 8.5	5 - 9
TDS, mg/L	412 – 652	381 – 637	500
SS, mg/L	380 – 767	86 – 812	50
BOD_5, mg/L	240 – 376	102 – 330	20
$N\text{-}NH_4$, mg/L	38 – 66	20 – 43	ND
$N\text{-}NO_3^-$, mg/L	0 - 1,2	0.01 – 2.9	30
$P\text{-}PO_4^{3-}$, mg/L	3.1 - 4.1	2.79 – 33.5	6
Total Coliform, MPN/100 mL	$1.4*10^7 – 1.5*10^8$	$7.3*10^5 – 1.3*10^7$	$1*10^3$

*Source: MOST, 2000

Results in Table 4.5 indicated low removal efficiency of monitored septic tanks. The inlet and out let are not much different. The small difference of SS, BOD and ammonia was observed in the influent and effluent. However, almost all parameters in the effluent are higher than Vietnamese standard. Due to its effluent is always connects to sewer. Therefore, it seemed as a source of pollutant in Hanoi.

4.1.4 Septic tank

Septic tank is one of the classical wastewater treatment units. Up to now, the tank has been used very popularly in Vietnam. However, its construction does not conform to standard specifications. In addition, settling sludge in chamber is emptied after using 5-10 years instead of 1-2 years. Therefore, its treating efficiency is too low and concentration of pollutants in the effluent is high. Normally, effluent of septic tank always connects to the sewerage system. This water is combined with others wastewater in the sewage system. That lead environmental pollution was very seriously.

At present, Institute of Standardized Construction and Ministry of Construction promulgated a typical design for septic tank. This design included total 26 types of septic tank with different volume from 1.5 m^3 to 20 m^3. The detail of design and dimensions is described in the Appendix B. This type of septic tank was introduced in the first part of the typical designs about medium and small wastewater treatment system in Vietnam.

According to Ha (2002), septic tank has simple structure. It is used to serve as an onsite treatment system for house, building, collective quarter that has total flow lower than 30 m^3/day. It can be rectangular or circular. It is divided into 2 compartments. It can be used for pretreatment of domestic wastewater from toilet. Normally, first compartment (counted from influent) is largest. Its volume is about 70% of total volume, due to almost all solid is settled in it. Its function is not only for solid settle but also holding sludge formed from degradation of waste. The second compartment is around 30% of total volume. In general, its depth is from 1.0 to 3.0 meters depending on the number of toilet it serve. The depth of water inside is not lower than 0.75 meters but not higher than 1.8 meters. The width and length of the basic septic tank are 0.9 and 1.5 meters respectively. The dimensions for some basic septic tanks are shown in the Table 4.6 below.

Table 4.6 Examples of basic dimensions for some septic tanks

Volume (m^3)	**Dimensions (meters)**			
	Length of first compartment	Length of second compartment	Width	depth
2.0	2.4	0.9	0.9	1.0
2.5	2.6	1.0	1.0	1.0
3.0	2.2	1.1	1.1	1.1
5.0	2.4	1.2	1.2	1.2
10.0	3.0	1.5	1.5	1.5

The hydraulic retention time of wastewater in the septic can be 1 to 3 days. Due to low velocity, almost all suspended solid will be settled in the first compartment. The solid removal efficiency can achieved from 40% to 60% depend on operation and management. Gases such as CH_4, H_2S etc were formed as a result of anaerobic degradation. Small solids were attached on to gas bubbles. When gas rise up and escape, these particles will be formed scum layer on the top in side tank. The thickness of this layer can be from 0.3 to 0.5 meter. Sludge is drawn periodic, normally 1 to 2 years. Total only 80% of sludge is taken out, 20% is remained as source of microorganism for degradation after withdrawing sludge.

Table 4.7 Examples about volume of septic tank for different building

Number of bed rooms in House	**Number of apartments in Building**	**Basic volume of septic tank, m^3**
2		2.8
3		3.8
4	2	4.5
	5	8.5
	8	11.5
	10	13.5

According to the Vietnamese standard TCXD 51:1984, the volume of septic tank should not lower than total flow in 1 to 2 days of the house or building. It should be avoided if design tank with volume larger than 25 m^3. When total flow is higher, it is better if separate in to multiple tanks.

4.1.5 Research contributions

Hanoi environment is polluted very seriously and reasons can be improper maintenance and no expanding of sewer system, with discharge of untreated wastewater into cannels and lakes inside city. Nowadays, most of households and building installed septic tanks to treat wastewater before discharge to sewer system. However, the removal efficiency of these septic tanks is low, hence number of septic tank is increased with time to replace for other type of sanitation systems, but the pollution is not decreased.

The objectives of this study are mention on decentralized treatment system and improvement of effluent septic tank by using MBR technology. The results of this study will help to improve quality of effluent septic tank. This also reduces the quantity of pollutants discharge into environment through domestic wastewater. The lower quantity of pollutants discharge is fewer breakdowns inside sewer system and less pollution in lakes and cannels in the city. These will help more improvement of environment in city when channels and lakes are used to discharge and transfer of wastewater inside city.

4.2 Septic tank monitoring in Thailand

4.2.1 Introduction

Klong 4 located about 50 km north of Bangkok is a suburban area in Klong Luang - Pathumthani province, Thailand. Total population is 8235 peoples. The people live based on agricultural activity. Land is divided about 50% for agriculture and 50% for residence. There is a clean water supply system with coverage for 100% resident area. Inhabitant has enough clean water for domestic activities. There is no sewer system and wastewater is discharged directly to cannels or water body which is close to the residences. For sanitation system, all household have water seal latrine. Most of it is pour and connected to septic tank. The most of tanks is circular with one or two compartments, detail design is described in Appendix D and map of monitored area is shown in the Appendix C.

In this study, a total of 10 septic tanks were chosen to monitor for 6 months. Among the 10, 8 septic tanks were monitored at normal operation mode while 2 remained at sludge withdrawn mode. For normal operation mode, samples were taken without any modification in present operation condition of septic tanks. For sludge withdrawn mode, all sludge inside septic tank was withdrawn. Samples were taken once and twice a month for normal mode and sludge withdrawn mode, respectively, to monitor the variation of effluent quality. The monitor results are shown in the Appendix E and discussed in following sections

4.2.2 Normal operation mode

Normally, pH and DO were measured immediately after samples taken out of septic tanks. The monitored results of pH are presented in Figure 4.3, based on this Figure the value of pH is in the range of 7.1 to 8.2, but the predominant range is 7.2 to 7.8. This is the same the value reported by EPA (2002) and Zhang et al. (1996). The results of pH between monitoring times of each location are not much different, they are around 0.3. This variation is normal because pH is very sensitive to temperature and influent quality. Between locations, pH values is significantly different, this is due to habit of the user.

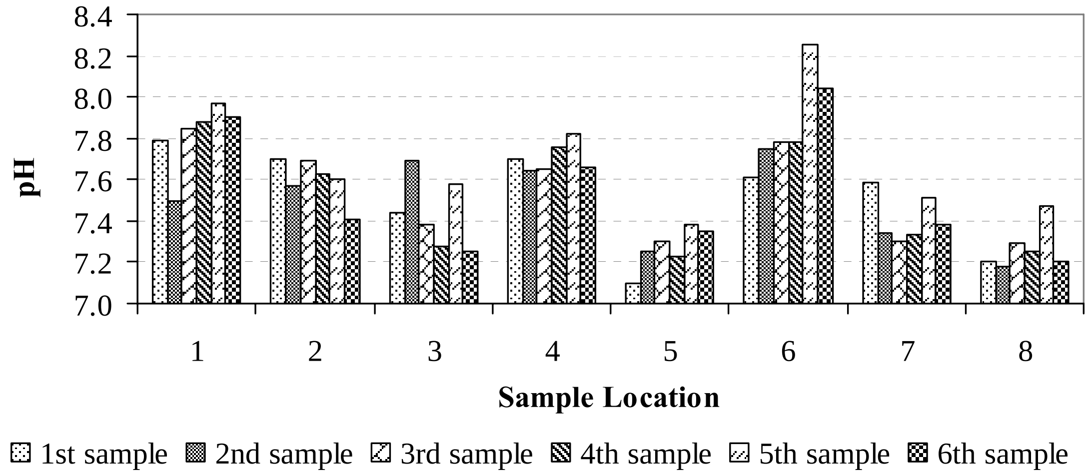


Figure 4.3 pH in monitored septic tanks

The results of DO in Figure 4.4 shown that almost of them are lower than 0.5 mg/L. This is due to the septic tank is closed and high concentration of organic matter so the anaerobic condition was created in all the tanks

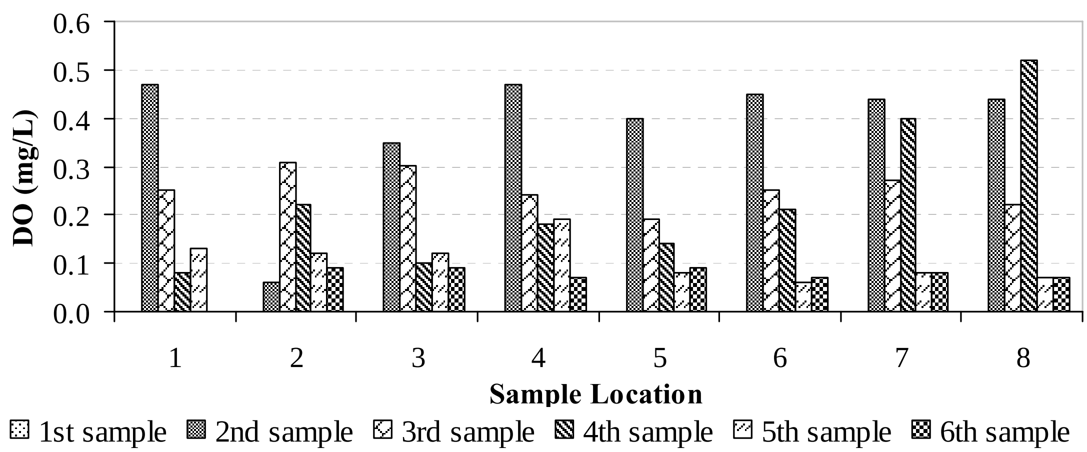


Figure 4.4 DO in monitored septic tanks

Inside septic tank, anaerobic degradation of organic matter occurred and suspended solids were results of this process. Figure 4.5 depicts the results of SS concentration in the monitored septic tanks. Based on this Figure, SS values varied from 25 mg/L to 720 mg/L. However, predominant values were in the range from 150 mg/L to 500 mg/L, which are higher than posted values by Zhang et al. (1996). This value is different from location to location, some tanks were low whereas others were high. Most of SS values between monitoring times of each locations varied, others remained steady. This is due to inlet quality changed between monitoring times.

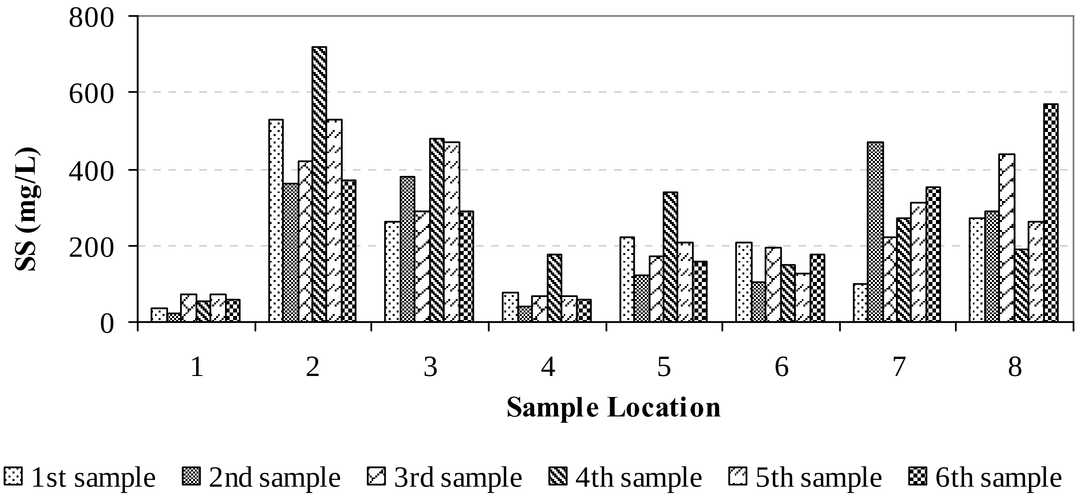


Figure 4.5 Suspended solid in monitored septic tanks

The tanks with high concentration of SS such as location No 2, 3, 7 and 8, a hard and thick scum layer occurred. This could be a result of an improper operation including late withdrawal of sludge. The volume of tank is limited, while sludge is accumulated with time. SS increases day by day in these tanks, which makes septic tanks to be overload and was failed. High levels of SS may affect the infiltration capacity in subsurface disposal areas (Charles et al., 2005), this prevented the leach of wastewater inside these septic tanks.

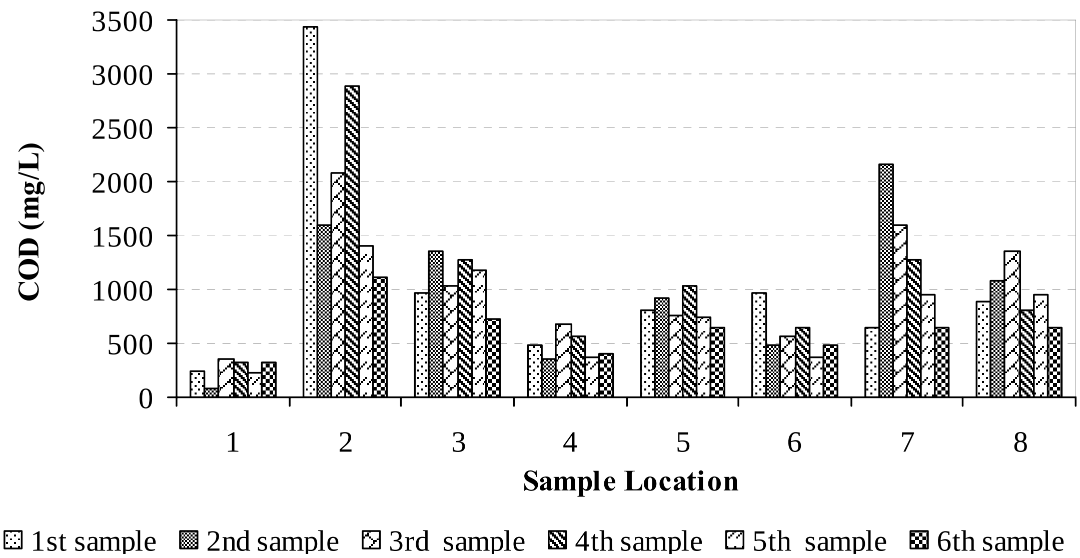
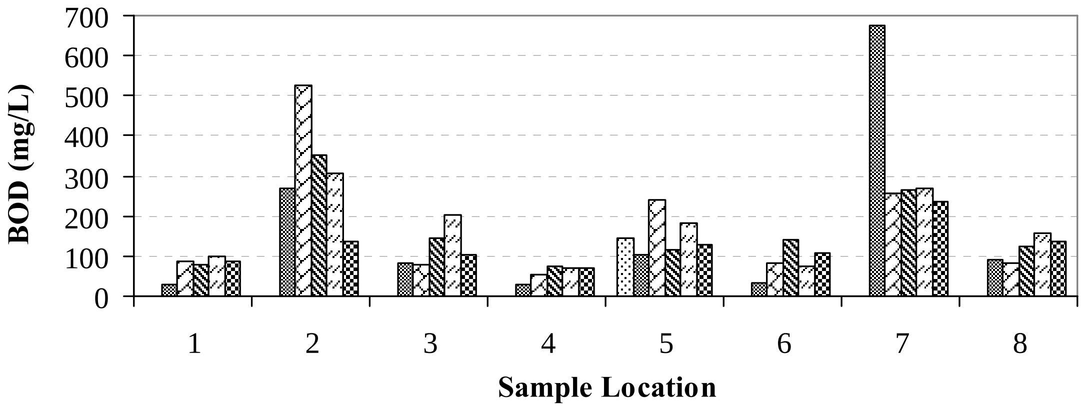


Figure 4.6 COD in monitored septic tanks.

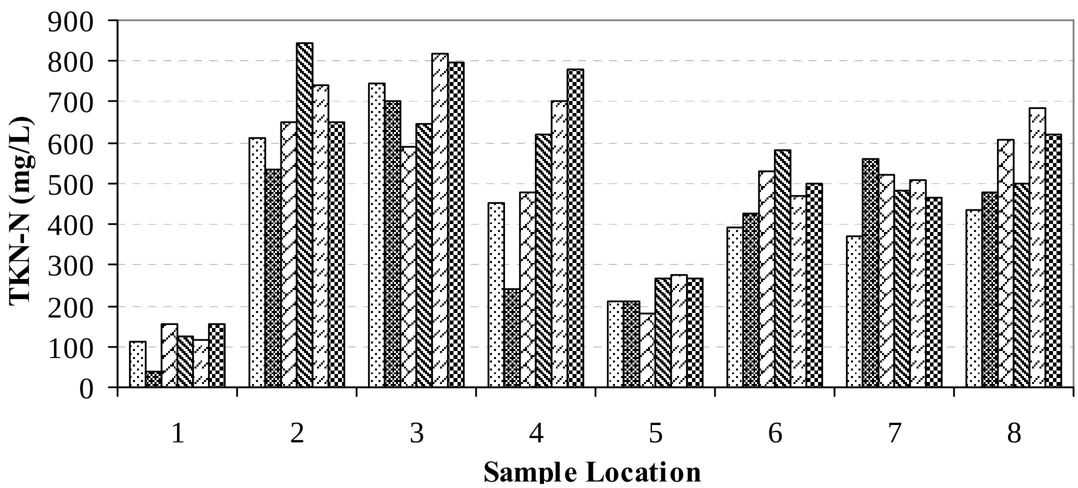
BOD and COD are important parameters when evaluating effluent quality of septic tanks. The COD monitoring result is presented in Figure 4.6. Based on this Figure, the values of COD are 300 mg/L to 1500 mg/L. These are higher than results reported by Van Cuyk et al. (2001) and Zhang et al. (1996). The difference occurred from tanks to tanks and between monitoring times. Some samples had wide range of COD, such as septic tank in location 2 and 7 that could be affected by influent quality and suspended solids content. Whereas others were quite steady.



1st sample 2nd sample 3rd sample 4th sample 5th sample 6th sample

Figure 4.7 BOD in monitored septic tanks

The results of BOD is described in Figure 4.7, based on this Figure, most of values of BOD varied in the range of 50 mg/L to 300 mg/L. However, high value was measured at location No 2 and 7 with values of 550 mg/L and 675 mg/L, respectively. The results of BOD in this study are low comparing with the posted reports (Charles et al., 2005; Van Cuyk et al., 2001). The BOD in a septic tank was not different between months and locations, it was steady during monitor time.



1st sample 2nd sample 3rd sample 4th sample 5th sample 6th sample

Figure 4.8 TKN in monitored septic tanks

For TKN, results are showed in Figure 4.8, the highest value achieved is 842 mg/L at location No 2 and lowest value is 39.2 mg/L at location No 1. However, almost all values are in the range of 200 mg/L to 700 mg/L. They are higher than 60 mg/L reported by Van Cuyk et al. (2001); and Crites and Tchobanoglous (1998). Some septic tanks were steady values such as Location No 1 and No 5, but others were not.

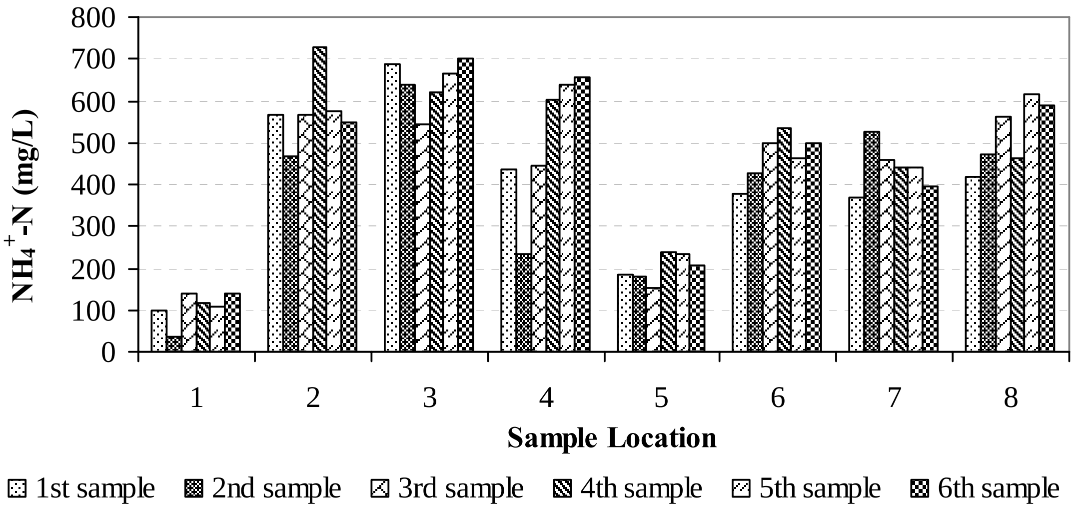


Figure 4.9 Ammonia in monitored septic tanks

Similar to TKN, the ammonia content in these septic tanks were also high. As depicted in Figure 4.9, the highest value was 720 mg/L in Location No 2 and lowest value was 30 mg/L in location No 1. Almost all Locations got values in the range of 150 mg/L to 600 mg/L. These values were higher than that in the posted documents (EPA, 2002; Crites and Tchobanoglous, 1998). However, these values were equal to the influent of septic tank reported by Zhang et al. (1996). The results indicated that ammonia content was about 70 to 100% of TKN. The variation was not much between different months.

Based on results described in Figure 4.5-4.9, there was a relationship between SS, COD, BOD, TKN and NH_4^+, the higher value of SS is, the higher values of COD, BOD, TKN and NH_4^+ are and vice versa. This means that the values of these parameters depend on the organic matters content in the influent of septic tank.

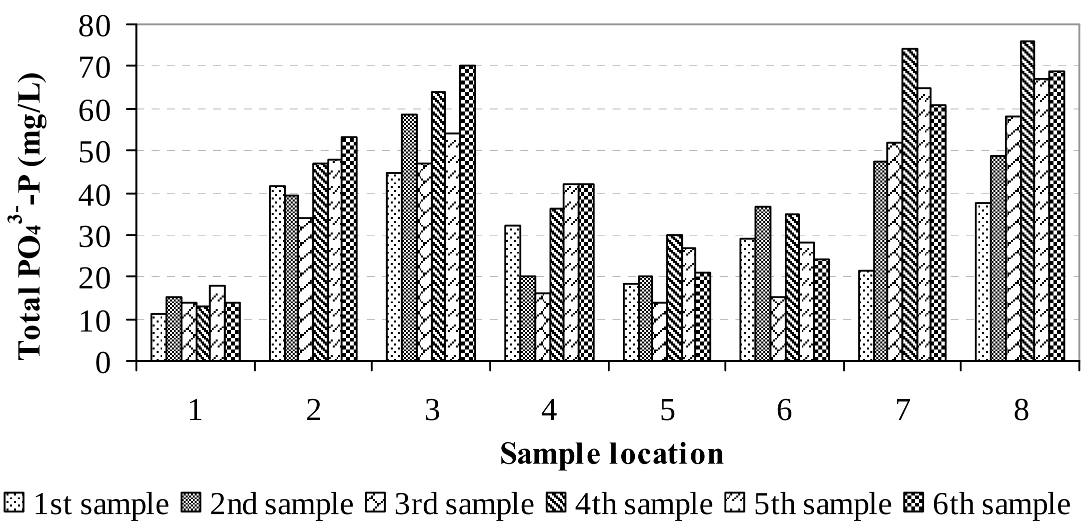


Figure 4.10 Total phosphates in monitored septic tanks

Figure 4.10 shows the monitoring results of total phosphate. The steady results were observed in septic tank no 1, the others were varying with values of 15 mg/L to 50 mg/L. These results were much higher than that in the other reports (Charles et al., 2005; EPA, 2002; Van cuyk, 2001). The pour latrine is very popular in this area, so the water used for flushing containing phosphate detergent is unavoidable. This leads to high concentration and variation of phosphate in the monitored septic tanks.

In conclusion, the monitoring results in this study are compared to the other reports and presents in the Table 4.8 below. Based on this Table, the results of some parameters in this study was higher than that in the former reports such as SS, COD, TKN, ammonia and total phosphate. Whereas, BOD was not much different and these values were too low compare to the posted value in the Zhang et al. (1996).

Table 4.8 The comparison of monitoring results with other reports

Parameters	**This study**	**EPA, 2002**	**Van cuyk, 2001**	**Crites and Tchobanoglous, 1998**	**Zhang et al., 1996**
pH	7.2 – 7.8	6.4 – 7.8	-	-	7.8
SS	150 – 500	40 - 350	69	40 -140	256
COD	300 – 1500	-	386	250 - 500	-
BOD_5	50 – 300	46 - 156	227	150 - 250	576
NH_4-N	150 – 600	-	47	30 - 50	462
TKN	200 – 700	19 - 53	57	50 - 90	-
Total PO_4^{3-}-P	15 – 50	7.2 - 17	4.6	12 -20	-

All values are mg/L except pH

Table 4.9 depicts quality of effluent septic tanks at two selected places in this study. Based on these results, it is found that poor effluent quality was observed in both places. Wastewater contains high value of SS, BOD. The same value and variation of BOD found in septic tanks in both places. However, variation of pH and SS in effluent septic tanks in Hanoi is wider than that in suburban of Bangkok. In addition, SS in septic tanks in Hanoi is quite high compare to value in suburban of Bangkok. The high concentration of ammonia found in septic tanks in suburban of Bangkok and it varies in the wide range. Contrariwise, ammonia varies in the small range and low value found in septic tanks in Hanoi.

Table 4.9 Effluent septic tank of selected areas in this study

Parameters	**Suburban of Bangkok**	**Hanoi**
pH	7.2 – 7.8	6.1 – 8.5
SS	150 – 500	86 – 812
COD	300 – 1500	-
BOD_5	50 – 300	102 – 330
NH_4-N	150 – 600	20 – 43
TKN	200 – 700	-
Total PO_4^{3-}-P	15 – 50	2.7 – 33.5

All values are mg/L except pH

The differences in effluent of these septic tanks in two selected places in this study are normal. There are some reasons that could be cause of differences. The first, septic tanks in two places are different in term of geometry, the details of design of two types septic tank clearly explained in the Appendix B and C. The second is the difference of number of compartment. The septic tanks in Hanoi have two compartment whereas in suburban area in Bangkok are single compartment. The other reasons that make effluent septic tank in two places can be type of latrine, amount of water using for each flushing and quality of flushing water. In general, many factors affect on the effluent quality of these septic tanks. In short, effluent quality is poor and contaminated in both places and it should not discharge directly into sewer, water body or leaching to the soil.

4.2.3 Sludge withdraw mode

In order to observe the treatment process occurring with septic tank, sludge was withdrawn to create a start up position of septic tank. In this study, two septic tanks had withdrawn sludge. The monitored samples were taken after sludge withdrawal at frequency once per 2 weeks. The results were follows.

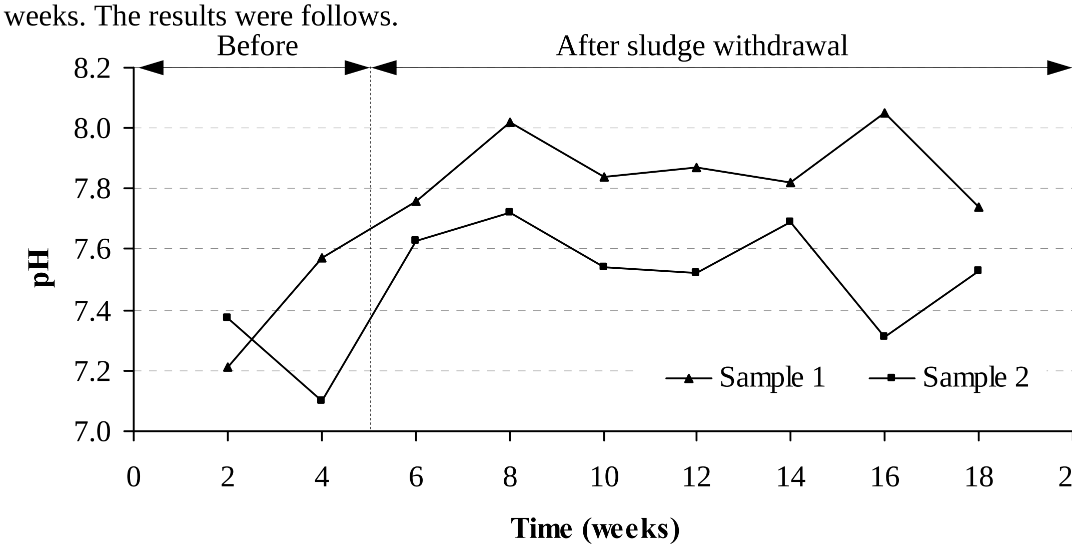


Figure 4.11 pH varies in withdrawn sludge septic tanks

Before withdrawn sludge, pH was different between two septic tanks and two monitor times. After sludge withdrawal, it still different, however, pH in both septic tank was increased and higher than before. The variation of pH was not much after sludge withdrawal, it was around 0.2 to 0.3. Figure 4.11 presents the variation of pH through monitored time.

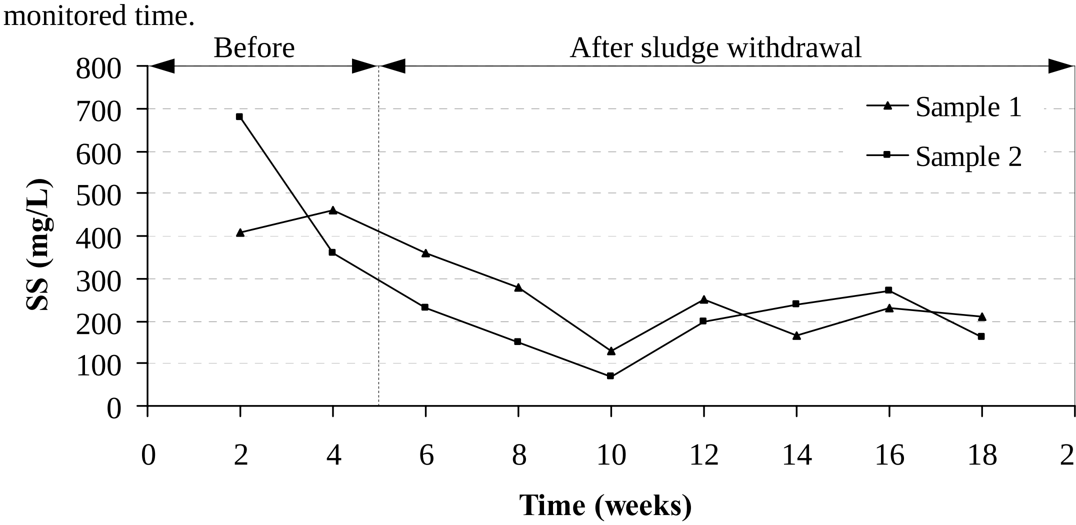


Figure 4.12 Variation of SS in withdrawn sludge septic tanks

SS was different between two septic tanks and two monitoring time before sludge was withdrawn as shown in Figure 4.12. It was two week after sludge withdrawn, measured value of SS decreased, but only 100 mg/L for each. The decreasing trend continued in the 4 weeks afterward. After week 8, SS slightly increased in both septic tanks and it was slightly various in the weeks after.

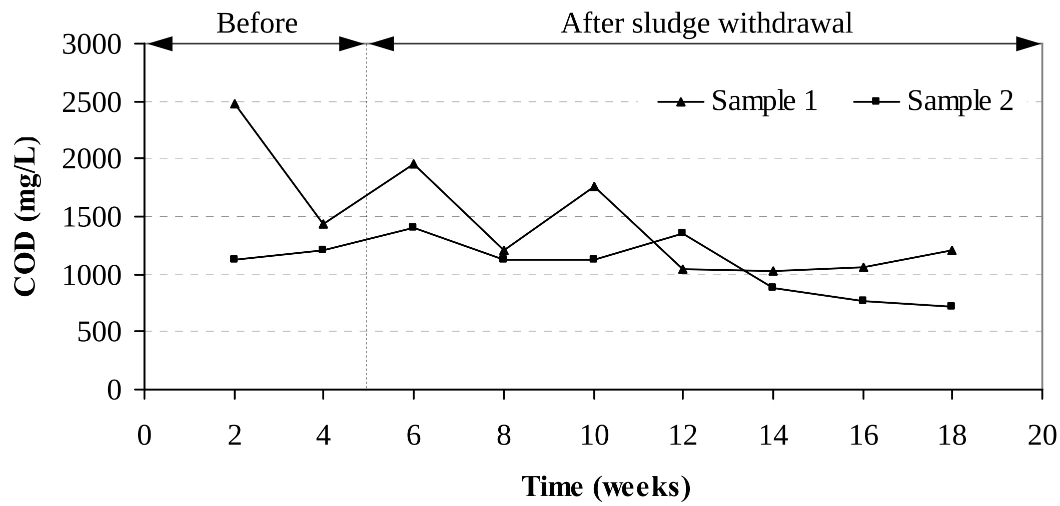


Figure 4.13 COD varies in withdrawn sludge septic tanks

The variation of COD was presented in Figure 4.13. It was different from SS and pH, COD changed not much after sludge withdrawal. It was slightly increased after 2 weeks. However, it decreased and slight varied in the weeks afterward. The variation was only around 200 mg/L. COD value in both septic tanks was around 1000 mg/L. The wide variation of COD in septic tank No 1 may effected by influent quality.

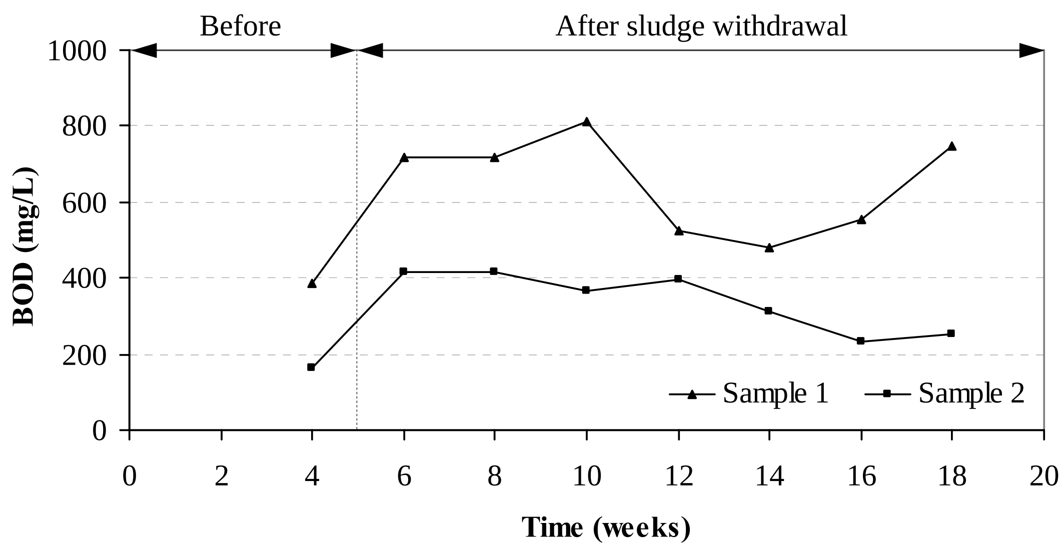


Figure 4.14 Variation of BOD in withdrawn sludge septic tanks

For BOD, Figure 4.14 presents the variation of BOD before and after sludge withdrawal. The value of BOD before was low than after sludge withdrawal. BOD in both septic tanks is reduced with time. Results showed that it take more than 8 weeks for microorganism to grow in order to recover septic tank to normal operational condition. The results indicated that BOD decreased in both septic tanks after 8 weeks of sludge withdrawal. However, the increase of BOD in septic tank No 1 at the last monitor may effected by influent and because at that monitoring time COD increased too.

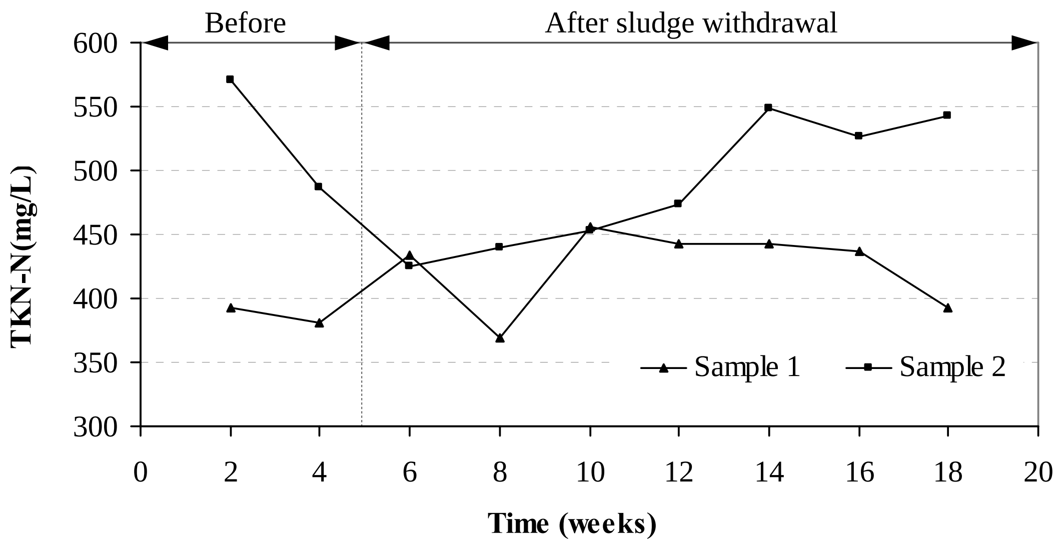


Figure 4.15 TKN varies in withdrawn sludge septic tanks

Figure 4.15 presents the variation of TKN in two sludge withdrawn septic tanks. After sludge withdrawal, TKN concentration in both septic tanks was the same. Its values in the septic tank No 1 were slightly different during monitor time, while values were increased with time in septic tank No 2. At the last monitoring time, TKN in two septic tanks became more different. In septic tank No 1, it started decreasing after long time steady. Nevertheless, in septic tank No 2, it became stable after 8 continuously weeks increase.

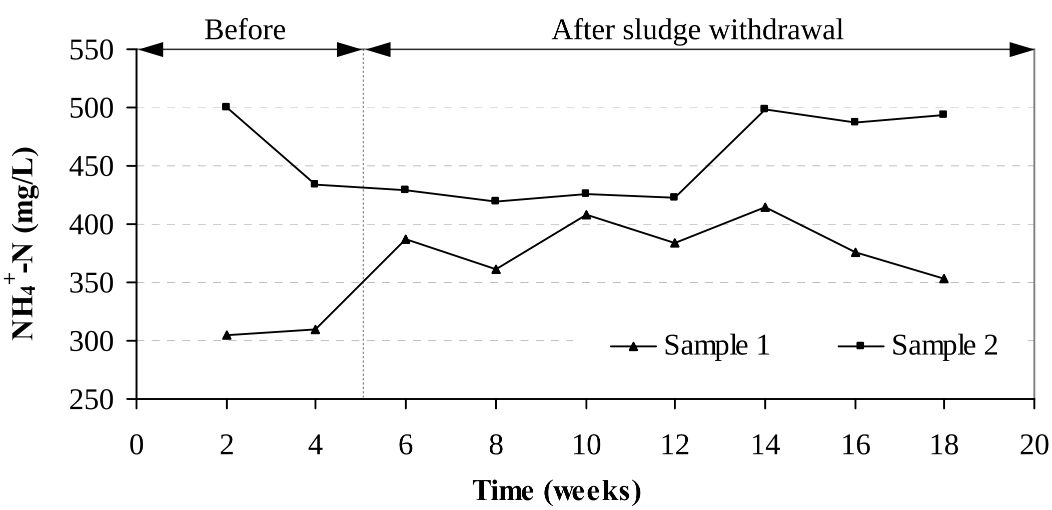


Figure 4.16 Ammonia varies in withdrawn sludge septic tanks

Ammonia in both septic tanks is slightly different as shown in Figure 4.16. After sludge withdrawal, ammonia was around 400 mg/L. However, 8 weeks afterward, ammonia increased in the septic tank No 2, but in septic tank No 1, its value is steady. This can be explained that after sludge withdrawal TKN was accumulated in septic tank. The anaerobic degradation slowly started inside septic tank and after 8 weeks, the degradation achieved to steady state. At that time, most of TKN was converted to ammonia that made ammonia concentration in septic tank increase. TKN and ammonia reached steady state after 10 weeks sludge withdrawal. For the septic tank in location No 1, ammonia and TKN changed not similar to septic tank in location No 2. The reason can be expressed by influence of influent quality.

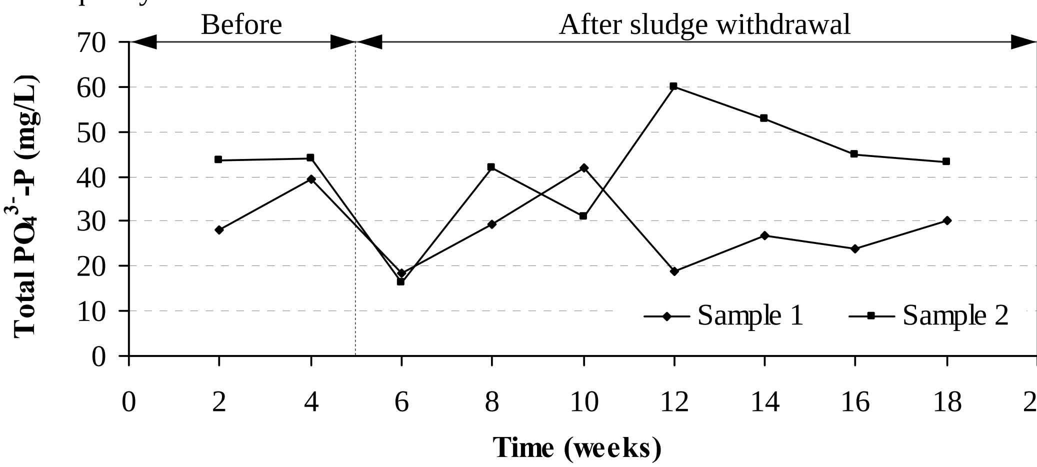


Figure 4.17 Variation of total Phosphate before and after sludge withdrawal

After sludge withdrawal, phosphate in two septic tanks was low, it lower than 20 mg/L. However, it started increasing in 4 weeks afterward. The results are showed in Figure 4.17. After 8 weeks, phosphate in both septic came to steady state. For phosphate as mentioned above, it is very difficult to estimate due to flushing water content phosphate detergent.

Based on the results described from Figure 4.11 to 4.17 it concludes that the results of pH, COD, BOD, SS, NH_4^+, TKN and Total phosphate changed after sludge withdraw comparing with previous one. It took about 8 weeks to return to normal condition as previous case. However, the effects to the change include many factors such as environmental conditions, influent quality of septic tank. In some case, the variation of quality of septic tank did not follow any trends, it makes more difficult to evaluate based on one factor.

4.3 Filtration test

This test measured a series of parameters to evaluate the influence of particulates in supernatants on filtration process such as particle size distribution, capillary suction time etc. Filtration test is very important for this research because it considers on influence of particles in supernatants on filtration process. It will help to predict and explain how long the filtration process will be.

The wastewaters to be used for this test were taken from domestic wastewater in AIT. Effluent septic tank was taken from Vietnamese restaurant and domestic wastewater was taken from physical plant.

The first parameter to be determined is SS. Figure 4.18 shows SS concentration of various wastewaters were used for filtration test. The SS of effluent septic tank was higher that that in domestic wastewater from AIT. The reason is domestic wastewater in AIT was diluted by wastewater from laboratories. The supernatants after settling 2, 3 and 4 hours were not much different in term of SS concentration. The SS reduced very fast at 2 initial hours and slow after wards.

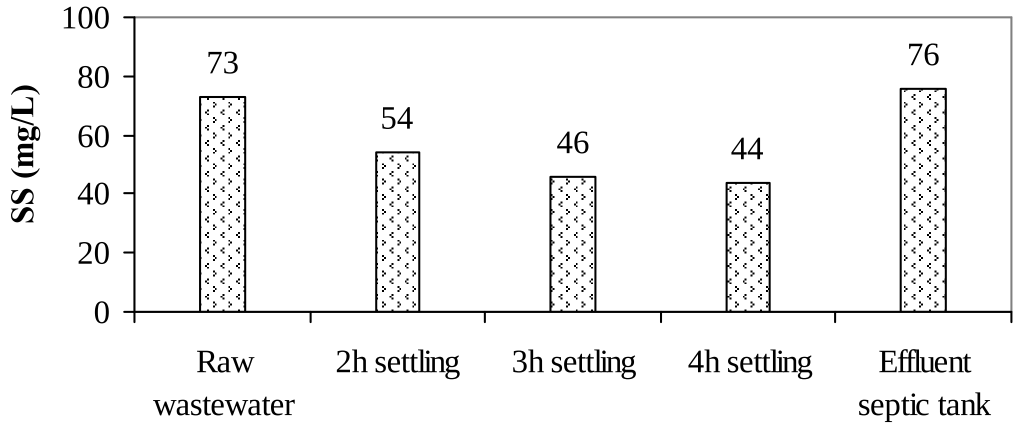


Figure 4.18 SS in various wastewaters

4.3.1 Particle size distribution

Data on Table 4.10 presents particle size distribution in various wastewaters. The results indicated that longer settling time the smaller particle size in the supernatant. Size of particles in effluent septic tank is smaller than raw wastewater but higher than wastewater after 2 hours settling time. The distribution of particles in raw domestic wastewater was the widest range than others. Its particle size was distributed in the range from 0.1 µm to 500 µm. For effluent septic tank, the distribution of particles was narrow range compare to raw domestic wastewater. The particle size of supernatant after 4 hours settling was smallest range. It distributes in the range from 0.1 µm to 120 µm. The detail information of result measurement is shown on Appendix F.

Table 4.10 Particle side distribution of various wastewaters

Sample	Mean diameter (µm)	Standard deviation (µm)	Uniformity
Raw wastewater	76.18	1.75	1.45
Wastewater after 2h settling	36.63	1.48	0.97
Wastewater after 3h settling	26.05	0.48	0.78
Wastewater after 4h settling	24.70	0.66	0.70
Effluent septic tank	52.32	1.50	1.18
Anaerobic mixed liquor	21.28	0.26	0.52
Aerobic mixed liquor	50.20	0.58	0.87

The uniformity of particles in raw wastewater was lowest. The highest uniformity was particles in supernatant after 4 hours settling. It is true because after 4 hours settling duration, almost all coarse particles settled. There are only small and light particles, which cannot settle and retained in supernatant. The measured results indicated that these particles were equivalent in term of diameter. The results also showed that particle uniformity of effluent septic tank was only higher than that of raw wastewater. The detail information regarding particle size distribution of various supernatants shows in the Table 4.10 above.

For mixed liquor, the results indicated that size of particle in anaerobic reactor was smaller than that in aerobic reactor. The distribution of particle size in anaerobic reactor was narrower than in aerobic reactor. The uniform size of particle in anaerobic reactor was also higher than that in aerobic reactor. Results showed that particle size in anaerobic reactor was smallest compare to the measured wastewater.

4.3.2 Capillary suction time

Mixed liquor of aerobic and anaerobic reactors were measured CST. The results indicated that value in aerobic reactor was higher than that in anaerobic reactors. That mean dewater ability of sludge in anaerobic reactor is lower than that in aerobic reactor. Figure 4.19 shows the measured results in these reactors. The MLSS of aerobic and anaerobic reactors were 4620 mg/L and 2830 mg/L, respectively.

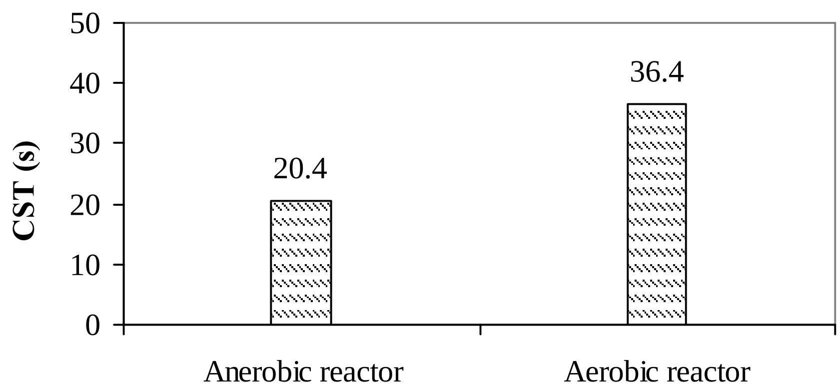


Figure 4.19 CST of mixed liquor in aerobic and anaerobic reactor

4.3.3 Membrane fouling index

The results of MFI measurement are shown in the Appendix G

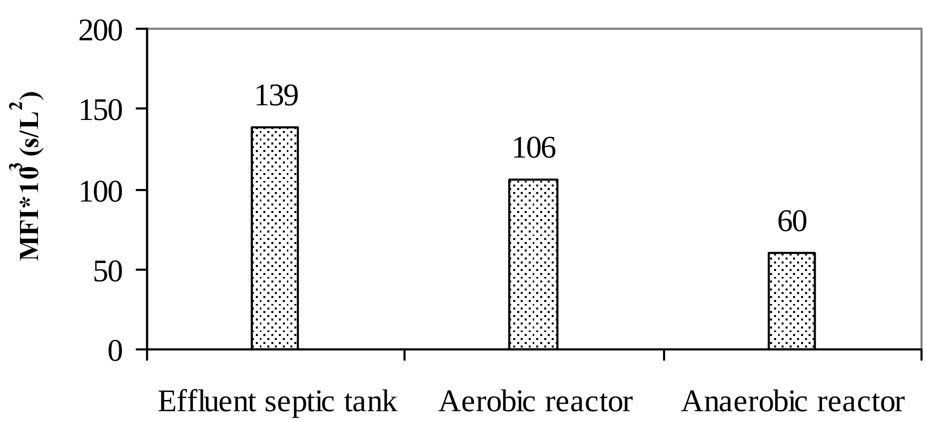


Figure 4.20 MFI of various wastewaters

Figure 4.20 presents the measured results of MFI from various wastewaters. Effluent septic tank was highest potential on membrane fouling than others. It was around 2 times higher than AIT raw wastewater. Although, its SS was equal to SS of raw wastewater. However, its particle size was higher than size of particle in reactors but lower than raw wastewater. Its MFI was $130*10^3$ s/L^2 and highest MFI value compare to measured wastewater.

For mixed liquor in two reactors, MFI of aerobic process was higher than that of anaerobic process. Its MFI was lower than wastewater from effluent septic tank. Nevertheless, mixed liquor in anaerobic was contrary. Its MFI value was lowest compare to others.

4.3.4 Specific resistance

The specific resistance values of correlative wastewater were calculated and listed in details in Appendix G. In short, specific resistance of given wastewater were presented Figure 4.21 below.

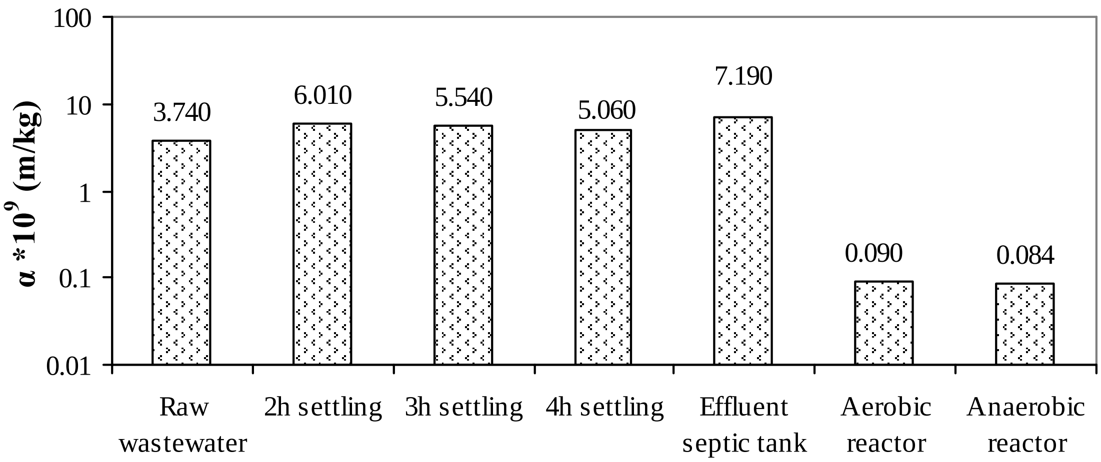


Figure 4.21 Specific resistance of various wastewaters

The specific resistance of mixed liquor in anaerobic was lowest. The specific resistance of mixed liquor in aerobic and anaerobic reactor was almost same. The highest value was effluent of septic tank. It was around 100 times higher than mixed liquor in anaerobic reactor. Specific resistance of effluent septic tank was about twofold of raw wastewater. For raw wastewater, its specific resistance was lower than settled wastewater. The values of wastewater after settling duration are close to that of effluent septic tank. The longer settling time the lower specific resistance, however, values are not much difference.

4.4 Laboratory scale experiment

In order to select suitable process that can be used for effluent septic tank treatment, an experiment with two processes were performed in ambient laboratory. The experimental results will be presented and discussed in this section.

4.4.1 Initial membrane resistance

For membrane, the initial resistance must be measured prior each run. This measurement not only uses to know the smallest difference of pressure applying on to membrane but also the effectiveness of the chemical cleaning. In this research, two membranes were used for two processes. The variations of flux were measured with change of trans-membrane. This relationship seemed linear and results presented in the Appendix H

The others resistances also measured when working with membrane. In the Appendix G, cake resistance for both processes also presented. The results were calculated through fouling resistance and total resistance.

4.4.2 Trans-membrane pressure variation

The Hg manometer was used to measure trans-membrane pressure for each run of experiment. TMP variation was recorded daily. Experiment was stopped when trans-membrane pressure reached 60 kPa (equal to 450 mm Hg). Membrane was taken out and cleaned by tap water and chemical solution. The recovery of flux after cleaning must be 80% or higher to show proper chemical cleaning. The variation of TMP during experiment in both processes is shown in the Figures 4.22 to 4.25 below.

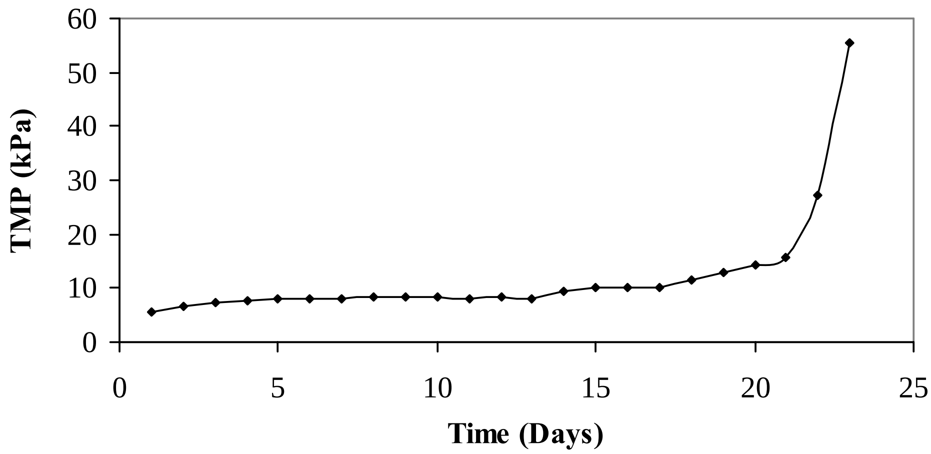


Figure 4.22 TMP variation in aerobic reactor run at HRT = 16 hours

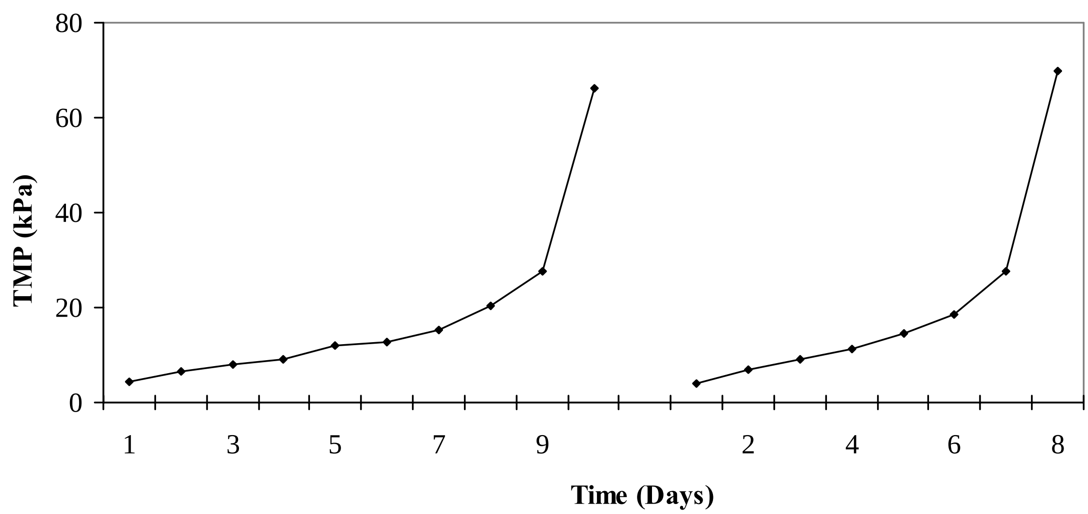


Figure 4.23 TMP variation in aerobic reactor run at HRT = 8 hours

For aerobic process, when maintaining HRT at 16 hours, TMP was very stable. System operated more than 20 day with small variation of TMP. After 22 days, TMP started increase and it was rapid increasing on the day of 23. When HRT decrease to 8 hours, TMP was less stable. It increased day by day. The results indicated that TMP increased around 1 kPa per day. However, in the first run, TMP was rapid increase after 9 days and in the second run, it was 7 days.

In short, membrane was clogged after 23 days operating when HRT of aerobic process maintenance at 16 hours. When HRT decrease to 8 hours, membrane could not operate longer than 9 days. During experimental performance, the TMP was measured at constant of flux.

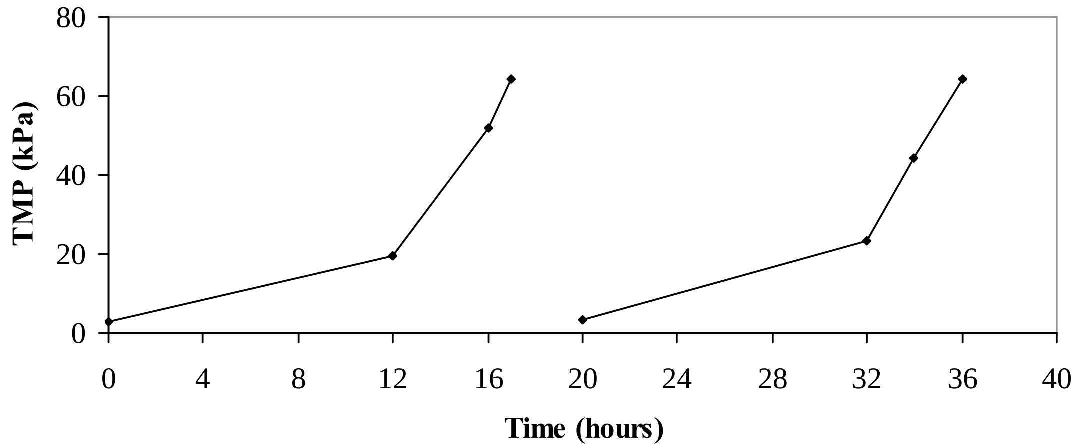


Figure 4.24 TMP variation in anaerobic reactor at HRT = 16 hours

Difference from aerobic, TMP of anaerobic process increased very fast. In the initial 12 hours, TMP increased 1-2 kPa per hour. However, TMP increased to 60 kPa within 3 hours afterward. Membrane was clogged after only 15 to 17 hours of operation. Figure 4.25 shows the variation of TMP when HRT maintenance at 16 hours.

When decrease HRT to 8 hours. The increase of TMP was faster. The results showed that TMP continuously increase during reactor run. Especially, it was rapid increase after 4 hours run. Membrane was clogged after 6 hours operation.

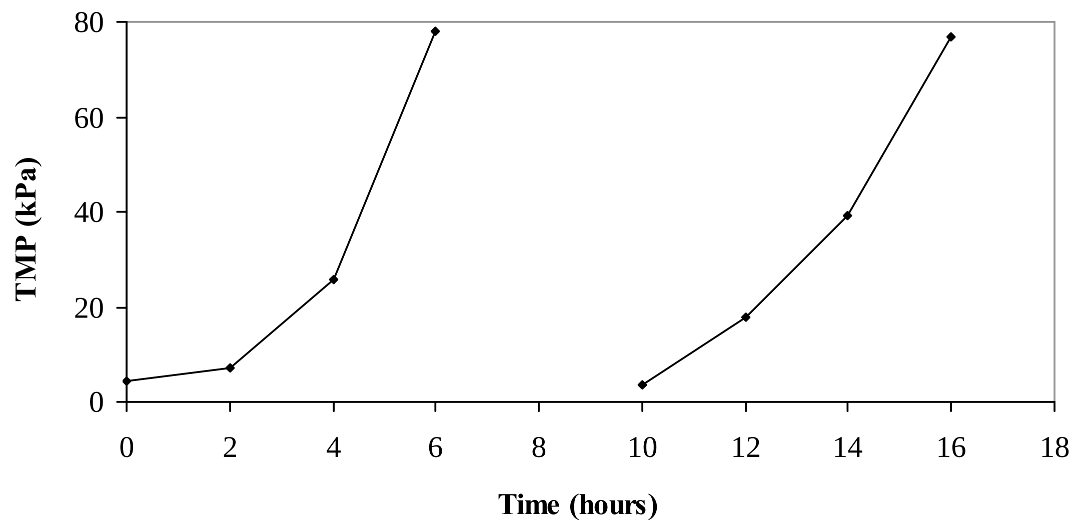


Figure 4.25 TMP variation in anaerobic reactor at HRT = 8 hours

4.4.3 Sludge and hydraulic retention time

In order to get efficiency when using actual wastewater from effluent septic tank for membrane bioreactor system, high sludge age was used for these reactor run. SRT was kept at 30 days for both reactors (Adam et al., 2002; Kraume et al., 2005). Based on volume of reactor, volume of sludge was calculated to discharge daily. Amount sludge daily discharge was 0.37L and 0.39L for aerobic and anaerobic reactor, respectively.

Two various values of HRT were performed for both processes, that is 16h and 8h. The flux was calculated and pump was controlled to suck a desired of flux. Flux was adjusted correlative with above HRT, for aerobic reactor at 14.5 mL/min and 28 mL/min, for anaerobic at 15 mL/min and 29 mL/min, respectively. Timer controlling was set at 8 minutes on and 2 minutes off in cycle.

4.4.4 MLSS variation

The processes used synthetic wastewater to start up. After 10 days, actual wastewater from effluent septic tank was used to replace for synthetic wastewater. Due to wastewater low in BOD and COD, MLSS for HRT = 16 hours was stared at low concentration in order to get sufficient of F/M (Metcalf & Eddy, 2003; Henze et al, 2002). The variation of MLSS and MLVSS showed in the Figure 4.26. MLSS was reduced in the first week and it was continuous a little reduced in the two weeks afterwards. However, after 5 weeks, it was rapid reduced and membrane was clogged at that time. The reason of reducing MLSS can be insufficient supply of food for microorganism to growth. This can be endogenous respiratory phenomenon due to long HRT maintenance in the process.

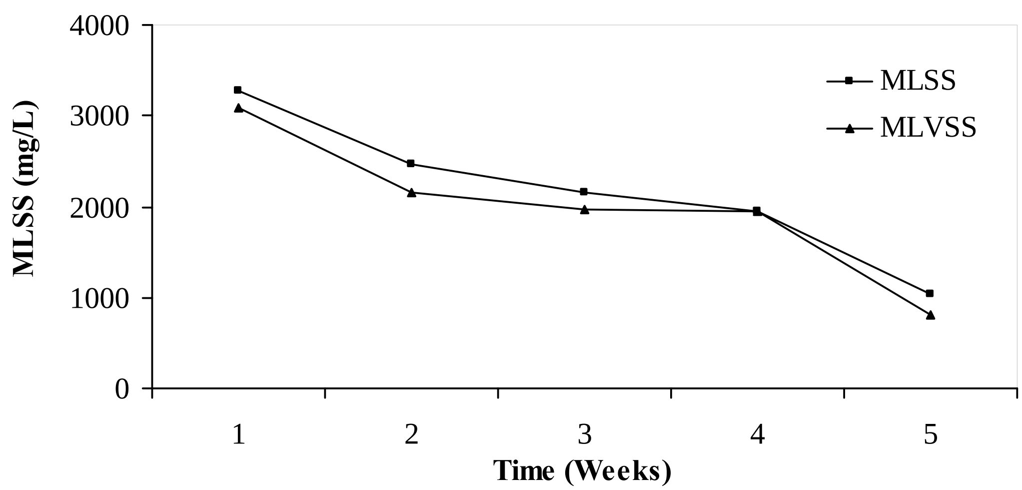


Figure 4.26 MLSS variation in aerobic reactor at HRT = 16 hours

In order to solve problem of reducing MLSS, HRT was reduced to 8 hours. However, to get removal efficiency of process, MLSS was increased from 3000 mg/L to 6000 mg/L. Although, HRT reduced, MLSS still reduced with running time. Membrane was clogged very fast. Due to short running process, MLSS was measured at frequency once per 3 days. Figure 4.27 shows the variation of MLSS in aerobic process when HRT maintenance at 8 hours.

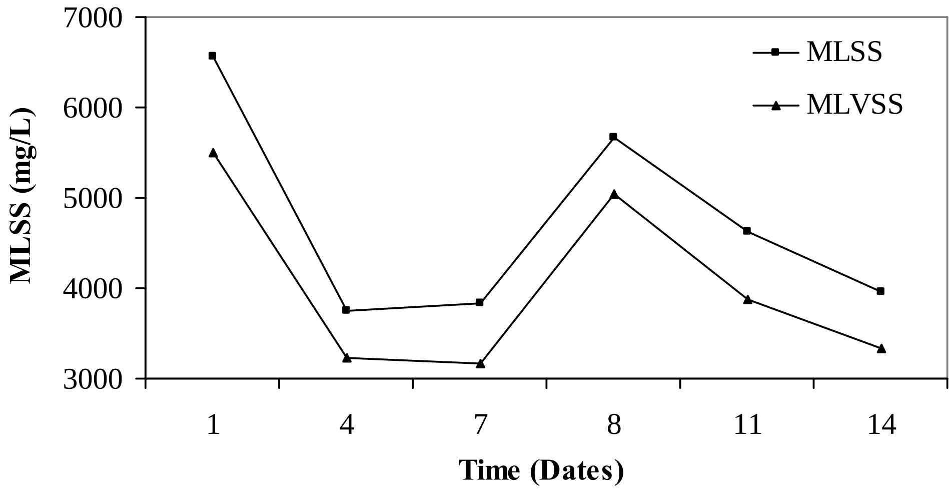


Figure 4.27 MLSS variation in aerobic reactor at HRT = 8 hours

For anaerobic process, membrane was rapid clogged, so MLSS only measured at starting and finishing time. The variation of MLSS and MLVSS in anaerobic reactor was listed in the Table 4.11 below.

Table 4.11 Variation of MLSS and MLVSS in anaerobic reactor

Sample	HRT	MLSS	MLVSS
start	16	3,130	2,960
end		1,860	1,820
start	16	3,260	3,130
end		1,980	1,920
start	8	3,020	2,950
end		2,070	1,960
start	8	2,840	2,740
end		1,950	1,870

The results indicated that MLSS was reduced during reactor run. The observed results indicated that a portion of sludge attached on the wall of reactor and a portion was floated, that led MLSS decrease in experiment. This phenomenon was also occurred in aerobic reactor. In aerobic reactor, air used to aerate, bubble moved up and it made a thin foam layer on the surface of reactor. A portion of sludge was brought by air bubble and attached on the wall of reactor when bubble exploded. This portion was decay due to not contact to the food source for long time. That could be the other reason of activated sludge decrease in these processes beside the insufficient of F/M.

4.4.5 pH and oxygen variation

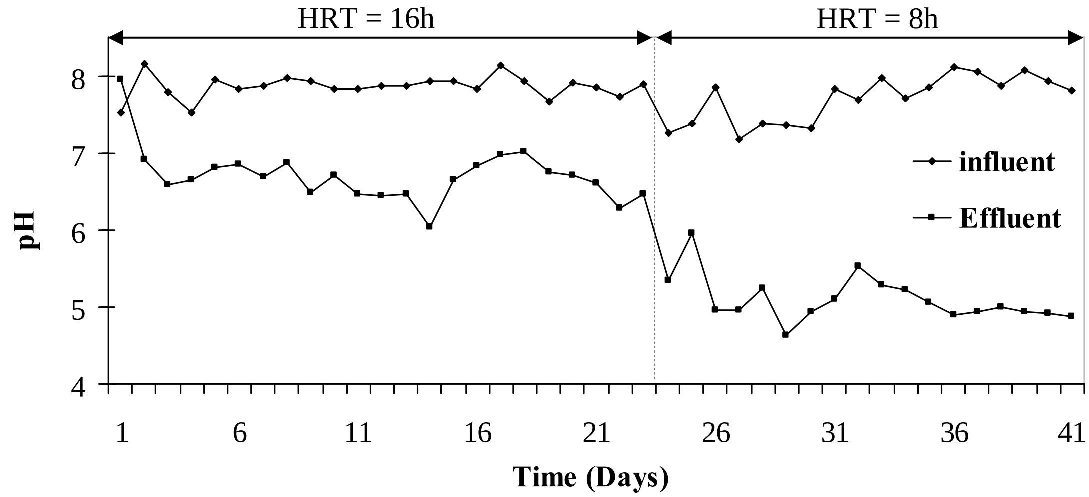


Figure 4.28 pH variation in aerobic process

The variation of pH in aerobic reactor shows in Figure 4.28. pH of effluent was lower than influent. Due to oxidation of organic matters and nitrification, that led pH decreasing in reactor and effluent (Henze et al., 2002). When loading rate increased (HRT decrease to 8h), the longer nitrification more pH decrease. It is very clearly in the Figure 4.29, effluent pH at HRT = 8h was lower than that of HRT = 16h.

For dissolved oxygen, in aerobic process, airflow was maintained at 1L/Liter of reactor per minute. The DO in reactor varied from 6 mg/L to 7 mg/L. When MLSS increase, HRT reduced and airflow was still kept at 1L/Liter of reactor per minute. The DO was reduced and it varied from 5 mg/L to 6 mg/L. Because media was used for this process, air flow was kept at this flow to prevent media settling down. If airflow was not sufficient, the penetration of oxygen will be reduced and some parts in reactor will be anaerobic condition. The DO also very sensitive with ambient temperature, so it varied in reactor is not abnormal.

For anaerobic process, it was very difficult to observe the variation of pH and DO inside reactor due to short running time. The COD and other parameters were only measured at beginning and finish of reactor run. The analyze results are presented in the part water quality after treatment and Tables 4.10 below.

4.4.6 Organic matter removal

The wastewater used for experiment was effluent septic tank. The COD varied between 150 mg/L and 350 mg/L. At the beginning of first run, effluent COD was quiet high when COD influent increase. However, removal efficiency still got higher 60%. The removal efficiency increased with time and it was around 80%. Effluent COD achieved a steady value with 30 mg/L after 10 days. When HRT reduced to 8h, effluent COD still stable at 30 mg/L and removal efficiency always higher than 70%, although influent quality varied. The Figure 4.29 depicts the variation of COD in aerobic process.

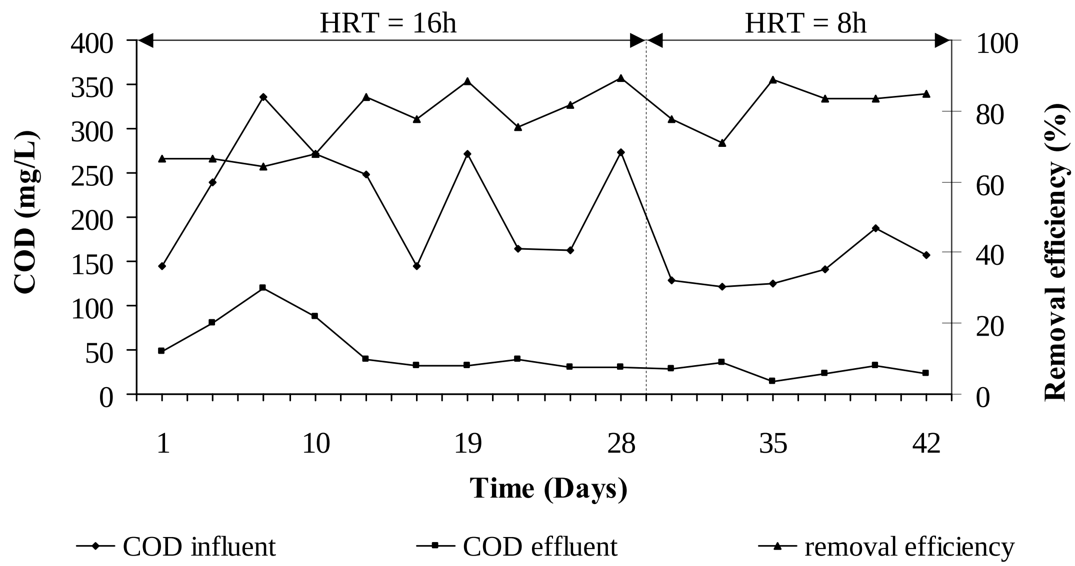


Figure 4.29 COD removal in aerobic process

4.4.7 Nitrogen removal

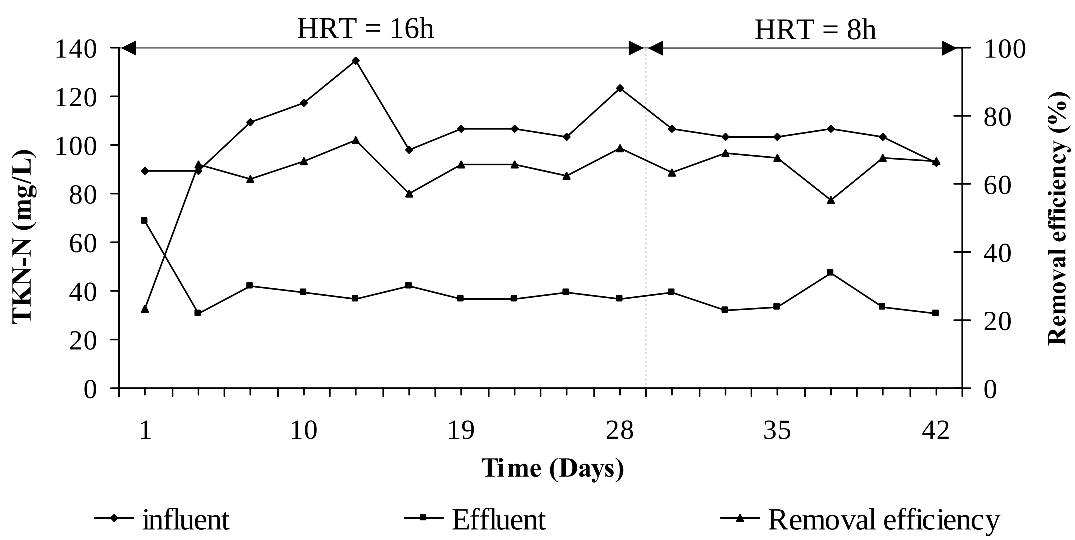


Figure 4.30 TKN variations and removal efficiency in aerobic process

Figure 4.30 and 4.31 showed TKN and ammonia removal in aerobic process. In the beginning, removal efficiency was not much, both TKN and ammonia in the effluent was high. However, after 4 days of first run, removal efficiency increased clearly. TKN and ammonia in influent varied from 80 mg/L to 130 mg/L and from 80 mg/L to 100 mg/L, respectively. The results indicated that concentration of ammonia and TKN in the effluent was equal, that mean there was no organic nitrogen in the effluent. The TKN and ammonia in the effluent were around 40 mg/L. The removal efficiency of ammonia was stable at 60% for both HRT (at 16h and 8h). The removal efficiency varies between 50% and 70 %. The change of HRT was not effect on removal efficiency for both TKN and ammonia.

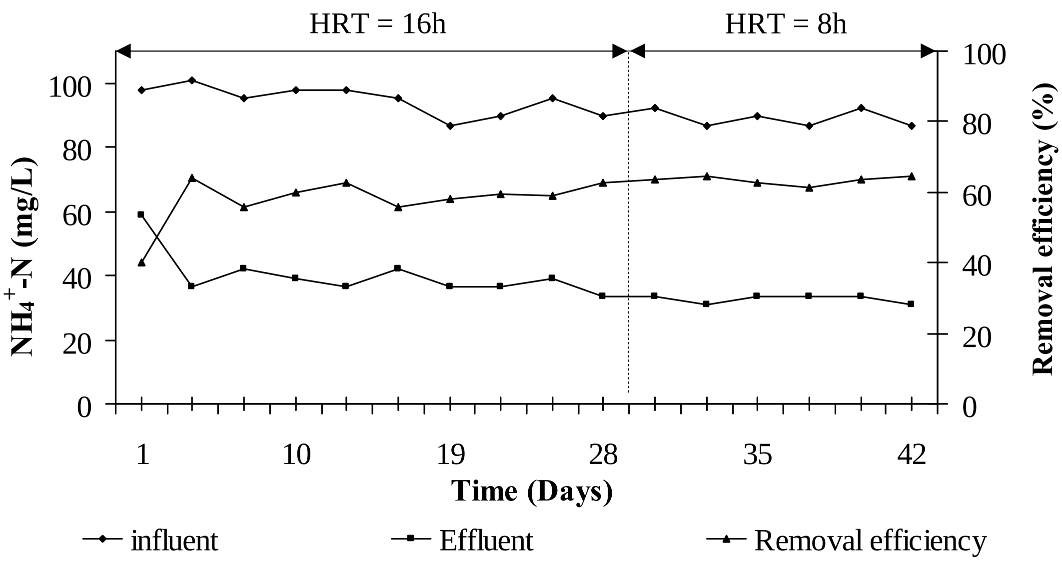


Figure 4.31 Ammonia variations and removal efficiency in aerobic process

4.4.8 Phosphate removal

The Figure 4.32 shows the phosphate variation in aerobic process. The results showed that phosphate was not removal in aerobic process. Phosphate in the effluent was not stable. For HRT at 16h, the long HRT made the lack of food and endogenous respiratory phenomenon was occurred. Phosphate in effluent was higher than influent at some points. For HRT = 8h, Phosphate concentration in effluent and influent was almost equal. There was not any phosphates is removal.

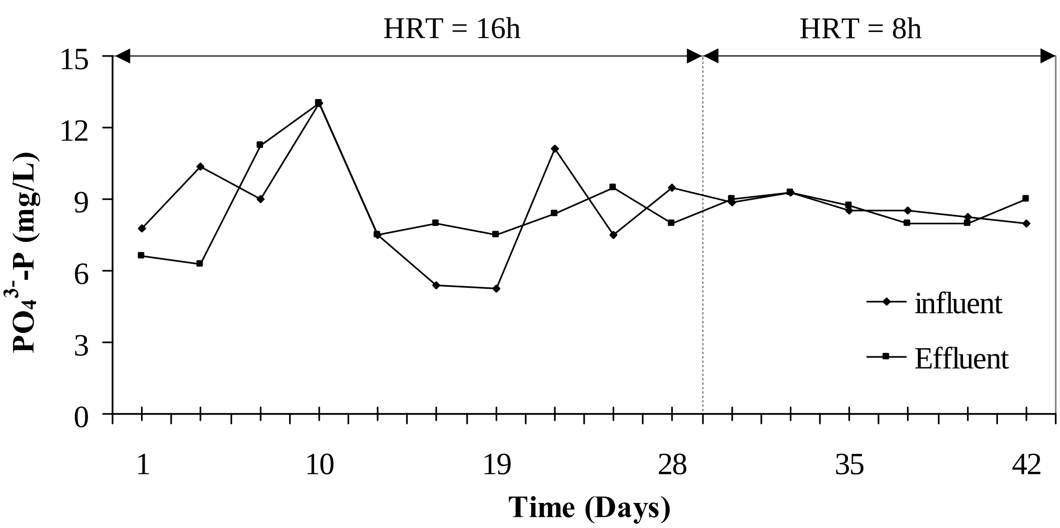


Figure 4.32 Phosphate variations in aerobic process

4.4.9 Water quality after treatment

The Table 4.12 shows the wastewater quality in anaerobic process at different HRT. The results indicated that there is no removal of TKN, ammonia and phosphate. The concentration of these parameters in the influent and in the effluent was not different. For

COD, the difference was observed in the influent and effluent. This amount of COD could be contributed by SS, due to membrane was used to separate solid from effluent.

Table 4.12 Wastewater quality in anaerobic process

Parameters	HRT = 16 hours		HRT = 8 hours	
	Influent	Effluent	Influent	Effluent
pH	7.8 – 7.9	7.4 – 7.8	7.4 – 7.6	7.4 – 7.5
COD, mg/L	180	129	197 -228	98 - 114
TKN, mg/L	126	126	131	123
NH_4^+, mg/L	120	120	120 – 123	117 – 120
PO_4^{3-}, mg/L	9.9	10.2	9.0 – 10.0	9.5 – 10.0

Table 4.13 Water quality after treatment at different HRT

Parameters	HRT = 16 hours		HRT = 8 hours		Appropriate range*
	Aerobic	Anaerobic	Aerobic	Anaerobic	
pH	6.0 – 7.0	7.4 – 7.8	4.6 – 5.9	7.4 – 7.5	-
COD, mg/L	29 – 40	129	13 – 35	98 - 114	ND
TKN, mg/L	33 – 42	126	30 – 47	123	10 - 30
NH_4^+, mg/L	30 – 42	120	30 – 33	117 – 120	ND
PO_4^{3-}, mg/L	6.2 – 13.0	10.2	8.0 – 9.0	9.5 – 10.0	0.1 - 20

*Reference: Asano (1998).

The results in the Table 4.13 show wastewater quality after treatment. This Table also presents the reference values that are quality of treated wastewater reuse for irrigation or gardening. At HRT = 16 hours, the effluent of aerobic is acceptable with TKN in the effluent a little higher than reference. The similar results also achieved with HRT = 8h.

However, wastewater quality after treatment in anaerobic process at both HRT = 16h and 8h was not good. It was almost high in all parameters. This wastewater could not meet the values in the reference and it cannot reuse or discharge to environment. You et al (2005) affirmed that anaerobic could not stand alone because its effluent does not meet required discharged standards.

Chapter 5

Conclusion and recommendations

5.1 Conclusion

The environmental pollution in Hanoi is very serious. Almost all cannels, lakes are polluted by untreated wastewater. Due to rapid urbanization in Hanoi and over load of sewer system, Hanoi needs an appropriately wastewater treatment. The application of decentralized treatment is very suitable in Hanoi, where new urbanization areas have been building for expanding of city.

The number of new urban area has been increasing and quantity of septic tank is increasing with number of building. In order to reduce the pollution of environment that occurred in the old section of city, ministry of construction promulgated the standardize septic tank. The aim is reduced pollutant concentration from domestic wastewater by improved removal efficiency of septic tank. Nowadays, this standard is applied for all new buildings.

Monitoring of septic tanks in suburban area of Bangkok was carried out for 6 months duration. Total 10 septic tanks were sampled and analyzed. Type of these septic tanks is circular with single compartment. The analyzed results indicated that wastewater quality of these septic tanks do not meet required discharged standard. Almost all tanks were high in term of SS, COD, TKN and ammonia.

It was found that the variation of pH in septic tanks within range of 7.1 to 8.2 and DO was lower than 1 mg/L in all samples. SS values changed from 24 mg/L to 720 mg/L, high SS concentration was found in almost all septic tanks. COD values distributed in the wider range, from 80 mg/L to 3500 mg/L. It was found that about 85% of total samples have COD value higher than 350 mg/L. BOD values were very low compare to COD values, while these values are high in term of wastewater quality. Highest BOD value was 675 mg/L and most of BOD values are higher than 100 mg/L and lower than 250 mg/L. The low ratio of BOD/COD makes more difficult for biological treatment. The concentration of TKN and ammonia were very high. Most of TKN and ammonia are higher than 200 mg/L and 150 mg/L, respectively. Total phosphate varies in the range of 10 mg/L to 70 mg/L and most of values are higher than 20 mg/L.

The sludge withdrawal was carried out in two septic tanks. After sludge withdrawal, pH increased around 0.2 to 0.5, while SS was decreased. 6 weeks afterward, SS achieved a quite varying value. Nevertheless, COD was not different after sludge withdrawal, it started to decrease 8 weeks afterward.

The monitoring was carried out at two modes of operation of septic tank and the monitoring results indicated that wastewater quality did not meet required effluent standard. These septic tanks were failed in term of design and treatment efficiency. The application of this type of septic tank to treating of human excreta was not achieved desire effects.

The filter test was performed with various wastewaters. The particle size distribution of different wastewater was measured. The results indicated that particle in anaerobic mixed liquor has smallest mean diameter, it is only 21.28 μm. Effluent septic tank and aerobic mixed liquor have same mean diameter, while its particle size is higher than that in supernatant of wastewater after settling duration. In regards to membrane fouling index, the highest potential was found in effluent septic tank while value for raw wastewater is half of that in effluent septic tank.

The experimental investigations were contacted at anaerobic and aerobic MBR process. It was found that membrane was rapid clogged on anaerobic process. Membrane was clogged after 16h and 6h at HRT as 16h and 8h, respectively. Longer time of experimental performance was found in aerobic process. Membrane was clogged after 10 and 25 days at HRT as 8h and 16h, respectively.

There was no removal efficiency in anaerobic process. TKN, ammonia and total phosphate were not different after treatment. There was about 50% of COD removal, this could be COD distributed by suspended solids.

The removal efficiency was higher in aerobic process. COD removal efficiency was more than 60% at HRT as 16h and it increased to higher 80% at HRT as 8h. TKN and ammonia was removal higher than 60% in aerobic process. However, there was no removal of total phosphate in this process.

Effluent quality of aerobic process was compared with values in reference to find the suitability of wastewater for reuse in selected options. Consequently, effluent from aerobic process had more potential to reuse in agriculture and unrestricted purposes.

5.2 Recommendations for further studies

Planning monitors the effluent quality of other types of septic tanks such as two or more compartments and rectangular septic tanks. There are scarce of documents regarding septic tank effluent.

In order to know the variation of effluent septic tanks, the long term monitoring should be carried out. The monitoring variation of effluent quality within a day should be performed with two or more compartments septic tanks.

Membrane fouling was problem in experiments. The other technology should be mentioned in order to solve fouling problem in experiment such as increase volume of media use or air distribution at membrane compartment.

The treatment systems afterward should mention on treatment of wastewater high in term of SS, COD, TKN and ammonia but low of BOD.

The combination of nitrification and denitrification in a system should be concerned to improve removal efficiency of nitrogen compounds and COD in effluent septic tank.

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Appendices

Appendix A

Design of reactors for experiment

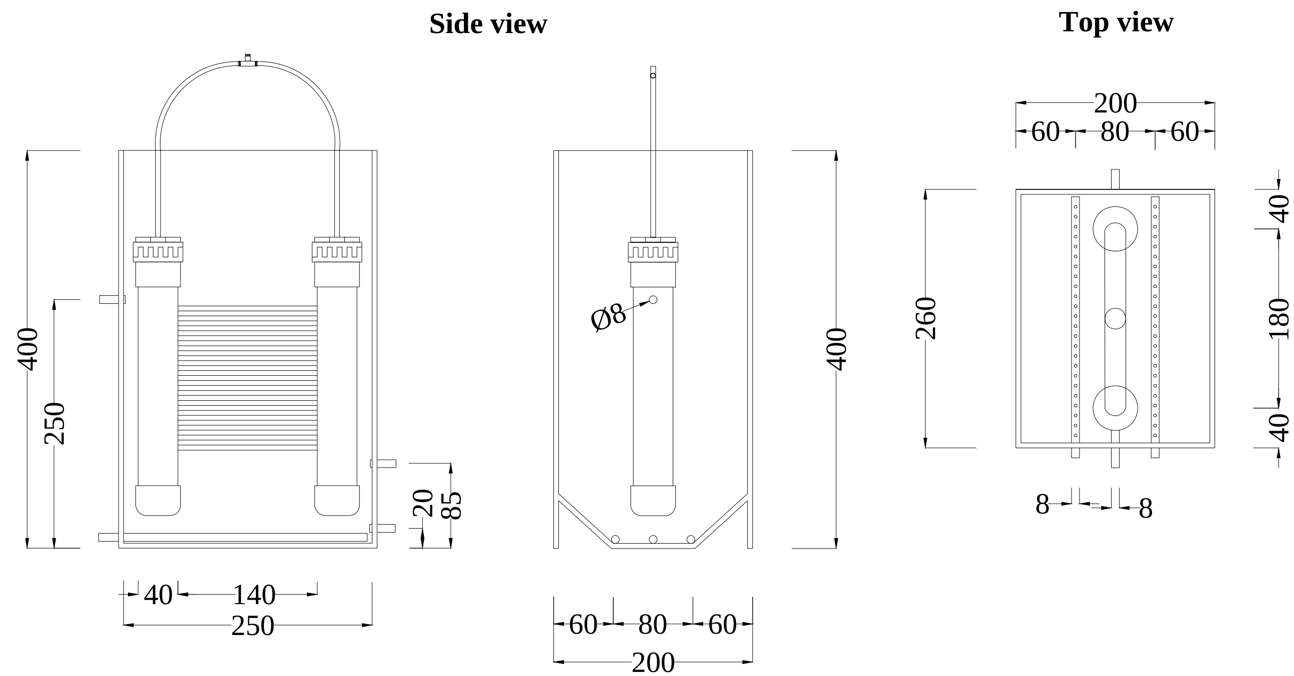


Figure A-1 Dimensions of anaerobic reactor
(All dimensions are millimetre)

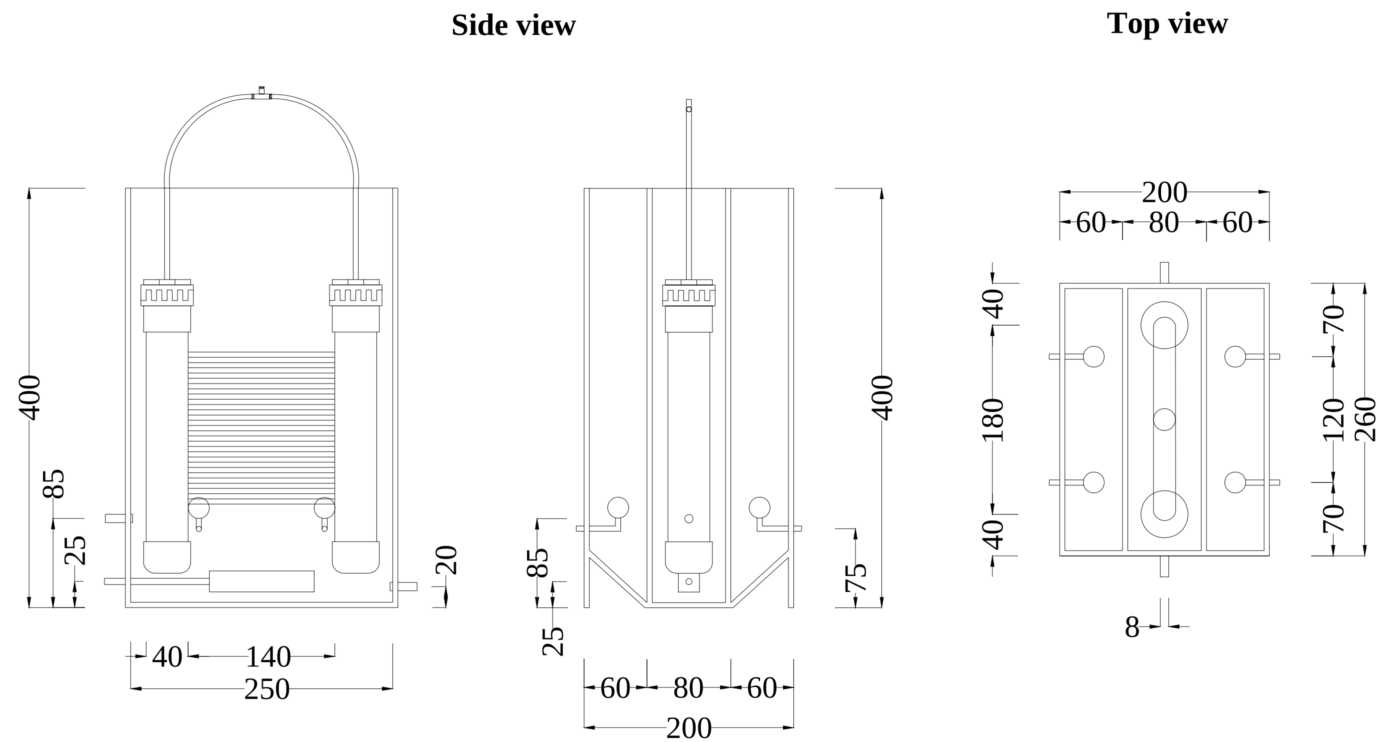


Figure A-2 Dimensions of aerobic reactor
(All dimensions are millimetre)

Appendix B

Design of 2 compartments septic tank in Hanoi

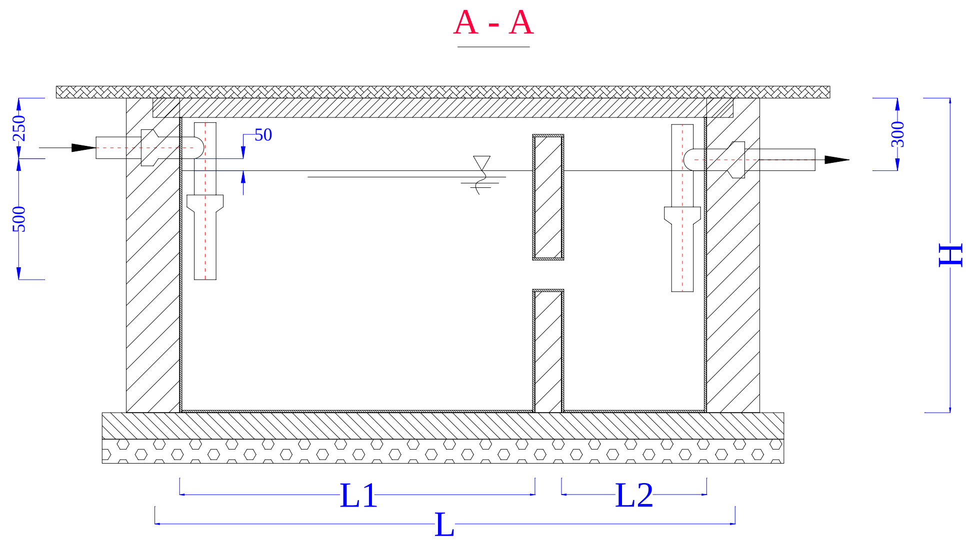


Figure B – 1 Side view of 2 compartments septic tank

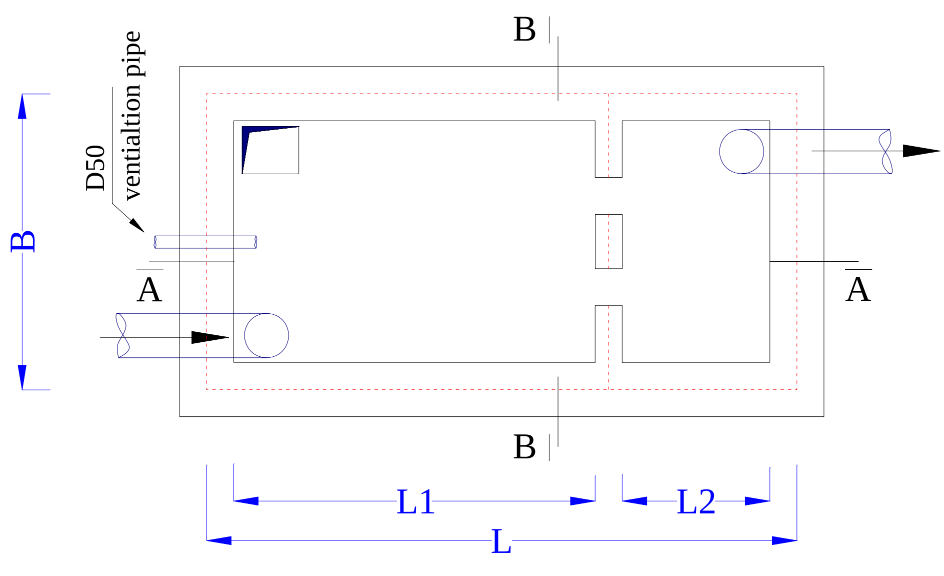


Figure B – 1 Top view of 2 compartments septic tank

B - B

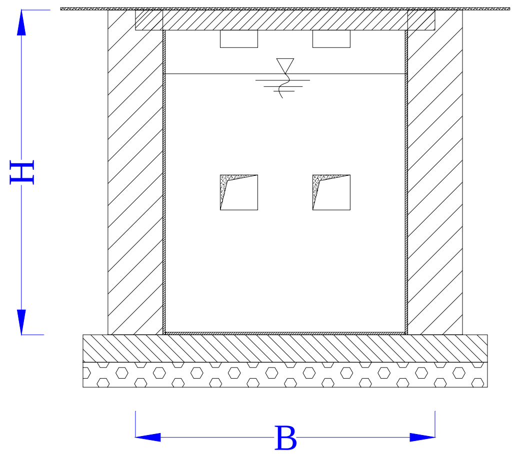


Figure B – 1 Rear view of 2 compartments septic tank

Table B – 1 Dimensions of septic tanks

No	**Type** (m^3)	**B** (m)	**H** (m)	**L** (m)	**L1: L2** (m)	**Working volume** (m^3)
1	1.5	1.2	1.3	1.8	0.87 : 0.6	1.44
2	2.0	1.2	1.3	2.4	1.47 : 0.6	2.02
3	3.0	1.2	1.3	3.5	2.37 : 0.9	3.20
4	3.0	1.8	1.3	2.4	1.47 : 0.6	3.27
5	5.0	1.8	1.3	3.6	2.37 : 0.9	5.16
6	7.5	1.8	1.3	4.8	3.26 : 1.1	6.9
7	5.0	2.4	1.3	3.0	1.97 : 0.7	5.23
8	7.5	2.4	1.3	4.2	2.76 : 1.0	7.37
9	10.0	2.4	1.3	5.4	3.66 : 1.3	9.72
10	7.5	3.6	1.3	3.0	1.97 : 0.7	8.43
11	10.0	3.6	1.3	3.6	2.47 : 0.8	10.3
12	15.0	3.6	1.3	4.8	3.26 : 1.1	13.8
13	20.0	3.6	1.3	6.6	4.56 : 1.6	19.5

Appendix C

Map to access to monitoring area in Thailand

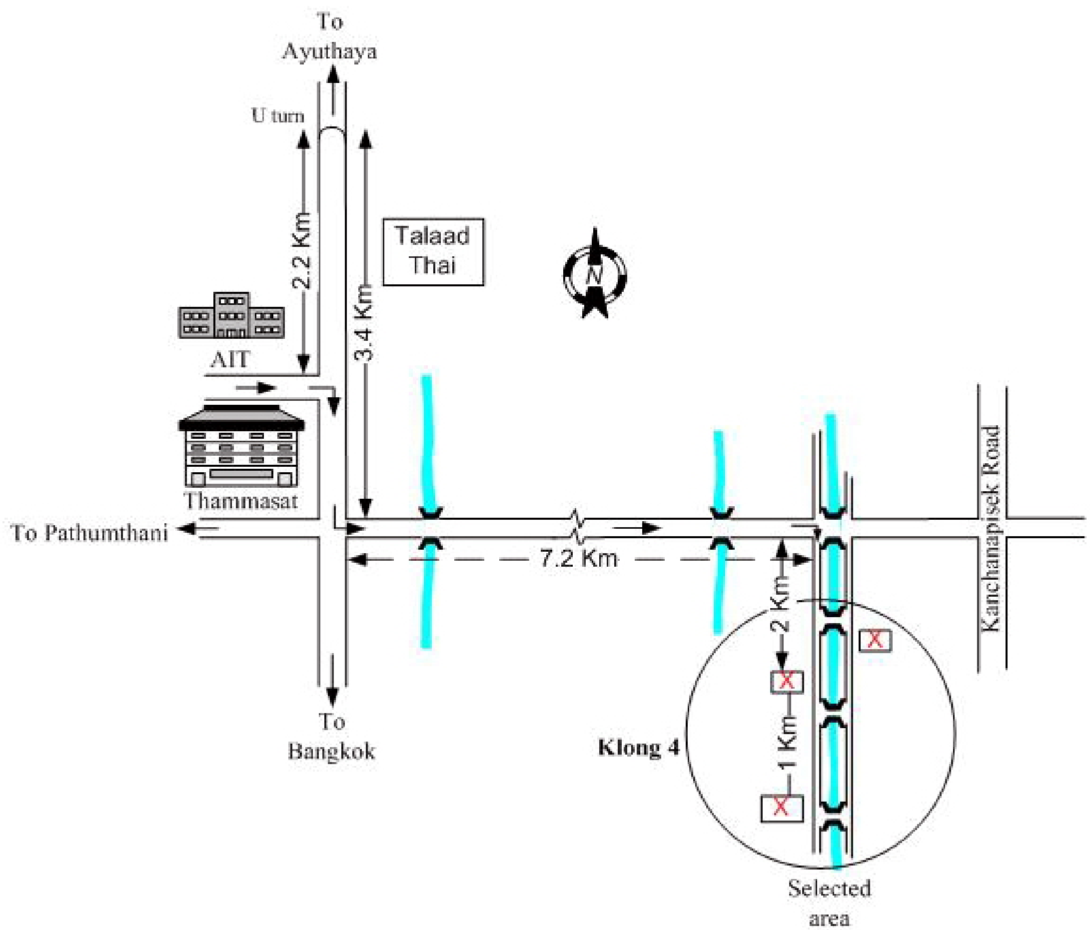
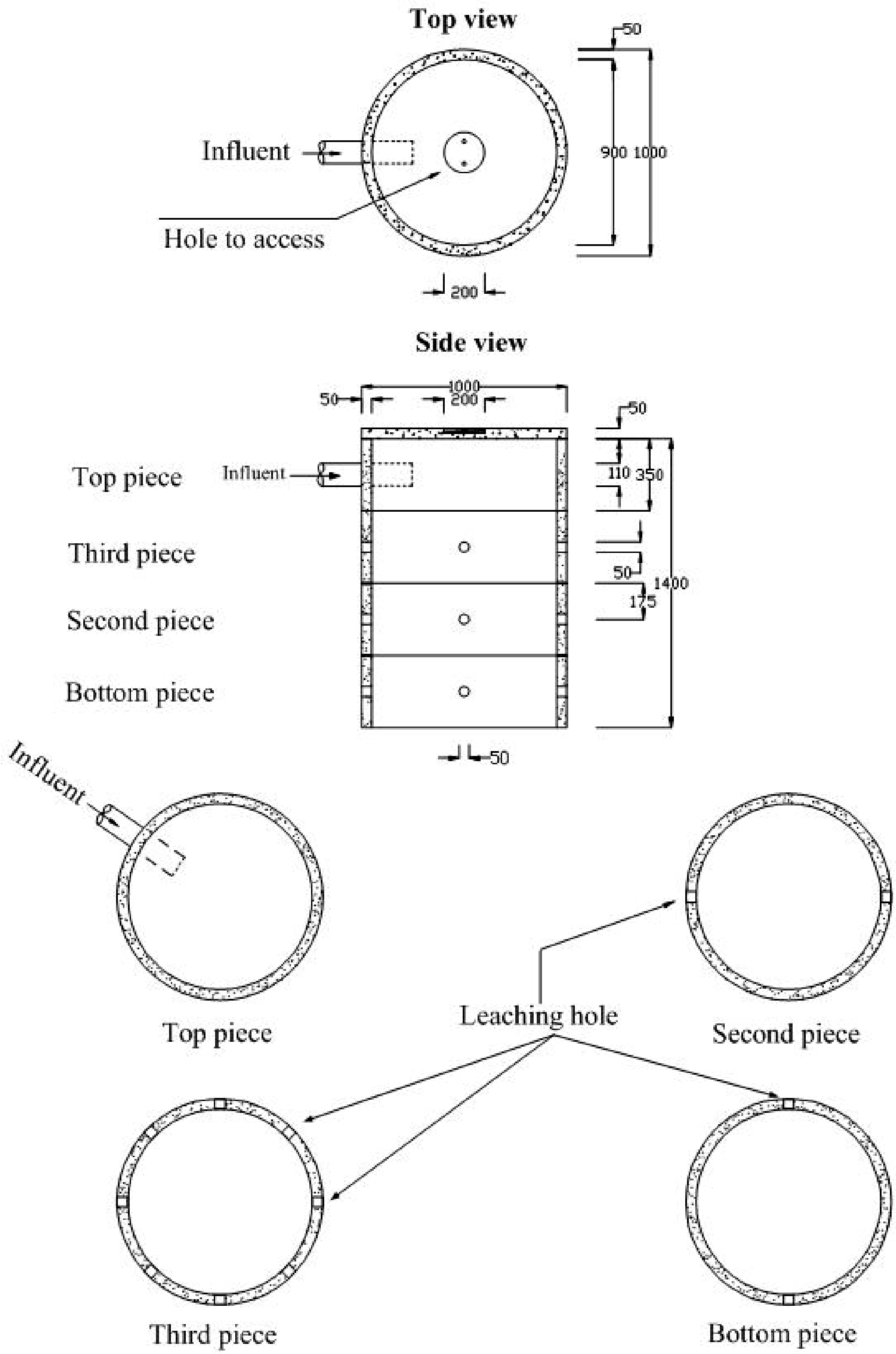


Figure C-1 Map to access to selected monitor area in Klong Luang

Appendix D

Design of septic tank in monitor area in Thailand



Note: All dimensions are millimetre

Appendix E

The monitoring results

Table E-1 Wastewater quality in septic tanks at normal operational mode

Sapling time	Sample location	pH	DO	SS	COD	BOD	NH_4-N	TKN-N	PO_4-P
Oct	1	7.79		38	288		101	111	11.0
	2	7.70		530	3,440		564	610	41.5
	3	7.44		260	800		686	745	44.8
	4	7.70		76	416		438	454	32.0
	5	7.10		220	736		185	213	18.5
	6	7.61		207	704		377	393	29.0
	7	7.59		100	416		370	370	21.5
	8	7.20		273	672		419	437	37.5
Nov	1	7.50	0.47	24	80	28	35	39	15.0
	2	7.57	0.06	360	1,600	270	466	532	39.5
	3	7.25	0.35	380	1,360	84	637	700	58.5
	4	7.64	0.47	40	360	30	234	241	20.0
	5	7.69	0.40	120	920	104	181	213	20.0
	6	7.75	0.45	105	480	35	426	426	36.5
	7	7.34	0.44	471	2,160	675	525	560	47.3
	8	7.18	0.44	290	1,080	90	472	476	48.8
Dec	1	7.85	0.25	72	352	86	138	154	14.0
	2	7.69	0.31	420	2,080	525	568	650	34.0
	3	7.38	0.30	290	1,040	78	543	591	47.0
	4	7.65	0.24	67	680	54	445	479	16.0
	5	7.30	0.19	170	760	240	152	179	14.0
	6	7.78	0.25	195	560	84	498	529	15.0
	7	7.30	0.27	220	1,600	258	456	521	52.0
	8	7.29	0.22	440	1,360	81	563	608	58.0
Jan	1	7.88	0.08	56	320	80	118	126	13.0
	2	7.63	0.22	720	2,880	354	728	843	47.0
	3	7.28	0.10	480	1,280	147	622	644	64.0
	4	7.76	0.18	175	560	75	602	619	36.0
	5	7.23	0.14	340	1,040	114	238	266	30.0
	6	7.78	0.21	150	640	141	535	580	35.0
	7	7.33	0.40	270	1,280	264	440	482	74.0
	8	7.25	0.52	190	800	126	465	498	76.0

Date	Sample location	pH	DO	SS	COD	BOD	NH_4-N	TKN-N	PO_4-P
Feb	1	7.97	0.13	72	221	98	106	118	18.0
	2	7.60	0.12	530	1,398	306	577	739	48.0
	3	7.58	0.12	470	1,178	204	666	818	54.0
	4	7.82	0.19	68	368	72	638	700	42.0
	5	7.38	0.08	210	736	183	235	274	27.0
	6	8.25	0.06	128	368	75	465	470	28.0
	7	7.51	0.08	310	957	270	442	510	65.0
	8	7.47	0.07	260	957	159	616	683	67.0
Mar	1	7.90	0.00	60	320	89	137	157	14.0
	2	7.41	0.09	370	1,120	135	549	650	53.0
	3	7.25	0.09	290	720	102	700	795	70.0
	4	7.66	0.07	60	400	71	655	778	42.0
	5	7.35	0.09	160	640	129	207	269	21.0
	6	8.04	0.07	175	480	108	498	498	24.0
	7	7.38	0.08	353	640	237	398	465	61.0
	8	7.20	0.07	570	640	138	588	622	69.0

All value are mg/L accept pH

Table E-2 Wastewater quality in septic tanks at sludge withdrawal mode

Sampling time	Sample location	pH	SS	COD	BOD	NH_4-N	TKN-N	PO_4-P
1	1	7.21	410	2,480		305	392	28.0
	2	7.37	680	1,120		500	571	43.5
2	1	7.57	460	1,440	384	310	381	39.5
	2	7.10	360	1,200	165	435	487	44.0
Sludge withdrawal								
3	1	7.76	360	1,960	720	386	434	18.5
	2	7.63	230	1,400	414	428	426	16.5
4	1	8.02	280	1,200	720	361	370	29.5
	2	7.72	150	1,120	414	420	440	42.0
5	1	7.84	130	1,760	810	409	456	42.0
	2	7.54	70	1,120	366	426	454	31.0
6	1	7.87	250	1,051	525	384	442	19.0
	2	7.52	200	1,351	396	423	473	60.0
7	1	7.82	165	1,030	480	414	442	27.0
	2	7.69	240	883	312	498	549	53.0
8	1	8.05	230	1,056	555	375	437	24.0
	2	7.31	270	774.4	234	487	526	45.0

Sampling time	**Sample location**	**pH**	**SS**	**COD**	**BOD**	**NH_4-N**	**TKN-N**	**PO_4-P**
9	1	7.74	210	1,200	750	353	392	30.0
	2	7.53	160	720	252	493	543	43.0

All value are mg/L accept pH

Appendix F

Results of particle size distribution measurement

ศูนย์เทคโนโลยีโลหะและวัสดุแห่งชาติ
สำนักงานพัฒนาวิทยาศาสตร์และเทคโนโลยีแห่งชาติ
114 อุทยานวิทยาศาสตร์ประเทศไทย ถนนพหลโยธิน
ตำบลคลองหนึ่ง อำเภอคลองหลวง จังหวัดปทุมธานี 12120
โทรศัพท์ : (662)564-6500 โทรสาร : (662)564-6501-5
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MTEC No. 1400/49

Report of Samples Analysis

Issued Date	:	28 March 2006
Customer	:	Nguyen Hoai Nam Asian Institute of Technology P.O. Box 4, Klong Luang, Pathumthani 12120 Tel : 0 2524 7733
Serviced by	:	Powder Characterization Laboratory, Central Laboratory, National Metal and Materials Technology Center
Date received	:	24 March 2006
Date analyzed	:	24 March 2006
Sample	:	Sludge in waste water 7 samples.
Identification no.	:	None.
Instrument	:	Mastersizer S, Malvern Instruments
Test method	:	Light scattering technique.
Conditions	:	He-Ne laser source, λ= 633 nm, Beam Length : 2.40 mm. Particle size range analysis : 0.05 – 900 μm. Small sample dispersion unit (MS1) Dispersing medium : De-ionized water Sample refractive index : 1.5259 (as default) Number of experiments : 3 Laser power : 80.0
Sample preparation	:	Put this sample into small sample dispersion unit (MS1) and analysis.
Technical Terms	:	**Obscuration :** value at particle come cover to laser beam (percent), ranging from 10 – 30%. **Residual :** on error value of analysis. This value should be less than 5%. **D [4, 3] :** mean diameter value by volume. **D [3, 2] :** mean diameter value by surface area. **D [v, 0.1] :** 10 volume percent less than or equal to a given diameter. **D [v, 0.5] :** 50 volume percent less than or equal to a given diameter, median diameter. **D [v, 0.9] :** 90 volume percent less than or equal to a given diameter. **Span :** the width of the distribution which is independent of median size (D [v, 0.5]). **Uniformity :** a measure of the absolute deviations from the median(D [v, 0.5]). **Specific S.A. :** specific surface area, calculated from density and D [3, 2] of a sample.

Samples detail :

Sample Number	Detail
1	Real domestic waste water
2	Waste water after 2h setting
3	Waste water after 3h setting
4	Waste water after 4h setting
5	Water in anaerobic reactor
6	Water in aerobic reactor
7	Effluent septic tank

Results :

The results of the particle size and size distribution of samples are shown in table 1 – 7 and the attachment No. 1 – 21.

Table 1 Mastersizer S results of Sample No.1

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	78.46	7.58	42.13	206.12	4.71
	2	78.34	7.35	40.84	204.95	4.84
	3	75.28	7.09	40.19	197.36	4.73
2	1	76.73	6.96	39.70	207.91	5.06
	2	74.10	6.77	39.04	194.58	4.81
	3	73.48	6.59	38.41	195.96	4.93
3	1	75.99	6.61	38.69	206.98	5.18
	2	77.49	6.48	38.32	210.05	5.31
	3	75.72	6.06	36.93	209.93	5.52
Mean		76.18	6.83	39.36	203.76	5.01
STD		1.75	0.47	1.55	6.11	0.28
RSD%		2.30	6.88	3.94	2.30	5.59

Table 2 Mastersizer S results of Sample No.2

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	38.15	3.59	27.10	74.95	2.63
	2	37.89	3.46	26.65	74.86	2.68
	3	38.08	3.36	26.30	74.70	2.71
2	1	37.05	3.21	25.78	72.82	2.70
	2	36.59	3.22	25.50	72.50	2.72
	3	35.50	3.08	25.07	70.28	2.68
3	1	33.72	2.91	24.50	67.14	2.62
	2	35.53	2.90	24.47	70.00	2.74
	3	37.15	2.72	24.23	72.26	2.87
Mean		36.63	3.16	25.51	72.17	2.71
STD		1.48	0.29	1.03	2.63	0.07
RSD%		4.04	9.18	4.03	3.64	2.58

Table 3 Mastersizer S results of Sample No.3

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	26.18	2.48	21.58	54.34	2.40
	2	26.02	2.44	21.53	53.99	2.39
	3	26.79	2.28	21.22	54.07	2.44
2	1	25.66	2.39	21.14	53.13	2.40
	2	26.45	2.17	20.90	53.64	2.46
	3	25.17	2.27	20.72	52.11	2.41
3	1	26.40	2.10	20.60	53.15	2.48
	2	25.93	2.11	20.45	52.43	2.46
	3	25.87	2.08	20.28	52.24	2.47
Mean		26.05	2.26	20.94	53.23	2.44
STD		0.48	0.15	0.46	0.84	0.04
RSD%		1.84	6.64	2.20	1.58	1.64

Table 4 Mastersizer S results of Sample No.4

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	25.71	2.72	21.88	52.65	2.28
	2	25.23	2.60	21.49	51.63	2.28
	3	25.37	2.65	21.41	51.76	2.29
2	1	24.99	2.57	21.21	51.23	2.29
	2	24.61	2.48	20.91	50.43	2.29
	3	24.47	2.41	20.75	50.21	2.30
3	1	24.20	2.35	20.53	49.61	2.30
	2	23.88	2.28	20.26	48.96	2.31
	3	23.86	2.24	20.22	48.98	2.31
Mean		24.70	2.48	20.96	50.61	2.30
STD		0.66	0.17	0.58	1.30	0.01
RSD%		2.67	6.85	2.77	2.57	0.43

Table 5 Mastersizer S results of Sample No.5

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	21.67	6.25	19.87	39.70	1.68
	2	21.59	6.22	19.78	39.56	1.69
	3	21.50	6.20	19.70	39.38	1.68
2	1	21.32	6.17	19.50	39.09	1.69
	2	21.21	6.14	19.41	38.82	1.68
	3	21.16	6.13	19.36	38.76	1.69
3	1	21.11	6.11	19.29	38.68	1.69
	2	20.99	6.08	19.16	38.50	1.69
	3	20.94	6.06	19.10	38.43	1.70
Mean		21.28	6.15	19.46	38.99	1.69
STD		0.26	0.06	0.27	0.46	0.00
RSD%		1.22	0.98	1.39	1.18	0.00

Table 6 Mastersizer S results of Sample No.6

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	50.16	9.52	36.74	98.86	2.43
	2	50.57	9.44	36.30	98.87	2.46
	3	49.53	9.26	35.62	99.61	2.54
2	1	51.07	9.27	35.64	100.15	2.55
	2	49.72	9.30	35.65	99.64	2.53
	3	49.50	9.16	35.16	98.28	2.54
3	1	50.66	9.15	35.21	98.99	2.55
	2	50.73	9.16	35.26	100.74	2.60
	3	49.86	9.06	34.77	98.80	2.58
Mean		50.20	9.26	35.59	99.33	2.53
STD		0.58	0.15	0.61	0.77	0.05
RSD%		1.16	1.62	1.71	0.78	1.98

Table 7 Mastersizer S results of Sample No.7

No.of measurement	Sub-run	D [4,3] (μm)	D (v,0.1) (μm)	D (v,0.5) (μm)	D (v,0.9) (μm)	Span
1	1	51.81	1.80	35.36	115.86	3.22
	2	53.54	1.84	34.80	118.59	3.36
	3	53.53	1.78	33.86	122.74	3.57
2	1	52.68	1.72	33.23	122.78	3.64
	2	54.73	1.71	32.87	131.78	3.95
	3	49.91	1.59	30.95	118.86	3.79
3	1	52.22	1.57	30.90	127.70	4.08
	2	51.80	1.55	30.65	127.68	4.12
	3	50.63	1.46	28.37	124.22	4.33
Mean		52.32	1.67	32.33	123.36	3.78
STD		1.50	0.13	2.27	5.11	0.37
RSD%		2.87	7.78	7.02	4.14	9.79

Note : The specific surface area is inapplicable unless the density of a sample is known.

Interpretation/Opinions : None

Attached pages :

The attachment number	Detail
1 – 3	Mastersizer-S results of Sample No.1
4 – 6	Mastersizer-S results of Sample No.2
7 – 9	Mastersizer-S results of Sample No.3
10 – 12	Mastersizer-S results of Sample No.4
13 – 15	Mastersizer-S results of Sample No.5
16 – 18	Mastersizer-S results of Sample No.6
19 – 21	Mastersizer-S results of Sample No.7

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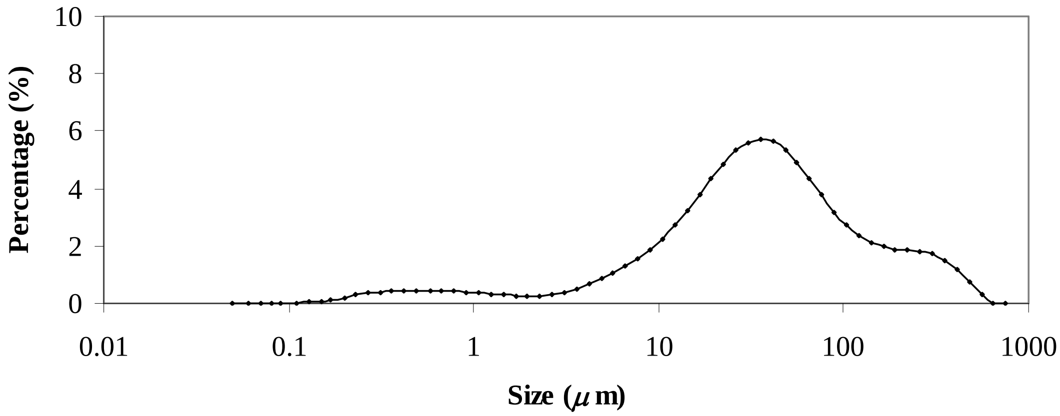

Figure F-1 Particle size distribution in raw domestic wastewater

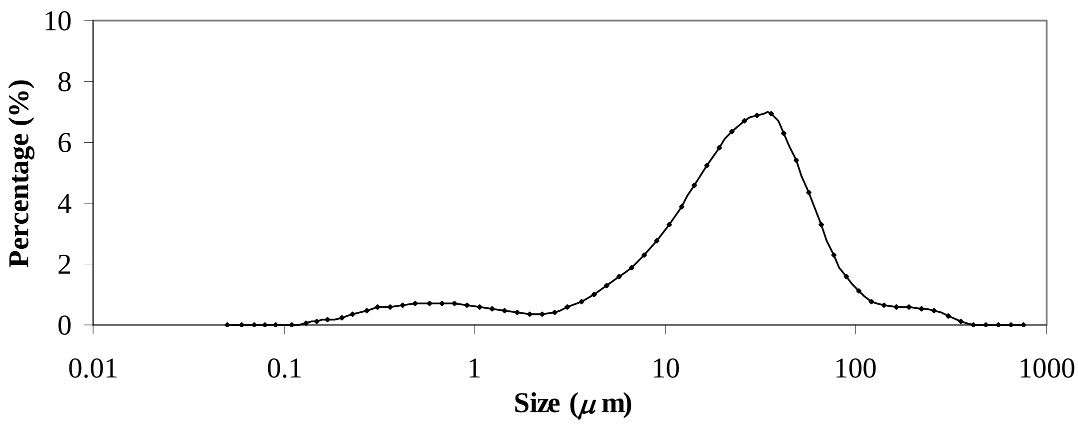

Figure F-2 Particle size distribution in supernatant of domestic wastewater after 2 hours settling

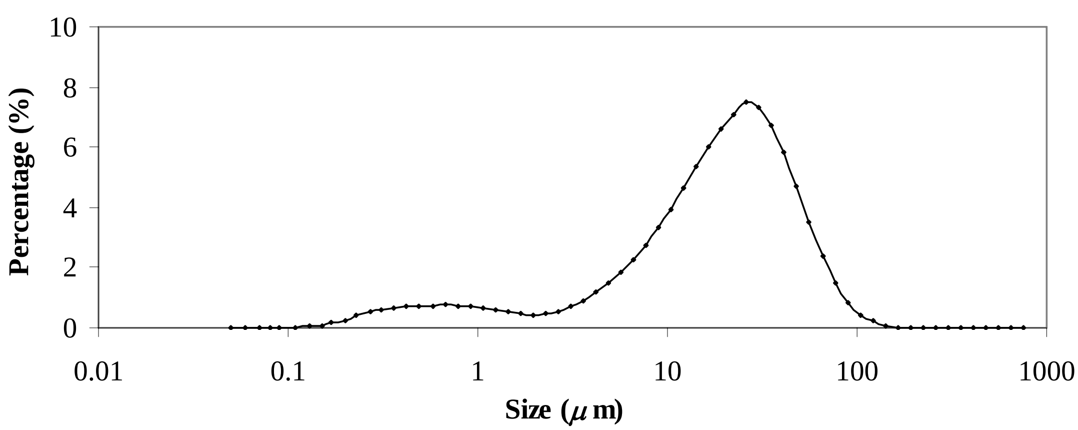

Figure F-3 Particle size distribution in supernatant of domestic wastewater after 3 hours settling

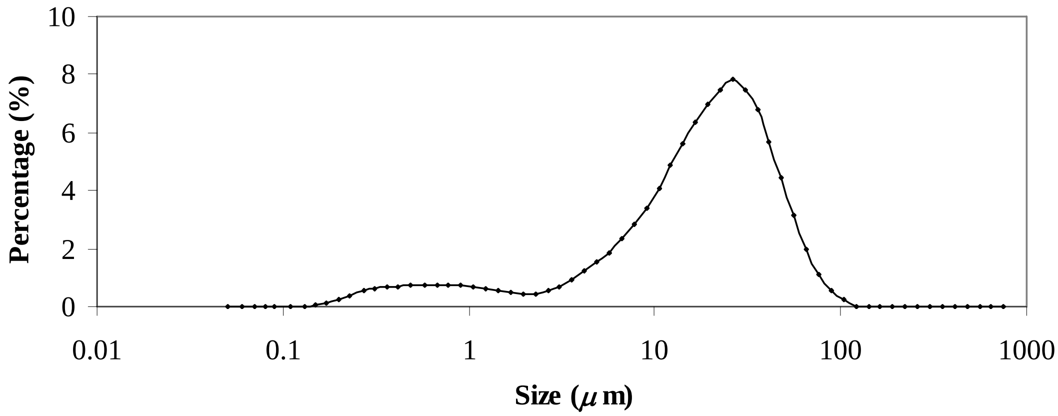

Figure F-4 Particle size distribution in supernatant of domestic wastewater after 4 hours settling

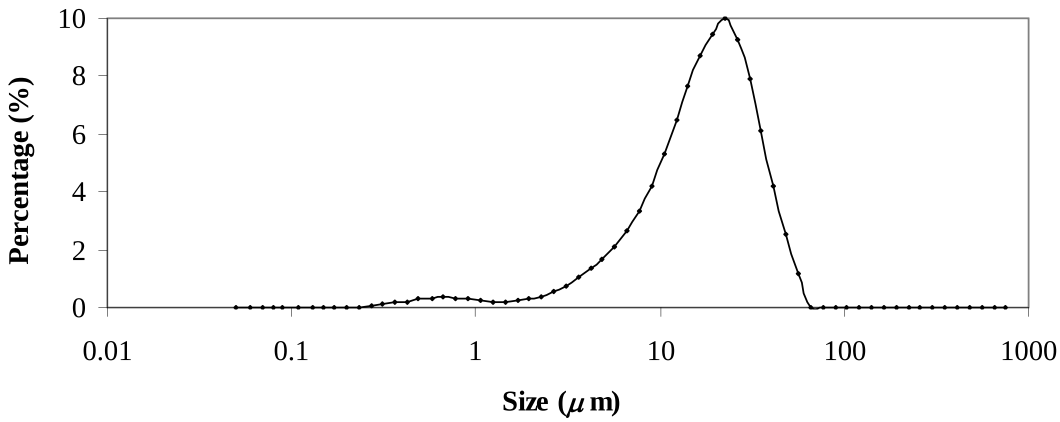

Figure F-5 Particle size distribution in mixed liquor in anaerobic reactor

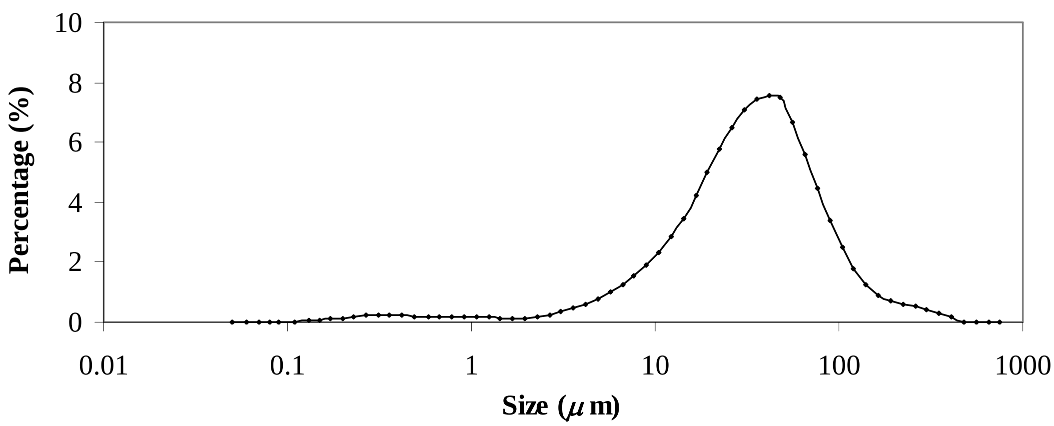

Figure F-6 Particle size distribution in mixed liquor in aerobic reactor

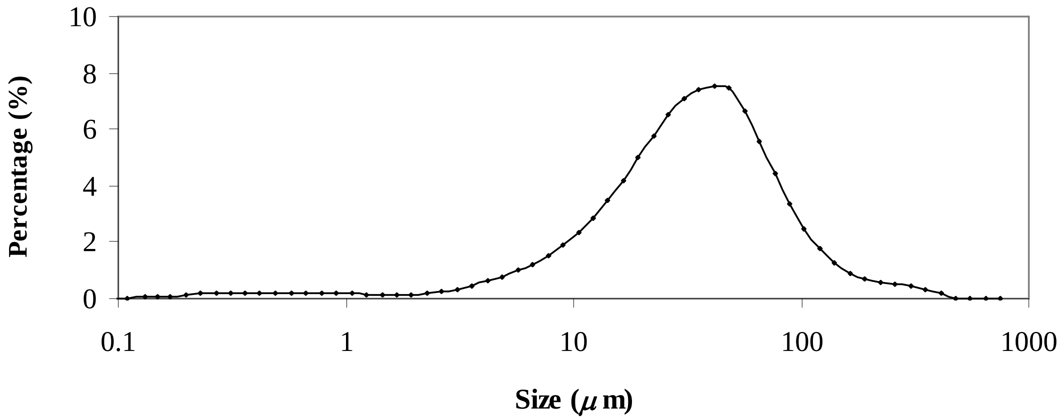

Figure F-7 Particle size distribution in effluent septic tank wastewater

Appendix G

The MFI and Specific resistance measurement of various wastewaters

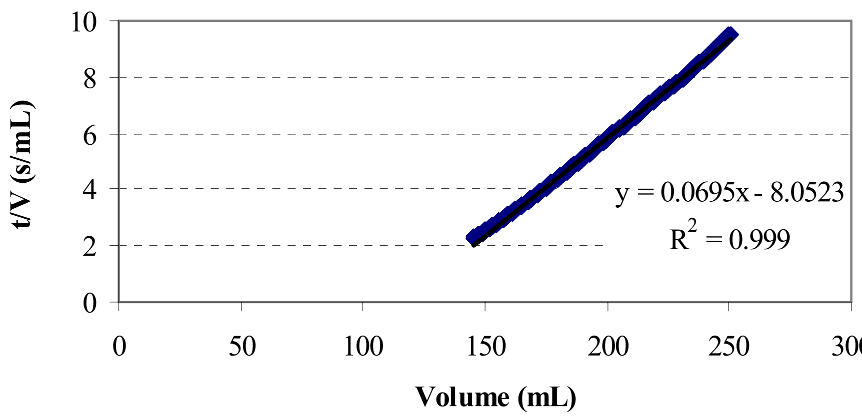

Figure G-1 Raw wastewater

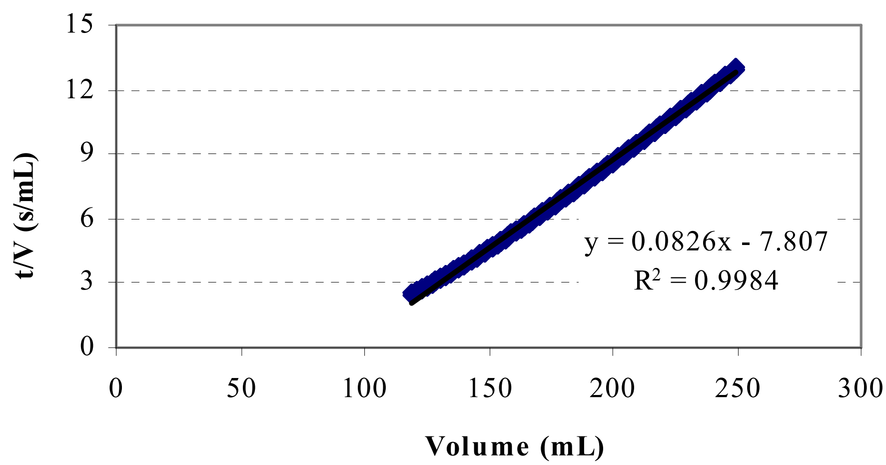

Figure G-2 Wastewater after 2 hours settling

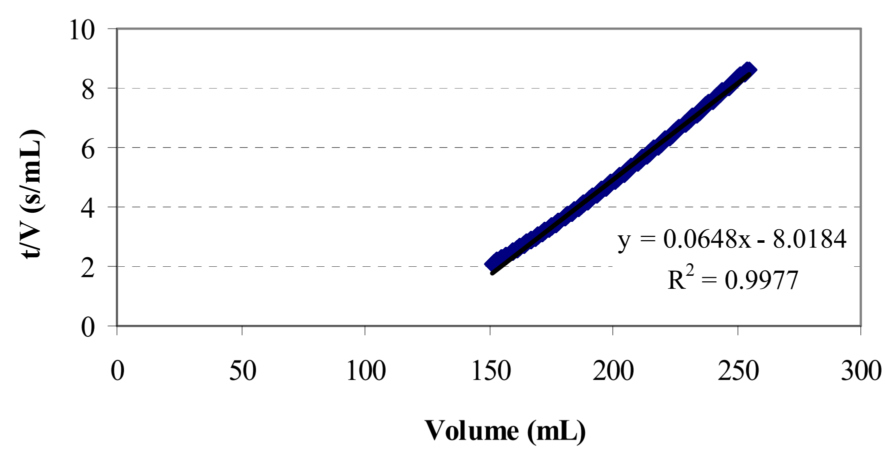

Figure G-3 Wastewater after 3 hours settling

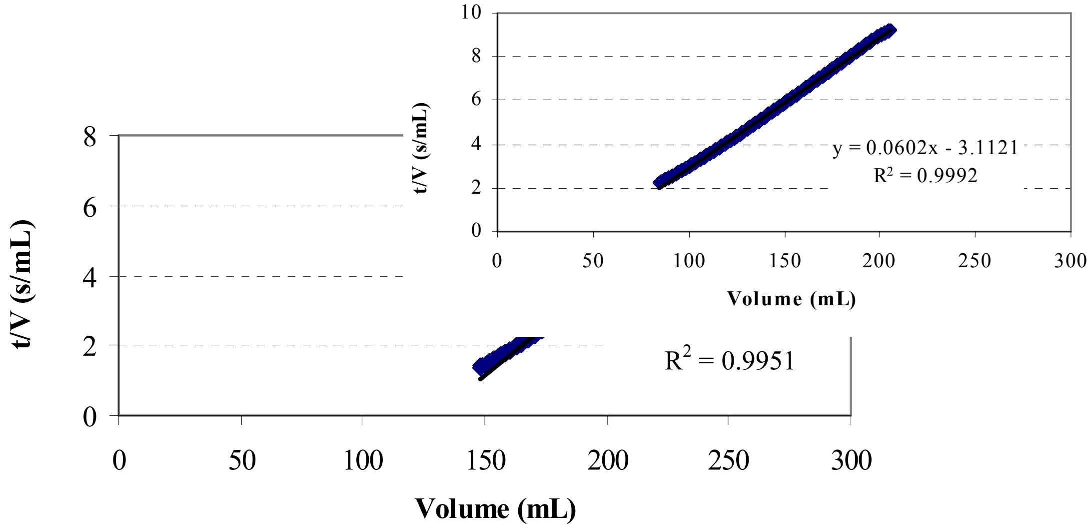

Figure G-4 Wastewater after 4 hours settling

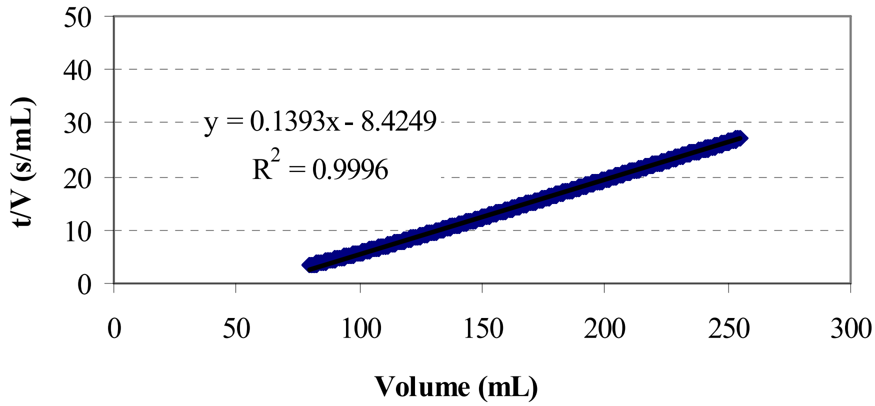

Figure G-5 Effluent septic tank

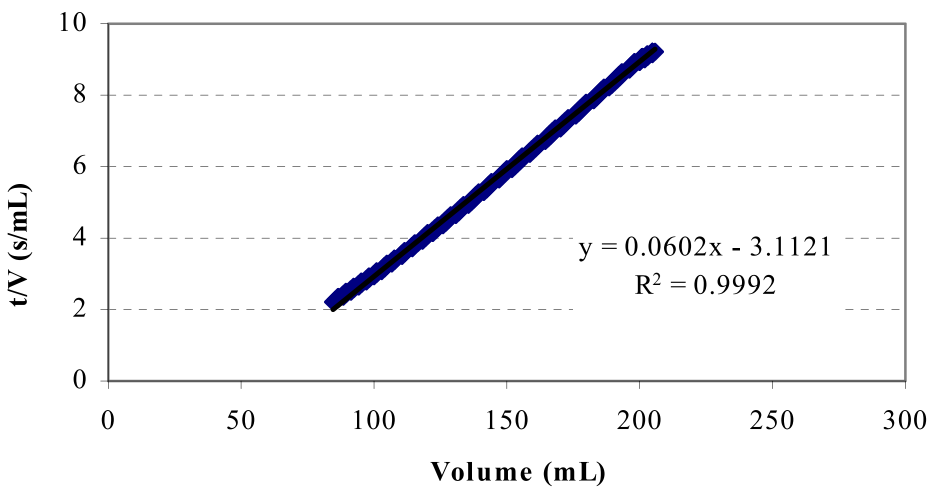

Figure G-6 Mixed liquor of anaerobic reactor

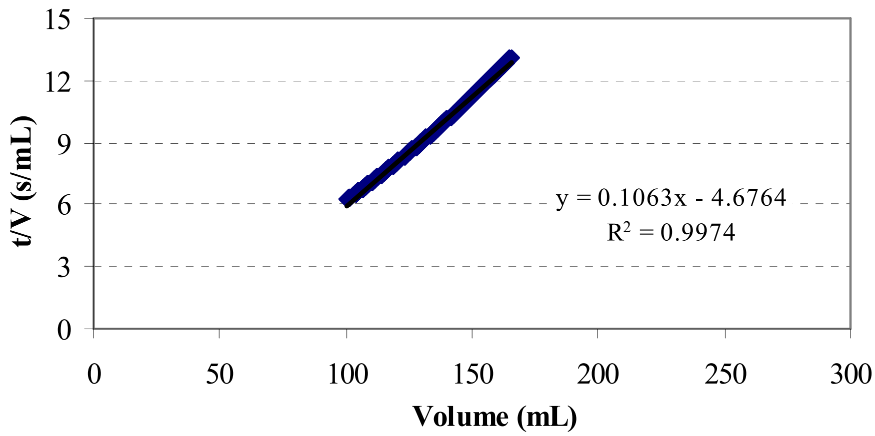

Figure G-7 Mixed liquor of aerobic reactor

MFI values were extracted from slope of curves in the figure G-1 to G-7 of correlative wastewater.

Based on MFI results measured above, specific resistance was calculated by formula:

$$\alpha = \frac{2 * A^2 * \Delta P}{\mu * C_b} * \frac{t/V}{V} \qquad \text{Equation G-1}$$

Which:

α	Specific resistance, m/kg
A	Membrane working area, m^2
ΔP	Different pressure, Pa
t	Time measurement, s
V	Volume of measuring water, m^3
μ	Water viscosity at 25 ^{0}C, N.s/m^2
C_b	MLSS concentration, kg/m^3

For this experiment

$A = 41.8*10^{-4}$ (m^2)

$\Delta P = 100$ (kPa)

$\mu = 0.890*10^3$ (N.s/m^2)

The specific resistance of these wastewaters was presented in the table G-1

Table G-1 MFI and specific resistance of various wastewaters

Sample	MFI (t/L^6)	α (m/kg)
Raw wastewater	$6.95*10^4$	$3.74*10^9$
Wastewater after 2 hours settling	$8.26*10^4$	$6.01*10^9$
Wastewater after 3 hours settling	$6.48*10^4$	$5.54*10^9$
Wastewater after 4 hours settling	$5.67*10^4$	$5.06*10^9$
Effluent septic tank	$1.39*10^5$	$7.19*10^9$
Mixed liquor of anaerobic reactor	$6.02*10^4$	$8.36*10^7$
Mixed liquor of aerobic reactor	$1.06*10^5$	$9.02*10^7$

Appendix H

Initial membrane resistance

Table H-1 Initial measurement of aerobic membrane

Flux (mL/min)	TMP (mm Hg)	Filtration flux ($L/m^2.h$)	TMP (kPa)
18	30	5.4	4.0
68	40	20.4	5.3
152	50	45.6	6.7
224	60	67.2	8.0
305	70	91.5	9.3
403	80	120.8	10.7

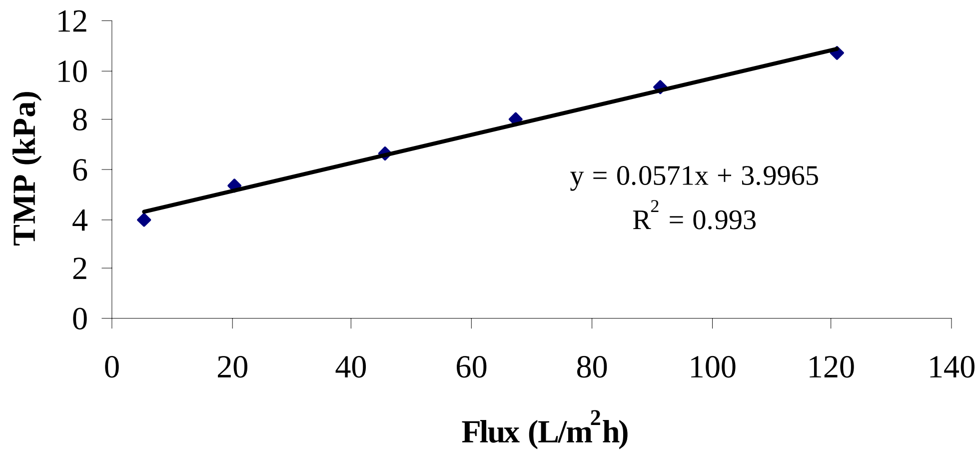

Figure H-1 Initial membrane resistance of membrane use for aerobic process

R_m= 0.0571 ($kPa/L/m^2h$)*10^3 (Pa/kPa)*3600 (s/h)*1000 (L/m^3)/0.798*10^3 (Ns/m^2)

R_m = 2.576*10^{11} (m^{-1})

Similar measurement for R_t and R_f The results for experiment as below:

R_t = 3.777*10^{12} (m^{-1})

R_f = 2.036*10^{12} (m^{-1})

Use formula $R_t = R_m + R_c + R_f \Rightarrow R_c = R_t - (R_m + R_f)$

R_c = 3.777*10^{12} (m^{-1}) – (2.576*10^{11} (m^{-1}) + 2.036*10^{12} (m^{-1}))

R_c = 1.483*10^{12} (m^{-1})

Table H-2 Initial measurement of anaerobic membrane

Flux (mL/min)	TMP (mm Hg)	Filtration flux (L/m^2.h)	TMP (kPa)
15	31	4.5	4.1
61	40	18.3	5.3
113	50	33.9	6.7
175	59	52.5	7.9
253	69	75.9	9.2
318	79	95.3	10.5

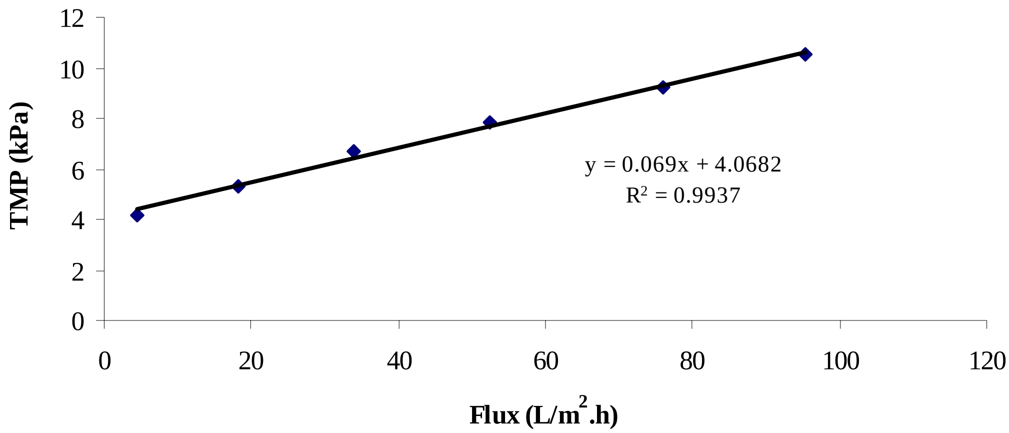

Figure H-2 Initial membrane resistance of membrane use for anaerobic process

R_m= 0.069 (kPa/L/m^{2}h)*10^3 (Pa/kPa)*3600 (s/h)*1000 (L/m^3)/0.798*10^3 (Ns/m^2)

R_m = 3.113*10^{11} m^{-1}

Similar measurement for R_t and R_f The results for experiment as below:

R_t = 1.515*10^{13} (m^{-1})

R_f = 8.7023*10^{11} (m^{-1})

Use formula $R_t = R_m + R_c + R_f \Rightarrow R_c = R_t - (R_m + R_f)$

R_c = 1.515*10^{13} (m^{-1}) – (3.113*10^{11} (m^{-1}) + 8.7023*10^{11} (m^{-1}))

R_c = 1.397*10^{13} (m^{-1})

Appendix K

The analyze results of experiments

Table K-1 pH TMP and flux of aerobic process

Date	Influent		Effluent				
	pH	T^0	pH	T^0	DO mg/L	TMP kPa	Flux $L/m^2.h$
3-Jan	7.53	18.9	7.95	25.7	6.44	5.60	4.1
7	8.16		6.92	25.2	3.78	6.53	4.2
10	7.78	25.9	6.58	25.0	4.60	7.33	4.2
11	7.53	24.5	6.65	24.8	4.50	7.60	4.2
12	7.96	23.9	6.82	22.2	5.00	8.13	4.2
13	7.83	25.6	6.86	25.2	4.20	8.13	4.1
14	7.87	26.0	6.69	27.3	4.20	7.86	4.1
15	7.98	27.3	6.87	24.3	6.50	8.53	4.1
16	7.94	28.7	6.49	27.7	6.90	8.26	4.1
17	7.82	27.4	6.71	25.4	6.80	8.53	4.1
18	7.83	30.9	6.47	27.7	6.70	7.86	4.1
19	7.86	32.1	6.44	27.9	6.70	8.26	4.1
20	7.86	31.5	6.47	27.6	6.60	7.86	4.1
21	7.92	28.0	6.03	25.3	6.90	9.33	4.2
22	7.93	29.5	6.65	25.6	7.05	10.00	4.2
23	7.82	27.0	6.84	26.5	6.40	10.26	4.2
24	8.14	28.7	6.97	25.2	5.70	10.26	4.2
25	7.94	26.7	7.02	23.9	6.90	11.60	4.2
26	7.67	26.8	6.75	23.8	6.20	12.93	4.2
27	7.9	27.1	6.70	23.7	6.30	14.26	4.2
28	7.85	26.9	6.61	23.8	6.10	15.59	4.2
29	7.72	27.8	6.28	25.3	5.70	27.19	4.2
30	7.88	27.7	6.47	25.5	6.00	55.32	4.2
9-Feb	7.25	30.3	5.34	25.9	6.1	4.40	8.1
10	7.38	25.5	5.95	25.9	6.0	6.40	8.1
11	7.85	29.8	4.96	26.9	4.6	7.86	8.1

Date	Influent		Effluent				
	pH	T^0	pH	T^0	DO mg/L	TMP kPa	Flux $L/m^2.h$
12	7.18	27.2	4.95	25.1	5.1	9.06	8.1
13	7.39	24.8	5.25	22.1	6.6	12.00	8.1
14	7.36	28.3	4.63	25.8	5.3	12.80	8.1
15	7.31	29.2	4.94	26.9	5.7	15.20	8.1
16	7.83	28.1	5.09	26.7	5.8	20.26	8.1
17	7.68	26.1	5.52	24.4	5.6	27.59	8.1
18	7.97	26.1	5.29	26.8	6.5	66.24	7.5
27	7.7	28.1	5.22	26.7	6.1	3.87	8.1
28	7.84	27.7	5.05	26.9	5.5	6.93	8.1
1-Mar	8.11	29.4	4.9	27.1	4.8	9.06	8.1
2	8.05	29.6	4.93	27.4	5.1	11.20	8.1
3	7.86	24	4.99	25.3	5.4	14.53	8.1
4	8.07	28.4	4.93	26.4	5.7	18.39	8.1
5	7.93	27.6	4.91	25.5	5.8	27.59	8.1
6	7.81	29.2	4.87	27.2	5.6	69.71	7.2

Table K-2 MLSS and MLVSS in aerobic reactor

Date	MLSS	MLVSS	Percentage
28-Dec	3,280	3,090	94
4-Jan	2,470	2,150	87
11	2,150	1,960	91
18	1,940	1,940	100
25	1,040	800	77
12-Feb	6,560	5,510	84
15	3,750	3,230	86
18	3,840	3,170	83
1-Mar	5,660	5,040	89
4	4,620	3,880	84
7	3,950	3,340	85

Table K-3 The treated wastewater quality in aerobic process

Date	Influent				Effluent			
	COD	TKN	NH_4^+	PO_4^{3-}	COD	TKN	NH_4^+	PO_4^{3-}
28-Dec	320		58.8	14.25	64		50.4	28.00
1-Jan	144		53.2	8.75	24		50.4	17.50
4	144	89.6	98.0	7.75	48	68.6	58.8	6.62
7	240	89.6	100.8	10.37	80	30.8	36.4	6.25
11	336	109.2	95.2	9.00	120	42.0	42.0	11.25
14	272	117.6	98.0	13.00	88	39.2	39.2	13.00
17	248	134.4	98	7.50	40	36.4	36.4	7.50
20	144	98	95.2	5.40	32	42.0	42.0	8.00
23	272	106.4	86.8	5.25	32	36.4	36.4	7.50
26	164	106.4	89.6	11.12	40	36.4	36.4	8.40
29	162	103.6	95.2	7.50	29.5	39.2	39.2	9.50
1-Feb	274	123.2	89.6	9.50	29.5	36.4	33.6	8.00
12	128.8	106.4	92.4	8.87	28.6	39.2	33.6	9.00
15	121.2	103.6	86.8	9.25	35.2	32.2	30.8	9.25
18	125.3	103.6	89.6	8.50	13.9	33.6	33.6	8.75
1-Mar	141.1	106.4	86.8	8.50	23.5	47.6	33.6	8.00
4	188.2	103.6	92.4	8.25	31.3	33.6	33.6	8.00
7	156.8	92.4	86.8	8.00	23.5	30.8	30.8	9.00

Table K-4 pH TMP and flux of anaerobic process

Date	Run time hours	Influent		Effluent				
		pH	T^0	pH	T^0	DO mg/L	TMP kPa	Flux $L/m^2.h$
14-Jan	0						2.93	4.35
	12						19.33	4.20
	16						51.98	4.20
	17	7.87	26	7.82	31.9	0.5	64.51	4.20
14-Mar	0						3.47	4.5
	12						23.33	4.5
	14						44.39	4.2
	16	7.89	32	7.41	33	0.04	64.25	4.2
19-Mar	0						4.40	8.7
	2						7.20	8.4
	4						25.86	8.7
	6	7.56	30.2	7.41	32.6	0.03	77.84	7.8
22-Mar	0						3.47	8.7
	2						17.86	8.7
	4						39.19	8.4
	6	7.47	25.9	7.45	30.7	0.05	76.64	7.8

K-4 The treated wastewater quality in anaerobic process

Date	Influent				Effluent			
	COD	TKN	NH_4^+	PO_4^{3-}	COD	TKN	NH_4^+	PO_4^{3-}
1-Jan	144.0		53.2	8.8	64.0		58.8	14.7
14-Mar	182.4	126.0	120.4	9.9	129.2	126.0	120.4	10.2
19	197.6	131.6	120.4	9.9	114	123.2	117.6	9.5
22	228	131.6	123.2	10.0	98.8	123.2	120.4	10.0

Some pictures in search

Untreated domestic wastewater discharge into cannel

Polluted cannel

Septic tank in Hanoi Vietnam

Septic tank in suburban of Thailand

Monitored septic tank in Klong 4 in Pathumthani – Thailand

Taking samples

Sludge withdrawal

Experimental setup in ambient laboratory

Anaerobic reactor

Aerobic reactor

Membrane before cleaning

Membrane after cleaning