

**MAXIMIZING RESOURCE RECOVERY: LIFE CYCLE COMPARISON OF
WATER QUALITY IMPACTS ON NON-POTABLE WATER REUSE AND
ENERGY RECOVERY.**

By

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The final copy of this thesis has been examined by the signatories, and we
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ABSTRACT

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Maximizing resource recovery: Life cycle comparison of water quality impacts on non-potable water reuse and energy recovery

Thesis directed by Assistant Professor Sherri Cook

Given the gap between water supply and demand, recycling of wastewater, including nonpotable applications, can improve water availability. Mainstream anaerobic processes are gaining importance due to reduced energy and cost demands as compared to conventional process. This study's goal is to compare mainstream aerobic and anaerobic water reuse treatment trains to identify criteria for improving the sustainability of water recycling. The functional unit is the production of nonpotable reuse water over 40 years from 20 million gallons per day of medium strength wastewater. The comparison of water reuse systems consisting of anaerobic (ANA) and aerobic (AER) processes were focused on the different chemical and energy demands for each treatment scenario. These demands were translated into 10 aggregated environmental impact categories using life cycle inventory data and the Tool for Reduction and Assessment of Chemicals and Other Environmental Impacts (TRACI) assessment method.

When comparing ANA and AER baseline scenarios, it was found that using chlorine disinfection had the largest impacts, compared to UV. Therefore, UV disinfection was used for further study. The AER scenario was best in 6 out of 10 environmental impact categories as compared to ANA, mostly due to offsetting chemical fertilizer production with biosolids land application, and negative impacts mostly due to aeration energy. ANA had benefits from energy production during mainstream wastewater treatment but large negative impacts due to high alum doses for coagulation. Additional scenarios included evaluating high strength wastewater,

which showed the significance of alum dosing on environmental performance; maximum dissolved methane recovery, which showed great improvements in ANA performance; and best and worst case ANA operations, which showed that ANA could be better than AER if optimized. These results were used to set criteria for technology performance and support model-based experimental design. Overall, the advancement of ANA technologies and employment of resource recovery can minimize the relative environmental impacts of conventional nonpotable reuse treatment systems.

TABLE OF CONTENTS

CHAPTER I: INTRODUCTION.....	1
CHAPTER II: LITERATURE REVIEW.....	3
Life Cycle Assessment (LCA).....	3
Water reuse	5
Reclaimed water for irrigation	6
Energy Recovery with Mainstream Anaerobic Processes	7
Impact of water quality.....	9
CHAPTER III: METHODOLOGY.....	10
Context.....	10
LCA Methodology.....	10
Treatment Scenarios.....	13
AER Scenarios	14
AER Performance	14
ANA Scenarios	21
CHAPTER IV: RESULTS.....	33
AER Scenarios.....	33
ANA Scenarios.....	35
ANA vs. AER	36
Dissolved Methane	39
Temperature model for ANA system.....	40
AnMBR Operational Uncertainty	41

High Strength Wastewater	43
Effluent nitrogen for water reuse standards	46
Enhanced primary treatment for better removal of organic carbon	47
Breakeven points for ANA and AER system:.....	47
Membrane Scouring.....	47
Dissolved Methane:	47
CHAPTER V: FUTURE WORK	48
References	49
APPENDIX A – DETAILED OXYGEN REQUIREMENT CALCULATIONS	54
APPENDIX B – ENERGY CONSUMPTION FOR ANAEROBIC SYSTEM.....	56
APPENDIX C – MEMBRANE CALCULATION	60
APPENDIX D - DEEP BED ANTHRACITE FILTER DESIGN	61

LIST OF TABLES

Table 1. Non-potable water reuse standards.....	7
Table 2. Influent domestic wastewater characteristics.....	12
Table 3. Description of all scenario evaluated in this study.....	13
Table 4. Treatment performance for AER-HRAS Scenarios (no nitrification).....	16
Table 5. Treatment performance for AER-NO ₃ Scenarios (nitrification).	16
Table 6. Treatment performance for ANA Scenarios	22
Table 7. Experimental data analysis for multiple experimental AnMBR systems, operated at mesophilic conditions.....	24
Table 8. Estimated minimum alkalinity as CaCO ₃ required to maintain pH AT 7.0 as a function of temperature and % carbon dioxide gas.....	26

LIST OF FIGURES

Figure 1. Phases of life cycle assessment methodology	3
Figure 2. Distinguishing between mid point and end point categories.....	5
Figure 3. Process flow diagram for anaerobic and aerobic water reuse systems.....	12
Figure 4. Comparison of four AER scenarios (with or without nitrification; chlorine or UV disinfection).....	34
Figure 5. Percent unit process contribution for GWP for all AER scenarios normalized to AER-HRAS+UV. The black box shows the net environmental impact.....	35
Figure 6. Disinfection contribution for anaerobic scenarios normalized to ANA+UV.	36
Figure 7. Comparison of ANA and AER shows the impact associated with AER to be best in 6 out of 10 TRACI categories.....	37
Figure 8. Percent process contribution for carcinogenic impact category for ANA and AER normalized to baseline ANA. The black box shows the net environmental impact.....	38
Figure 9. Percent process contribution for eutrophication impact category for ANA and AER normalized to baseline ANA. The black box shows the net environmental impact.....	39
Figure 10. Percent process contribution for GWP for maximum dissolved methane recovery. The black box represents the net environmental impact.....	40
Figure 11. Comparison of 'Best Case - ANA' with baseline ANA scenario.	42
Figure 12. Comparison of ANA-baseline, ANA-best case and ANA-worst case with AER	43
Figure 13. Contribution due to aeration and coagulation on AER and ANA high strength water reuse systems.	44
Figure 14. Carcinogenics (a) and global warming potential (b) impacts as a function of the ANA alum dose for treating high strength wastewater.....	45

Figure 15. Relative environmental impact with alum dose of 120 mg/l for ANA system, normalized to AER.	46
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CHAPTER I: INTRODUCTION

The increasing water demands of a rapidly growing global population are increasing the need for water recycling. The use of treated wastewater for beneficial purposes such as agriculture, landscape irrigation and direct and indirect potable use (Tchobanoglous, 2014) are looked upon as an attractive alternatives to freshwater use. Water reuse is mainly divided into two major categories: (1) non-potable water reuse and (2) potable water reuse. Potable water reuse is further divided into indirect potable reuse and direct potable reuse. Out of many non-potable water reuse applications in the USA, landscape irrigation (e.g., parks and golf courses), agricultural irrigation, and industrial applications (e.g., cooling towers) are the most common (Schimmoller and Kealy, 2014). Indirect potable reuse is the augmentation of a drinking water source (surface or groundwater) with reclaimed water followed by an environmental buffer that precedes drinking water treatment. Direct potable water includes the direct introduction of reclaimed water into the drinking water treatment plant (US EPA, 2012). Water reclamation has become an attractive option in conserving and extending available water resources by potentially: (1) replacing the low-quality water supply by reclaimed water, (2) expanding available water sources by providing alternative water supply to meet present and future needs, (3) reducing the flow of nutrients and contaminants directly into the water body, and (4) maintaining the balance of aquatic ecosystems by minimizing the diversion of freshwater (Asano et al., 2007). Around 70% of the total global freshwater withdrawals is used for irrigation(UNESCO, 2012). In United states the percent of freshwater withdrawals for irrigation purposes is 37% (Geological Survey, 2005); therefore, recycling of wastewater for fulfilling the demands of irrigation / agricultural water would be an attractive option to conserve the freshwater.

Water and energy are highly interconnected, and high consumption of water results in higher energy demands (e.g., collection, distribution, and treatment) (Crone et al., 2016). At an average energy cost of \$0.075 per kWh, the total cost for providing safe drinking water and wastewater treatment is approximately \$7.5 billion per year (Energy, 2014). Water reuse technologies have high energy and chemical demands, especially when using conventional aerobic process; studies have found that many water reuse treatment scenarios result in overtreatment and avoidable resource consumption for these water reuse systems (Schimmoller and Kealy, 2014; Tran K. et al., 2016). Reducing net energy requirements for wastewater treatment and water reuse can be achieved by recovering the energy in wastewater. In addition, domestic wastewater is now being considered as a source of resources (e.g., water, energy, nutrients) instead of waste (Mccarty et al., 2011). To achieve resource recovery, the use of mainstream anaerobic processes have been found to be a possible replacement for aerobic treatment (Kim et al., 2011; Mccarty et al., 2011; Shoener et al., 2014).

Water reuse systems are complex systems and changing the main wastewater treatment biochemical environment will have many impacts on downstream treatment (water quality, chemical and energy use etc.) Analyzing the water reuse system through life cycle approach will help understand the flexibility for design consideration to optimize the system. Therefore, a systems approach and life cycle assessment methodology was used to understand the impact of various components associated with water reuse system (e.g. chemicals and energy) along with achieving agricultural water reuse standards.

CHAPTER II: LITERATURE REVIEW

Life Cycle Assessment (LCA)

Several environmental assessment management techniques are being used to study and analyze the impact of a product or process on the environment. A few of them are, risk assessment, environmental performance evaluation, environmental auditing, environmental impact assessment, and life cycle assessment (LCA). LCA is a standardized methodology to estimate the environmental aspects and impacts of a product or activity and the methodological framework is described in (ISO, 1997). LCA studies the aspects of environmental impact through a product's life from raw material acquisition through production, use, and disposal. The general category that LCA addresses are related to human health, resource use, and ecological consequences. The results developed from LCA study shall be used as a part of much more comprehensive decision process or used to understand the broad or general tradeoffs between compared studies. The LCA process consists of four main phases – goal and scope definition, inventory analysis, impact assessment and interpretation of results as shown in Figure 1.

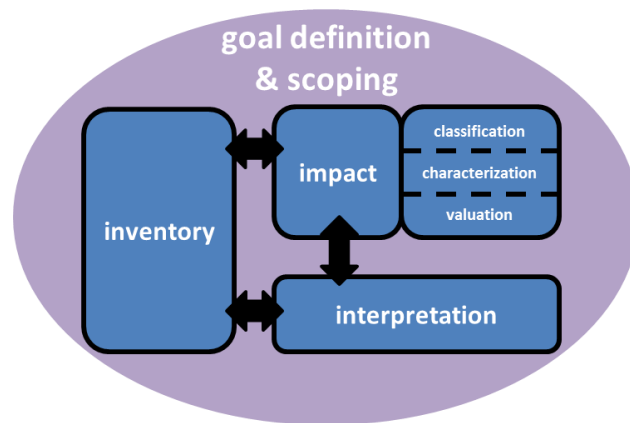


Figure 1. Phases of life cycle assessment methodology

During the goal and scope definition phase, the goal of the LCA is described and the scope is defined by determining the functional unit and system boundary, including

unit process(es), chemical production, energy requirements. The life cycle inventory (LCI) analysis phase involves data collection from various resources to quantify relevant inputs and outputs of the system. Inputs in terms of required raw materials, energy, and chemicals are used to translate those activities into life cycle emissions (using readily available databases, such as Ecoinvent). The output available from such analysis will give the data, which includes hundreds of chemical emissions released to the soil, water, and air. Various LCI databases are found in Simapro: Ecoinvent v3.1 LCI database, Agri-footprint LCI database, European reference Life Cycle Database (ELCD) and U.S. Life Cycle Inventory Database (USLCI) to name a few.

The third phase is Life Cycle Impact Assessment (LCIA). It evaluates the significant impact of emissions using the output from the life cycle inventory analysis. It involves analyzing and characterizing the results within various impact categories. The basic elements of this LCIA are classifying the LCI data to impact categories (based on environmental mechanisms and damage) and then characterization of these chemicals to an equivalent indicator compound in each category; this step involves multiple characterization models, specific to the LCIA method used. Midpoint category impacts are considered to be links in the cause-effect chain (environmental mechanism) of an impact category, prior to the endpoints, at which characterization factors or indicators can be derived to reflect the relative importance of emissions or extraction. Common examples for midpoint category include ozone depletion, global warming potential, and photochemical smog (Bare et al., 2003). Endpoint characterization factors are calculated to reflect differences stressors at an endpoint in a cause-effect chain. They are characterized in relevance to the social understanding of the final effects. This should be understood that, in some impact categories, more than one endpoint measure exists. An example for such impact can

be observed in the context of ecosystem effects, were measures include the Potentially Affected Fraction (PAF) of species and the Potentially Disappeared Fraction (PDF) of species (Bare et al., 2000). A higher level of uncertainty is observed in end point categories as compared to midpoint categories. Midpoint modelling minimizes assumptions and simplifies the communication of categories by being more comprehensive than model coverage for end point estimation (Bare et al., 2003).

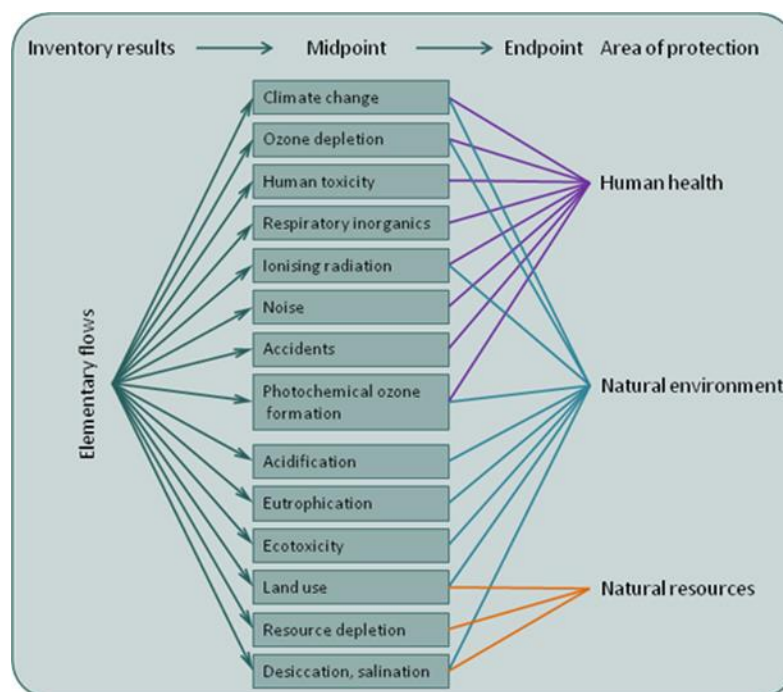


Figure 2. Distinguishing between midpoint and end point categories.

Water reuse

Due to increase in population growth and water consumption, freshwater availability is decreasing. To address this increasing demand of water, water reuse has been practiced for many years (Tchobanoglous et al., 2003). Major pathway for water reuse includes groundwater recharge, irrigation, industrial use and surface water replenishment. However, the degree of treatment required in individual water and wastewater reclamation facilities vary mainly on its reuse application depending on

the water quality requirements (Asano and Levine, 1996). Also domestic wastewater, is been looked upon as a great resource for water, energy and nutrients like nitrogen (N) and phosphorus (P). Hence, the reuse of domestic wastewater for landscape and crop irrigation is being widely accepted to make use of the fertilizing elements of the wastewater (Mccarty et al., 2011).

Reclaimed water for irrigation

In 2005 the freshwater withdrawal for irrigation alone was around 37% and 62% of all freshwater withdrawals excluding thermoelectric withdrawals. Irrigation is the second largest category of water reuse after thermoelectric (Kenny et al., 2009). The non-potable use of reclaimed water is gaining importance mainly due to the potential of resource recovery from wastewater. The use of reclaimed water for irrigation of food crops is prohibited in some states, while others follow irrigation of food crops with reclaimed water only if the crop is to be processed and not eaten raw. Depending on the type of crop or type of irrigation, states treatment requirements range from secondary treatment and disinfection, to advanced chemical treatment downstream like coagulation - flocculation and filtration (US Environmental Protection Agency, 2012). The treatment design and water quality requirements for various non-potable reuse standards including agricultural reuse – food crops are as shown in **Table 1**.

A number of treatment technologies have been implemented for potential non-potable water reuse projects. Advanced tertiary treatment in terms of granular media filtration, membrane systems and cloth filters have been proved to satisfy the filtration requirement for non-potable systems. Non-potable reuse application based scenario studied by (Schimmoller and Kealy, 2014) for landscape irrigation compares a granular media filtration to two membrane-based approaches. Also, (Sheikh et al., 1990) includes the use of advanced tertiary treatment in form of coagulation,

clarification, filtration and disinfection for reclamation study of wastewater for agricultural purposes.

However, the traditional approach of using the conventional water reuse treatment facilities are energy intensive, produce large quantities of residuals and fail to recover the potential resources from wastewater (Mccarty et al., 2011; Smith et al., 2012). The energy associated with aeration in conventional process accounts for about half of the total energy required for wastewater treatment plant. Even having developed the anaerobic digestion for solids handling system might recovery energy in form of biogas, but it can only satisfy up to quarter to half of plants' need (Mccarty et al., 2011).

Table 1. Non-potable water reuse standards. (US Environmental Protection Agency, 2012)

	Unrestricted use	Restricted use	Agricultural reuse – food crops	Agricultural reuse – non-food crops
Biological oxygen demand (BOD), mg/l	≤ 10	≤ 30	≤ 10	30
Total suspended solids (TSS), mg/l	30	≤ 30	5	30
Total Nitrogen (TN), mg/l	< 10	< 10	10	10
Turbidity, NTU	≤ 2	-	2	10
Disinfection, Chlorine residual, mg/l	>1	>1	>1	>1
Disinfection, UV, MJ/cm ²	-	-	100	-
Bacterial Indicators, mL	23/100	$\leq 200/100$	23/100	$\leq 200/100$

Energy Recovery with Mainstream Anaerobic Processes

Different anaerobic processes are gaining attention due to their capability to recovery energy in form of biogas (Mccarty et al., 2011; Smith et al., 2014, 2012). The well-established upflow anaerobic sludge blanket (UASB) and expanded granular sludge bed (EGSB) reactor configurations largely meet the required effluent standards using

anaerobic process (Smith et al., 2012). Anaerobic membrane bioreactors (AnMBR) coupled with membranes is another emerging technology that was found to be capable of achieving high effluent quality (Hu and Stuckey, 2006; Martin et al., 2011; Martinez-Sosa et al., 2011; Wen et al., 1999). Use of granular activated carbon coupled with AnMBR has found to be used to minimize the membrane fouling associated with developing increased flux (Kim et al., 2011).

To understand the energy model for AnMBR system, various configuration of operating parameters has been studied. The experimental data from (Mei et al., 2016) for various temperature (35°C, 25°C and 15°C) shows that energy neutral system can be achieved at temperature of 35°C, flux ranging from 8.8 to 10.5 L/m².h with hydraulic retention time of 5.8 – 4.8 h, equivalent to the organic loading rate (OLR) of 1.49 to 1.78 kg COD/(m³.d). Experimental energy demand values as reported in literature for AnMBR system with biogas sparging and without dissolved methane recovery are approximately in the range of 0.25 to 1.00 kWh/m³ (Liao et al., 2006), while some studies have reported the values to be in the range of 0.69 to 3.41 kWh/m³ (Li et al., 2016; Martin et al., 2011). Expected energy consumption for recovery of dissolved methane is 0.05 kWh/m³ (McCarty et al., 2011; Seib et al., 2016). Experimental data for comparison of effect of SRT on methane yield shows that infinite SRT helps achieve higher methane yield as compared to SRT of 30-100 days (Huang et al., 2011, 2008). With low HRT (< 8hours) and high SRT values, the slow growing anaerobic microbial populations are found to be effective in treating medium strength wastewater (Smith et al., 2012).

The full scale application of this AnMBR has not been implemented in real world, however (Smith et al., 2014) studies the complete wastewater treatment using AnMBR. Granular media filtration for efficient solids removal for reclaimed water production needs to be taken into consideration when thinking of water reuse

applications. However, for organic carbon removal for downstream process, high alum dose was required for conventional aerobic process. Alum dose of 50 – 200 mg/l with polymer dose of 0.2 mg/l was considered while studying the reclamation facility at Monterey (Sheikh et al., 1990). Relatively high amounts of alum dose can be expected for anaerobic effluent due to different nature of organic matter.

Impact of water quality

The influence of water quality on treatment plant operation was studied by (Santana et al., 2014). While this study was for drinking water treatment plant, the results showed the influent water quality to be responsible for about 14.5% of the total operational embodied energy associated with the treatment process. It also highlighted the fact that majority of the energy associated was due to high amount of chemicals used in the treatment process. While considering the high degree of treatment requirement for wastewater reuse operation, the need to understand the importance and effect of water quality helps to motivate the current research. The potential of using anaerobic processes, mainly AnMBR's to minimize the net energy associated with the reclaimed water helps to develop the water reuse model.

CHAPTER III: METHODOLOGY

Context

The system boundary for the water reuse treatment scenarios included wastewater process (conventional aerobic and mainstream anaerobic processes), downstream water reuse treatment scenarios consisting of coagulation and granular media filtration and disinfection in form of UV and chlorination and solids handling system. Energy consumption associated with all these unit processes was considered to evaluate the life cycle environmental impacts. Energy production was considered to calculate the benefit associated with energy production from mainstream ANA process and digestion of AER solids. Environmental impacts for reclaimed water effluent was out of the scope of this research.

The application of this reclaimed water shall vary according to its location. For rural implementation were the land application site might be close enough to the water reclamation facility, for urban application the distance for hauling might be a good consideration for analysis. These won't affect the analysis scenario wise, but will have significant impact considering the implementation of reclaimed water. The main application of this reclaimed water was considered for agricultural purpose. Irrigation, not considered as a year round application at some places proper storage of the reclaimed water need to be considered. This storage might have additional benefit of virus log inactivation (Bahri, 1999).

LCA Methodology

This study used comparative life cycle assessment (LCA) methodology following the ISO 14040 framework (ISO, 1997). The functional unit is the production of nonpotable reuse (NPR) water over 40 years from 20 million gallons per day of medium strength wastewater (Table 2). The system boundary included wastewater bioreactor processes followed by water reuse treatment processes as well as solids management;

it included the production and transportation of any chemicals (e.g., alum, chlorine, lime, polymer, etc.) needed during treatment, electrical energy used for treatment (e.g., aeration energy, UV lamp energy, etc.), energy production offsets (due to methane generation during mainstream anaerobic treatment and solids anaerobic digestion), and chemical fertilizer production offset (due to biosolids land application). The pumping of raw wastewater to and from the reuse treatment facility was assumed to be the same for all scenarios. Chemical fertilizer production offsets due to NPR water irrigation, infrastructure, and coagulated sludge disposal were out of the scope of this project; these were qualitatively considered when comparing treatment alternatives. The two main treatment trains had either mainstream anaerobic (ANA) processes or mainstream, conventional aerobic (AER) processes, as shown in Figure 33.

Life cycle inventory data for chemical (alum, lime, sodium hypochlorite, citric acid, polymer, and chlorine) production, membrane material (Polyvinylidene fluoride for ANA membranes), fertilizer (nitrogen and phosphorus) production, and electrical energy was collected from the Ecoinvent (Swiss Centre for Life Cycle Inventories, 2014) and US-EI (Earthshift, 2014) databases. The environmental impacts were evaluated using the Tool for Reduction and Assessment of Chemicals and Other Environmental Impacts (TRACI) (Bare et al., 2003) assessment method. TRACI has 10 midpoint impact categories: ozone depletion (kg CFCs eq), global warming potential (kg CO₂ eq), acidification (kg SO₂ eq), eutrophication (kg N eq), smog (kg O₃ eq), respiratory effects (kg PM_{2.5} eq), carcinogenics (CTUh), non-carcinogenics (CTUh), ecotoxicity (CTUe), and fossil fuel depletion (MJ surplus).

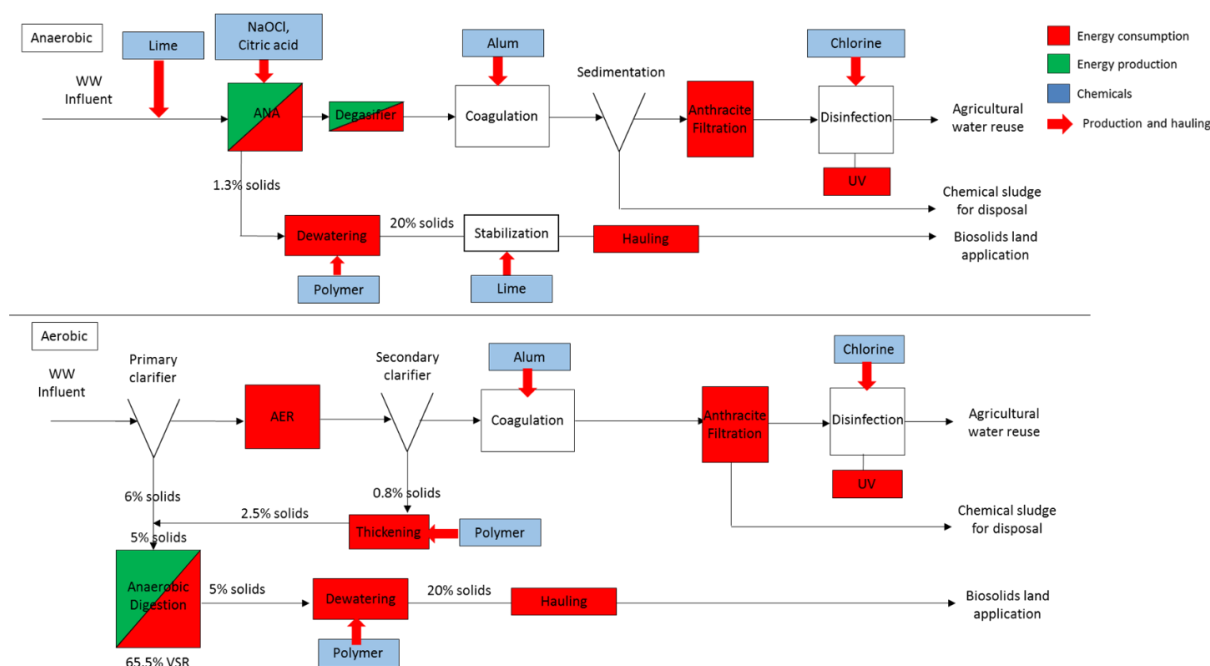


Figure 3. Process flow diagram for anaerobic and aerobic water reuse systems.

Table 2. Influent domestic wastewater characteristics

(Tchobanoglous et al., 2003).

Parameters	Medium strength wastewater	High strength wastewater
pH	7.5	7.5
Chemical oxygen demand (mg/l)	430	800
Biological oxygen demand (mg/l)	190	350
Total organic carbon (mg/l)	140	260
Total suspended solids (mg/l)	210	400
Total nitrogen (mg/l)	40	70
Total phosphorus (mg/l)	7	12
Sulfate (mg/l)	30	50
Alkalinity (mg CaCO ₃ /l)	200	400

Table 3. Targeted effluent standards according to Agricultural reuse-food crops. (US Environmental Protection Agency, 2012)

Parameters	Targeted standards
Biological oxygen demand (mg/l)	10
Total suspended solids (mg/l)	5
Total nitrogen (mg/l)	10
Turbidity (TNU)	2
Chlorine residual (mg/l)	1
UV disinfection (mJ/cm ²)	100
Bacterial Indicator (mL)	23/100

Treatment Scenarios

Two main treatment alternatives were considered for the comparison of water reuse treatment trains that had either mainstream anaerobic or mainstream aerobic wastewater processes (see Figure 3). In addition, the impact of changing disinfection technologies, operational parameters, and wastewater strength (Table 2) were also evaluated, as described in Table 4.

Table 4. Description of all scenario evaluated in this study.

Scenario #	Strength of wastewater	Scenario description
1	Medium strength	ANA + UV
2	Medium strength	ANA + Cl
3	Medium strength	AER-HRAS + UV
4	Medium strength	AER-HRAS + Cl
5	Medium strength	AER-NO ₃ + UV
6	Medium strength	AER-NO ₃ + Cl
7	Medium strength	ANA max CH ₄ + UV
8	Medium strength	ANA best case + UV
9	Medium strength	ANA worst case + UV
10	High strength	ANA + UV
11	High strength	AER + UV

AER Scenarios

AER Performance

When evaluating conventional aerobic treatment processes, two solids retention times and reactor operations for the conventional activated sludge basin were considered: high rate activated sludge (HRAS) with an SRT of 2 days (AER-HRAS), and conventional activated sludge with an SRT greater than 10 days and achieving nitrification (AER-NO₃). The influent parameters for each scenario were considered to be medium strength wastewater (Table 2). TSS and BOD removal in primary sedimentation for AER scenarios was considered to be 60% and 35%, respectively (Tchobanoglous et al., 2003). Corresponding COD and TOC values at primary sedimentation effluent were calculated from average BOD/COD and BOD/TOC of 0.50 and 1.00 for effluent after primary sedimentation (Tchobanoglous et al., 2003). No removal of TN and TP were considered in primary sedimentation. COD removal in HRAS process was assumed to be 70% as found in (Jimenez et al., 2015). Corresponding BOD and TOC values at HRAS effluent were calculated from ratios for BOD/COD and BOD/TOC of 0.2 and 0.35, respectively for final effluent (Tchobanoglous et al., 2003). Percent BOD reduction for HRAS was hence calculated and equal % removal of TSS was considered for HRAS, which was around 88%. Percent removal for nitrogen was calculated from Equation 1:

Equation 1. Total nitrogen in effluent for HRAS (Grady et al., 2011)

$$TN_{eff} = TN_{in} - 0.087 \times Y_{HRAS} \times i_{O_{XB,T}} \times COD_{removed}$$

Where:

$S_{N,a}$: Nitrogen in effluent (mg/l)

TN_{in} : Influent nitrogen concentration (mg/l)

Y_{HRAS} : Sludge yield for HRAS, 0.36 kg VSS /kg COD removed

$i_{O_{XB,T}}$: Mass of COD in solids, 1.42 g COD/g VSS

% removal for phosphorus was not accounted for in HRAS process. The turbidity value at HRAS effluent was calculated from TSS/turbidity ratio of 2.2 for settled

secondary effluent (Asano et al., 2007). COD and TSS removal of 50% and 35% were assumed during coagulation respectively (Diamadopoulos et al., 2007). However, experimental analysis need to be carried out to understand the exact behavior of organic matter removal for water reuse systems using alum coagulation. Once TSS value was calculated from percent removal, TSS/turbidity ratio of 2.2 was applied to calculate the turbidity at pre-filtration effluent (Asano et al., 2007). BOD and TOC value for filtered effluent were calculated using ratios of BOD/COD and BOD/TOC of 0.2 and 0.35 respectively (Tchobanoglous et al., 2003). No organic removal in granular media filtration was assumed. The turbidity of influent water at granular media filtration should be in the range of 5-7 NTU to get turbidity of < 2 NTU at filter outlet (Asano et al., 2007). Since the influent turbidity for filtration unit was assumed to always be < 7 NTU, the effluent turbidity criteria shall be taken care of. However, to specify the turbidity of filtered effluent, with coagulation and filtration unit, 98% turbidity is assumed according to (Adin and Asano, 1998).

For AER-NO₃ process, similar % removal of BOD and TSS is assumed in primary sedimentation. The BOD removal of 93% is assumed in bioreactor (Environmental Protection Agency, 1997) with nitrification. Corresponding COD and TOC values at bioreactor effluent were calculated using the ratios as stated for HRAS. Nitrogen in effluent of bioreactor was calculated from

Equation 2. Total nitrogen in AER-NO₃ effluent (Grady et al., 2011)

$$TN_{eff} = TN_{in} - S_{N,a}$$

Where,

TN_{eff}: TN in the effluent (mg/l)

TN_{in}: Influent TN to bioreactor (mg/l)

S_{N,a}: Nitrogen removed in the solids (mg/l)

Phosphorus removal in bioreactor was not assumed for this study. Similar percent removal of TSS and COD was assumed for coagulation, as assumed for HRAS system. For turbidity removal in granular media filtration, same assumptions were

considered as HRAS. The water treatment performance of these main configurations are summarized in Table 5 and

Table 6.

Table 5. Treatment performance for AER-HRAS Scenarios (no nitrification).

Parameters	Influent	Primary sedimentation effluent	HRAS effluent	Filtration pre-treatment effluent	Granular Media filtration effluent
pH	7.5	7.5	7.5	7.5	7.5
COD (mg/l)	430	250	75	37	37
BOD (mg/l)	190	125	15	7.5	7.5
TOC (mg/l)	140	125	42	20	20
TSS (mg/l)	210	85	10	7	7
TN (mg/l)	40	40	32	32	32
TP(mg/l)	7	7	7	7	7
Turbidity (NTU)	-	-	5	3.2	0.1

Table 6. Treatment performance for AER-NO3 Scenarios (nitrification).

Parameters	Influent	Primary sedimentation effluent	CAS-NO3 effluent	Filtration pre-treatment effluent	Granular Media filtration effluent
pH	7.5	7.5	7.5	7.5	7.5
COD (mg/l)	430	250	45	25	25
BOD (mg/l)	190	125	10	5	5
TOC (mg/l)	140	125	25	13	13
TSS (mg/l)	210	85	6	5	5
TN (mg/l)	40	40	7.5	7.5	7.5
TP (mg/l)	7	7	7	7	7
Turbidity (NTU)			3	1.8	< 0.1

Wastewater Processes

The amounts of oxygen required for aeration in the AER-HRAS and AER-NO3 scenarios were calculated using following equations (Grady et al., 2011):

Equation 3. Required oxygen for heterotrophs (Grady et al., 2011)

$$RO_H = F \times (S_{SO} + X_{SO} - S_S) \left[1 - \frac{(1 + f_d \times b_H \times \theta_C) Y_{H,T} i_{O,XB,T}}{1 + b_H \times \theta_C} \right]$$

Where:

F: Influent flowrate of wastewater (m³/day)

S_{SO}: Slowly biodegradable substrate (mg COD/l)

X_{SO}: Readily biodegradable substrate (mg COD/l)

S_S: Soluble substrate concentration (mg COD/l)

f_d: fraction of biomass leading to debris, 0.20 mg TSS/mg TSS

b_H: decay coefficient for heterotrophs, 0.22 /day

θ_C: Solids retention time

Y_{H,T}: Yield for heterotrophic biomass, 0.50 mg TSS/mg COD

i_{O,XB,T}: Mass of COD in solids, 1.20 mg COD/mg TSS

Equation 4. Required oxygen for nitrification (autotrophs) (Grady et al., 2011)

$$RO_A = F \times (S_{N,a} - S_{NH}) \left[4.57 - \frac{(1 + f_d \times b_A \times \theta_C) Y_{A,T} i_{O,XB,T}}{1 + b_A \times \theta_C} \right]$$

Where:

S_{N,a}: Influent nitrogen concentration

S_{NH}: Effluent ammonia concentration, assumed as 0 mg/l

b_A: decay coefficient for autotrophs, 0.12 /day

Y_{A,T}: Yield for autotrophic biomass, 0.20 mg TSS/mg N

Equation 5. Aeration energy (Tchobanoglous et al., 2003)

$$P_w = \frac{WRT_2}{29.7ne} \left[\left(\frac{P_2}{P_1} \right)^{0.283} - 1 \right]$$

Where:

P_w: Power required for aeration (kW)

W: Weight of oxygen flow (kg/s)

R: Engineering gas constant, 8.314 (J/mol K)

T₂: Effluent temperature (K)

P₂: Absolute effluent pressure (Pa)

P₁: Absolute influent pressure (Pa)

n: (k-1)/k, k is the specific heat capacity of air 8.14 J/kg K

e: Efficiency of compressor, 0.8

Water Reuse Processes

Processes downstream of mainstream anaerobic treatment included alum coagulation and granular media filtration. Two coagulants were considered for this study, alum and ferric chloride. However, aluminum salts have higher advantage than ferric salts for removal of organic matter. The nonsorbable fraction of DOC is typically higher for alum salts than ferric salts. Hence, at relatively low doses of alum, large amount of sorbable organic matter can be removed (Edwards, 1997). No specific model was used to calculate the coagulant dose due to insufficient, limited data. The Edwards model is used for TOC values less than 10 mg/l and therefore the model was not used for wastewater TOC values (Table 5 and Table 6). The experimental literature had an estimated alum dose for an effluent from a conventional activated sludge (nitrifying) process as 5 to 50 mg/l (Sheikh et al., 1990). This range was used for the ARE-NO₃ scenarios. Since the amount of alum needed depends on the amount of organic matter, the alum dose assumed for the AER-HRAS system was 1.5 times larger than the nitrifying process, which matches their difference in organic carbon removal efficiencies (Table 5 and Table 6). Resulting in an alum dose range of 8 to 85 mg/l for the AER-HRAS scenarios.

Deep-bed anthracite media filtration was considered for granular media filtration (Schimmoller and Kealy, 2014). Deep bed filters are used because they allow suspended solids in the liquid to be filtered to penetrate farther into the filter bed, hence extending the filter run length (Asano et al., 2007). Total filtration area required for filtration operation was calculated using total flow rate of wastewater into the filtration system and assumed filtration rate of 15 m/h (Kawamura, 1991). The filtration area of a single filter cell was assumed to be 35 m² (Asano et al., 2007). The total number of filters were calculated depending on the total filter area required and filter area of single cell. This number was checked using an empirical formula for

calculating number of filter with respect to flow rate to each filter (Kawamura, 1991). The dimensions for a single filter were based on typical length to width ratios of 3:1 for granular media filters (Kawamura, 1991). The production of anthracite media was included in the system boundary. Assuming a bulk density of 800 kg/m³, the amount of anthracite required was calculated from volume of anthracite in single filter. The media bed height of 1.5 m was assumed for calculating the volume of anthracite in a single filter (Asano et al., 2007). No replacement of anthracite media was assumed.

The energy needed for backwashing the filters was considered to calculate the energy consumption of filtration system. The backwash water flowrate was assumed to be 45 m/h (Kawamura, 1991). The head achieved during backwashing was assumed as 8 m (Kawamura, 1991). The duration was 10 minutes, twice a day, which was assumed to be typical for backwashing a deep-bed anthracite filters. Disinfection in the forms of chlorine and ultraviolet (UV) disinfection were considered. The chlorine dose was calculated based on the concentration of organic matter and ammonia present in the effluent. The chlorine demand considered for TOC was in the ratio of Cl₂: TOC = 1.5:1 and for ammonia was Cl₂: NH₃ = 8:1 (Howe et al., 2012). The UV dose was kept constant at 100 MJ/cm², based on regulations (Table 1). The energy required for UV disinfection was calculated depending on energy required per lamp and number of lamps required per MGD of wastewater flow. Energy required per lamp was assumed to be 3.2 kW/lamp and number of lamps per MGD of influent flow was assumed to be 2.5 lamps/MGD (Trojan Technologies, n.d.)

Solids Management

The solids production for AER-HRAS was assumed to be 0.36 kg/kg COD removed (Jimenez et al., 2015). The solids handling systems were similar for both AER scenarios. The solids characteristics were based on typical values (Tchobanoglous et al., 2003): waste activated sludge was 0.8% dry solids and specific gravity was 1.005;

primary sludge was 6% solids and specific gravity was 1.02. The secondary solids (waste activated sludge) were thickened before being combined with primary solids. The thickened sludge was 2.5% solids. The energy required for thickening was calculated to be 75,800 kWh/yr using energy model developed by (Guest, 2012). Acetonitrile was assumed to be a representative compound used as polymer during thickening, and the dose was assumed to be 3.5 g/kg dry solids (Tchobanoglous et al., 2003). The combined sludge was fed to an anaerobic digester. The volatile solids reduction during digestion was assumed to be 65% (Tchobanoglous et al., 2003). A portion of COD which is volatile in nature is accounted for the methane production, hence the mass of volatile solids is assumed to be equal to COD removed. (Tchobanoglous et al., 2003). Methane production was calculated assuming theoretical gas production yield of 0.40 m³ CH₄/kg COD removed (Tchobanoglous et al., 2003). The total electrical energy produced from methane was calculated assuming a 21% conversion efficiency from methane to electric energy (Kim et al., 2011). The energy consumed by anaerobic digester was calculated using 88.56 kWh of energy consumed per dry ton of solids fed to the anaerobic digester (Hospido et al., 2005). Percent solids of the anaerobic digestion effluent was assumed as 5% (Tchobanoglous et al., 2003), and these solids went to a centrifuge for dewatering. The final percent dry solids were assumed to be 20%. Acrylonitrile was also used as a polymer during dewatering, and the amount of polymer was 4 g/kg of dry solids (Tchobanoglous et al., 2003). Energy consumption by centrifuge was calculated as 224,900 kWh/yr (Guest, 2012).

The solids production for AER-NO₃ was assumed to be 0.45 kg/kg BOD removed (Tchobanoglous et al., 2003). Similar percent solids for secondary activated sludge and primary sedimentation were assumed as for HRAS scenario. However, due to different amount of solids yield for AER-NO₃, the energy consumption values

calculated for gravity belt thickeners and centrifuge changed for AER-NO₃. The energy consumption for thickener was calculated to be 63,500 kWh/yr (Guest, 2012). The energy consumption for centrifuge was calculated as 217,500 kWh/yr (Guest, 2012). All of the biosolids were land applied. Their nitrogen and phosphorus contents were used to calculate the offset of chemical fertilizers. For dewatered biosolids, 5.5% and 3% of nitrogen and phosphorus was assumed respectively (Sullivan et al., 2007). Biosolids application rate usually depends on plant-available nitrogen. Typical plant-available nitrogen is considered as 35% and phosphorus as 40% from biosolids application (Sullivan et al., 2007).

ANA Scenarios

ANA Performance

The water quality for ANA process is as given in Table 7. The influent was considered to be medium strength wastewater (Table 2). The COD removal for ANA was assumed as 92%, which was based on experimental data (Table 8). The corresponding value of BOD and TOC effluent for ANA was assumed considering the ratio of BOD/COD and BOD/TOC as 0.20 and 0.35 for final effluent (Tchobanoglous et al., 2003). These ratios were stated for aerobic effluent, however detailed organic experimental data for ANA effluent is not available, so these ratios were assumed to be similar. Further experimental analysis is needed to understand the exact composition of ANA effluent. Total nitrogen and phosphorus concentration were determined using experimental data from Table 8. COD and TSS reduction due to alum coagulation was assumed to be 50 % and 35%, respectively (Diamadopoulos et al., 2007). However, the exact percent removal of organic matter and solids shall need to be evaluated experimentally for ANA systems, due to expected difference in organic composition of ANA effluent. Turbidity was calculated using the ratio for TSS/turbidity of 2.2 (Tchobanoglous et al., 2003). The turbidity of influent water at

granular media filtration should be in the range of 5-7 NTU to get turbidity of < 2 NTU at filter outlet (Asano et al., 2007). Since the influent turbidity for filtration unit was assumed to be < 7 NTU, the effluent turbidity criteria was met. However, to specify the turbidity of filtered effluent, with coagulation and filtration unit, 98% turbidity is assumed according to (Adin and Asano, 1998).

Influent wastewater temperature and AnMBR operation temperature were assumed to be 35°C since the majority of experimental data is available for this temperature and the ANA effluent values were based on experimental data (Table 8). Energy for heating wastewater was not included in the scope of the project, but will be discussed qualitatively since environmental impacts will be larger due to this requirement.

Table 7. Treatment performance for ANA Scenarios

Parameter	Influent	ANA effluent	Pre-filtration treatment effluent	Granular media filtration effluent
pH	7.5	7.5	7.5	7.5
COD (mg/l)	430	35	17	17
BOD (mg/l)	190	7	< 5	< 5
TOC (mg/l)	140	20	10	10
TSS (mg/l)	210	11	7	7
TN (mg/l)	40	41	41	41
TP (mg/l)	7	6	6	6
Turbidity (NTU)		5	3	0.1

Wastewater Processes

Anaerobic membrane bioreactor (AnMBR) were considered for mainstream anaerobic process. Coupling of SRT and HRT was main design consideration to help achieve maintain high sludge concentrations in the reactor and also to decrease the reactor size. This can be achieved with using membrane technology along with anaerobic technology. The main advantages achieved with using AnMBR include high effluent

quality, rejection of microorganisms, retention of particulate and colloidal organics in the bioreactor (Lin et al., 2011; Martinez-Sosa et al., 2011; Mei et al., 2016). No primary sedimentation was considered upstream of the anaerobic bioreactor. AnMBR process were modeled using experimental data (Hu and Stuckey, 2006; Huang et al., 2008; Lin et al., 2011; Mei et al., 2016; Smith et al., 2014) as summarized in Table 8 and design literature (Grady et al., 2011; Guest, 2012; Tchobanoglous, 2014; Tchobanoglous et al., 2003). All experimental design values were used to develop ANA model considering values from Table 8.

For AnMBR operation, the minimum hydraulic retention time of 5 hours was considered because the range was 3 to 19 hours (Table 8) and a smaller value allows for smaller reactors highlighting the fact that the energy model for AnMBR considered the lower energy consumption. Experimental data on the effect of SRT on methane yield show that infinite SRT value helps achieve higher methane yield as compared to SRT of 30-100 days (Huang et al., 2011, 2008), so the operational assumptions were based on typical solids retention times (SRTs) around 200 days. Methane production and sludge yield were based on COD removal. COD percent reduction was assumed to be 92% of the influent COD concentration, which was an average value (Table 8). When sulfate is present in the wastewater, the amount of COD used for sulfate reduction was 0.89 g COD/g sulfate (Grady et al., 2011). The presence of sulfate in domestic wastewater reduces the COD available for methanogens because sulfate serves as an electron acceptor for sulfate reducing bacteria that consume the COD.

Table 8. Experimental data analysis for multiple experimental AnMBR systems, operated at mesophilic conditions.

	1 (Mei et al., 2016)	2 (Martinez- Sosa et al., 2011)	3 (Hu and Stuckey, 2006)	4 (Huang et al., 2011)	5 (Lin et al., 2011)
Type of wastewater	Synthetic WW	Urban WW + glucose	Synthetic WW	Synthetic WW	Domestic WW
Influent COD, mg/l	400	630	460	550	435
COD reduction, %	90%	87%	91%	97%	90%
TSS reduction, %	-	-	-	-	99.7%
TN reduction, %	-	-10%	-	-	4%
TP reduction, %	-	13%	-	-	12%
SGD, m3/m2/h	0.49	-	3.00	-	0.5
COD mass loading per filtration area, g COD/day/m2	77.34	78.86	110.40	68.75	104.33
Methane yield, l CH4/g COD removed	0.30	0.27	0.25	0.25	0.24
TMP, kPa	-	17.70	40.00	16.00	-
Temperature of influent, C	35	35	35	35	30
Temperature of AnMBR, C	35	35	35	35	30
BOD reduction, %	-	94%	-	-	-
HRT, hrs	5.2	19.2	3	8	10
SRT, days	50	-	-	infinite	-
COD mass loading, g COD/day	57	158	11.04	8.25	62.57
Organic loading rate, gCOD/l.day	1.64	0.85	3.68	1.65	1.00
Flux, LMH	9.65	7.00	8.13	7.90	12.00
Effective membrane filtration area, m2	0.74	3.50	0.10	0.12	0.60
Biogas circulation rate, m3/h	0.36	0.81	0.30	0.03	0.30
% dissolved methane	14%	23%	29%	29%	31%
Energy produced, kWh/m3	0.26	0.40	0.31	0.39	0.29

Organic loading rate was calculated based on influent flow (Q) and COD concentration (COD_{in}). For medium strength wastewater, the organic loading rate of 2.06 g COD/l/day was used considering an HRT of 5 hours. Typical OLR values used in literature vary from 0.85 to 3.68 g COD/l/day (Table 8) depending on corresponding HRT values. The OLR value chosen for this study was between the experimental range (Table 8). The membrane flux was assumed to be 10 L/m²/h (Smith et al., 2014), and typical values are from 9 to 12 L/m²/h (Huang et al., 2011; Lin et al., 2011; Mei et al., 2016). The specific gas demand (SGD) was assumed to be 0.23 m³/m²/h (Smith et al., 2014). The average experimental value for trans-membrane pressure (TMP) was assumed 27.5 kPa (Table 8). The total effective membrane area was the average value of 88 g COD/day/m², based on the experimental data range of 70 to 110 g COD/day/m² (Table 8). To calculate the permeate pumping flow rate, a single tank of AnMBR was assumed with 225 number of ZeeWeed 500-D cassette were considered, each had 48 number of ZeeWeed modules (Technologies, 2016). Dry weight of single module given was 28 kg. Assuming this dry weight to be the only weight of module, the total polymer weight required for AnMBR system was 302.4 tons. The membrane used for AnMBR system was Polyvinylidene difluoride (PVDF). The membrane life assumed is up to 10 years (Smith et al., 2014). Considering 40 years of timeframe and 10 year for membrane lifetime (Smith et al., 2014), the membranes were assumed to be replaced every 10 years. Hence the total polymer required for membrane was about 1210 tons. Biogas circulation rate for system was than calculated depending on SGD and effective membrane area provided.

Chemicals

Alkalinity was added in the form of calcium carbonate (CaCO₃) at the inlet of AnMBR to increase alkalinity and ensure proper microbial functioning. An alkalinity of 2000 to 4000 mg/l as CaCO₃ is required to maintain a neutral pH during ANA processes.

The amount of lime was calculated using design parameters in Table 9 (Tchobanoglous, 2014). It was assumed that the biogas would have a gas phase carbon dioxide of 25% (Hu and Stuckey, 2006). For 35°C and 25% CO₂, the minimum alkalinity required is 1609 mg CaCO₃/l; lime was added to supplement the raw wastewater alkalinity to meet this minimum value. The chemicals required for cleaning of membranes in AnMBR were sodium hypochlorite and citric acid. The sodium hypochlorite (12.5% solution) dose was 2.2 l/yr/m³/d and citric acid (100% solution) dose was 0.6 l/yr/m³/d (Shoener et al., 2016).

Table 9. Estimated minimum alkalinity as CaCO₃ required to maintain pH AT 7.0 as a function of temperature and % carbon dioxide gas (Tchobanoglous, 2014).

Temperature, °C	Gas phase, CO ₂ %			
	25	30	35	40
20	2040	2449	2857	3265
25	1913	2295	2678	3061
30	1761	2113	2465	2817
35	1609	1931	2253	2575
40	1476	1771	2066	2362

Energy model for AnMBR

Electrical energy production from methane was based on methane from biogas and dissolved methane (recovered during degasification). The average, experimental methane production as biogas was assume to be 0.26 l CH₄/g COD removed (Table 8). Since this value is less than theoretical methane production (0.40 l CH₄/g COD removed, at 1 atm and 35°C) (Tchobanoglous, 2014) and dissolved methane is a significant portion of methane produced (Gimenez et al., 2011; Martinez-Sosa et al., 2011; Smith et al., 2014) it was assumed that the AnMBR achieved theoretical methane production was produced and the amount not accounted for with biogas was dissolved. The concentration of dissolved methane was calculated using:

Equation 6. Dissolved methane concentration (Smith et al., 2014;

Tchobanoglous et al., 2003)

$$CH_{4diss} = \left(\frac{P_{CH_4}}{H_{CH_4}} \right) \times MW_{CH_4} \times M_{solution} \times 1.5 \times 1000$$

Where:

CH_{4diss} : Dissolved methane in effluent (mg/l)

P_{CH_4} : Partial pressure of methane, 0.55 atm (biogas composition is considered as 55% methane (Gimenez et al., 2011))

H_{CH_4} : Henry's constant of methane at 35°C, (49060 atm)

MW_{CH_4} : Molecular weight of methane (16 g/mol)

M_{sol} : Molarity of solution (55.6 mol/l)

1.5 is the oversaturation constant for methane (Smith et al., 2014)

1000 is the conversion of g to mg

The amount of dissolved methane per day was estimated using the density of methane as 0.66 kg/m³. Due to the global warming impact of methane, the recovery of dissolved methane is an important global issue. This study used degasifying membranes to simulate dissolved methane recovery (Bandara et al., 2011); and 35% recovery of dissolved methane has been observed experimentally for medium strength wastewater (Bandara et al., 2011). Different technologies for recovery of dissolved methane from effluent include physical gasification based on gas-liquid equilibrium, and mixing with gas or paddle and biological oxidation of dissolved methane (Bandara et al., 2011). However, for high wastewater strength with influent COD more than 1000 mg/l, more than 80% recovery of dissolved methane was observed (Bandara et al., 2011). Energy content of methane was considered to be 35.845 kJ/l CH₄ (Shoener et al., 2014) and the electrical energy conversion efficiency was 21% (Kim et al., 2011).

Energy consumption for AnMBR system consist of the following components:

1. Energy required for membrane scouring (E_s): Membrane scouring energy was calculated to be 0.21 kWh/m³ (Tchobanoglous et al., 2003). The specific gas

demand for membrane scouring was assumed to be 0.23 m³/m²/h (Smith et al., 2014).

2. Pumping energy requirements (E_p): The permeate pumping energy was calculated depending on trans-membrane pressure (TMP) across the membrane. The average TMP was assumed to be 27.5 kPa, a value obtained from experimental data (Hu and Stuckey, 2006; Martinez-Sosa et al., 2011; Shoener et al., 2014).
3. Mechanical mixing of AnMBR (E_M): Mechanical mixing in the anaerobic basin was considered to be in range of 8 to 13 kWh/m³ (Tchobanoglous et al., 2003). The average of 10.5 kWh/m³ was assumed for baseline scenario.
4. Energy required for dissolved methane recovery (E_{DM}): The average value of 0.035 kWh/m³ of dissolved methane recovered was assumed as per range observed in (Seib et al., 2016).

The influent temperature of wastewater was assumed to be 35°C, hence no energy consumption for heating of influent wastewater can be observed here. However, a considerable amount of energy is required for heating of the influent to mesophilic temperature to achieve anaerobic process. The model was run at different temperatures of influent wastewater for ANA system to study the effect of temperature on system analysis.

Table 10. Effect of influent temperature on energy consumption for heating of influent wastewater to 35°C.

System	Influent temperature (°C)	Energy associated with heating of influent (kWh/m ³)
ANA ₁₅	15	23.23
ANA ₂₅	25	11.62
ANA ₃₅	35	0.00

Water Reuse Processes

The downstream of mainstream anaerobic process consisted of alum coagulation, granular media filtration, and disinfection. The modeling assumptions were the same as the AER scenarios except for the following. Due to presence of high organic matter and high alkalinity from anaerobic effluent, the coagulant dose is expected to be more than as compared to aerobic effluent. The TOC for AnMBR effluent was 10 times higher than AER-NO₃ effluent (

Table 6 and Table 7), hence the alum dose for AnMBR effluent was assumed in the range of 50 to 500 mg/l. The baseline ANA scenario considers 50 mg/l. Due to the high amount of TOC and alum dose, in-line filtration was not considered as the coagulated organic matter would clog the downstream filter media. Hence, a sedimentation tank was provided. This chemical sludge was not considered with solids handling system for biosolids production due to high amount of chemical present in the sludge. The chemical sludge was disposed of and not consider in the system boundary since both systems will have sludge from backwashing and assumed to have similarly small contributions to overall environmental impacts compared to the other unit processes.

Solids Management

The sludge yield for AnMBR was very small, so lime stabilization after dewatering was used instead of anaerobic digestion (Smith et al., 2014). The sludge yield of 0.08 g MLSS/g COD removed was considered for the AnMBR (Tchobanoglous et al., 2003). Data on AnMBR sludge characteristics study are limited, so the values required for modelling of the solids handling system were assumed to be that of aerobic waste activated sludge process without sedimentation (Tchobanoglous et al., 2003): 1.3% dry solids with a specific gravity of 1.005. This sludge was then fed to a centrifuge for dewatering (same energy estimates as AER). Since the centrifuge influent solids were much lower than in the ANA scenarios, the dewatered cake was assumed to be 15%,

average of 10-20%, since the influent dry solids concentration was between 0.8-2.5% (Tchobanoglous et al., 2003). The amount of polymer used was also larger for ANA compared to AER. A value of 5 g/kg of dry solids has been used previously for AnMBR sludge range (Smith et al., 2014), so that was averaged with typical sludge requirements (1-7.5 g/kg of dry solids (Tchobanoglous et al., 2003)) to get a dose of 4 g polymer/kg dry solids.

Due to less amount of sludge produced in ANA compared to AER, anaerobic digestion was not used as stabilization process for ANA scenario. Also aerobic digestion, which will include significant amount of aeration energy was not considered for ANA scenario, as the main aim for this study was to maximize energy recovery from wastewater. Instead post-lime stabilization was used which does not include any energy intensive parameter. A typical lime dose of 300 g Ca(OH)_2 /kg dry solids was used (Tchobanoglous et al., 2003). Quicklime was used for chemical addition, because the exothermic reaction of quicklime and water can raise the temperature of mixture of above 50°C, sufficient to deactivate worm eggs. According to the stoichiometric equation of quicklime and calcium hydroxide, 1 kg of quicklime equivalents to 1.32 kg of calcium hydroxide (National Lime Association, 2016). Hence using 190 g Ca(OH)_2 /kg dry solids, equivalent amount of quicklime per day was calculated.

All of the biosolids were land applied as in the AER scenarios. The amount of nitrogen and phosphorus content in biosolids achieved were assumed to be different from AER sludge: 1.5% (average of range, 1 to 2%) nitrogen and 1% phosphorus (Sullivan et al., 2007). Typical plant-available nitrogen in biosolids was assumed the same as AER (35%), but the percent of phosphorus that is plant available was smaller (36% instead of 40%). The addition of lime, to increase AnMBR alkalinity and for solids stabilization, decreases the solubility of phosphorus in biosolids (Sullivan et al., 2007). The exact reduction in plant available phosphorus is not known, so a 10%

reduction was assumed (and the impact of this assumption need to be tested during the uncertainty and sensitivity analysis).

Operational Uncertainty

Three cases were studied for ANA bioreactor; (i) best case (with maximum energy production and minimum energy consumption), (ii) base case (according to baseline scenario) and (iii) worst case scenario (minimum energy production and maximum energy consumption).

Table 11. Operational uncertainty parameters for best case, base case and worst case ANA system

Parameters	Units	Best case	Base case	Worst case
COD reduction	%	97	92	87
TN reduction	%	4	-3	-10
TP reduction	%	13.4	13	11.6
TMP	kPa	15	27.5	40
SGD	m ³ /m ² /hr	0.1	0.23	1.2
COD mass loading per filtration area	g COD/day/m ²	110.4	88	70
Methane yield	L CH ₄ /g COD removed	0.3	0.26	0.24
Mixing energy in bioreactor	kWh/m ³	8	10.5	13
Recovery of dissolved methane	%	80	35	20
Energy consumed by dissolved methane recovery system	kWh/m ³	0.02	0.035	0.05
Sludge production	kg solids/kg COD removed	0.04	0.08	0.12

The range for COD, TN and TP reduction was assumed from different experimental studies (Table 8). The operational parameters like TMP, COD loading per filtration area and methane yield were also considered from the same experimental analysis done previously in literature (Table 8). SGD was assumed from values found by (Smith et al., 2014). The energy required for mixing in anaerobic bioreactor and for

recovering dissolved methane were both assumed from (Tchobanoglous et al., 2003) and (Seib et al., 2016) respectively. Dissolved methane recovery was assumed considering maximum 80% recovery with high strength wastewater from (Bandara et al., 2011). The sludge production from ANA bioreactor was assumed from (Tchobanoglous et al., 2003).

Alum coagulation for removal of organic matter

The organic matter in the effluent of ANA bioreactor is considered to be soluble due to membrane technology. (Kastl et al., 2004) talks about enhanced coagulation to be one of the best available technology for removal of dissolved organic matter from effluent. According to (Diamadopoulos et al., 2007) the filtered UASB effluent can achieve COD removal of 50 to 70%. The similar removal efficiency for ANA effluent was considered, as no data was available for ANA effluent coagulation. Experimental analysis for removal of ANA effluent organic matter need to be studied to have clear understanding of COD removal via coagulation.

For current study, alum coagulation was assumed to be in the range of 50 to 500 mg/l of alum according to (Diamadopoulos et al., 2007). As no data for ANA was achieved from literature, this values were considered for the model development.

CHAPTER IV: RESULTS

Data and graph interpretation: For each environmental impact graph, the data has been normalized to the stated scenario (baseline scenario); normalization is within each impact category and the magnitudes between categories is not relevant when using TRACI, a midpoint impact assessment method. To interpret this normalized data: within a category, if a scenario has a value greater than one, then that scenario has more/worse environmental impacts than the baseline scenario. If a scenario has a value less than one, then that scenario has smaller/better environmental impacts than the baseline scenario. If a scenario has a negative value (less than zero), then that scenario resulted in an environmental benefit in that category.

AER Scenarios

At first, the AER scenarios with the best environmental performance were evaluated by comparing the following 4 scenarios: AER-HRAS+UV, AER-HRAS+Cl, AER-NO₃+UV, and AER-NO₃+Cl. Figure 4 shows that HRAS systems perform better than conventional activated sludge systems that achieve nitrifications, using the same disinfection technology. This is due to high RO and lower solids (which means lower energy recovery during anaerobic digestion) for scenarios with nitrification (AER-NO₃+UV and AER-NO₃+Cl), as shown in Figure 5. Figure 5 shows the global warming potential (GWP) impact of all scenarios as a representative category. It shows that more than 95% of GWP impact is due to aeration energy for all but the AER-HRAS-Cl scenario, and that the nitrification scenarios had impacts that were 4 times as large as the HRAS systems due to the increased oxygen demand of nitrification. The AER-HRAS-Cl scenario had large impacts due to chlorine disinfection because of high effluent ammonia concentrations and corresponding chlorine demand. Also, since the offset of fertilizer production due to nonpotable reuse water irrigation was not included in the system boundary, the AER-nitrification

scenarios would even have large relative impacts since the HRAS system would have additional environmental benefits due to these offsets. Therefore, the baseline AER scenario used for comparison was the HRAS system that employed UV disinfection.

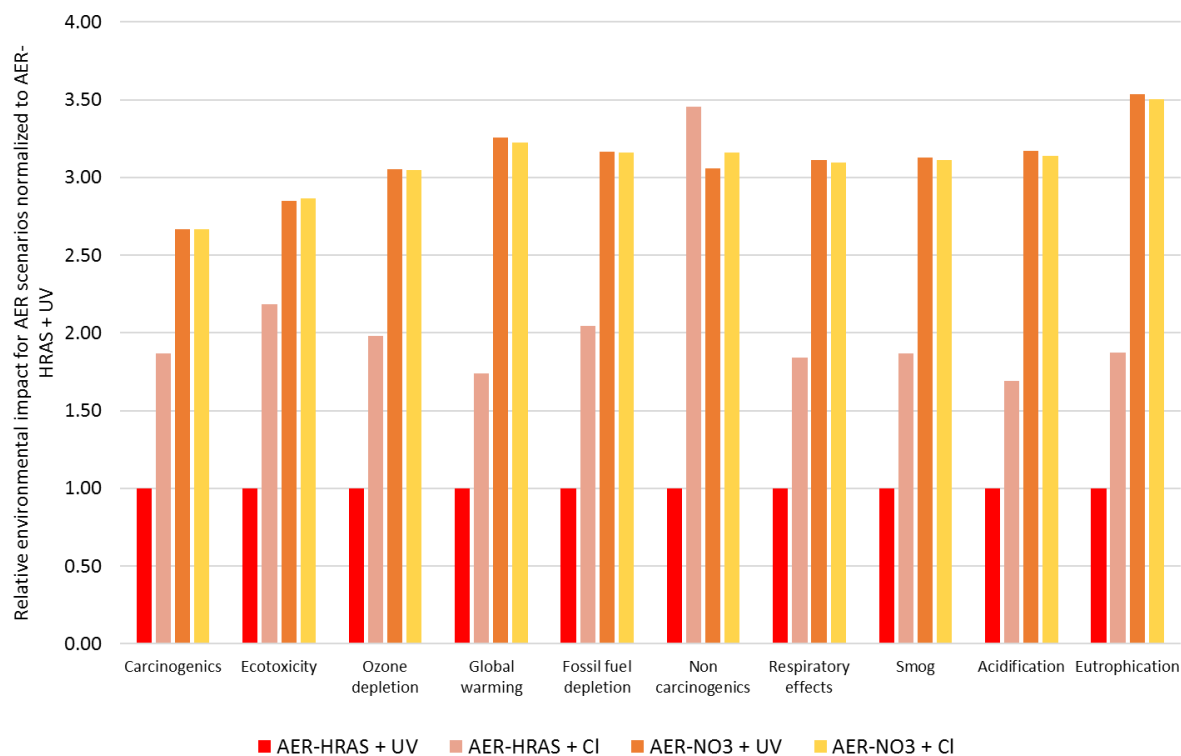


Figure 4. Comparison of four AER scenarios (with or without nitrification; chlorine or UV disinfection).

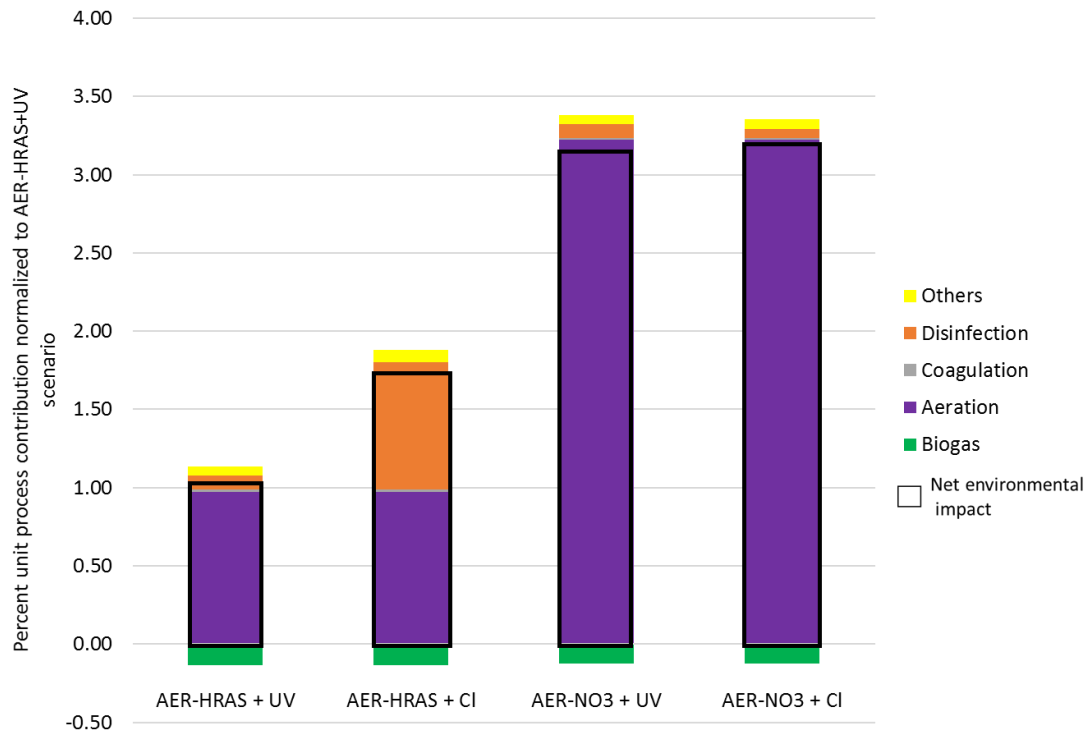


Figure 5. Percent unit process contribution for GWP for all AER scenarios normalized to AER-HRAS+UV. The black box shows the net environmental impact.

ANA Scenarios

Two main ANA scenarios were evaluated to determine which would have the best environmental performance (under typical conditions) and serve as the ANA baseline. The only difference was the type of disinfection: chlorine or UV disinfection. Figure 6 shows that coupling a mainstream anaerobic process with chlorine disinfection can have large environmental consequences due to the high chlorine demand of ammonia, which was the same trend seen with the AER-HRAS scenarios. Therefore, the baseline ANA scenario will use UV disinfection.

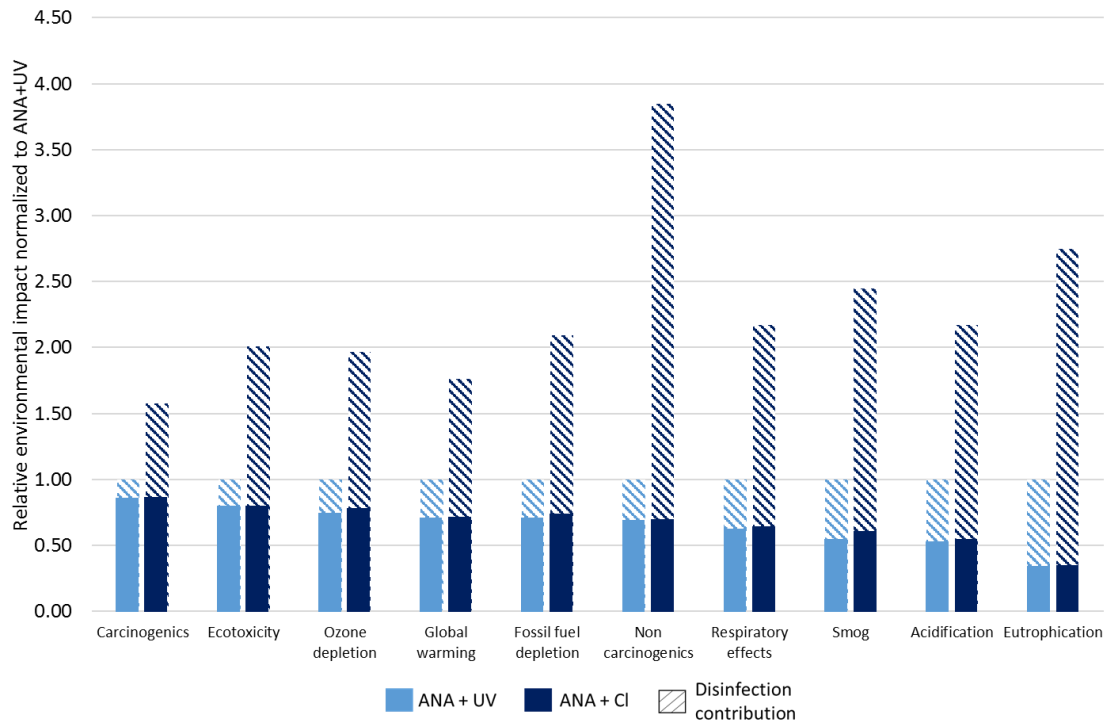


Figure 6. Disinfection contribution for anaerobic scenarios normalized to ANA+UV.

ANA vs. AER

Figure 7 shows the comparison of the baseline ANA and baseline AER scenarios. AER was found to be best in 6 out of 10 TRACI categories (carcinogenics, ecotoxicity, ozone depletion, global warming potential, fossil fuel depletion and non-carcinogenic) as compared to ANA. ANA was found to be best in the remaining 4 categories (respiratory effects, smog, acidification and eutrophication). Out of the 10 impact categories, AER had the smallest relative carcinogenic impact (50% of ANA's carcinogenic impact) and the largest relative eutrophication impact (160% of ANA's impact). The contribution of all processes to these two categories is shown below to help explain the overall trends between ANA and AER baseline scenarios.

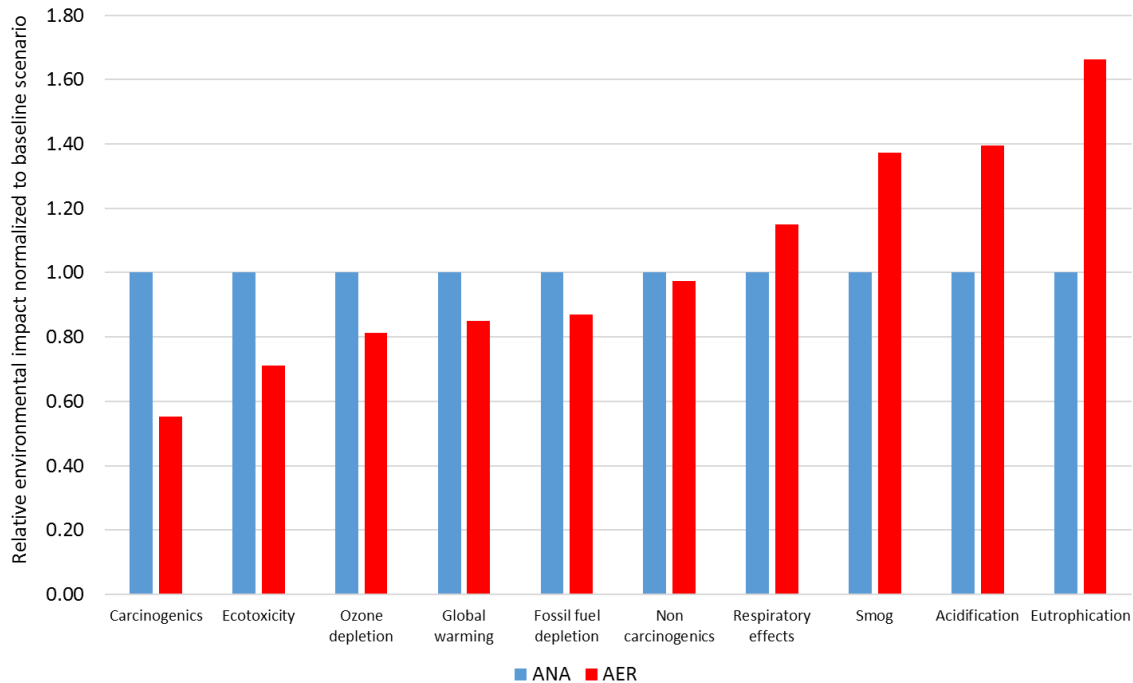


Figure 7. Comparison of ANA and AER shows the impact associated with AER to be best in 6 out of 10 TRACI categories.

Figure 8 shows the contribution of each process to carcinogenic impacts. ANA was worse due to impact of alum coagulation, followed by energy associated with membrane scouring for AnMBR and operational energy for AnMBR which includes influent and effluent pumping, mechanical mixing, energy for degasification, chemicals (lime, sodium hypochlorite and citric acid) required during AnMBR processes and membrane material (Polyvinylidene fluoride). The environmental benefit was achieved due to energy production during mainstream anaerobic process. The AER scenario had lower environmental impacts than ANA scenario due to better water quality and a correspondingly lower alum dose. The figure also shows that the AER aeration energy had the largest contribution to carcinogenics, followed by coagulation. An environmental benefit due to energy production from anaerobic digestion of solids was achieved for AER scenario.

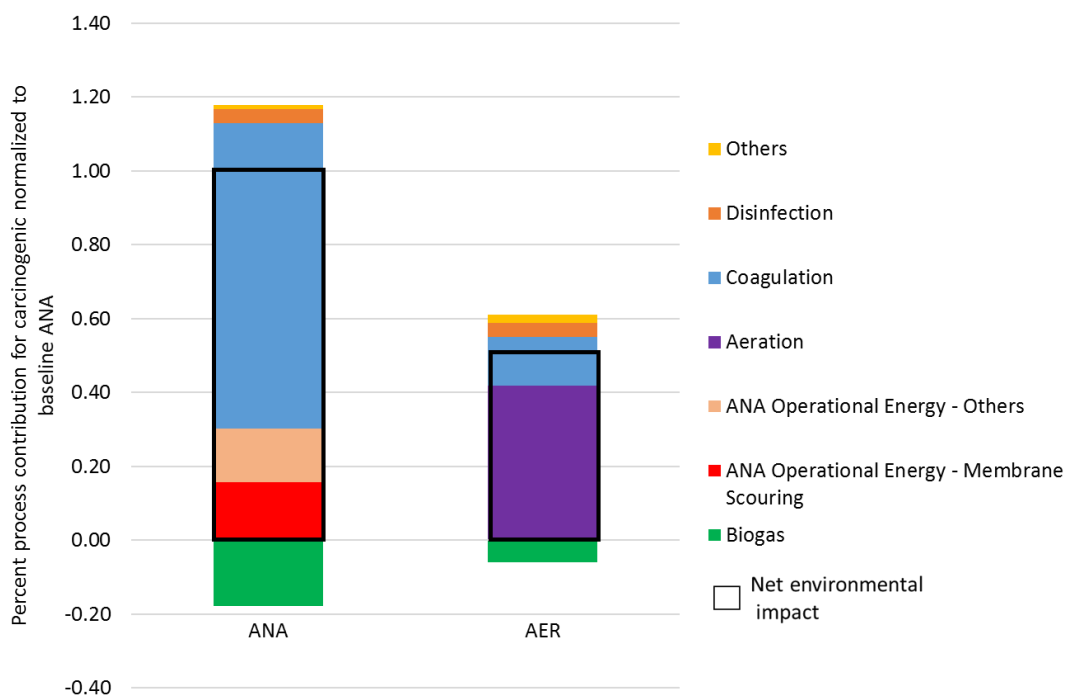


Figure 8. Percent process contribution for carcinogenic impact category for ANA and AER normalized to baseline ANA. The black box shows the net environmental impact.

Figure 9 shows the process contribution for the eutrophication impact. Unlike the carcinogenic impact for the ANA which had alum coagulation contribution to be large, eutrophication had the largest contribution due to membrane scouring, followed by operational energy associated with AnMBR, coagulation, and UV disinfection. Environmental benefits included energy production and fertilizer offset associated with biosolids land application. For the AER scenario, aeration was still the main contributor to impacts, followed by UV disinfection. There were environmental benefits due to fertilizer offset and biogas production during solids anaerobic digestion. Fertilizer offset for AER was much higher than ANA because the AER-HRAS system results in higher solids production than the ANA systems. These comparison does not clearly mention which scenario can be environmentally beneficial, hence we looked at different scenarios and operations to evaluate the ANA technology.

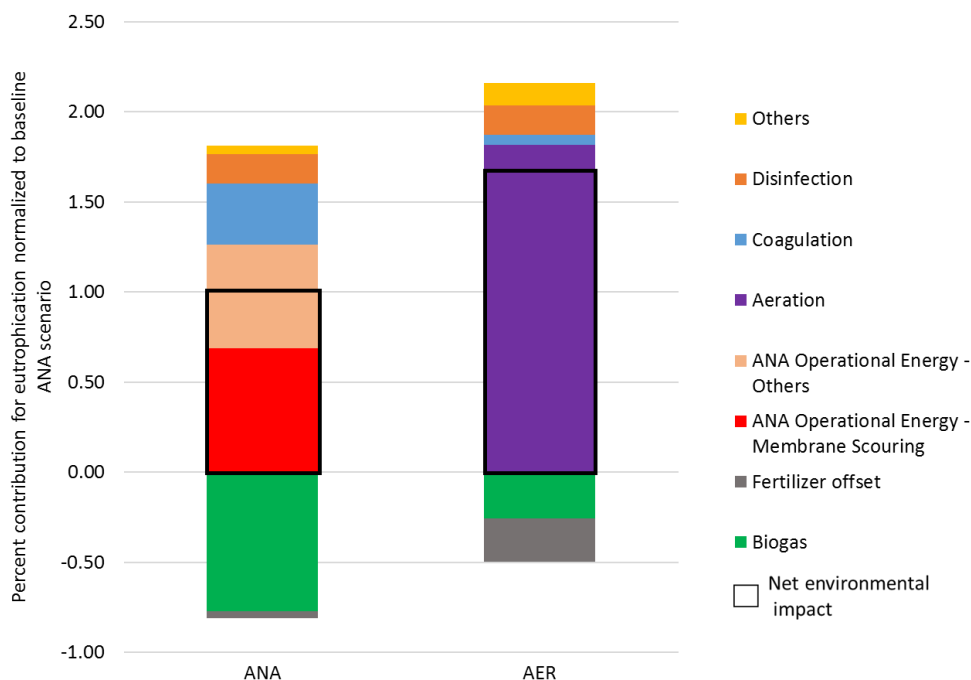


Figure 9. Percent process contribution for eutrophication impact category for ANA and AER normalized to baseline ANA.

Dissolved Methane

The unit process breakdown for global warming potential was considered again in Figure 10 as a representative category; it shows that dissolved methane emissions have a large contribution for ANA scenarios. The impact of methane is about 25 times worse than carbon dioxide. Therefore, another scenario was analyzed that looked at an increase in the amount of dissolved methane that is recovered from the effluent. When dissolved methane recovery was increased from 35% (baseline) to 80% (max CH₄ recovery), the importance of recovering dissolved methane becomes clear. The impact caused due to emission of dissolved methane in effluent is reduced by nearly 80% and the net GWP impact decreases by around 50% when recovery of dissolved methane is increased from 35 to 80%; this large change is due to fewer dissolved methane emissions and increased energy production from the recovered dissolved methane. The max CH₄ recovery scenario not only decreases the overall GWP impact

as compared with baseline ANA scenario, but also decreases the impacts of all the other nine categories due to increased energy production.

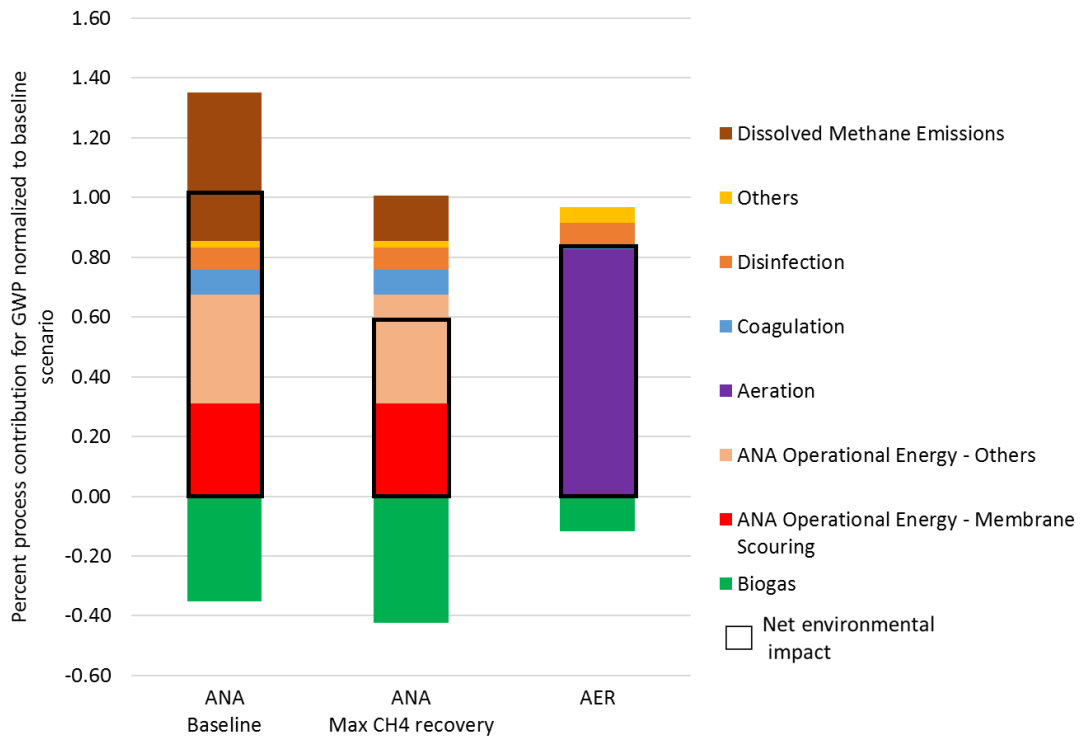


Figure 10. Percent process contribution for GWP for maximum dissolved methane recovery. The black box represents the net environmental impact.

Temperature model for ANA system

Energy consumption associated with varying influent temperature for ANA system can be found in Figure 11. As compared to baseline scenario of ANA for influent temperature of 35°C, the system with influent temperature as 15°C can be found to have maximum environmental impact as compared to system with 25°C as influent temperature. The main reason associated with this increase in environmental impact is due to additional amount of energy required to heat the influent temperature till 35°C to achieve mesophilic condition for proper anaerobic operation.

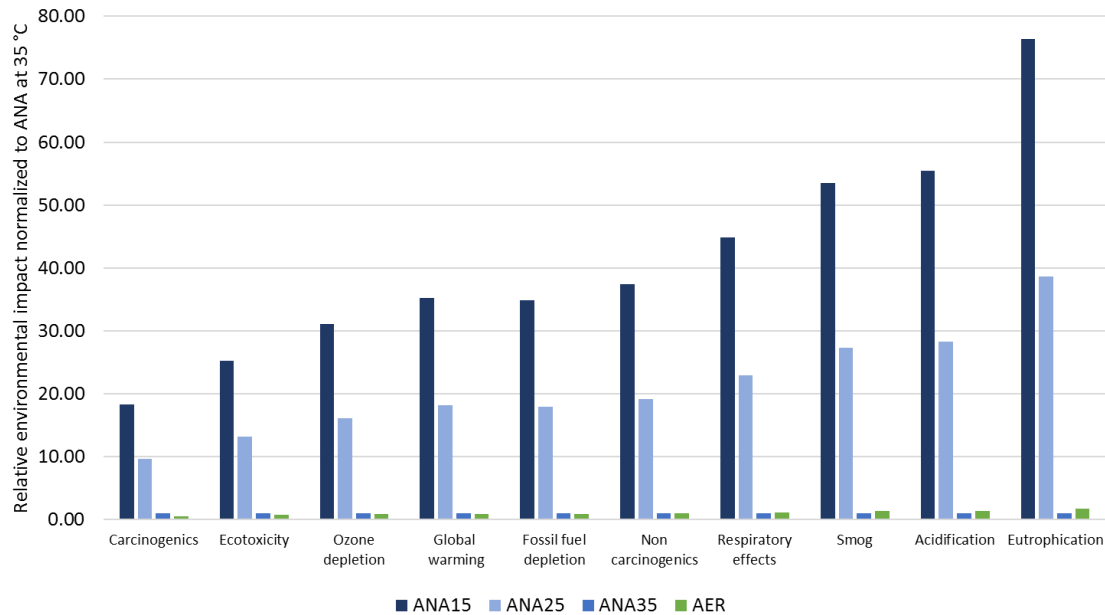


Figure 11. Effect of varying influent temperature on ANA energy consumption

AnMBR Operational Uncertainty

The next scenarios analyzed were “best case” and “worst case” ANA scenarios that focused on uncertainty in operating an AnMBR and understanding the importance of operating the AnMBR system. The “best case” scenario evaluated maximum energy production (biogas and maximum dissolved methane recovery from AnMBR effluent, as observed experimentally (Table 8) with minimum AnMBR energy consumption as well as minimum sludge yield, which means minimum energy consumption for solids handling. As expected, the environmental impacts for the ‘Best Case – ANA’ scenario (Figure 12) were lower than the baseline ANA scenario. The “worst case” ANA scenario had minimum energy production (minimum methane yield with minimum dissolved methane recovery) as well as maximum energy consumption and sludge yield.

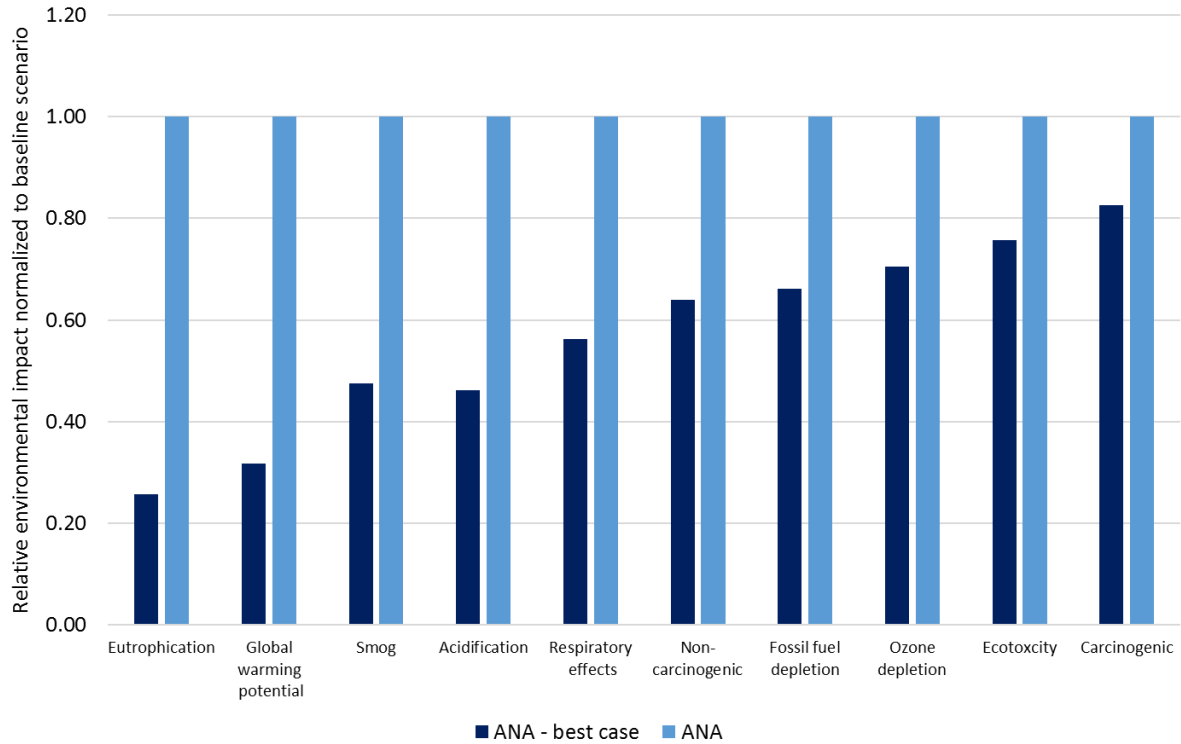


Figure 12. Comparison of 'Best Case - ANA' with baseline ANA scenario.

To evaluate all three ANA scenarios, they were normalized to the AER baseline scenario (red line) in Figure 13. As observed in Figure 13 the worst case ANA scenario had highest environmental impacts in all 10 categories (i.e., it was worse than baseline ANA and baseline AER scenarios). In addition, it can be seen that the best case ANA scenario (maximum energy production, minimum energy requirement and sludge) has the best environmental performance in 8 out of 10 impact (i.e., it is better than the baseline AER scenario in the majority of categories). This is sufficient to understand the importance of operating AnMBR system efficiently to make it comparable with the conventional technology for developing sustainable water reuse system. Optimized operation of an ANA system has potential to be an environmentally sustainable alternative to conventional AER systems (Figure 13).

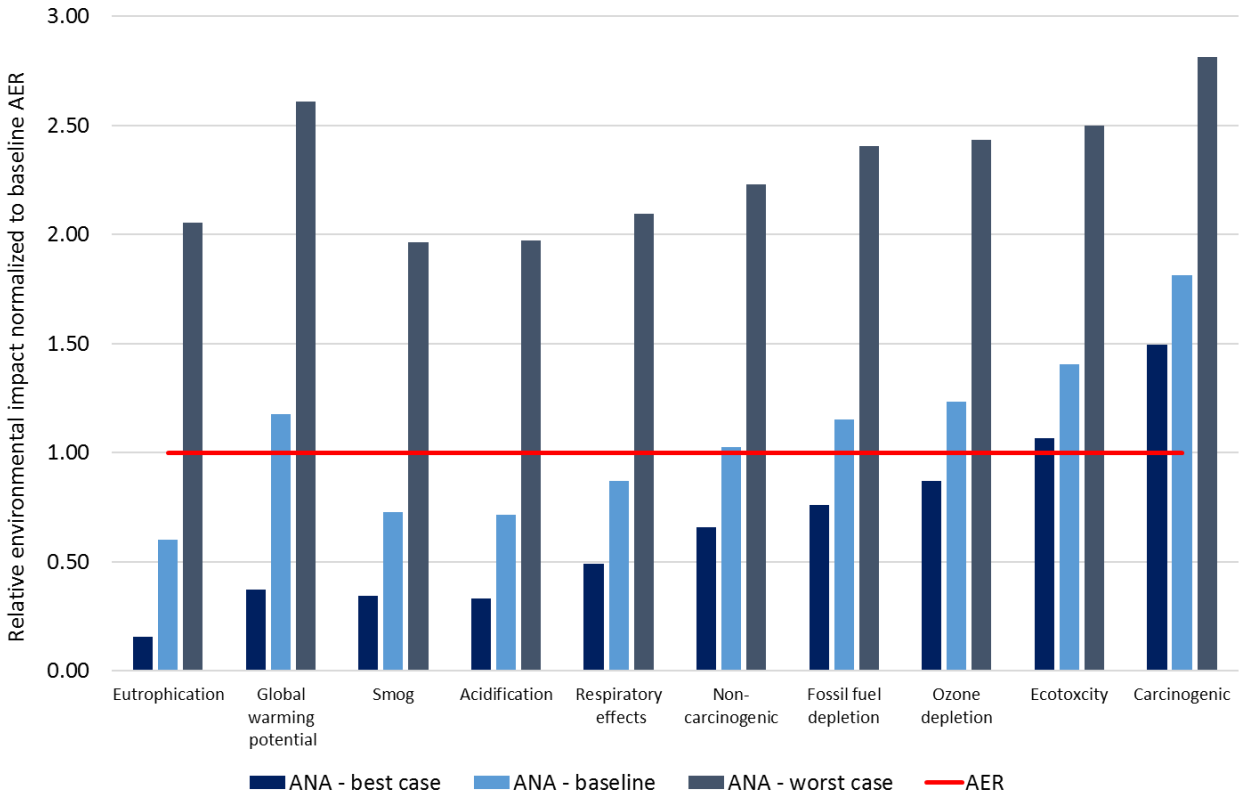


Figure 13. Comparison of ANA-baseline, ANA-best case and ANA-worst case with AER

High Strength Wastewater

The impact of water quality was studied by applying this mathematical model to high strength wastewater. When treating high strength wastewater, the results initially showed that the ANA system had worse impacts than AER in all 10 impact categories (Figure 14). This was due to a high, assumed alum dose for the AnMBR effluent. The AER scenario also had increased impacts due to aeration, but they were not as pronounced and the impact due to ANA coagulation (Figure 14). For Figure 14, the dark blue hatched line shows the contribution impact due to high dose of alum coagulation for efficient removal of organic matter for ANA system. While red hatched lines show the impact due to high amount of energy required for aeration for AER system. The alum dose assumed for the ANA high strength wastewater scenario was 500 mg/l, which was the highest alum dose found in literature. However, future

experimental work to develop the coagulation model for ANA effluent system will improve this evaluation since the results are very sensitive to this dose assumption.

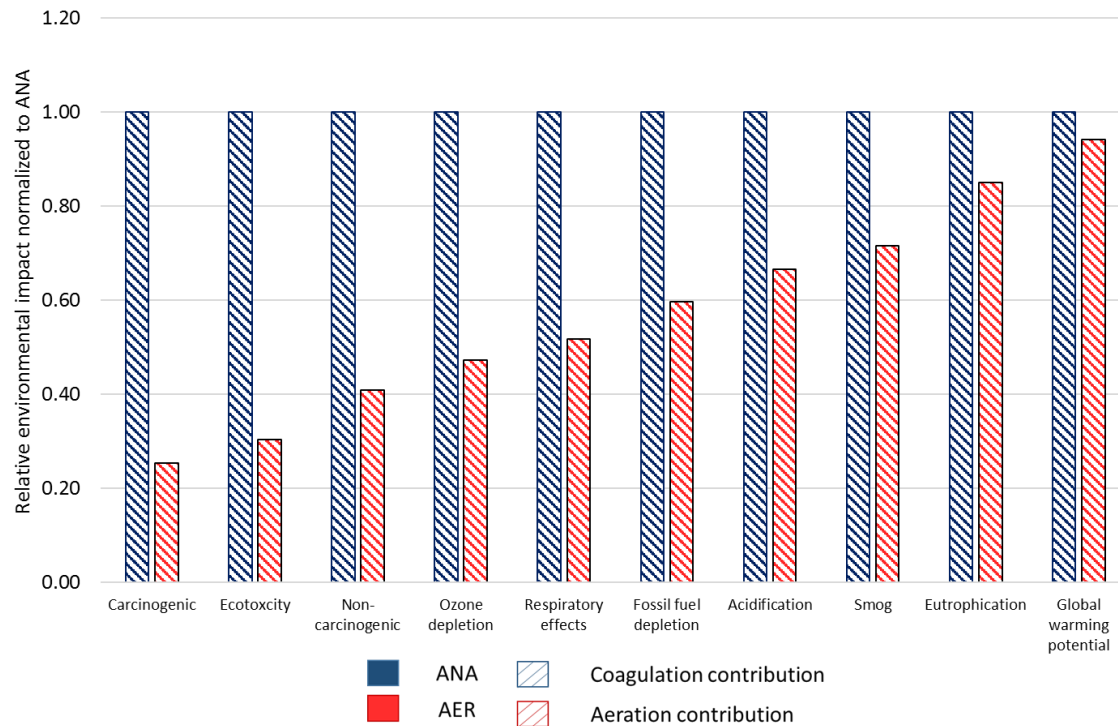


Figure 14. Contribution due to aeration and coagulation on AER and ANA high strength water reuse systems.

To help understand what alum doses for ANA system will be sufficient to have similar or less environmental impact than AER, we evaluated the impact of alum dose on ANA impacts for carcinogenics (Figure 15a), which had the largest contribution from coagulation, and global warming potential (Figure 15b), which had the smallest contribution. As observed in Figure 15, an ANA alum dose of 120 mg/l would result in the same carcinogenic impacts of the AER scenario (with an alum dose of 85 mg/l). The breakeven ANA alum dose for global warming impacts, though, was 400 mg/l since the ANA and AER global warming impacts were similar and the alum contribution to impacts was smaller (so a small change makes a small difference, where a small change of alum dose for carcinogenics made a large difference). Hence,

an alum dose of 120 mg/l to 400 mg/l was found to have shown similar environmental impact for ANA as compared to AER system for high strength wastewater. Figure 16 shows the comparison between high strength wastewater treatment when using ANA or AER main stream treatment, when the ANA alum dose was 120 mg/l. Figure 16 shows that this smaller alum dose resulted in the ANA scenario to be better than the AER scenario in all 10 categories. However, future experimental research is needed to determine the actual alum dose needed since the assumed alum doses might not efficiently remove all required organic matter from the effluent stream.

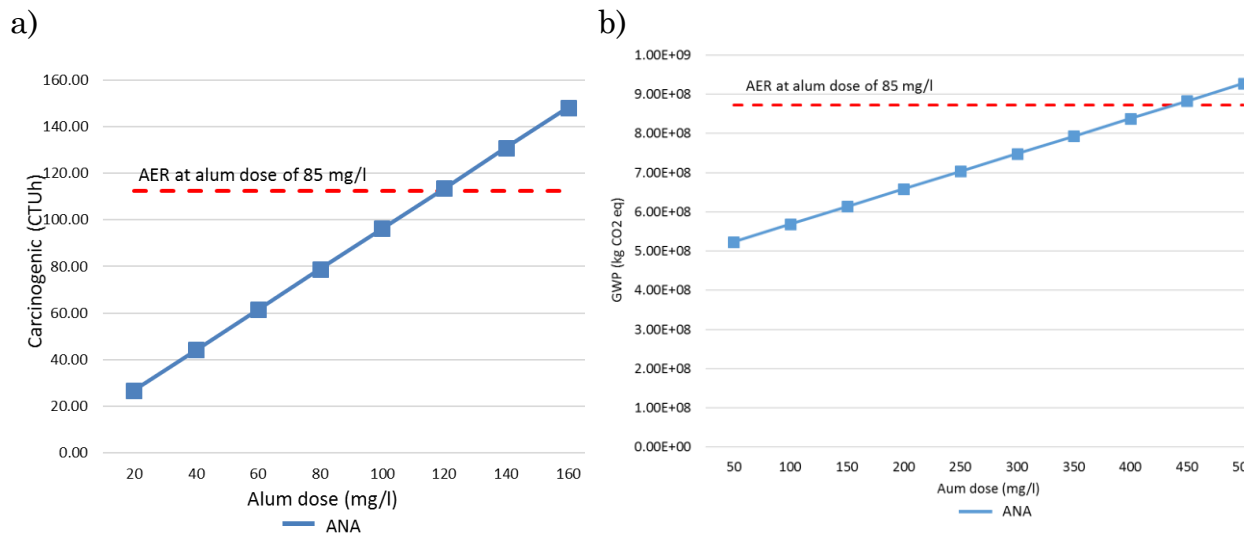


Figure 15. Carcinogenics (a) and global warming potential (b) impacts as a function of the ANA alum dose for treating high strength wastewater.

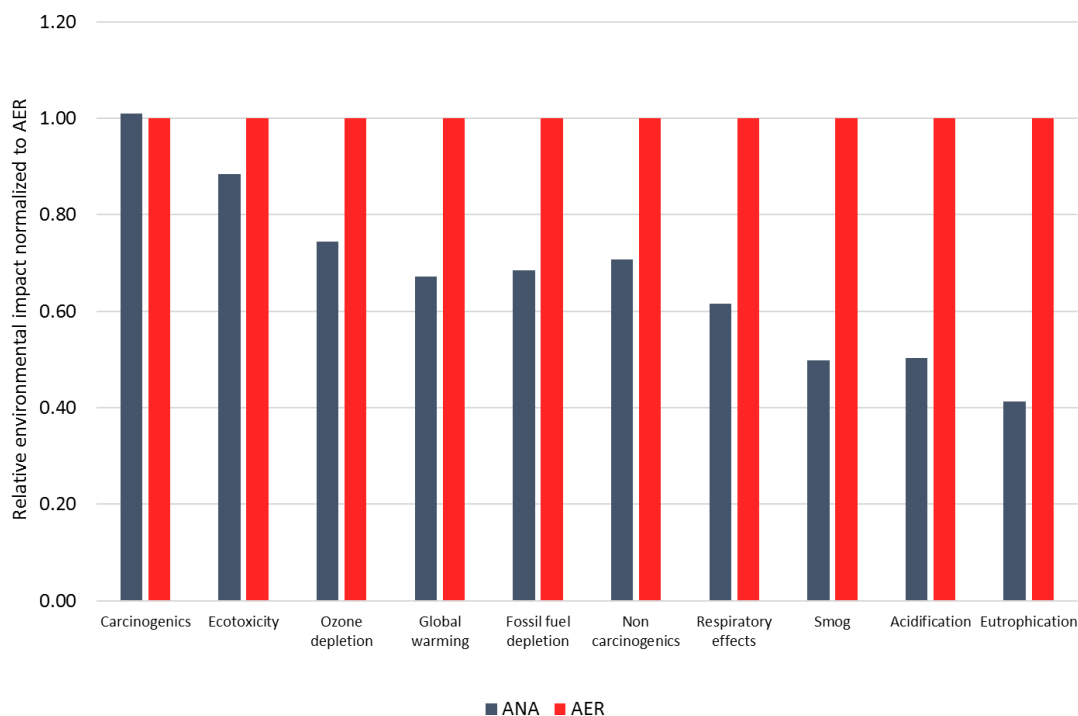


Figure 16. Relative environmental impact with alum dose of 120 mg/l for ANA system, normalized to AER.

Effluent nitrogen for water reuse standards

For AER-HRAS system, the effluent nitrogen was about 32 mg/l as NH₃-N and for AER-NO₃ the effluent nitrogen was about 8.3 mg/l as NO₃-N for medium strength wastewater. According to

Table 3, the required effluent concentration for agricultural water reuse should be less than 10 mg/l and none of the AER system satisfies the reuse standards for nitrogen. The reason being, as denitrification was not considered during the biological process, nitrogen either in the form of ammonia or nitrified nitrogen shall be available in effluent. Nitrogen present in form of ammonia will be directly available for plant, and the remaining nitrogen (except ammonia nitrogen used by plant) might be considered as agricultural run-off. The environmental impact for the nitrogen run-off was not included in the current system boundary, but need to consider to evaluate the system configuration.

Enhanced primary treatment for better removal of organic carbon

To have efficient removal of organic carbon from the water reuse system, various types of flocculants (ferric chloride, alum, lime and anionic polyelectrolyte) have been used (Jiménez and Chávez, 1997). The organic carbon effluent with such enhanced primary treatment can improve the water quality downstream and hence can help achieve the water reuse standards (Wang et al., 2009).

Breakeven points for ANA and AER system:

Membrane Scouring

According to Figure 8 and Figure 9, it is clearly observed that after alum coagulation, the second important unit process that has huge environmental impact was energy associated with membrane scouring. For ozone depletion impact, calculated values show that, 31% reduction in membrane scouring energy would make ANA similar to AER system in terms of ozone depletion impact. While to reduce the impact for fossil fuel depletion, 58% reduction in membrane scouring energy will be required to have similar impact as AER system. Recycling of the mixed liquor through the membrane module, helps create enough turbulence to enhance the back transport of foulants from its surface, thus reducing membrane fouling (Martin et al., 2011). Also increasing the flow velocity results in higher shear stress, thus reducing membrane fouling (Liao et al., 2006).

Dissolved Methane

Impact associated due to dissolved methane emission can be seen in global warming potential impact, as seen in Figure 10 along with energy consumption due to membrane scouring. To have similar global warming impact for ANA as AER, 52% of dissolved methane shall be recovered. This is 48% more than the baseline value of 35% recovery of dissolved methane. 100% recovery of dissolved methane shows AER to be worse than ANA system almost by 200%.

CHAPTER V: FUTURE WORK

The future work includes experimental analysis of mainstream anaerobic bioreactor to understand the different operational parameters associated with efficient bioreactor performance. This shall include studying specifically the water parameters like TOC and alkalinity to understand the coagulation mechanism for removal of organic matter from the ANA effluent. These experimental data shall then give proper analysis for design consideration which can then give more realistic results associated with anaerobic systems.

Proper disinfection model for reclaimed water reuse for application purpose shall need to be studied to minimize the impacts associated with run-off of nutrients as well as to study the impact of organic matter applied to agricultural land. Selection of disinfection process need to be optimized to have sustainable environmental unit process.

Agronomics for the water reuse treatment trains was not considered for this research. However, understanding the plant available nutrients and nutrients fed to the agricultural land will help optimize the upstream water reuse scenarios, depending on amount of nutrients required for particular crops. Also for biosolids land application, appropriate nutrients shall be considered to optimize the design considerations.

Using mainstream anaerobic process for potable water reuse might be a good option to consider for minimizing the energy associated with conventional aerobic process. But this might include downstream energy intensive process to achieve stringent water quality required for potable reuse application.

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APPENDIX A – DETAILED OXYGEN REQUIREMENT CALCULATIONS

Two aeration process considered for AER and AER-NO₃ scenarios differ in their SRT values to allow nitrification. The SRT value for AER system was considered to be 2 days, sufficient enough for the growth of heterotrophs in the bioreactor; but not sufficient for nitrifying bacteria. The oxygen required for heterotrophic bacteria is given as

Equation 7. Heterotrophic oxygen requirement for AER system (Grady et al., 2011)

$$RO_H = Q \times (S_{SO} + X_{SO} - S_S) \left[1 - \frac{(1 + f_D \times b_H \times \theta_C) Y_{H,T} i_{O_{\frac{O}{XB,T}}}}{1 + b_H \times \theta_C} \right]$$

Where:

RO_H: Heterotrophic oxygen requirement (kg O₂/day)

Q: Influent flow of wastewater (m³/day)

S_{SO}: Readily biodegradable substrate (mg COD/l)

X_{SO}: Slowly biodegradable substrate (mg COD/l)

S_S: Soluble substrate (mg COD/l), calculated using Equation 8

f_D: 0.20 mg TSS/mg TSS

b_H: 0.22 day⁻¹

θ_C: 2 days

Y_{H,T}: 0.50 mg TSS/mg COD

i_{O/XB,T}: 1.2 mg COD/mg TSS

Equation 8. Soluble substrate (Grady et al., 2011)

$$S_S = \frac{K_S \left(1/\theta_C + b_H \right)}{\mu_H - \left(1/\theta_C + b_H \right)}$$

Where:

K_S: 20 mg COD/l

θ_C: 2 days

b_H: 0.22 day⁻¹

μ_H: 8.8 day⁻¹

S_{SO} + X_{SO} is assumed to be total biodegradable COD influent to the bioreactor.

The SRT value for AER-NO₃ system considered is 10 days, which gives sufficient time for nitrifying bacteria to grow inside the bioreactor. The oxygen required for

AER-NO₃ is than calculated by using Equation 7, except for θ_c which is 10 days for AER-NO₃ (Grady et al., 2011). The nitrifying bacteria consumes significant amount of oxygen for their growth, and the amount required is calculated from Equation 9

Equation 9. Autotrophic oxygen requirement for AER system (Grady et al., 2011)

$$RO_A = Q \times (S_{N,a} - S_{NH}) \left[4.57 - \frac{(1 - f_D \times b_A \times \theta_c) Y_{A,T} \times i_{O/XB,T}}{1 + b_A \times \theta_c} \right]$$

Where:

RO_A: Autotrophic oxygen requirement (kg O₂/day)

Q: Influent flow of wastewater (m³/day)

S_{N,a}: Influent nitrogen available for nitrifiers (mg N/l)

S_{NH}: Ammonia concentration in wastewater (mg N/l)

f_D: 0.20 mg TSS/mg TSS

b_A: 0.12 day⁻¹

θ_c : 10 days

Y_{A,T}: 0.20 mg TSS/mg COD

i_{O/XB,T}: 1.2 mg COD/mg TSSF:

The energy consumption calculation for aeration was than calculated using Equation 10 and Equation 11

Equation 10. Power required for aeration (Tchobanoglous et al., 2003)

$$P_w = \frac{WRT}{29.7ne} \left[\left(\frac{P_2}{P_1} \right)^{0.283} - 1 \right]$$

Where:

P_w: Required power (kW)

W: Weight of air flow (kg/s), calculated using Equation 7 and Equation 9.

R: Engineering gas constant (8.314 J/mol K)

T: Absolute temperature of AER system (293 K)

n: (k-1)/k where k is specific heat capacity of air (k=1.4 J/kg.K)

e: efficiency of compressor (80%)

Equation 11. Energy consumption for blower

$$E_w = \frac{P_w}{Q}$$

Where:

P_w: Power required for aeration calculated using Equation 10 (kW)

Q: Influent flow to the bioreactor (m³/hr)

APPENDIX B – ENERGY CONSUMPTION FOR ANAEROBIC SYSTEM

Energy consumption for anaerobic system consists of four different operational parameters associated with functioning of the anaerobic system.

1. Energy consumption for membrane scouring (E_s): Membrane scouring is associated with the mixed liquor suspended solids, particle size distribution and extra-polymer substances. However, all of these physiological characteristics properties of bulk solids change according to operational conditions of the anaerobic system (Martinez-Sosa et al., 2011). Fouling is therefore one of the major operational areas for anaerobic systems that is unpredictable and is poorly understood (Martinez-Sosa et al., 2011; Smith et al., 2014). Energy required for membrane scouring was calculated using Equation 12

Equation 12. Power consumption due to membrane scouring (Tchobanoglous et al., 2003)

$$P_s = \frac{WRT}{29.7ne} \left[\left(\frac{P_2}{P_1} \right)^{0.283} - 1 \right]$$

Where:

P_s : Power consumption due to membrane scouring (kW)

W : Weight of biogas circulated to avoid fouling of membrane (kg/s)

R : Engineering gas constant (8.314 J/mol K)

T : Absolute temperature of anaerobic system (K)

n : $(k-1)/k$, where k is the specific heat capacity of biogas (1.2 kJ/kg K)

e : Efficiency of blower (assumed to be 80%)

P_1 : Absolute inlet pressure assumed to be 1 atm

P_2 : Absolute outlet pressure ($P_2 = 10000h + P_1$), where h is the head loss across the AnMBR system and assumed to be 2.5 m

29.7 is the unit conversion

Energy consumed by membrane scouring is calculated as 0.21 kWh/m³ for baseline scenario.

2. Energy required for pumping (E_p): Pumping energy consist of influent and effluent pumping. Depending on hydraulics of the wastewater treatment plant, influent pumping can be avoided but for current research influent pumping of the wastewater is considered. Power consumed by influent pumping was than calculated using Equation 13.

Equation 13. Power consumed by each influent pump (Tchobanoglous et al., 2003)

$$P_{P1} = \frac{Q\rho gh}{\eta}$$

Where:

Q: Influent flow of wastewater to AnMBR system (m³/h)

ρ : Density of biogas (1.15 kg/m³)

g: Acceleration due to gravity (9.81 m/s²)

h: Headloss across AnMBR system (assumed to be 2.5 m, depending on maximum ZeeWeed*50D cassette height of 2.5 m)

η : Efficiency of compressor (assumed to be 80%)

Influent pumping energy consumption is calculated as 0.008 kWh/m³.

Effluent pumping was calculated from flow of wastewater through each pump.

Effluent pumping depends on total dynamic head across the anaerobic system

(Shoener et al., 2014). Dynamic head was calculated using Equation 14

Equation 14. Total dynamic head across AnMBR (Shoener et al., 2014)

$$TDH = H_{ts} + H_{sf} + H_{df} + TMP$$

Where:

TDH: Total dynamic head across AnMBR system (m)

H_{ts} : Total static head (assumed to be 1.0 m for this research, but shall depend on hydraulics of AnMBR and effluent discharge of AnMBR)

H_{sf} : Suction friction head (m) calculated as $H_{sf} = 3.02LV^{1.85}C^{-1.85}D^{-1.17}$

Where: L: Length of the suction pipe (assumed to be 10 m)

V: Velocity in the suction pipe (should be in range of 0.91 to 2.44 m/s, for this research velocity is assumed to be 2.44 m/s)

C: Hazen William coefficient (assumed to be 110)

D: Diameter of the suction pipe (assumed as 0.3 m, maximum pipe diameter available)

H_{df} : Discharge friction head (m) calculated as $H_{df} = 3.02LV^{1.85}C^{-1.85}D^{-1.17}$

Where: L: Length of the discharge pipe (assumed to be 10 m)

V: Velocity in the discharge pipe (should be in range of 0.91 to 2.44 m/s, for this research velocity is assumed to be 2.44 m/s)

C: Hazen William coefficient (assumed to be 110)

D: Diameter of the suction pipe (assumed as 0.3 m, maximum pipe diameter available)

TMP: Trans-membrane pressure across the membrane in anaerobic bioreactor (for baseline scenario trans-membrane pressure is considered as 27.5 kPa as given in Table 11 the values of which are dependent on experimental values found in literature for AnMBR (Table 8))

Brake horse power (BHP) is calculated from wastewater flow and total dynamic head as calculated from Equation 14.

Equation 15. Brake horsepower for effluent pump (Shoener et al., 2014)

$$BHP = \frac{Q \times TDH}{\eta}$$

Where:

Q: Effluent wastewater flow through effluent pump (m³/h)

TDH: Total dynamic head (m)

η : Efficiency of pump (assumed as 0.8)

Energy consumption of the effluent pump is than calculated from Equation 16.

Equation 16. Energy for effluent pump

$$E_{p2} = \frac{BHP}{\eta}$$

Where:

BHP: Brake horsepower of the pump

η : Efficiency of motor (assumed as 0.7)

Energy consumed by effluent pumping is calculated to be 0.02 kWh/m³.

3. Energy required for heating of influent wastewater (EH): Depending on the temperature of the influent wastewater, energy required to increase the temperature of the influent wastewater up to mesophilic temperature of the anaerobic system was considered. For this research the influent temperature of wastewater was considered as 35°C. Hence no energy for heating was accounted for this research. However, variation in influent temperature of wastewater was studied to understand the

operational uncertainty of the anaerobic model. The energy required for heating was calculated using Equation 17

Equation 17. Energy required for heating of influent wastewater

$$E_H = C_p \times \rho_w \times \Delta T$$

Where:

C_p : Specific heat capacity of water (4.2 kJ/kg.K)

ρ_w : Density of water at 35°C, temperature of anaerobic system (995.7 kg/m³)

ΔT : Difference in temperature of influent wastewater and anaerobic system (K)

4. Energy for mechanical mixing in bioreactor (EM): Typical values for mechanical mixing in anaerobic bioreactor is 8 to 10 kW/10³m³ (Tchobanoglous et al., 2003). Average value of 10.5 kW/10³m³ was assumed for this research, which equals to 0.053 kWh/m³.

5. Energy required for degasifier (EDM): Value of 0.035 kWh/m³ is assumed for recovery of dissolved methane (Seib et al., 2016)

The total energy consumed for ANA system is then calculated by adding all the 5 components.

$$E_{cons} = E_S + E_{p1} + E_{p2} + E_H + E_M + E_{DM}$$

APPENDIX C – MEMBRANE CALCULATION

Polyvinylidene difluoride (PVDF) is the most common membrane material used for AnMBR systems (Shoener et al., 2014). The same is considered for this research. ZeeWeed 500D-cassette (Technologies, 2016) is considered for AnMBR. Number of modules (#modules) in each cassette available according to manufacture are considered as 48. Type of each module is ZeeWeed 500D-Module with product name as 370. Nominal membrane surface area of each module is 34.4 m². Whereas dry weight of single module is 28 kg. This dry weight of module is assumed to be entirely of the membrane material (PVDF), hence 28 kg of PVDF is assumed to be used for single module of PVDF membrane.

Membrane surface area per cassette is then calculated from (34.4 m² X 48 nos) which is 1651 m². Total number of cassette required for 20 MGD was then calculated using Equation 18

Equation 18. Number of cassette in ANA system

$$\#cassette = \frac{COD \text{ loading rate} (g \frac{COD}{day}) / COD \text{ mass loading per filtration area} (\frac{g \frac{COD}{day}}{m^2})}{Membrane \text{ surface area per cassette (m}^2\text{)}}$$

Amount of polymer required for AnMBR system for 20 MGD of influent wastewater is calculated from Equation 19.

Equation 19. Amount of polymer required for membrane system

$$Amount \text{ of polymer} = Dry \text{ weight of single module} \times \#modules \times \#cassette$$

1210 tonnes of PVDF is required for 20 MGD of medium strength wastewater for 40 years with membrane life assumed as 10 years (Smith et al., 2014).

APPENDIX D - DEEP BED ANTHRACITE FILTER DESIGN

Deep bed anthracite filters were considered for downstream water reuse treatment trains. Total filtration area is calculated from Equation 20

Equation 20. Total filtration area

$$A_T = \frac{Q}{V_f}$$

Where:

A_T : Total filtration area (m²)

Q : Influent wastewater flow (m³/h)

V_f : Filtration rate (range of 10 to 30 m/h is generally used for tertiary system (Kawamura, 1991), value of 15 m/h is assumed for current research)

Filtration area of single filter cell (A_f) assumed was 35 m² (range of 25 to 100 m² can be used for filter cell (Kawamura, 1991). Total number of filters required for 20 MGD is calculated using Equation 21.

Equation 21. Number of filters

$$n_f = \frac{A_T}{A_f}$$

Media height of 1.8 m is assumed from the range of 0.8 to 2.0 m (Kawamura, 1991). 2.4 m of water depth is considered above filter media (range of 1.8 m to 2.4 m is general range of water depth for deep bed granular media filter (Kawamura, 1991). Amount of anthracite required for 20 MGD is then calculated from Equation 22.

Equation 22. Amount of anthracite required

$$M_{anthr} = \rho_{anthr} \times V_{anthr}$$

Where:

M_{anthr} : Amount of anthracite required (kg)

ρ_{anthr} : Bulk density of anthracite (800.925 kg/m³)

V_{anthr} : Volume of anthracite (m³; calculated from depth of filter media and filter area)

Amount of anthracite required for treating 20 MGD of medium strength wastewater is calculated to be 305 tons for 40 years of timeframe. No replacement of anthracite is considered for this research.

Backwashing with water is considered twice a day. Backwash water required for entire flow is calculated from Equation 23.

Equation 23. Backwash water flowrate

$$Q_{BW}^{water} = A_T \times Q_B$$

Where:

Q_{BW}^{water} : Backwash water flowrate (m³/h)

A_T : Total filtration area (m²); calculated from Equation 20

Q_B : Backwash water flow rate (45 m/h is assumed for baseline scenario)

Backwash water required for single filter is then calculated as Equation 24.

Equation 24. Backwash water for single filter

$$Q_{BW}^{filter} = \frac{Q_{BW}^{water}}{n_f}$$

Power consumed by backwash water pump is calculated using Equation 25.

Equation 25. Power consumption for backwash water pump

$$P_{BW} = \frac{Q_{BW}^{filter} \rho g h}{\eta}$$

Where:

P_{BW} : Power consumed by backwash pump (kW)

Q_{BW}^{filter} : Backwash water for single filter (m³/h)

ρ : Density of water (1000 kg/m³)

g : Acceleration due to gravity (9.81 m/s²)

h : Backwash pressure head (assumed as 8 m (Kawamura, 1991))

Backwash energy for 40 years is then calculated as 209140 kWh.