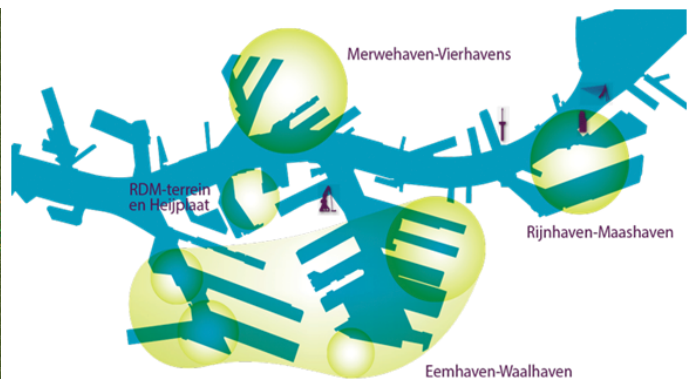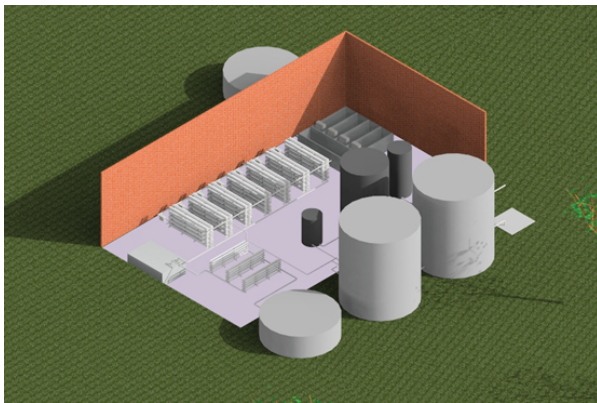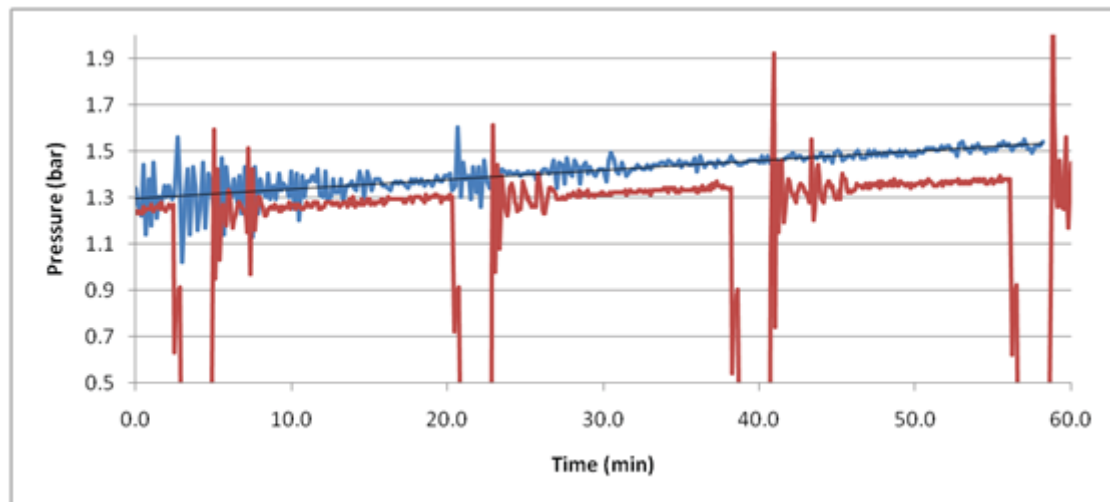# Development of an innovative wastewater reuse plant for the RINEW project

Joeri Legierse

June 2013

Challenge the future

Development of an innovative wastewater reuse plant for the RINEW project

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Abstract

Increasingly, harbour activities in the harbour of Rotterdam are moving west towards the Northsea (Maasvlakte I & II). This impoverishes harbour areas close to Rotterdam like the Stadshavens area. The municipality of Rotterdam is trying to revitalize these areas by transforming them into sustainable living and working communities. Evides N.V. is participating in this by developing an innovative wastewater reuse plant which will reuse the wastewater from the Stadshavens area by treating it to demi-water standards, while also recovering nutrients and energy. The objective of the research described in this thesis is to develop several sustainable and innovative treatment scenarios for the wastewater reuse plant, test relevant innovative technologies on lab scale and present a conceptual design of the wastewater reuse plant.

Conventional wastewater treatment plants treat the wastewater under both aerobic and anaerobic conditions. Especially the aeration needed for aerobic treatment has a high energy consumption. With the development of new treatment techniques like ceramic nanofiltration and forward osmosis it is now possible to replace the aerobic treatment used in conventional wastewater treatment plants by these technologies. This saves energy and gives a smaller installation footprint (no settling phases are necessary). Four scenarios were developed, two using combined collection of wastewater, while the other two have a separate collection of grey water (from washing) and black water (from toilets). One of the combined collection scenarios and one of the separate collection scenarios uses ceramic nanofiltration (CNF) as a first treatment step, while the other two use forward osmosis (FO) as a first treatment step. After the first treatment step, the water quality is sufficient to produce demi-water by using the reverse osmosis process (RO). In the separate collection scenarios, the grey water is treated similarly to the wastewater treated in the combined collection scenarios.

Nutrients and energy are recovered from the reject water of the CNF and FO processes by applying anaerobic digestion in the form of a UASB reactor. This process produces sludge and biogas from the biological material in the wastewater. After this step, magnesium ammonium phosphate precipitation and SHARON®-Anammox® processes are applied to recover ortho-phosphate and ammonium from the wastewater. With the separate collection scenarios, the black water is treated similarly to the concentrate water treatment in the combined collection scenarios.

After comparing the scenarios based on energy consumption, demi-water production and demi-water quality, it was chosen to apply separate collection of wastewater, using CNF as a primary treatment step. The CNF process was tested on lab scale to review its performance when treating either raw wastewater or grey wastewater. Results indicated a better permeate quality and reject composition when treating grey water, but more fouling was observed, compared to raw wastewater. However, it is estimated this is controllable by applying regular backwashing and chemical cleaning. This research has shown that membrane processes can be used as a viable replacement for the aerobic processes which are used in conventional wastewater treatment plants. By using membrane processes instead of aerobic processes, less energy is consumed, while still achieving reliable effluents and reuse products.

Based on the research a conceptual design of the wastewater treatment plant was made which consisted of a building which contains all treatment processes except the grey and black wastewater buffers, the demi-water storage and the digestion process tanks which are located outside of the building.

Preface

This thesis is the result of MSc. research conducted at Delft University of Technology, faculty of Civil Engineering and Geosciences, Department of Sanitary Engineering. The objective of this research was the development of sustainable and innovative treatment scenarios for the wastewater of the Stadshavens area in Rotterdam and testing of innovative techniques used in the scenarios on lab scale. A conceptual design of a treatment facility was also made. This research is part of the RINEW project, initialised by Evides N.V.

During my graduation work I was supported by many people, whom I would like to thank for their help, support and advice. First off I would like to thank my graduation committee, consisting of LuukRietveld, Bas Heijman, Jan-Willem Mulder and Hans Vrouwenvelder. Luuk, my professor, thanks for all the feedback and enthusiasm shown for my research. Bas, my daily supervisor, thanks for the advice, feedback, help and unconditional support you've shown me throughout my graduation work. I would also like to thank Bas van Eijk, Jan-Willem Mulder and other members of the RINEW Project team for their feedback and providing a fresh look on the research. Also thanks to Ran Shang, for the help and advice you've given me with my lab scale experiments, I hope you will be successful in continuing my research. Special thanks goes out to my friend Jurgen Kil, for some much appreciated help with the 3D modelling of my design. I would also like to thank Dhr. Quatfass (Qua-Vac B.V.), Mw. Wiersma (Desah B.V.) and Dhr. Ramaekers (Triqua B.V.) for their time, information and input. Finally I would like to thank my girlfriend, family and friends who have always supported me.

Joeri Legierse
Delft, June 2013

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List of abbreviations

CD	Capacitive Deionization
CIP	Cleaning In Place
CNF	Ceramic Nanofiltration
COD	Chemical Oxygen Demand
DOC	Dissolved Organic Carbon
EB	Enhanced Backwash
EFC	Eutectic Freeze Crystallization
FO	Forward Osmosis
HRT	Hydraulic Retention Time
MAP	Magnesium Ammonium Phosphate Precipitation
MF	Micro Filtration
MWCO	Molecular Weight Cut Off
NF	Nano Filtration
NTU	Nephelometric Turbidity Unit
RO	Reverse Osmosis
TMP	Trans Membrane Pressure
UASB	Upflow Anaerobic Sludge Bed
UF	Ultra Filtration
WWTP	Waste Water Treatment Plant

1   Introduction

1.1   History of Rotterdam harbour

The city of Rotterdam is, because of its harbour activities, one of the Netherlands most well-known cities. Until 1970, a coalition between the municipality of Rotterdam and the port authority drove the massive growth of the harbour. However, from 1970 onwards, the focus of the municipality shifted more and more towards creating good living conditions for its inhabitants and the economic aspects of the harbour often came second to this (Kreukels and Wever 1996). As a result, expansion of the harbour within the city limits was restricted. Another problem was the fact that the ships commuting the harbour got bigger, this meant that the Nieuwe Waterweg had to be dredged out. To overcome these problems, it was decided to construct the Maasvlakte. This area, situated between Rotterdam and the North Sea, was converted into a massive expansion of the harbour. Because of its direct proximity to the North Sea, it was easier to construct deeper waterways to accommodate larger ships. After its construction was finished in 1973, it ensured that the Port of Rotterdam could keep on competing with other harbours in Europe and the world (Port of Rotterdam 2013).

Figure 1 Overview Maasvlakte (right area) and Maasvlakte 2 (left area) (www.maasvlakte2.com)

For twenty years the harbour kept slowly growing, however halfway through the nineties it became obvious that more space was again needed. This resulted in plans being drafted to construct the Maasvlakte 2 (see Figure 1). Plans were made for the Maasvlakte to be expanded in the direction of the sea. In 2008, the construction of the Maasvlakte 2 was started, and is scheduled to be completed in 2013. Between 2013 and 2030, more expansions will be completed, depending on the demand of harbour space that is needed. With its first stage complete, harbour activities will shift from close to the city of Rotterdam towards the North Sea (Havenbedrijf Rotterdam NV. 2013). With this shift of activities, comes a decrease in the use of the docks near the city. This creates opportunities to transform and modernise these areas into a sustainable living and working environment.

1.2 Stadshavens

One of the main areas that is assigned to be transformed is the area Stadshavens. The north side of this area is currently partly in use by several fruit- and juice industries, while the south side is characterized by large scale container transhipments. With the completion of the Maasvlakte 2, these container transhipments will move away from the Stadshavens area, creating opportunities for transformation in that area (Port of Rotterdam 2012).

Figure 2 Overview Stadshavens area (www.stadshavensrotterdam.nl)

To accommodate this transformation, the municipality of Rotterdam and the Port Authority of Rotterdam started the project Stadshavens. The aim of this project is to create a sustainable mixture of high quality harbour activities and a sustainable living and working environment in the Stadshavens area (Municipality of Rotterdam 2010). In the project, the area has been subdivided into 4 smaller areas (also see Figure 2):

- Merwehavens-Vierhavens
- RDM-Terrain
- Waalhaven-Eemhaven
- Rijnhaven-Maashaven

Each of these areas will be given a variety of functions.

1.2.1   Merwehaven-Vierhavens

The Merwehaven-Vierhavens area will accommodate floating communities and its main focus will be on re-inventing delta-technology, crossing borders and sustainable mobility. One of the driving forces behind this focus will be the establishing of a Climate Campus, where knowledge and experiences in the fields of sustainable energy use and water management can be shared between engineering firms, educational institutions and other interested parties (Projectbureau Stadshavens Rotterdam 2009).

Figure 3 Floating community in the Merwehaven (www.stadshavensrotterdam.nl)

There are also plans to locate the National Water Centre in this area, which will be a showcase of Dutch water legislation, ingenuity and experience (Rotterdam Climate Initiative 2010).The fruit and juice industries that are currently located in the Merwehaven-Vierhavens area will move to the south side of the river (Eemhaven-Waalhaven) as soon as the large container transhipments are relocated to the Maasvlakte 2. Some of the old buildings will be converted into art studios, while others will be demolished to create parks and accommodate infrastructure. With the docks no longer being used, the water becomes available as a building area for floating communities (see Figure 3). The nature of these transitions and its focus makes this area an ideal sandbox for innovative water and sustainable energy use.

1.2.2   RDM terrain

The RDM terrain housed until recently the Rotterdamse Droogdok Maatschappij (hence the abbreviation). The municipality of Rotterdam plans to transform this area into a place where research, design and manufacturing come together (thus keeping the same abbreviation). This is amongst other things accomplished by erecting an RDM campus, where companies and schools will offer technical internships and learning experiences for students, ranging from welding, to water management and energy transition (Projectburea Stadshavens Rotterdam 2009).

1.2.3   Waalhaven-Eemhaven

The Waalhaven-Eemhaven area is currently home to a variety of harbour functions but its main function is that of an intra-European and deep sea container hub. The deep sea container activities will move to the Maasvlakte 2, creating space for modernizing the smaller harbour activities. Also, space becomes available for office buildings along the waterside and floating communities on the water itself (Projectbureau Stadshavens Rotterdam 2009).

1.2.4   Rijnhaven-Maashaven

This area is currently used by barges and deep sea vessels to anchor. As in the other Stadshavens areas, these activities will slowly move towards the Maasvlakte. The municipality of Rotterdam plans to make this area its showcase for the entire Stadshavens Project (Projectbureau Stadshavens Rotterdam 2009). They hope to recreate the success of the Kop van Zuid Project (Doucet, Van Kempen and Van Weesep 2010), in the Rijnhaven and as such this area will become the most urban of the four.

Figure 4 Artist impression floating communities in the Maashaven (www.stadshavensrotterdam.nl)

Along the waterside of the Rijnhaven, various office and high rise apartment buildings will be constructed, while in the Maashaven the water will be used for floating communities (see Figure 4). These floating houses will be as energy neutral as possible, with solar panels for electricity, while rest-heat from the remaining industries will be used for heating.

1.3   Project RINEW

In the Merwehaven-Vierhavens area the focus of the municipality of Rotterdam is renewing delta-technologies with an emphasis on sustainability and innovation. Evides N.V. as the primary supplier of drinking and industry water in the area, has chosen to participate in the Stadshavens project by developing a wastewater treatment system that can reclaim demi-water from the wastewater, while recovering energy and nutrients. Because of the growing shortage of fresh water, innovations are necessary to limit the amount of fresh water that is

fed back into the rivers as treated wastewater. By reusing the wastewater, more fresh water is available for drinking and industrial water production. By installing a pilot-project in the Merwehavens-Vierhavens area, innovative techniques for wastewater reclamation can be tested. This pilot can be scaled up towards a full treatment plant to treat and reclaimwastewater from the entire Stadshavens area.

1.4   Thesis objective

As part of the development of a pilot for the Stadshavens area, this thesis covers the development and comparison of possible scenarios which can accommodate the project goals mentioned in the previous paragraph. Water reuse will be the main focus, with energy and nutrient recovery as secondary focus points. It is chosen to reclaim the wastewater as demi-water, since reusing it as drinking water will present a possible threat to public health and would make the project too complex (Mulder and van Eijk 2011). Apart from the development and comparison of the scenarios, the most promising scenario will be further investigated at lab scale after which a preliminary design will be presented to give an insight into the scale of a possible future wastewater treatment installation for the Stadshavens area.

From the thesis objective, the following research questions are formulated:

1.   *What combination of source wastewater and innovative technologies is best suited to produce demi water and recover energy and nutrients from the wastewater of the Stadshavens area in Rotterdam?*

2.   *How does ceramic nanofiltration operate when fed with grey and raw wastewater with a recovery of 80% and what is the composition of the permeate and concentrate under both these conditions?*

1.5   Thesis outline

In the second chapter of this thesis, various processes which are used during the development of the scenarios will be briefly explained. The third chapter contains a scenario study into the development of various wastewater treatment scenarios for the Merwehaven-Vierhavens area. In this chapter, a comparison between these scenarios is also given based on their energy consumption and demi-water production and quality that is expected. In the fourth chapter, results of lab-scale experiments of the treatment of grey water and raw wastewater are presented and discussed. The fifth chapter presents a rough preliminary design of a possible treatment plant using the most promising scenario. The sixth chapter contains the conclusions of this thesis and some recommendations for future research.

2 Theory of treatment processes

In this chapter, a short description is given of the various technologies used in the scenario study (MAP, CNF, FO, RO, Sharon-Anammox& Anaerobic digestion). These technologies are divided into membrane, biological and chemical processes.

2.1 Membrane processes

Ceramic nanofiltration, forward osmosis and reverse osmosis are all membrane based processes. In essence, water is pumped through a membrane (which acts as a filter) where the pore size of the membrane determines which substances it rejects. The membrane processes are classified as follows (with decreasing pore size): microfiltration, ultrafiltration, nanofiltration, forward osmosis and reverse osmosis (Eddy 2004). An overview of the pore sizes of the various membrane processes and the substances that are rejected can be seen in Figure 5. Apart from membrane processes, Figure 5 also shows the pore size of conventional filtration processes like sand filtration. In general, more pressure is required when the pore size decreases, while the quality of the permeate improves.

Figure 5 Overview of the various membrane processes, their pore sizes and rejected substances (van Dijk, et al. 2009)

2.1.1 Membrane modules

Micro and ultrafiltration are primarily applied by using capillary modules (see Figure 6) . These modules consist of hundreds or sometimes thousands of tubes with a small diameter ranging from 0.1 to 5 mm (capillaries) (Lenntech 2013). Water to be filtered is pumped through the tubes of the module. The sides of these tubes contain the pores which reject the substances while water that passes through the pores is collected in an inner tube. This process creates a concentrated water stream that contains all the substances, and a clean water stream (permeate). These modules can also be operated using a so called 'dead-end' configuration, this means the small tubes are open on one side, and closed on the other, effectively filtrating all the water without any water loss. This does however mean the rejected substances cannot be transported out of the module without stopping the filtration process and cleaning the module. If no dead-end configuration is used, the substances will be washed out of the module with the concentrated stream. This way, less cleaning is needed.

Figure 6 Capillary module (van Dijk, et al. 2009)   *Figure 7 Spiralwound module (van Dijk, et al. 2009)*

Unlike micro and ultrafiltration, nanofiltration and reverse osmosis are primarily applied using spiral wound modules (see Figure7). The reason for this is the lower flux of water through the membrane (because of the smaller pore size). When applying spiral wound modules, the filtration area per volume is larger, compared to using capillary modules. This does however come at the expense of a greater susceptibility to fouling and scaling. The spiral wound module essentially consists of membrane sheets which are spirally wound around the permeate tube. Between these sheets, thin layers of spacers are applied to facilitate the flow of water. Because the spacers are so thin, more substances will remain behind on the membrane instead of being flushed out. This increases the susceptibility to fouling of the membranes.

2.1.2 Cleaning of membranes

During the treatment process, some of the substances that are rejected by the membrane will not be washed away by the flow of water past the membrane. These substances will form a layer on the membrane, blocking the pores. This will increase the membrane resistance and thus the pressure required to achieve the necessary permeate flow. This layer can consists of biological material growth (fouling) or calcium hydroxide (scaling). With the more open membranes (like microfiltration membranes), particles can also block the pores. With capillary membranes, a backwash can be applied to remove this layer. During a backwash, water is pumped from the permeate side to the feed water side, hence lifting or breaking the layer of fouling and/or scaling. This method is particularly effective when combined with a forward flush (Enhanced Backwash). During a forward flush, the flow of water through the capillaries is increased to scour the fouling and/or scaling from the membrane. Depending on the effectiveness of the backwash, a frequent chemical cleaning might also be necessary to remove any substances that are not removed during a backwash. During a chemical cleaning, the membrane is soaked in different chemicals. Firstly NaOH is used to remove any biological material growth, secondly HCl is used to remove any scaling, and last, NaOCl is used to remove any substances that remained. This is called a cleaning in place (CIP). Because these chemicals need time to react with the fouling and scaling, CIP requires a certain soaking time depending on the severity of the fouling and scaling. This means it is only used as a last resort, since during the time the CIP takes place, the membrane will be out of operation. A graphical representation of the combination of enhanced backwashing and CIP is shown in Figure 8.

Figure 8 Membrane resistance as a function of time during treatment with the application of enhanced backwashes (EB) and cleaning in place (CIP)

2.1.3 Ceramic nanofiltration

As mentioned before, sufficient pre-treatment is normally necessary for nanofiltration to protect the membrane spacers from clogging with particles. However, with the development of ceramic nanofiltration modules, the pretreatment is no longer necessary. Ceramic membranes are capable of withstanding higher temperatures and chemical concentrations than their polymeric counterparts (the membranes mentioned in the two previous paragraphs are all polymeric membranes). Also because they are not spiral wound, they contain no spacers and thus they are less susceptible to clogging. Ceramic membranes consist of long modules, with smaller tubes inside. Much like the capillary membranes used in ordinary micro and ultrafiltration, these tubes have diameters ranging from 2 to 10 mm (Inopor 2013), depending on the type of ceramic membrane. Here also, the smaller the diameter of the capillaries, the larger the internal surface area of the module. Unlike micro and ultrafiltration however, these nanofiltration membranes are also capable of removing divalent ions like calcium, magnesium and even some mono-valent ions like ortho-phosphate. This could make them ideal for wastewater treatment, where they can possibly be used directly on raw wastewater (after screening).

2.1.4 Reverse osmosis

As can be seen from Figure 5, the pressure required to operate reverse osmosis is much larger than the pressure required for the other membrane processes. This is partly caused by the higher resistance of the pores but also by the difference in salt concentration. Because reverse osmosis is capable of removing almost all salts from water, the osmotic pressure on the feed side is higher than the osmotic pressure on the permeate side. Because of this osmotic pressure, the water on the permeate side wants to flow back through the membrane. This process is called osmosis (see Figure 9 and Figure10). By applying sufficient pressure on the feed side of the membrane, this osmotic pressure difference can be overcome. This is why the process is called reverse osmosis.

2.1.5 Forward osmosis

Forward osmosis is actually just osmosis. However, to prevent confusion compared to reverse osmosis it was given the name forward osmosis. This process is basically the same as reverse osmosis (with the same kind of spiral wound modules). However, instead of applying pressure on the feed side, a solution with a high salt concentration (>35 g/l) is recirculated on the permeate side of the membrane. When applied on feed waters with a low salt concentration (like wastewater), this salt solution will draw water through the membrane from the feed to the permeate side because of the osmotic pressure difference. However, because the permeate water is now mixed with the salt solution, the salt solution will become diluted, decreasing the osmotic pressure difference. To overcome this (and to reclaim the permeate water), reverse osmosis can be applied after the forward osmosis process to separate the permeate water from the salt solution.

| Figure 9 Osmotic pressure balance (van Dijk, et al. 2009) | Figure 10 Reverse osmosis (van Dijk, et al. 2009) |

2.2   Biological processes

Biological processes are often applied in wastewater treatment. Mostly these processes are driven by bacteria which transform or remove biological material from the wastewater. Among these processes, the removal of COD and ammonium is almost always carried out by biological processes. One of these processes is the anaerobic digestion of thickened wastewater (or sludge from settling processes). The thickened wastewater is rich in ammonium, phosphate and has a high COD value (indication of biological material in wastewater). This makes it very suitable for anaerobic digestion. Another biological process is the SHARON[®]-Anammox[®] process. This process is capable of removing ammonium from water.

2.2.1   Anaerobic digestion

During the anaerobic digestion, the organic material present in wastewater is degraded and biogas is produced (van Lier, Mahmoud and Zeeman 2008). Apart from producing biogas, mineralized compounds like ortho-phophate and ammonium are released from their organically bound form. The process itself is built up of various smaller processes that intertwine (see Figure 11). The four main processes that drive the anaerobic digestion are hydrolysis, acidogenesis, acetogenesis and methanogenesis. Each of these processes is driven by different bacteria which convert the organic material into smaller substrates. The process also produces granular sludge, which contains some of the mineralized compounds. To run optimally, the influent water is usually heated to 30 ℃.

2.2.2   SHARON®-Anammox®

The SHARON[®]-Anammox[®] process, developed in 2001 (van Dongen, Jetten and van Loosdrecht 2001) is a combination of a partial nitrification process (SHARON[®]) and anoxic ammonium oxidation process (Anammox[®]). During the SHARON[®] process, ammonium is oxidized to nitrite. The effluent of this process is ideally suited for the Anammox[®] process. In the Anammox[®] process, ammonium, in combination with nitrite is converted into dinitrogen gas. It is capable of removing up to 80% of ammonium from wastewater. Since it's ideal operating temperature lies between 30 ℃ and 40 ℃, it is often used after anaerobic digestion.

Figure 11 Reactive scheme of anaerobic digestion (van Lier, Mahmoud and Zeeman 2008)

2.3   Chemical processes

2.3.1   Magnesium ammonium phosphate precipitation

MAP is a process to (primarily) remove ortho-phosphate from wastewater. When magnesium, ammonium and ortho-phosphate are present in wastewater with a pH range of 8 to 10, they will crystallize to the salt $MgNH_4PO_4$ (struvite) (Schulze-Rettmer 1991). The reaction does require certain conditions, like the presence of magnesium, ammonium and ortho-phosphate in a 1:1:1 molecular ratio. This means that for every gram of ortho-phosphate converted into struvite, 0.19 grams of ammonium and 0.26 grams of magnesium need to be present in the wastewater. Often, the magnesium concentration is the limiting factor, hence decreasing the efficiency of the ortho-phosphate removal process. Therefore, magnesium usually has to be added to improve the ortho-phosphate removal. Struvite is a valuable resource which can be used as a fertilizer in agriculture.

3   Scenario study

This chapter will cover the development of several innovative scenarios for the treatment of the wastewater of the Stadshavens area. The goal of the scenario study is to investigate which combination of source water and innovative technologies can lead to certain end product(s). The main focus will be on the production of demi-water. Secondary goals are the recovery of energy and nutrients from the wastewater.

3.1   Current reuse of wastewater for industrial purposes

Traditional wastewater treatment is often comprised of a primary settling phase, followed by biological treatment and reclamation of the biological material in a secondary settling phase. After the secondary settling phase, the water is usually disposed of onto surface water (Metcalf 2004). However, the effluent of a wastewater treatment plant (WWTP) is a viable source for the production of demi-water through dual membrane processes(del Pino and Durham 1999). Dual membranes are needed as the wastewater effluent is rich in organic carbon, phosphorus and nitrogen. Combined with high water temperatures, this can lead to bio-fouling on the reversed osmosis (RO) membranes (Shang, et al. 2011). Hence either microfiltration (MF) or ultrafiltration (NF) with polymeric membranes is often used as pretreatment for RO to remove these substances from the WWTP effluent. Examples of wastewater reuse through the use of dual membranes in the Netherlands are the DECO (Terneuzen) and DWP Sas van Gent plant. The DECO plant uses municipal WWTP effluent to produce demi-water through the use of microscreening, ceramic microfiltration and RO (see Figure 12). The DWP Sas van Gent plant uses WWTP effluent from a starch production company to produce demi water through the use of multimedia-filtration, ultrafiltration and RO (see Figure 13). Another example is the Purewater Factory in Emmen, which has a similar setup.

Figure 12 Schematic process diagram of the DECO plant (Shang, et al. 2011)

Figure 13 Schematic process diagram of DWP Sas van Gent (Shang, et al. 2011)

The type of setups described above require effluent from an existing WWTP plant since raw wastewater would clog the spacers of the polymeric membranes. However, with the introduction of new treatment techniques like ceramic membranes and forward osmosis (Lutchmiah, et al. 2011), it is expected to be possible to use raw wastewater as a direct source for the production of demi-water. This omits the use of a conventional wastewater treatment plant, hence decreasing costs and needed space (see Figure 14). Also, apart from using anaerobic treatment steps, conventional WWTP plants also use aerobic treatment processes which require aeration. Aeration is a very energy consuming process. By circumventing the necessity of aeration, the energy consumption of the treatment can possibly be lowered.

Figure 14 Schematic process diagram of wastewater treatment with CNF and RO or FO and RO

3.2 Development of preliminary scenarios

3.2.1 Source water, demi water production and energy recovery

When considering the development of possible treatment scenarios, the composition of the wastewater (the source) is an important factor to consider since this will have a large influence on the layout and composition of the scenarios. This composition of the wastewater is largely influenced by the manner in which the wastewater is collected. In the past 20 years, rainwater has been collected and disposed separately from domestic wastewater because it is relatively clean, compared to domestic wastewater. Domestic wastewater is composed of both grey (washing, laundry, washing) and black water (toilet). Apart from collecting this domestic wastewater in a combined state, it is also possible to collect the grey and black water separately. From this, 2 scenarios can already be created, one where the wastewater is collected in a combined state and one where grey and black water are collected separately.

By collecting the black and grey water separately, 2 different streams are created. The black water contains urine, faeces and optionally kitchen waste, while the grey water stream contains water from bathing and laundry. These 2 streams have a completely different composition, the black water stream is small and contains most of the nutrients and has the largest energy potential, while the grey water stream is large and contains relatively clean water with a smaller energy potential (Elmitwalli and Otterpohl 2007). Also, pharmaceutical residues and pathogens are only present in the black water stream, hence the grey water stream will not have to be treated for these. Since black water is very concentrated and thick, it is usually collected using a vacuum sewer system. This opens up the possibility of installing vacuum toilets which only need 1 litre of water during flushing, compared to the $5 - 7$ litres a normal toilet uses. This can save up to 14.6 m^3 of water per person per year (Zeeman, et al. 2008). Also, by installing kitchen grinders, kitchen waste can be added to the black water stream. Because of the high energy potential of black water, it can be used as a direct source for anaerobic digestion. The addition of kitchen waste can even increase its biogas production by a factor 2 (Kujawa-Roeleveld, et al. 2005).

Figure 15 Separate collection of grey & black water (lecture notes Wastewater Treatment – TU Delft)

It is also possible to collect urine separately from faeces. From this urine, energy could be recovered by means of a microbial fuel cell, which when combined with the magnesium ammonium phosphate precipitation (MAP) process can also recover a large part of the ortho-phosphate as struvite (Zang, et al. 2012). However, collecting the urine separately is expensive since it requires another sewer pipe, while its benefits are small. Hence the option to also collect urine separately is not taken into consideration.

Apart from distinguishing 2 different source water collection systems, the composition of the scenarios is also dependent on their ability to produce demi-water and recover energy. The recovery of nutrients will be dealt with in a later stage. With the dual-membrane treatment of raw wastewater, the first membrane step is to prevent bio-fouling on the RO-membrane and to remove particles that might clog the RO-membrane. Normally, micro- or ultrafiltration membranes are used as a first treatment step because polymeric nanofiltration membranes have very fine pores and are often composed of very small capillaries or spiral-wound sheets which tend to clog rapidly when fed with particle rich feed water like raw wastewater (Metcalf 2004). However, newly developed techniques like ceramic nanofiltration and forward osmosis have a different composition (see chapter 2), possibly making them suitable to directly treat raw wastewater. It is expected that by using ceramic nanofiltration or forward osmosis instead of micro- or ultrafiltration a cleaner permeate can be produced, resulting in a better performance of the subsequent reverse osmosis membranes.

This first membrane step results in a large clear water stream suitable for RO-treatment, and a small waste stream that contains most of the biological material and suspended solids. This small waste stream also contains the largest part of the energy potential in the wastewater and is suitable for anaerobic digestion (Rao, et al. 2010). The biogas produced by the anaerobic digestion can be used to provide energy for the other treatment steps. To protect the membranes in the first step, a sieve (1 mm in case of CNF, 0.25 mm in case of FO) is placed in front of the treatment to remove the largest particles and debris from the raw wastewater.

Based on this information, four preliminary scenarios are proposed (see Figure 16, Figure 17, Figure 18 and Figure 19).

Figure 16 Preliminary scenario 1: Ceramic nanofiltration (CNF)

In scenario 1 and 2 the wastewater is collected with a combined sewer after which the water is sieved. After passing through the sieves, the water is collected in a buffer tank. In scenario 1, it is chosen to apply ceramic nanofiltration (CNF) as a first membrane treatment step. The small thickened wastewater (concentrate) stream from the CNF process is fed to an anaerobic digester (UASB) to recover biogas from the reject water. This process results in a small sludge stream, while the larger digested water stream is fed back into the buffer tank. Meanwhile, the permeate water stream from the CNF process is treated by reverse osmosis (RO) membranes to produce demi water. Apart from this, the RO process also produces a small mineral rich reject stream.

Figure 17 Preliminary scenario 2: Forward osmosis (FO)

In scenario 2, instead of applying CNF, forward osmosis (FO) is used as a first membrane step. Because of the configuration of the FO process, no reject water is produced (see chapter 2).

Figure 18 Preliminary scenario 3 - Source separation & CNF

In scenario 3 and 4, the wastewater is collected separately. Apart from this distinction, in scenario 3 the same treatment steps from scenario 1 are applied. However, because of the application of a separate sewer system and the composition of the grey and black water streams, the setup of the treatment steps is different. The black wastewater is directly fed to an anaerobic digester to recover biogas. From this process, the digested water is merged with the grey water stream. The resulting stream is sieved, after which the water is collected in a buffer tank. From the buffer tank, the water is pumped to the CNF and RO process. Similar to scenario 1, the concentrate stream from the CNF process is fed to the anaerobic digester.

Figure 19 Preliminary scenario 4 - Source separation & FO

Scenario 4 is a combination of scenario 2 and 3. The wastewater is collected separately and subsequently treated in an anaerobic digester and sieved, after which it is treated by the FO and RO process.

The reject water from the RO can also be treated further to produce process water or it can be reused for other purposes. This however, is only possible with the application of CNF as with the application of FO, a draw solution is used. The water recovery for CNF is estimated to be 80% (Futselaar, Schonewille and van der Meer 2002), while the water recovery of FO is estimated to be 70% (Holloway, et al. 2007).

3.2.2 Nutrient recovery

Wastewater contains many substances that have to be removed. When we consider the operation of the RO process, ammonium is an import substance to review since it is only removed for 99 % in the RO process (Bodalo, et al. 2005). This means that part of the ammonium will have to be removed before the water passes through the RO process. The SHARON®-Anammox®process is a combination of a partial nitrification process (SHARON®) and an anoxic ammonium oxidation (Anammox®) process. This process can remove more than 80% of the ammonium under low COD concentrations (van Dongen, Jetten and van Loosdrect 2001).

Apart from ammonium, an abundance of phosphate is also present in most domestic wastewaters. Most of this phosphate is present in the form of organic P. After hydrolysis in the anaerobic digestion, this organic P is released as ortho-phosphate. The Magnesium Ammonium Phosphate precipitation process (MAP) can remove phosphorus from water by converting magnesium ammonium and ortho-phosphate into struvite. This resource can be used as a fertilizer in agriculture (Bridger, Salutsky and Starostka 1962).

To find the optimal configuration of both nutrient removal processes in the scenarios, a model was created in Excel using removal percentages of phosphorus and ammonium found in literature (see Table 1).

Table 1 Removal percentages of the analyzed processes and their literature references

	Ortho-phospate removal	Ammonium removal	Reference
MAP	94%	1-10 %	(Munch and Barr 2001)
CNF	90%	12%	(Visvanathan and Roy 1997)&(Sayed, et al. 2007)
UASB	-28%	-15%	(Zeeman, et al. 2008)
SHARON-Anammox	-	80%	(van Dongen, Jetten and van Loosdrect 2001)
RO	99,9%	99%	(Bodalo, et al. 2005)
FO	99,6%	85%	(Holloway, et al. 2007)

Apart from these removal percentages, also influent concentrations of phosphorus and ammonium of grey, black and combined wastewater are needed to complete the model. These can be found in Table 2.

Table 2 Phosphate and ammonium concentrations in different wastewaters

	Ortho-phosphate	Ammonium	Reference
Grey	5,8	0,6	(Leal, et al. 2010)
Black	280	1400	(Zeeman, et al. 2008)
Combined	6.7	36.4	(Weij 2013)&(Brouwer and van de Giesen 2006)

Figure 20 Scenario 1 with ammonium, ortho-phosphate and flow indication by Excel model with recirculation

When applied, the model gives an indication of the concentrations of ammonium and ortho-phosphate in each stream for every scenario. Also, flows are given in the model as an indication of the amount of demi-water produced per scenario. All flows are derived from a starting flow of 1 L. When applied to the first scenario (see Figure 20), because of the recirculation of effluent from the anaerobic digester back to the CNF process, ortho-phosphate accumulates in the loop. Because of the low retention of ammonium by the CNF process, this does not occur with ammonium; there an equilibrium situation is reached. The same would occur for the other scenarios since the loop is present there as well. Apart from ortho-phosphate, nanofiltration also has a high removal rate of divalent ions such as calcium and magnesium (Eddy 2004). These ions would also accumulate in the loop since none are well removed by the anaerobic digestion process.

Since the water recovery of the CNF process is 80%, removing the recirculation of anaerobic digester effluent from the scenarios has a limited effect on the amount of demi-water that is produced. This would solve the problem of accumulation of particles. After removing the recirculation loop, the model gives better results, as can be seen in Figure 21.

Figure 21 Scenario 1 with ammonium, ortho-phosphate and flow indication by Excel model

Instead of recirculating the effluent from the anaerobic digester to the CNF process, it offers opportunities to reuse as process water after further treatment. As can be seen from Figure 21, the high ortho-phosphate and ammonium concentrations in the anaerobic digester effluent make this an ideal location to apply nutrient recovery. By placing the MAP process, followed by the SHARON®-Anammox® process, both nutrients can be recovered. The MAP process also requires magnesium to be present at a ratio of 1 to 3.91 mg/l ortho-phosphate. Wastewater contains approximately 15 mg/l of magnesium, of which 35 % is rejected by the CNF process (Llenas, et al. 2011). This means that 5 mg/l of magnesium will pass through to the MAP process. This is not enough to run the MAP process under optimal conditions, so magnesium will have to be added.

Figure 22 shows the model for scenario 1, with implementation of both processes. Also, the solids accumulated on the sieve can be fed into the anaerobic digestion process, to increase its biogas production.

Figure 22 Scenario 1 with nutrient recovery

When we apply nutrient recovery to scenario 2, it differs only slightly from scenario 1 since the only change is the replacement of the CNF process by the FO process. However, because of the operation of the FO-process, no RO rejection is present since this is fed back into the FO process to act as a draw solution. Applying the nutrient recovery to scenario 2 results in the model shown in Figure 23. As can be seen, the amount of demi-water produced for every liter of wastewater is lower compared to scenario 1 (0.7 L versus 0.76 L). This is caused by the lower recovery of the FO process. The water does however contain less ammonium, which is retained more by the FO process compared to CNF.

Figure 23 Scenario 2 with nutrient recovery

When nutrient recovery is applied to scenario 3, it will be done at the black water treatment side of the scenario since this side contains the streams with the highest concentrations of nutrients. Here also the recirculation loop is removed. However, the concentrate water from the CNF will still be added to the anaerobic digester to increase the reuse potential of the water.

Figure 24 Scenario 3 with nutrient recovery

As can be seen from Figure 24, the amount of demi-water produced in scenario 3 is similar to scenario 2. Its quality however, is much better. The cause of this is the separated collection. The black water contains most of the nutrients and is treated separately. When nutrient recovery is applied to scenario 4, the model is comparable to scenario 3. See Figure 25 for the model of scenario 4.

Figure 25 Scenario 4 with nutrient recovery

As with scenario 2, nutrients are accumulating in the draw solution of the FO-RO process. This draw solution will have to be regenerated periodically to remove these nutrients.

3.4   Overview final scenarios

Table 3 shows an overview of the water produced and its quality per scenario. The production of demi water and effluent has been expressed as a percentage of the inflowing wastewater. It can be seen that the percentages do not add up to 100 %. This is caused by the removal of sludge from the anaerobic digester, which still contains some water (10% of the digested water (Eddy 2004)).

Table 3 Overview water production & quality

	Demi water			Effluent		
Parameter	Production	PO4	NH4	Production	PO4	NH4
Unit	%	mg/l	mg/l	%	mg/l	mg/l
Scenario 1	76	<0.001	0.5	22	0.8	32.8
Scenario 2	70	<0.001	0.1	27	0.6	33.1
Scenario 3	70	<0.001	0.01	27	2.6	90.7
Scenario 4	64	<0.001	0.001	32	2.2	76.6

It is hard to compare the scenarios on these figures alone. To make a solid comparison, more information will be needed on the energy consumption of each scenario. This will be dealt with in the next section.

3.5   Energy consumption scenarios

When comparing the scenarios based on energy consumption, it is important to establish a comparable parameter for all the treatment processes that are used in the scenarios. This parameter is set to be kWh/m^3 total treated wastewater. For the scenarios with source separation of wastewater this means the energy consumption will be expressed as the consumption over the black water and grey water collectively. Also, in order to make a complete overview, the energy consumption of the wastewater collection and eventual water savings at the drinking water treatment plant will also have to be taken into consideration.

Furthermore, the energy consumption of treatment processes that are present in each scenario are discarded because they will be same for each scenario (i.e. MAP & SHARON®-Anammox® and pumping of water from/to the buffer tanks).

Table 4 Overview of energy consumption of each scenario at CNF recovery of 80% and FO recovery of 70% (kWh/m³ treated wastewater)

Process	Sc. 1	Sc. 2	Sc. 3	Sc. 4
Conventional sewer	0.08	0.08	0.07	0.07
Vacuum sewer	-	-	0.14	0.14
Kitchen grinders	-	-	0.11	0.11
Ceramic NF	0.8	-	0.8	-
Forward Osmosis	-	0.2	-	0.2
Digestion	-0.76	-0.84	-2.69	-2.69
Heating	1.20	2.24	-	-
RO	1.0	2.5	1.0	2.5
Savings on drinking water production	-	-	-0.2	-0.2
Total	2.32	4.18	-0.77	0.13

Table 4 shows the energy consumption of each scenario. The energy consumption of the conventional sewer is based on a daily wastewater production of 144 l/p/d (wRw 2002). This translates into 53 m³/p/year. Combined with the energy consumption of a conventional sewer of 4 kWh/p/y (Blom, et al. 2010). This gives us an energy consumption of 0.08 kWh/m³ transported wastewater. It is estimated that the energy consumption of CNF does not differ much from conventional polymeric nanofiltration (which has been extensively documented). This energy consumption lies around 0.8 kWh/m³ (Lazarova, Choo and Cornel 2012).

The biogas production is dependent on the COD values of the wastewater. On average 0.4 m³ of biogas is produced per kg of COD (van Lier, Mahmoud and Zeeman 2008). With average COD values in raw wastewater (dry weather flow) of 800 mg/l (Almeida, Butler and Friedler 1999) and a COD rejection by the CNF process of 87 % (Sayed, et al. 2007) and 99 % by the FO process (Holloway, et al. 2007) a balance can be made for the COD values of the influent of the anaerobic digestion process. A detailed calculation on the energy required for heating and the energy production through biogas can be found in Appendix 5. The difference in energy required for heating between the scenarios is caused by the differences in the amount of water that has to be treated for each scenario. Also, because the grey water has a higher temperature than combined wastewater (25℃ (Zeeman, et al. 2008)), the energy required for heating in scenario 3 and 4 is lower. Compared to other applications of reversed osmosis, its energy consumption is in this case rather low. The reason for this is the low ion/mineral content of the wastewater. Because of this low content, the required pressure and therefore the energy consumption is low. It is estimated to be 1.0 kWh/m³ filtered wastewater (Guillen and Hoek 2009).

The energy consumption of the second and fourth scenario is higher than the first and third scenario. This is caused by application of the FO process. This process, as explained in paragraph 2.1.5 requires a salt draw solution on its permeate side. The permeate of the FO mixes with the draw solution, decreasing its osmotic potential. To counter this effect, the osmotic pressure of the draw solution is kept constant by filtrating it with the RO process. However, because of the high concentrations of salt in the draw solution (35-60 mg/l (Holloway, et al. 2007)) the pressure required for the RO process is higher compared to the pressure required in the first scenario. Thus more energy is required to operate the RO process. This is approximately 2.5 kWh/m³, and the energy required for the FO process is approximately 0.2 kWh/m³ (Holloway, et al. 2007).

In scenario 3 and 4 a vacuum sewer is used to transport the black water to the treatment plant, while the grey water is transported using a conventional sewer. Also, the black water is collected using vacuum toilets, which leads to a drinking water saving of 14.6 m^3/p/y. This leads to an energy saving of approximately 0.2 kWh/m^3 transported wastewater (grey + black (Zeeman, et al. 2008)). The energy consumption of a vacuum sewer is 1.4 kWh per transported m^3 of black water (Quatfass 2012). This is very high compared to a conventional sewer system, which is a result of pumping to maintain a vacuum within the sewer system. However, only 10% of the water is black water, and thus the energy consumption over the total wastewater is only 0.14 kWh/m^3. The energy consumption of the conventional sewer is therefore also lower, since it only transports 90% of the total wastewater (0.07 kWh/m^3). This vacuum sewer also allows the installing of kitchen grinders, which can collect kitchen waste. The addition of this kitchen waste to the black water can increase the biogas production by a factor 2. The energy consumption of the kitchen grinders is approximately 0.11 kWh/m^3 (Zeeman, et al. 2008).

3.6 Overview energy consumption scenarios

Of all the scenarios, scenario 3 has the lowest overall energy consumption. This is mainly caused by the fact that only 10% of the water has to be heated for digestion, and the extra energy input from added kitchen waste. The same is also the case for scenario 4. It can also clearly be seen from the table that no external energy is required for heating of the water in scenarios 3 and 4. The water is heated up using the heat that is released by the conversion of biogas into electricity. In scenario 1 and 2, this energy is not sufficient and external energy will have to be used. To increase the production of biogas, the recovery of both the CNF and FO processes can be increased to 90 % recovery (see Table 5). Also, another possibility might be the application of an external (rest) heat source. In the Merwehaven-Vierhavens area where the wastewater is treated, an EON heating factory (for the warm water net of the city of Rotterdam) is also present. It might be possible to use rest heat from this factory to heat up the water for digestion. This would greatly benefit the energy recovery from the wastewater.

Also, compared to the production of demi-water from conventional WWTP plants, up to 4 kWh/m^3 wastewater (Blom, et al. 2010) is saved because no aerobic processes (and thus no aeration) are present in these scenarios.

Table 5 Overview of energy consumption of each scenario at CNF and FO recovery of 90 % (kWh/m^3 treated wastewater)

Process	Sc. 1	Sc. 2	Sc. 3	Sc. 4
Conventional sewer	0.08	0.08	0.07	0.07
Vacuum sewer	-	-	0.14	0.14
Kitchen grinders	-	-	0.11	0.11
Ceramic NF	0.8	-	0.8	-
Forward Osmosis	-	0.2	-	0.2
Digestion	-0.76	-0.84	-2.69	-2.69
Heating	0.06	-0.09	-	-
RO	1.0	2.5	1.0	2.5
Savings on drinking water production	-	-	-0.2	-0.2
Total	1.18	1.85	-0.77	0.13

3.7   Scenario comparison

Now that we have an indication of the energy consumption of each scenario and their water qualities and quantities of each scenario we can make a comparison. Although scenario 1 has the highest demi-water production, its demi-water also contains the highest concentration of ammonium. Furthermore, its energy consumption is high compared to scenario 3 and 4. Scenario 2 has a better water quality than scenario 1 but it has the highest energy consumption. This means the preliminary choice will be between scenario 3 and 4. Of the two, scenario 3 has the lowest energy consumption, indicating this will be the best choice.

It has to be mentioned though that this information is all based on a desktop study of various research papers related to the treatment processes. Therefore lab-experiments have to be conducted to see whether this scenario is the best choice in practice. The lab-experiments will be covered in the next chapter.

4 Experimental Research& Results

The experimental research is meant to give an insight into the functioning of one of the treatment processes discussed in the previous chapter. From the scenario comparison in the previous chapter it can be seen that the scenario using ceramic nanofiltration is most promising. It is therefore decided to investigate the behaviour of this treatment technique on lab scale.

4.1 Introduction

The experimental research was conducted at the sanitary engineering laboratory of Delft University of Technology. The aim of this experimental research was to observe the behavior of a ceramic nanofiltration membrane when it is directly fed with either grey water or strained raw wastewater. This behavior can influence our choice between a separate sewerage collection system and a mixed sewerage collection system. Also, water samples were taken to be able to determine the composition of the influent, concentrate and permeate. The composition of the concentrate will gain us some insight into the amount of biogas that can be produced.

4.2 Materials and methods

4.2.1 Membrane

The ceramic nanofiltration membrane used during the experiments was supplied by Inopor (Veilsdorf, Germany). The specifications of the membrane can be found in Table 6 and Figure 26.

Table 6 Membrane specifications

Parameter	Value
MWCO	450 Da
Material	TiO_2
Clean water flux (at 1 bar)	20 $L/m^2/h$
Surface area	0.25 m^2
Configuration	Inside-Out
Diameter (channels)	25 mm (3.5 mm)
Number of channels	19
Length	1200 mm

Figure 26 Membrane cross-section

4.2.2 Experiment overview & conditions

The following experiments were conducted:

- Water recovery of 80% for both grey and raw wastewater (experiment 1 and 2)

- Experiment with double flux with raw wastewater for fouling analysis (experiment 3)

- Experiment with half crossflow velocity with raw wastewater for fouling analysis (experiment 4)

- Short one pass experiment with raw wastewater for ion rejection determination (experiment 5)

If otherwise stated, the conditions of all experiments were as follows (see Table 7).

Table 7 Conditions of experiments

Parameter	Value
Feed configuration	Recirculation
Crossflow velocity (feed)	0.46 m/s (R = 5000)
Flux	20 L/m^2/h
Resulting permeate flow from flux	5 L/h
Temperature	21 ℃
Crossflow pressure	3.5 bar
Backwash frequency	15 min.
Backwash duration	1 min.
Backwash flux	28 L/m^2/h (flow of 7 L/h)
Crossflow during backwash (forward flush)	0.69 m/s (R = 7500)

After each experiment cleaning in place (CIP) was executed using a solution of NaOCl (1% Cl) where the membrane was soaked at least 1 hour. As a backwash water source, demi-water was used. As can be seen in Table 7, the crossflow velocity provides turbulent flow in the channels of the membrane. This is needed to scour the fouling of the membrane under normal operating conditions. A flux of 20 L/m^2/h was applied, which is normal for nanofiltration(Eddy 2004). A crossflow pressure of 3.5 bar was applied to maintain a steady flux during the experiments. An enhanced backwash (backwash + forward flush) was conducted every 15 minutes to remove any fouling that is formed on the membrane during operation. The used equipment only allowed for a maximum backwash flux of 28 L/m^2/h.

4.2.3 Collection of source water

Raw wastewater was collected behind the influent rosters (first step of treatment) of WWTP Harnaschpolder in Delft. The water was collected behind the rosters to protect the membrane during the experiments (the rosters have a distance of 4 mm between each plate). Grey water was collected at the community of Noorderhoek in Sneek (see Appendix 6). The water was pumped from a grey water collection tank. This water had not undergone any treatment.

4.2.4 Experimental setup

Figure 27 Diagram of the experimental setup used during the experiments

Figure 27shows the experimental setup used during the experiments. To reach a recovery of 80% in practice a tree-like setup is usually used. However, because the experiments have to be small scale, this situation was mimicked by recirculating the feed water back into the feed vessel. With a recirculation flow of 300 l/h and a permeate flow of 5 l/h, only 1.7 % of the water entering the membrane module passes through the membrane to the permeate side. The other 98.3 % was recirculated back to the feed vessel. Cooling of the feed water had to be applied because the feed pump heats the feed water during operation. For cooling, two spools attached to two cooling baths were applied. These kept the feed water temperature constant. This is necessary because the temperature can affect the viscosity of the water and hence the physical separation process within the CNF module.

Control of the experimental conditions (Table 7Table 7 Conditions of experiments) was carried out using an OSMO-Inspector (Convergence Beheer BV., Deventer). All flows in and out of the vessels, pump and membrane were controlled and adjusted from this machine. The OSMO-inspector also controlled the pressures applied to the flows. Since a constant crossflow pressure had to be used (because the OSMO-Inspector cannot operate on a variable crossflow pressure), the flux was kept constant by an automated valve on the permeate side. This valve opened slowly as the TMP increased. This is necessary because fouling will increase the membrane resistance and thus the TMP. By opening this valve slowly while the TMP increases, the flux was kept constant. The OSMO-Inspector and its setup screen are shown in Figure 28.

Figure 28 OSMO-Inspector and its setup screen

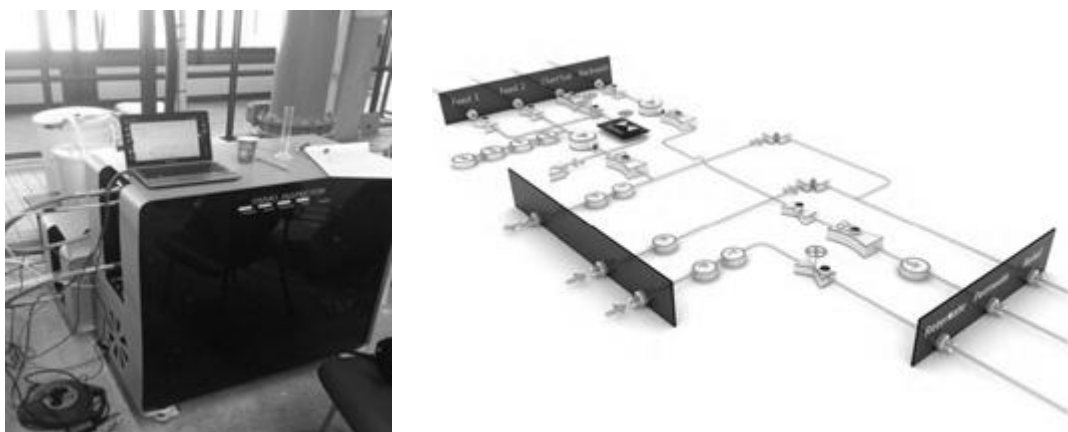### 4.2.5   Experiment 1 and 2 - 80 % Recovery

In experiment 1, the membrane module was fed with 50 L of raw wastewater (strained at 4 mm at the rosters of Harnaschpolder). Goal of the experiment was to observe the fouling and operation with a recovery of 80%. The conditions mentioned earlier in Table 7 were applied. During the experiment several samples were taken from the influent, concentrate and permeate to measure for water quality and composition. At the start of the experiment a sample was taken of the feed water. When 80% recovery was reached (40 L of permeate, 10 L of concentrate) samples were taken from the concentrate and permeate (mixed). The permeate sample had to be taken from the mixed permeate in the permeate vessel and not straight from the permeate outlet since the permeate quality would change during experiment. Because the feed water is concentrated during the experiment, the concentrations of substances in it change. This influences the permeate quality. In practice when a tree-like setup is used, the permeate from the first module will be mixed with the permeate from the last module as well, this is why a mixed permeate sample had to be taken instead of a sample straight from the permeate outlet.

Apart from water quality samples, several physical parameters were also monitored and recorded by the OSMO-Inspector during the experiments (see Table 8).

Table 8 Physical parameters monitored by the OSMO-Inspector

Flows	Pressure	Temperature
Feed	Feed	Feed
Concentrate	Concentrate (Crossflow)	
Permeate	Permeate	
Backwash		

In experiment 2, the membrane module was fed with grey wastewater (unstrained). This experiment was conducted in the same way as experiment 1.

4.2.6   Experiment 3 - Double flux

The goal of this experiment was to observe the fouling characteristics when a double flux was applied. The experiment was conducted in a similar manner as experiment 1 and 2. However the experiment was run only for a short time (30 minutes) since 80% recovery did not have to be reached. A flux of 40 L/m^2/h (10 L/h) was applied. The physical parameters shown in Table 8 were monitored and recorded again but no water quality samples were taken.

4.2.7   Experiment 4 - Lower crossflow velocity

The goal of this experiment was to observe the fouling characteristics when the crossflow velocity is decreased by half. Again, the experiment was conducted in a similar manner as

experiment 1 and 2, but only run for 30 minutes. A crossflow velocity of 0.23 m/s was applied. Here also the physical parameters shown in Table 8 were monitored and recorded but no water quality samples were taken.

4.2.8   Experiment 5 - Salt retention

The goal of the third experiment was to gain insight into the salt rejection of the membrane. The experimental setup was changed because a separate concentrate stream needed to be measured. In the previous experiments the feed water was recirculated, however this causes a constant change in the concentration of salts in the feed water as salt is retained in the concentrate and then fed back into the feed water vessel. Therefore, in this experiment, the concentrate was collected in a separate tank (see Figure 29). Because the membrane is cleaned in place with a solution of NaOCl (1% Cl) after each experiment, this solution is present in the membrane module before each experiment. To prevent cross-contamination, this solution has to be flushed out with demi-water. After this has been done, the demi water on the feed side of the membrane module is flushed out with raw wastewater. However, because the permeate side cannot be flushed with raw wastewater (it would pollute the permeate side of the membrane) the salt retention experiment is conducted for 20 minutes. With a flow of 5 L/h and an estimated volume of the permeate side plus hoses of 1 L, it will take a minimum of 10 minutes before the demi-water has been completely flushed from the permeate side.

Figure 29 Experimental setup salt retention experiment

To be sure no demi-water was left in the permeate side of the membrane module, the experiment was run for 20 minutes before taking samples from the feed vessel, concentrate outlet and the permeate outlet. Both the concentrate and permeate samples were not taken from the vessel because in this case the salt concentrations of the water after 20 minutes of running were needed, not those mixed over 20 minutes. Because the experiment had to run for 20 minutes, 120 L of raw wastewater was supplied by the feed vessel (instead of 50 L as in experiment 1 and 2). Also, no backwash was applied during the 20 minutes of running, since this would contaminate the permeate side with demi-water. Backwashes were applied both before and after the experiment, as was a CIP. The crossflow and crossflow pressure and permeate flow were kept at the same values as used in experiment 1 and 2 (see Table 7). The experiment was conducted at different pH values to evaluate the influence of the pH on the removal of ortho-phosphate since different forms of ortho-phosphate exist at different pH values (see Figure 30) (Zeng 2012).

Figure 30 Ortho-phosphate forms under different pH values (Zeng 2012)

4.2.9 Water quality measurements

The water quality samples taken in experiment 1 and 2 were measured for COD using the COD Cell Test (Spectroquant). Only soluble COD was measured since these tests are analyzed using a spectrophotometer and when particles are present in the solution, the measurement gives false readings. Therefore all samples were filtered with a 45 µm filter before measuring. DOC was measured using a TOC analyzer, with pre-filtration of samples using a 45 µm filter before measuring. These samples were also filtered with a 45 µm filter before measuring. The pH and conductivity were measured using a inoLab Multi 720 meter (WTW). The turbidity was measured using a 2100N Turbidimeter (Hach). The samples taken during experiment 5 were measured using a ProfIC ion chromatography system (Metrohm). This system is capable of measuring various ions, however the following ions were measured for the pre-filtered samples taken during experiment 5:

- Ammonium
- Ortho-Phosphate
- Nitrate
- Calcium
- Magnesium
- Chloride
- Sulfate

4.3 Results and discussions

4.3.1 Water recovery 80 %

The following paragraphs contain the physical results experiments 1 and 2.

Raw wastewater

Figure 31 shows the trans membrane pressure (TMP) during the 80 % recovery experiment. Clearly visible are the backwashes, which were executed every 15 minutes. Because of membrane fouling the TMP is increasing during each 15 minute run. A small part of this fouling is removed during each backwash. The effect of each backwash seems to be limited however, since only a small part of the fouling is removed. However, the amount of total fouling is low, especially considering the fact that raw wastewater was used.

Figure 31 Raw wastewater 80 % recovery TMP

Around 180 minutes the TMP shows a sudden increase from 0.75 bar to 1.2 bar. This is probably caused by the fact that the experiment was stopped overnight. During that time, bio growth may have occurred on the membrane hence increasing the TMP when the experiment was resumed the following morning. Also, the temperature slightly dropped (see Figure 32), which caused an increase in TMP. Also notable is the decrease in TMP after 550 minutes, there the temperature increased because the cooling system ceased to work, hence decreasing the TMP. The total increase of TMP over the entire experiment is about 1.5 bar (0.14 bar/hour). Data on the other physical parameters can be found in Appendix 1.

Figure 32 Raw wastewater 80 % recovery feed water temperature

Grey water

Figure 33shows the TMP of the grey wastewater 80 % recovery experiment (red). As can be seen the TMP increase during this experiment is much higher compared to the experiment with raw wastewater (blue). This is caused by the composition of the grey wastewater. Compared to raw wastewater, grey water contains less particles. It does however contain large amounts of humic acids (as reported by Nghiem, Oschmann and Schafer, 2005/2006), which combined with the presence of calcium and magnesium leads to the formation of a dense cake layer (Oschmann, Nghiem and Schafer 2005) (Nghiem, Oschmann and Schafer 2006). This dense cake layer causes a rapid increase of the TMP. It can be seen however that the backwash is better capable of removing this fouling compared to the backwash with raw wastewater. However, because the fouling rate is so high, the maximum TMP (with a crossflow pressure of 3.5 bar) was reached after 180 minutes.

Figure 33 Grey water 80 % recovery TMP (red) with raw wastewater 80% recovery TMP (blue)

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At that moment, the crossflow pressure was increased to 8.5 bar to overcome the fouling. This however had an exponential effect on the fouling as can be seen in Figure 33. When lowered to 7 bar, the maximum TMP was again reached and the permeate flow began to decrease sooner after each backwash. After 260 minutes it was decided to execute a CIP in order to be able to continue the experiment. The CIP was capable of removing all the fouling as can be seen in Figure 33. Because the concentration of substances in the grey water increase during the experiment, the CIP had to be conducted sooner after the first one, compared to the first CIP and the start of the experiment. More cleanings in place had to be conducted after 440, 460, 540 and 590 minutes. Figure 53 and Figure 55 in Appendix 2 show the permeate flow and feed pressure during the experiment. During the experiment the average increase of TMP was about 1 bar/h. However, after a certain threshold is reached (2.5 bar) the TMP is increasing exponentially. More data on the other physical parameters can be found in Appendix 2.
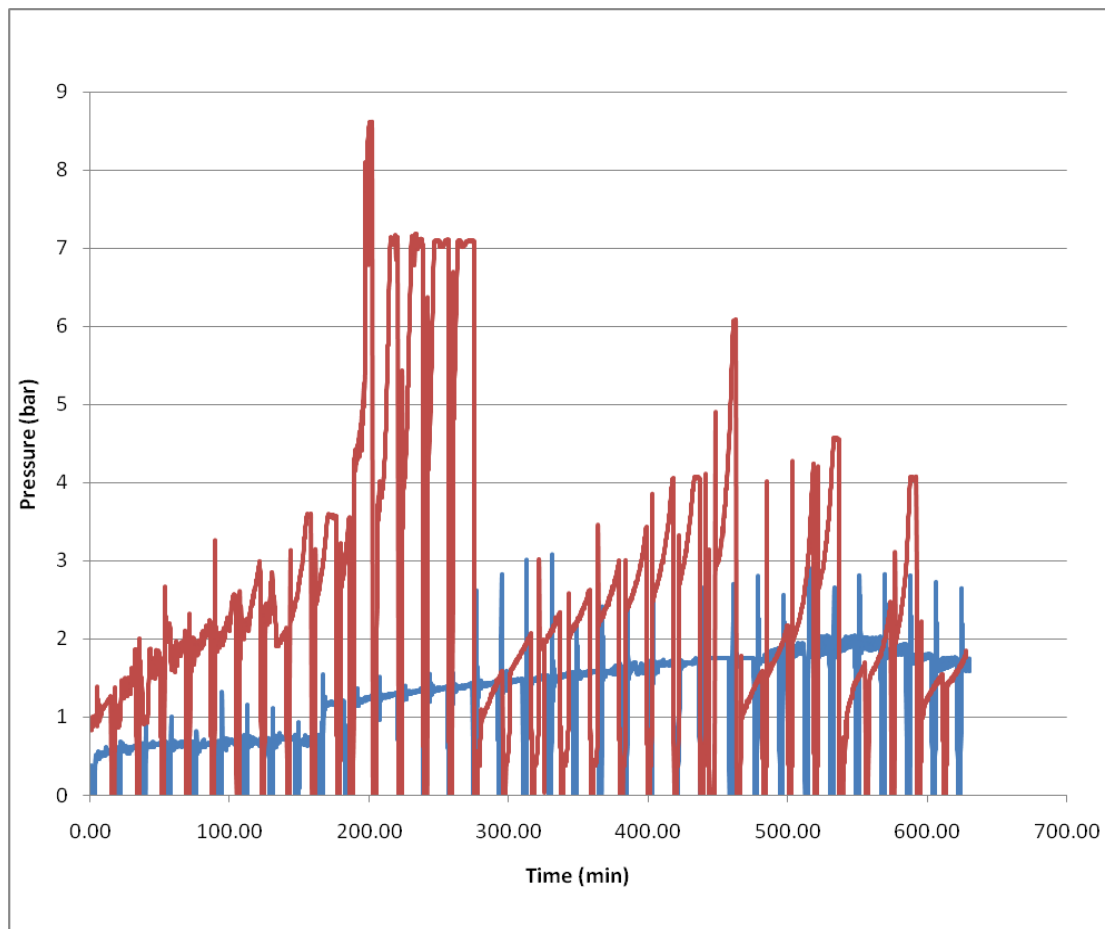
4.3.2 Double flux

Figure 34 shows the TMP of experiment 3. The flux during this experiment was set at 40 $L/m^2/h$, to analyze the effect of a double flux on the fouling of the membrane. Raw wastewater was used as feed.

Figure 34 Double flux TMP (blue) with raw wastewater experiment 1 TMP (red)

As can be seen from Figure 34, with a double flux, the TMP increase is about 3 bar/h. Compared to experiment 1, this increase in TMP is significantly higher. The reason no backwashes were executed is that the effect of the backwashes in experiment 1 was so low, the effect they would have in this experiment was considered negligible. At 37 minutes, the TMP shows a sudden increase. When we look at the permeate flow at that time (see Figure 35), we notice a sudden increase as well. After review of the permeate valve of the OSMO-Inspector once the experiment was finished, it was noticed that the valve was not operating properly. This caused the sudden increase in permeate flow and thus also the increase in TMP. The valve was fixed after this experiment. More data on the physical parameters of this experiment can be found in Appendix 3.

Figure 35 Double flux permeate flow

4.3.3 Low crossflow velocity

During this experiment the crossflow velocity was decreased by half to 0.23 m/s (150 L/h). Raw wastewater was used as feed.

Figure 36 Lower crossflow TMP (blue) with raw wastewater experiment 1 TMP (red)

As can be seen from Figure 36, the TMP increase during this experiment is small. Over one hour the increase is approximately 0.25 bar. This is almost twice as high compared to the TMP increase noticed in experiment 1. This seems to indicate that the crossflow velocity and fouling seem to be cross-linked. When the crossflow velocity is doubled, the fouling is decreased by half. More physical data on this experiment can be found in Appendix 4.

4.3.4   Salt retention

Since salt retention is constant with varying salt concentrations, the salt retention experiment is only conducted with raw wastewater. The salt rejection will be similar for grey water.

Table 9 Salt retention at different pH

Salt retention (%)	Sample time: 15 minutes		Sample time: 20 minutes	
	pH = 7	pH = 9	pH = 7.6	pH = 8.5
Chloride	0	0	0	0
Nitrate	-	-	-	-
Ortho-phosphate	99.5	68.8	94.3	81.5
Sulfate	51.6	14.5	5.1	4.7
Ammonium	8	2.6	8.4	17.1
Magnesium	2	13.8	1.9	5.5
Calcium	0.2	17.8	2	9.3

Table 9 shows the salt retention at different pH values. As can be seen, the removal of ortho-phosphate at pH 7 is 99.5%. This value seems unrealistic and contradicts values found in other literature (Zeng 2012) (Visvanathan and Roy 1997). A possible reason for this may be sample time. The sample was taken after 15 minutes, at that time, some demi-water from the flushing before the experiment may still be present at the permeate side of the membrane. This dilutes the sample, and thus increases the retention value of ortho-phosphate (and the other ions). Therefore 2 more experiments were executed with a later sample time (20 minutes). This gives an ortho-phosphate retention of 94.3 % which corresponds better with values found in literature. At a pH of 8.5 and 9 the concentration of ortho-phosphate is lower which suggest the retention is highest at a pH of 7.6. At a lower pH the retention will lower as well, as described by (Zeng 2012). The retention of chloride and sulfate and ammonium is low at all pH values. This is logical since these are mono-valent ions which are not well retained by nanofiltration membranes (Eddy 2004). The pH was increased during the experiment by the addition of NaOH. This reacted with the calcium and hydroxide forming $Ca(OH)_2$ and $Mg(OH)_2$. This explains the higher retention values of calcium and magnesium at higher pH. Nevertheless, the retention values of both calcium and magnesium are very low for nanofiltration. This may be caused by the high molecular weight cut off (400 Dalton) of the membrane which is high for nanofiltration. The reason no retention values of nitrate were found is probably because the concentrations in the samples were too low to be accurately measured.

4.3.5   Other water quality parameters

Water quality samples were taken during the recovery experiments (experiment 1 and 2). The samples were taken at different stages throughout the experiments (at 20%, 40%, 60% and 80% recovery).

Turbidity, pH and conductivity

Table 10 shows the results of the turbidity and pH measurements. The samples were taken at different stages throughout the experiments (at 20%, 40%, 60% and 80% recovery). The turbidity of some of the permeate samples is high (>1) while the values of other samples are below 1. After opening the membrane module, it was found that some of the messing coupling pieces had leached into the water on the permeate side under influence of the addition of NaOCl used for the CIP. This might explain some of the high turbidity values of the permeate.

Table 10 Turbidity, pH and Conductivity of grey water and raw wastewater at different recovery

Turbidity, pH & Conductivity	Grey water			Raw wastewater		
	NTU	pH	µS	NTU	pH	µS
Permeate 20 %	4.28	7.43	521	3.63	7.619	1036
Permeate 40 %	10.8	7.378	511	0.66	7.592	968
Permeate 60 %	5.57	7.477	595	0.45	7.636	940
Permeate 80 %	0.94	7.462	692	3.13	7.703	916
Influent	104	6.801	672	29.2	7.504	948
Influent 20 %	127	6.725	671	35.6	6.913	926
Influent 40 %	143	6.835	624	40	7.008	945
Influent 60 %	177	6.678	678	41.3	7.012	938
Influent 80 %	242	6.863	750	67	7.032	968

When observing the turbidity of the influent samples, an increase is visible. This is caused by the concentrating of the influent during the experiments. The turbidity values of the grey water are overall significantly higher than those of the raw wastewater. This is caused by the detergents that are present in the grey water, these scatter the light more than the particles in the raw wastewater which influences the turbidity readings. In the raw wastewater the detergents are more diluted and they tend to connect to the particles in the raw wastewater, whereas there are almost no particles present in the grey water. The pH is almost constant throughout the experiment. The conductivity retention of both grey and raw wastewater is close to 0, where grey water even has a negative retention of -3 %. This is most likely caused by taking the last permeate sample too close to the location where the concentrate stream enters the tank. With raw wastewater, a conductivity retention of 3.4 % was measured.

DOC

Table 11 shows the DOC concentrations and retention that were measured during the recovery experiments. The amount of DOC in the grey water is considerably higher than in raw wastewater. However, its retention is also much higher. A possible explanation for this is the dense cake layer that is formed during the filtration of grey water. This rejects more of the DOC, while the DOC passes more easily through the more coarse cake layer formed with raw wastewater filtration. The only outlying value is the concentration of DOC in the grey water influent at 80 % recovery. This is most likely a measurement error since the previous values show a constant increase and the permeate value at 80 % recovery remained constant compared to the value at 60 % recovery.

Table 11 DOC concentrations and retention measured during the recovery experiments

DOC concentrations (mg/l)	Grey water	Raw wastewater
Permeate 20% Recovery	1.3	40.1
Permeate 40% Recovery	1.2	39.5
Permeate 60% Recovery	0.7	38.0
Permeate 80% Recovery	0.5	48.0
Concentrate 0% Recovery	149.2	116.3
Concentrate 20% Recovery	150.2	109.4
Concentrate 40% Recovery	224.4	122.9
Concentrate 60% Recovery	248.6	136.9
Concentrate 80% Recovery	120.2	146.2
Retention %	99.7	67.0

COD

During the recovery experiments samples were taken to measure the soluble COD. The results are shown in Table 12. Although the concentrations show an increase of concentration at higher recovery, the overall retention of COD with both wastewaters is low compared to values found in literature (Sayed, et al. 2007). Also, there should be a correlation between the COD and DOC retention, which is not present. Whether this is a measurement/sampling error or an unexplainable phenomenon has to be checked in further research.

Table 12 COD concentrations and retention measured during the recovery experiments

COD concentrations (mg/l)	Grey water	Raw wastewater
Permeate 20% Recovery	68	80
Permeate 40% Recovery	181	83
Permeate 60% Recovery	189	80
Permeate 80% Recovery	199	80
Concentrate 0% Recovery	292	115
Concentrate 20% Recovery	487	143
Concentrate 40% Recovery	541	151
Concentrate 60% Recovery	645	172
Concentrate 80% Recovery	672	174
Retention %	45.5	44.3

5 Conceptual design

In this chapter, a design is presented for the production of demi-water from wastewater from the Stadshavens area. Since scenario 3 gave the lowest energy consumption, it is chosen to design a wastewater reclamation plant based on this scenario.

5.1 Wastewater production Stadshavens

Table 13 shows the expected wastewater production of the Stadshavens area. The total wastewater produced was estimated based on an expected number of dwellings of 5500 which equals approximately 12000 people (Mulder and van Eijk 2010).

Table 13 Wastewater production indication used in the design

Parameter	Value
Total wastewater produced (indication)	2000 m^3/d
Saved by using vacuum toilets	500 m^3/d
Total wastewater produced (scenario 3)	1500 m^3/d
Of which: Grey water	57.5 m^3/h (1380 m^3/d)
Of which: Black water	5 m^3/h (120 m^3/d)

5.2 Dimensions ceramic nanofiltration

Based on the flux used in the experiments (20 L/m^2/h), and an expected water recovery of 90 %, 52m^3/h of permeate water has to be produced. It was chosen to use the M37-19-25-L module produced by Inopor GmbH since the module used in the experiments was also provided by this company. The internal membrane area of this module is 9.28 m^2. Combined with the flux and the amount of permeate that has to be produced, it was calculated that 275 modules are needed to satisfy this demand. Since the length of each module is only 1.2 m, 5 modules are placed in a pressure vessel in series. With a water recovery of 8.3 % per pressure vessel, it was decided to construct 5 pressure vessels in series. By putting the pressure vessels in series, the necessary crossflow velocity drop is manageable while still saving energy because less pumps are needed compared to when each pressure vessel would be supplied by an individual pump.

In the experiments a crossflow velocity of 0.42 l/h was used, in order to achieve this value in all pressure vessels (the crossflow velocity drops because less water is transported in the 5th pressure vessel compared to the 1st), a crossflow velocity of 0.61 m/s is applied for the first pressure vessel. In total 11 series of 5 pressure vessels are needed to achieve 90 % recovery of water. However, extra series will be needed since during maintenance and backwashing some trains will be shut down. It was decided to construct a total of 14 series of 5 pressure vessels to accommodate the shutdown of series during backwashing and maintenance. This is of course also beneficial for the reliability of the water production. Racks are constructed which contain 2 series of 5 pressure vessels each. A total of 7 racks are needed. See Table 14 for an overview of the dimensions and design specifications.

Table 14 Overview dimensions and design specifications ceramic nanofiltration

Parameter	Value
Permeate flow	51750 L/h
Flux	20 L/m^2/h
Crossflow velocity	0.62 m/s
Modules per pressure vessel	5
Number of pressure vessels in series	5
Number of series	14
Number of racks	7

5.3 Dimensions reverse osmosis

Based on a flux of 20 L/m^2/h (Eddy 2004) and a recovery of 95%, 49000 L/h of permeate has to be produced. This is achieved by using spiral wound RO modules with a cross section diameter of 15 cm. Since RO modules have an internal area of approximately 1000 m^2/m^3 (van Dijk, et al. 2009), it was calculated that 140 RO modules are needed. Constructed in series of 6 modules per pressure vessel, 25 pressure vessels are needed. Here also, because of maintenance and cleaning, 5 extra pressure vessels are constructed. This gives a total of 30 pressure vessels. These pressure vessels are constructed on racks, with each rack containing 10 pressure vessels. This gives a total of 3 RO racks. Table 15 shows an overview of the dimensions and design specifications.

Table 15 Overview dimensions and design specifications reverse osmosis

Parameter	Value
Permeate flow	49000 L/h
Flux	20 L/m^2/h
Modules per pressure vessel	6
Number pressure vessels per rack	10
Number of racks	30

5.4 Dimensions anaerobic digestion

Both the reject water from the RO process and CNF process and the black water is pumped to the anaerobic digestion process. This gives us an inflow of 258 m^3/d. With a hydraulic retention time (HRT) of 7 days (Zeeman, et al. 2008), a required volume of the anaerobic digestion process of 1806 m^3 was calculated. By constructing 2 anaerobic digester tanks with a diameter of 10 m and a height of 11.5 m this volume is achieved. See Table 16 for an overview of the dimensions and design specifications.

Table 16 Overview dimensions and design specifications reverse osmosis

Parameter	Value
Inflow	258 m^3/d
HRT	7 days
Volume	2 x 903 m^3
Diameter	10 m
Height	11.5 m

5.5 Dimensions magnesium ammonium phosphate precipitation

The MAP process requires a HRT of approximately 2 hours (Wilsenach, Schuurbiers and van Loosdrecht 2007). With an inflow of 10 m^3/h, this translates into a necessary volume of 20.4 m^3. This is achieved by constructing a tank with a diameter of 2.6 m and a height of 4 m. See Table 17 for an overview of the dimensions and design specifications.

Table 17 Overview dimensions and design specifications MAP

Parameter	Value
Inflow	10 m^3/h
HRT	2 hours
Volume	20.4 m^3
Diameter	2.6 m
Height	4 m

5.6 Dimensions SHARON®-Anammox®

The SHARON®-Anammox® process requires 3 tanks. One for the SHARON® process and 2 for the Anammox® process. The HRT is 24 h, 4.5 h and 0.75 h respectively. With the same flow

of the MAP process of 258 m^3, the SHARON$^®$ tank requires a volume of 255 m^3, while the Anammox$^®$ process requires 2 tanks with a volume of 47 m^3 and 8 m^3 respectively. See table 5 for an overview of the dimensions and design specifications. See Table 18 for an overview of the dimensions and design specifications.

Table 18 Overview dimensions and design specifications SHARON-Anammox

Parameter	Value
Inflow	10 m^3/h
HRT SHARON$^®$	24 hours
HRT Anammox$^®$ 1	4.5 hours
HRT Anammox$^®$ 2	0.75 hours
SHARON$^®$ Volume	255 m^3
SHARON$^®$ Diameter	6 m
SHARON$^®$ Height	9 m
Anammox$^®$ 1 Volume	255 m^3
Anammox$^®$ 1 Diameter	3 m
Anammox$^®$ 1 Height	6.6 m
Anammox$^®$ 2 Volume	255 m^3
Anammox$^®$ 2 Diameter	1.5 m
Anammox$^®$ 2 Height	4.5 m

5.7 Dimensions buffers, pipelines and building

For both the black and grey wastewater an estimation was made of the necessary buffer capacity of the primary buffers. Considering a diurnal household pattern it was estimated that a grey water buffer of 400 m^3 and a black water buffer of 40 m^3 were sufficient to cover the diurnal differences with an extra 25% volume for calamities. All the pipelines were constructed with a diameter corresponding to a flow of 1 m/s to prevent the settling of particles. To place all the processes and equipment, a building was designed with a floor area of 1100 m^3 (26m x 42 m) and a height of 11 m to accommodate the SHARON$^®$ tank (the highest tank in the building). The buffers and the anaerobic digester are constructed outside of the building. See Figure 37 for a flow scheme of the design.

5.8 Costs

Because of the early stage the RINEW project is currently in, it was decided not to cover the costs of the various facilities used in this design. Costs often tend to have a drastic effect on decision making, while this part of the RINEW project is focusing on the use of innovative solutions, which are not necessarily the cheapest solutions.
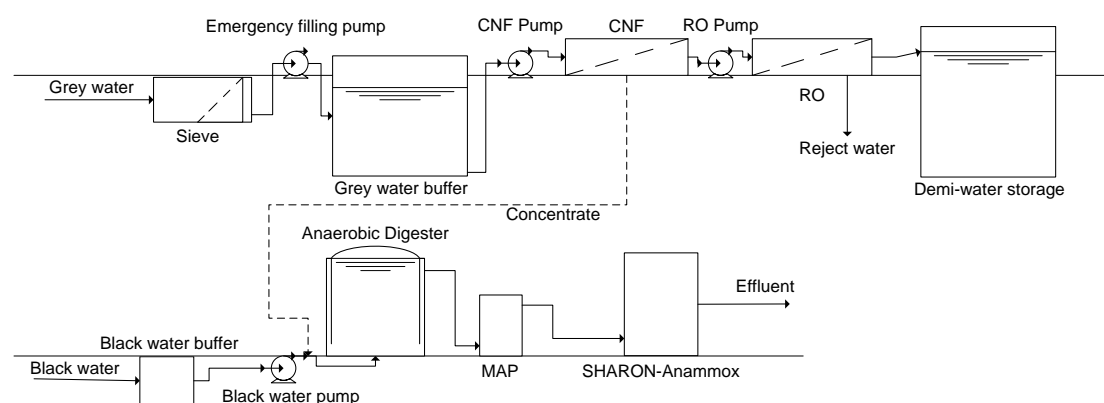

Figure 37 Flow scheme of the design

5.9   3D model of design

The following figures show a 3D model of the design. The design was constructed in Autodesk Autocad 2013 and resulted in a building with floor area of 42 x 24 meters and a height of 11 meters (to accommodate the SHARON® reactor and a lift to replace membrane modules).
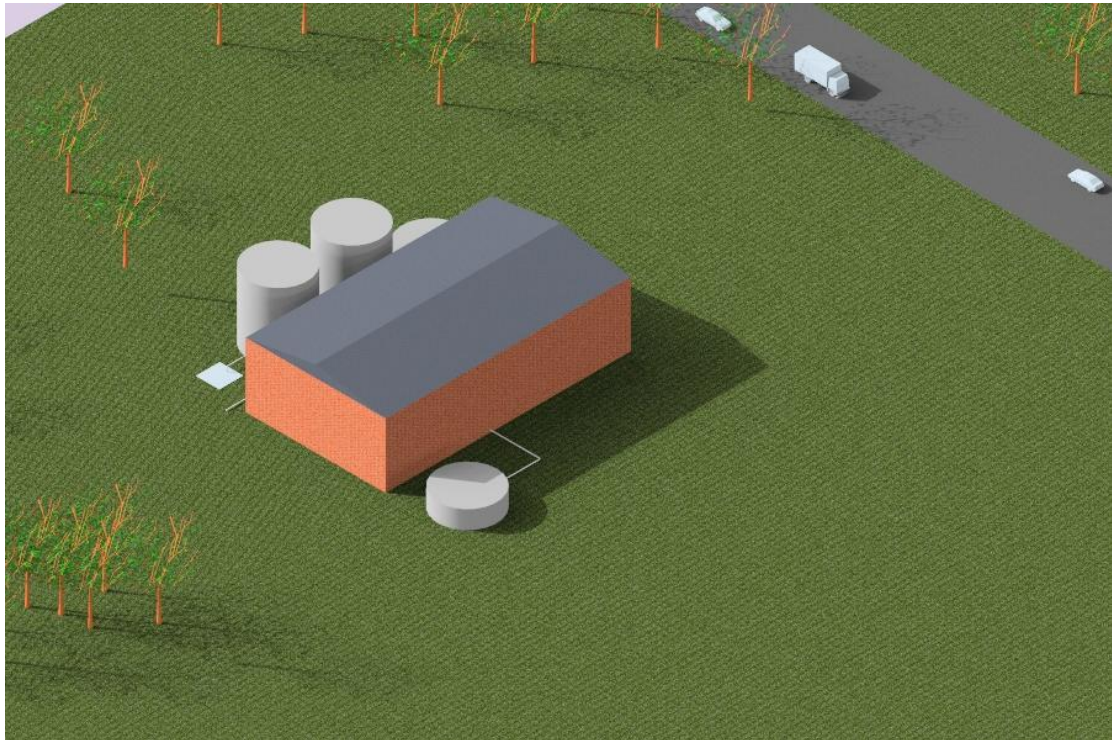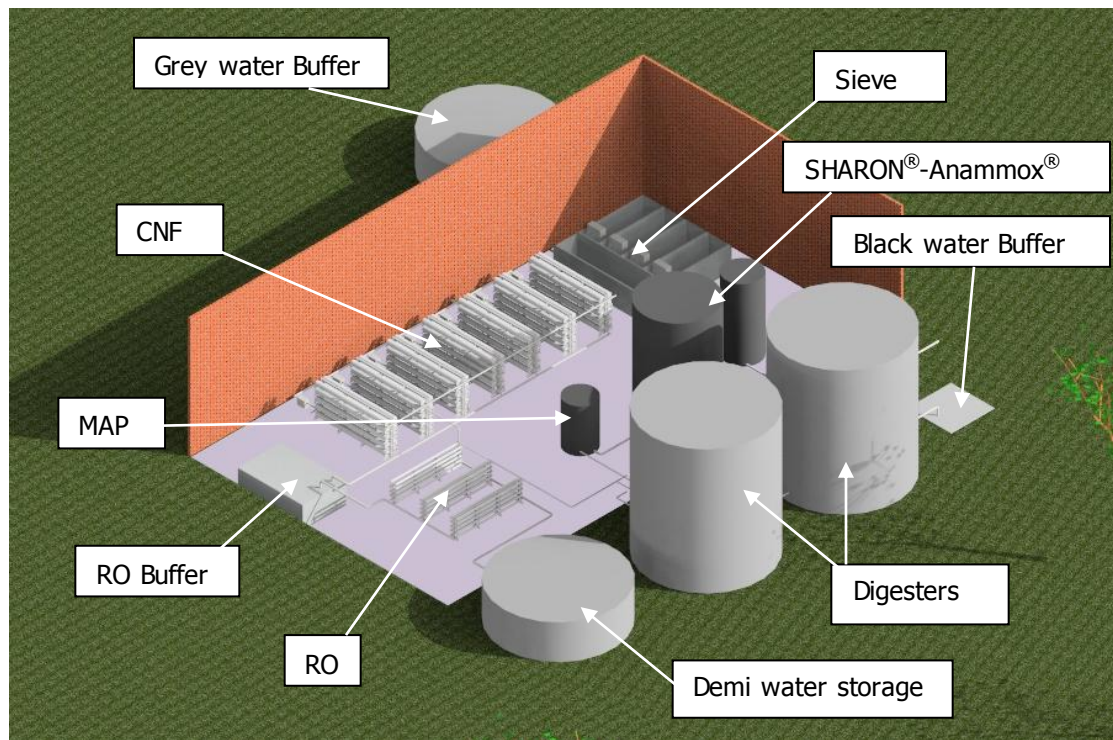

Figure 38 Overview of the treatment facility

Figure 39 Overview of treatment facility inside and outside

As can be seen from Figure 39, most treatment processes are located within the building, while the grey & black water buffer, the demi-water storage and the digesters are located outside. For the design, a spacious structure was chosen to facilitate the easy replacement and maintenance of the various systems. In a conventional wastewater treatment plant (WWTP) the water usually flows through most processes under the influence of gravity. In the Harnaschpolder WWTP for example, the sieves are located 10m above the ground. From there the water flows to the subsequent process steps, hence no pumps and thus no further energy is required to transport the water (apart from the pumps which transport the water up before the sieves). In the Stadshavens reuse plant this is not necessary because the first treatment step after the sieves is ceramic nanofiltration, for which pumps are needed to supply pressure and to be able to control the process. Hence all processes are located on the ground floor of the building.

Figure 40 See-through overview of the treatment facility

As can be seen from Figure 41 and Figure 42, more CNF than RO racks are present in the treatment plant while in conventional demi-water treatment facilities it's the other way around. This is caused by the fact that conventional polymeric nanofiltration membrane modules have a much higher surface to volume ratio than ceramic nanofiltration modules. Hence more volume is needed in the Stadshavens reuse plant.
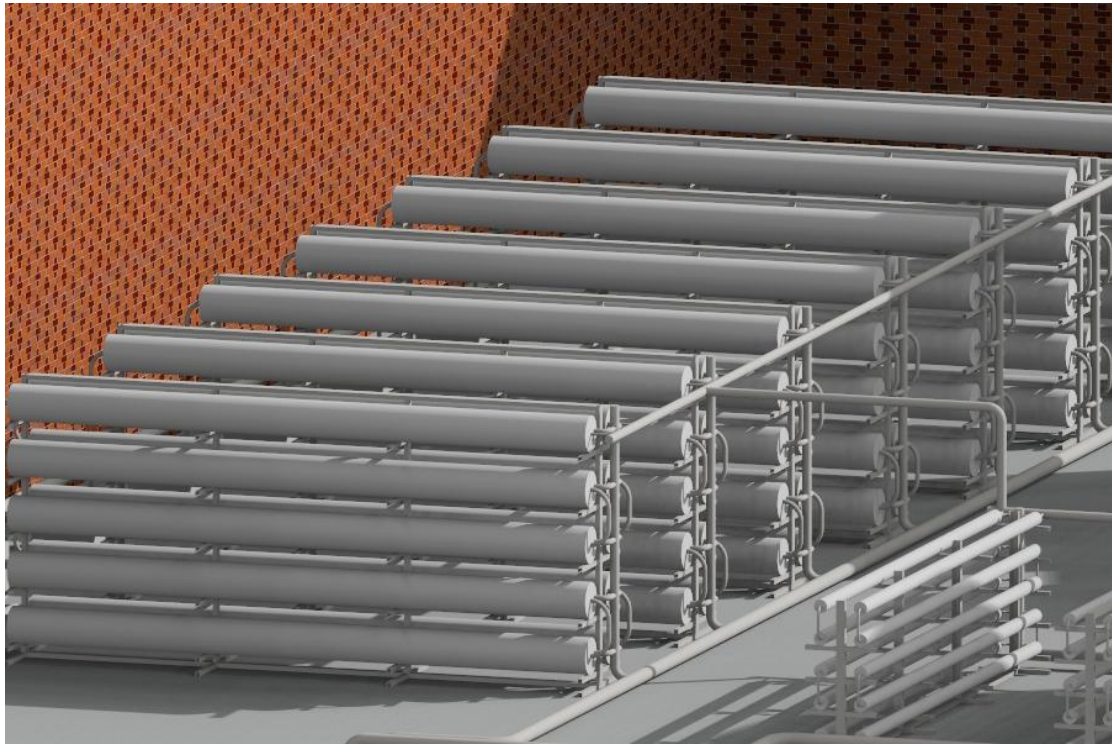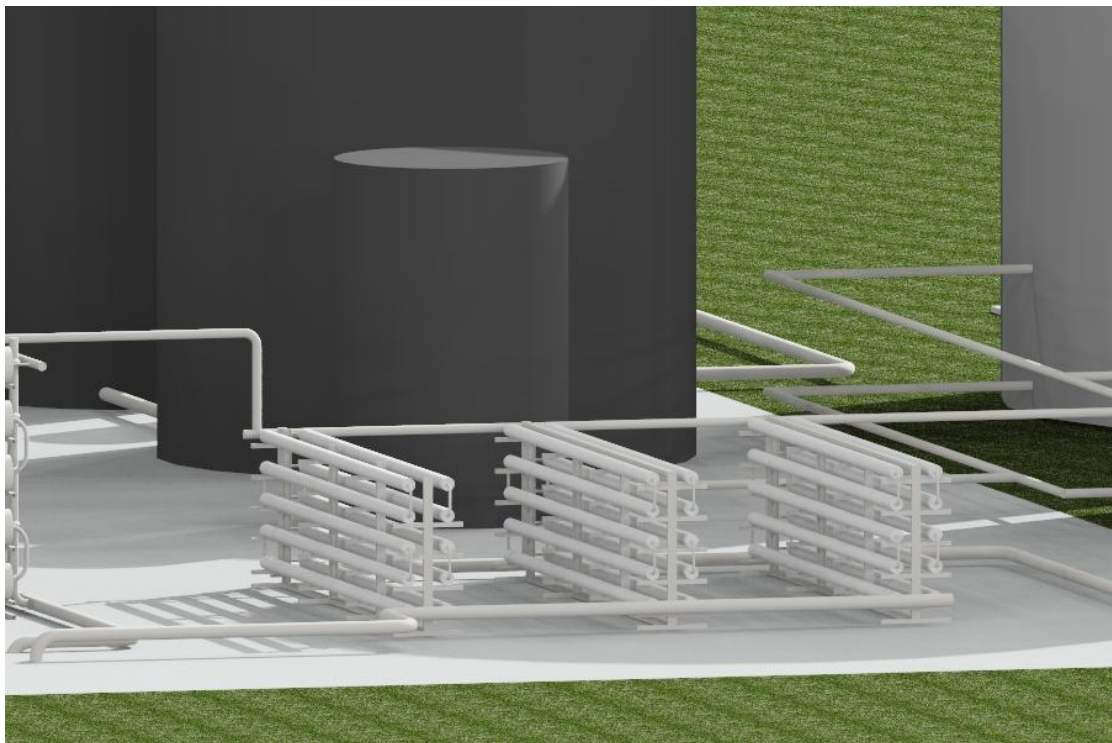
Figure 41 Close-up CNF racks

Figure 42 Close-up RO racks

6 Conclusions and recommendations

6.1 Conclusions

During the scenario study, 4 viable scenarios were developed to treat domestic wastewater into demi-water with nutrient recovery and low energy consumption. After comparison of the scenarios based on energy consumption it was concluded that the most viable and economical option (based on energy consumption) would be the application of scenario 3 (Figure 43). This scenario, with source separation through the use of a vacuum sewer for black water and a conventional sewer for grey water, is both energy efficient and capable of recovering the maximum amount of nutrients while still producing demi-water for reuse in industry. Also, the secondary effluent can possibly be reused after advanced treatment as process water, this will have to be further researched.

Figure 43 Scenario 3 - Energy efficient, recovery of nutrients and water reuse

After review of the applied technologies in the scenarios, a comparison was made to see which technology was most viable for small scale laboratory experiments. The conclusion of the comparison was that ceramic nanofiltration is both energy efficient and innovative, while it has a high research value.

When looking at the fouling mechanics, raw wastewater is preferable over grey water when ceramic nanofiltration is used as a first membrane step. Its fouling is easier to control, and less time is lost with backwashing. However the fouling which occurred when treating grey water was more easily removed by backwashing and since the backwash capability in the laboratory experiments was limited to 28 $L/m^2/h$, a backwash with a higher flux might have a bigger effect. This would increase the controllability of the fouling when treating grey water with ceramic nanofiltration. When considering the salt retention, retention of both magnesium and calcium are low. These are both substances that cause scaling on RO membranes, which is not preferable. However, the concentration are in a range that is manageable for the RO process. Apart from protecting the RO process from substances which are harmful for its operation, the CNF process also thickens the water stream that is rejected so it can be digested. In that process biogas is formed.

The DOC concentrations in the influent of the anaerobic digester are a good indication of the biogas production efficiency. With grey water, 99,5 % of DOC was rejected by the CNF process, which is a very positive indication for the biogas production in the digester. Less DOC was rejected when the CNF process was treating raw wastewater, possibly indicating a

lower biogas production. Also, because with the application of source separation, kitchen waste can be added to the black water, its biogas production will increase even more. So although the fouling control at the treatment of grey water may be lower compared to raw wastewater, its characteristics allow for more energy recovery. And since the fouling control when treating grey water may be improved by using a higher backwash flush, it is concluded that scenario 3 is indeed the most viable option.

6.2 Recommendations

It is recommended to intensify research on the fouling mechanics of the treatment of grey water using ceramic nanofiltration. Especially the application of a higher backwash flush or the addition of a chemically enhanced backwash. This can be achieved by saturating the backwash water with CO_2 while feeding the crossflow side with an acid solution. When backwashing in this configuration, the CO_2 will come out of its solution face to form gas bubbles underneath the fouling layer on the membrane. This can have a positive effect on the efficiency of a backwash.

More research is also needed into the effect of pH on the rejection of ions by the CNF process. Some of the results presented in this thesis contradict previously documented research. Apart from the ion rejection, also the exact composition of the concentrate stream of the CNF process will have to be researched further to get a better indication of its viability as influent for an anaerobic digester.

After the recovery of minerals in the RO process, the rejection water might be suited for reuse as process-water (less pure than demi-water) after further treatment. Since especially the salt concentrations in the rejection water will be high, it might be suitable for treatment by eutectic freezing crystallization (EFC) or Capacitive Deionization (CD), this has to be further researched however.

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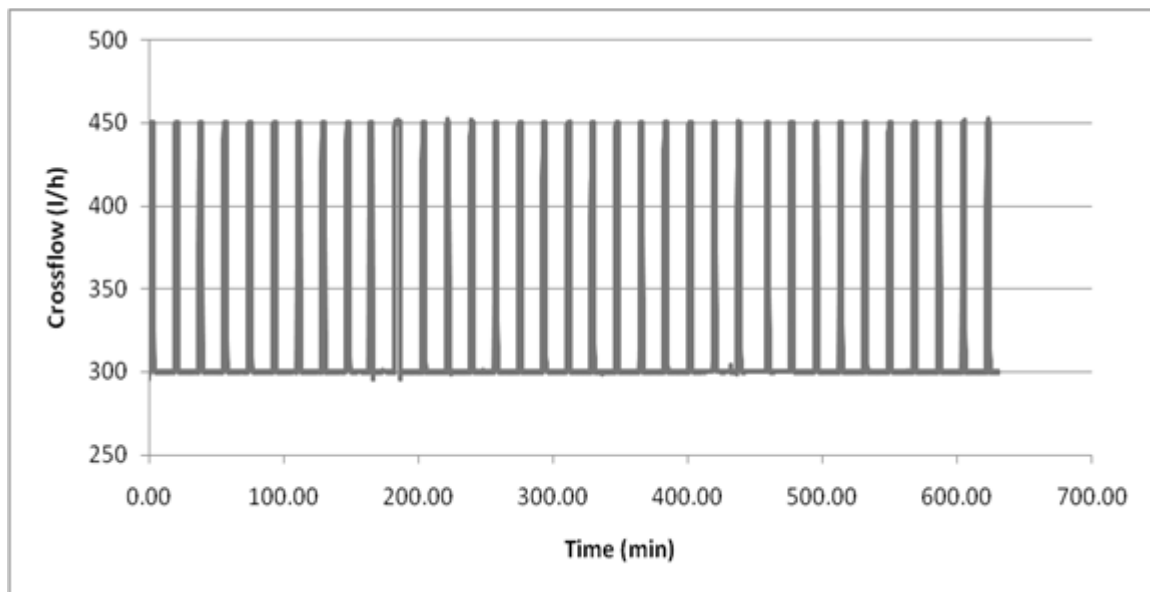
Appendixes

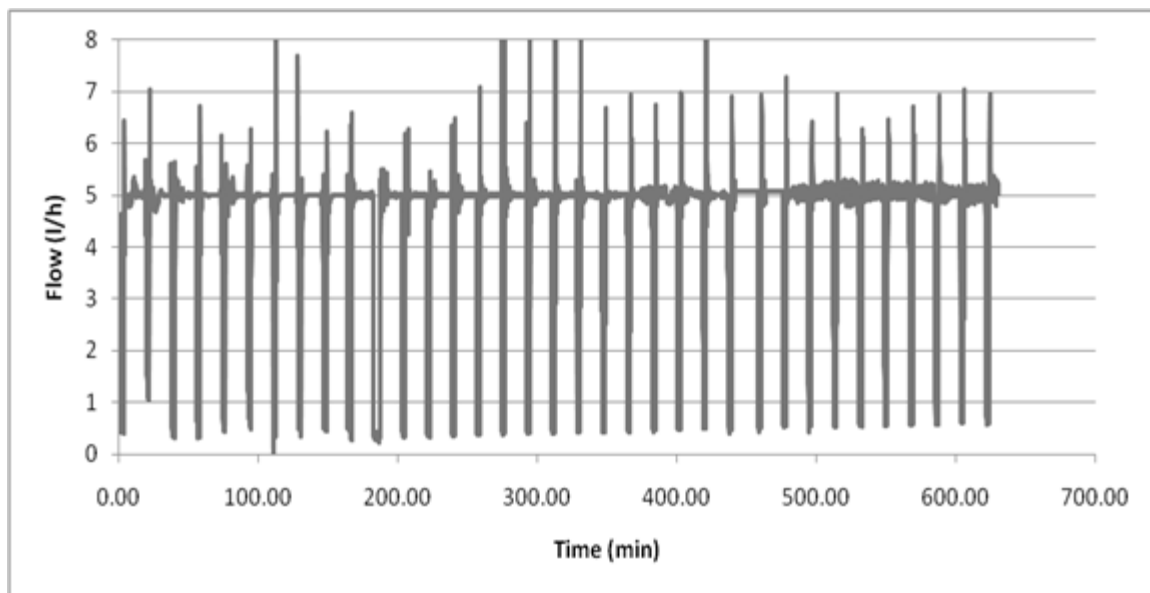# Appendix 1   Results experiment 1

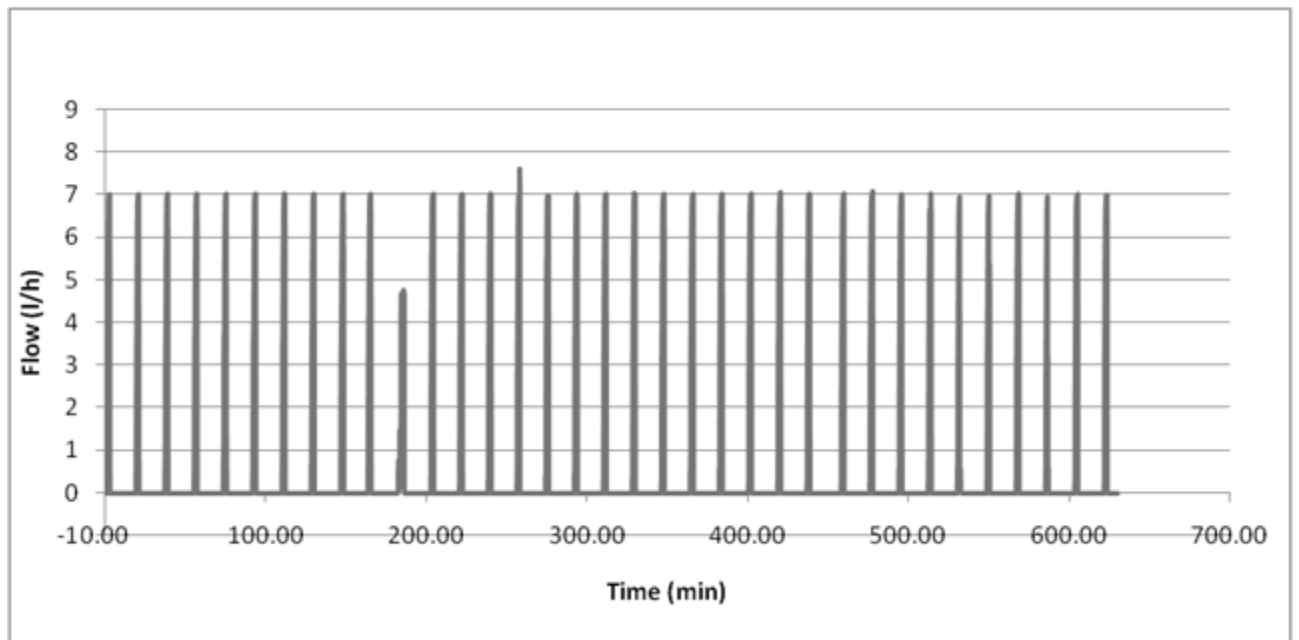*Figure 44 Crossflow*

Figure 45 Permeate flow
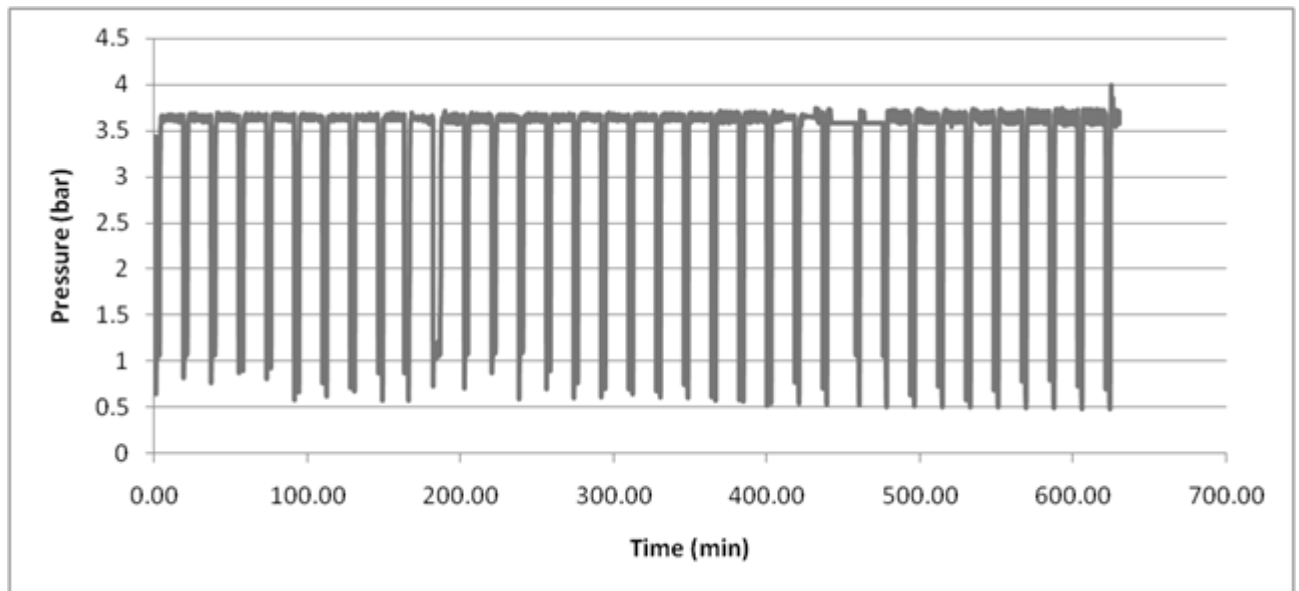
Figure 46 Backwash flow
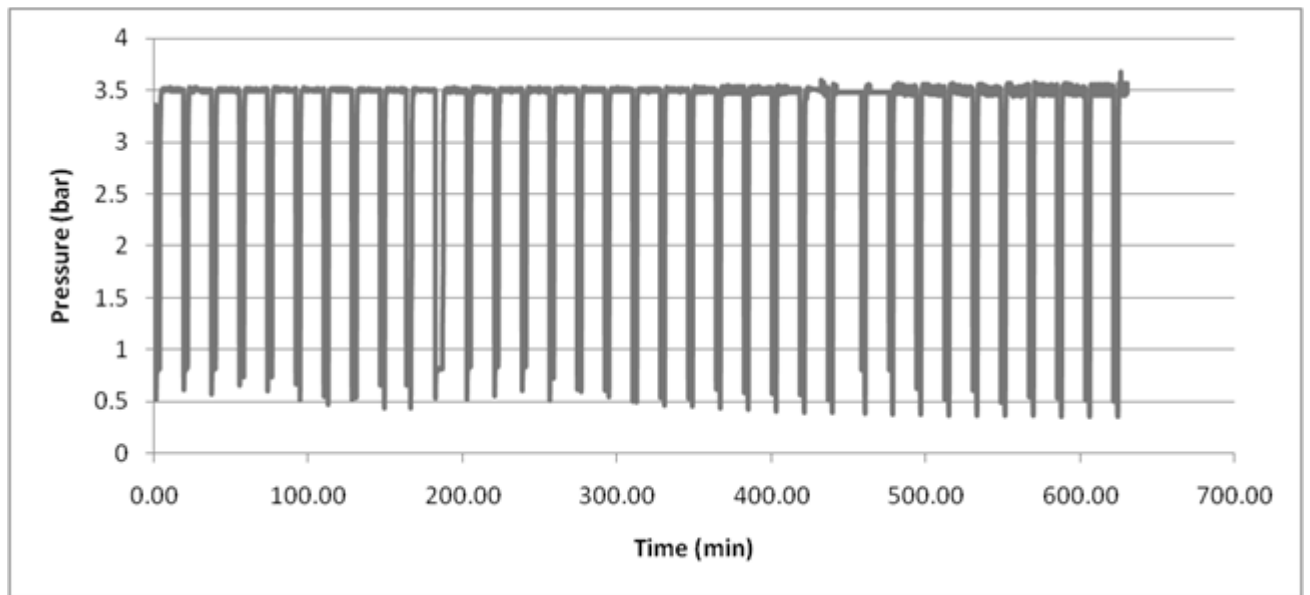

Figure 47 Feed pressure
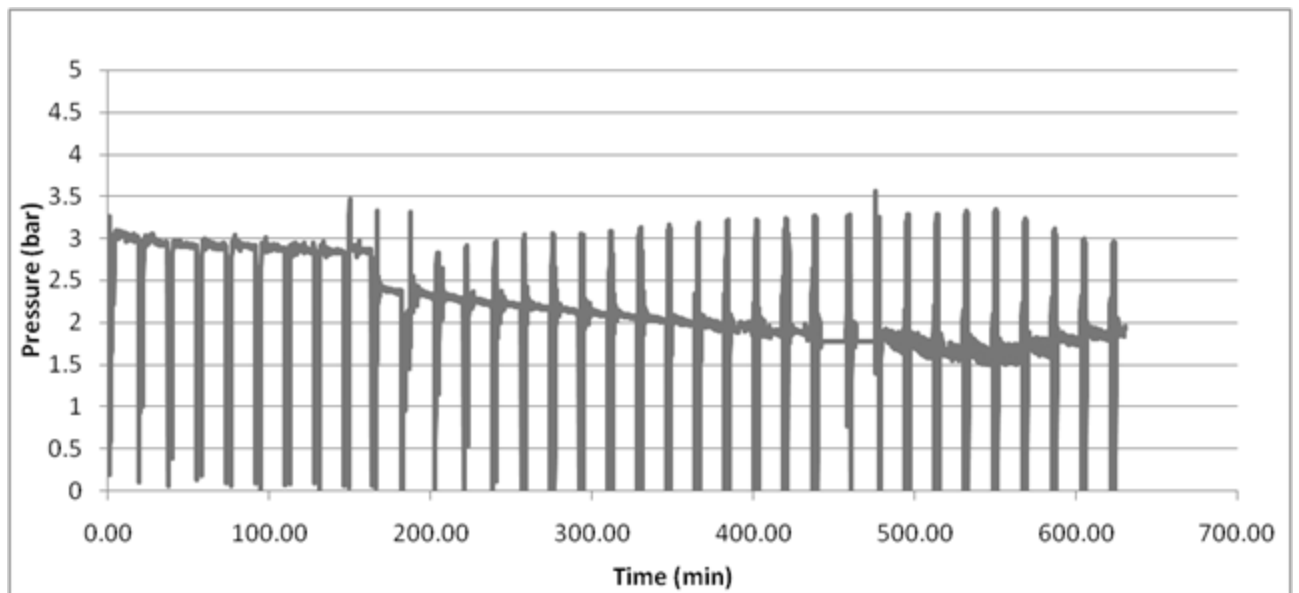
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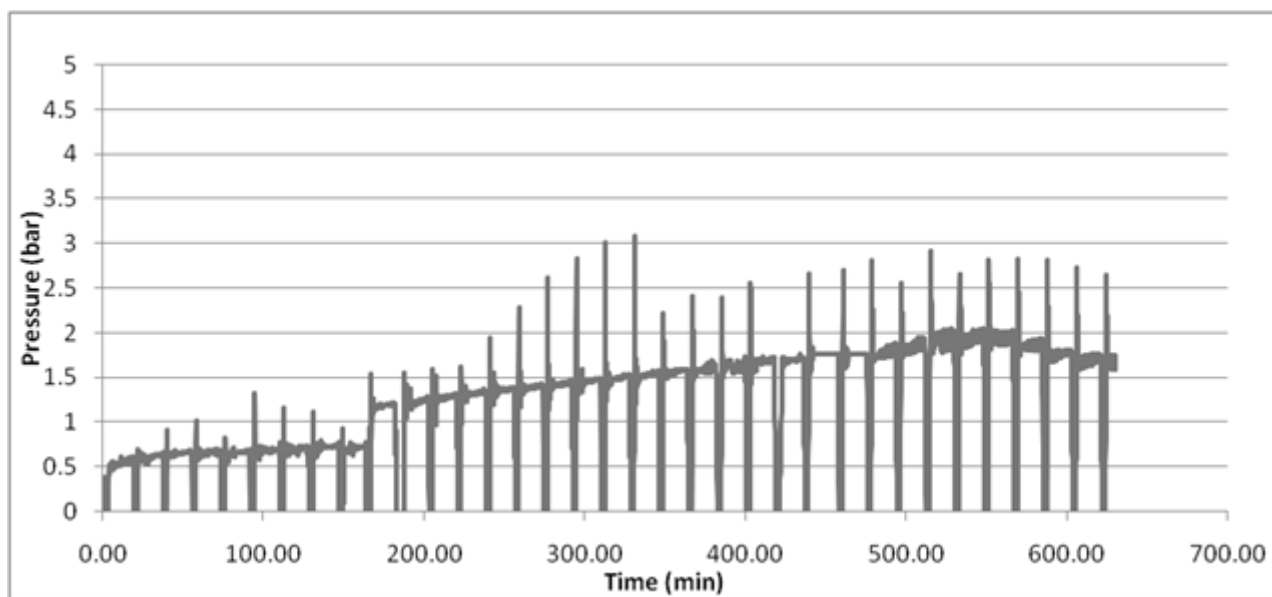

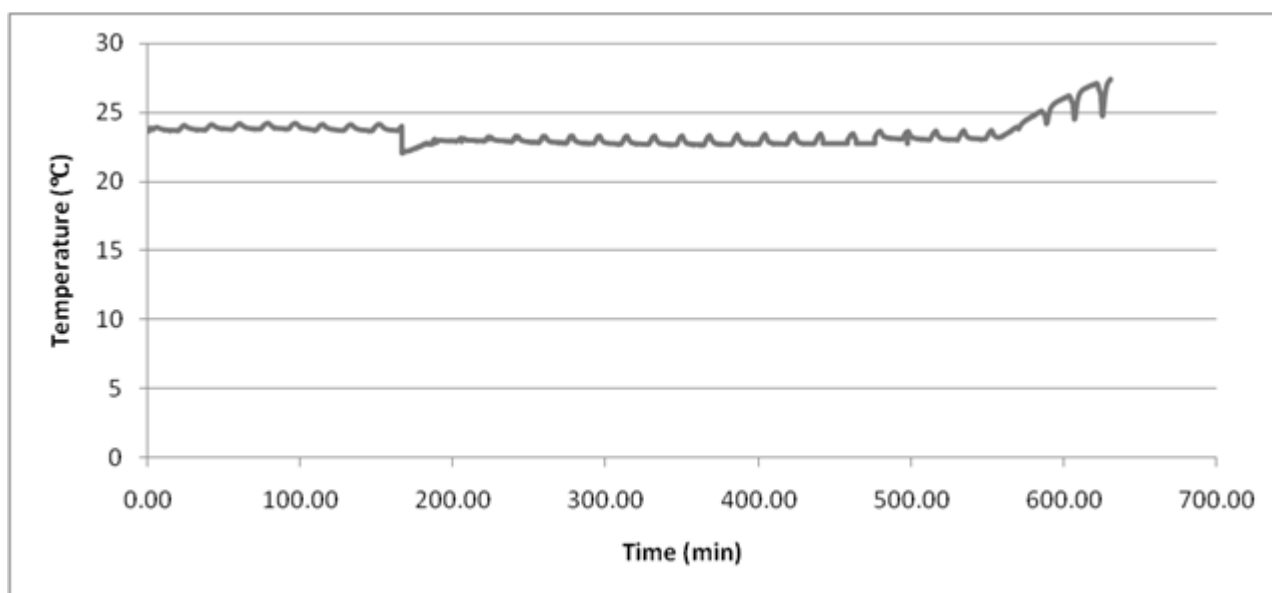*Figure 49 Permeate pressure*

Figure 50 TMP

Figure 51 Feed temperature
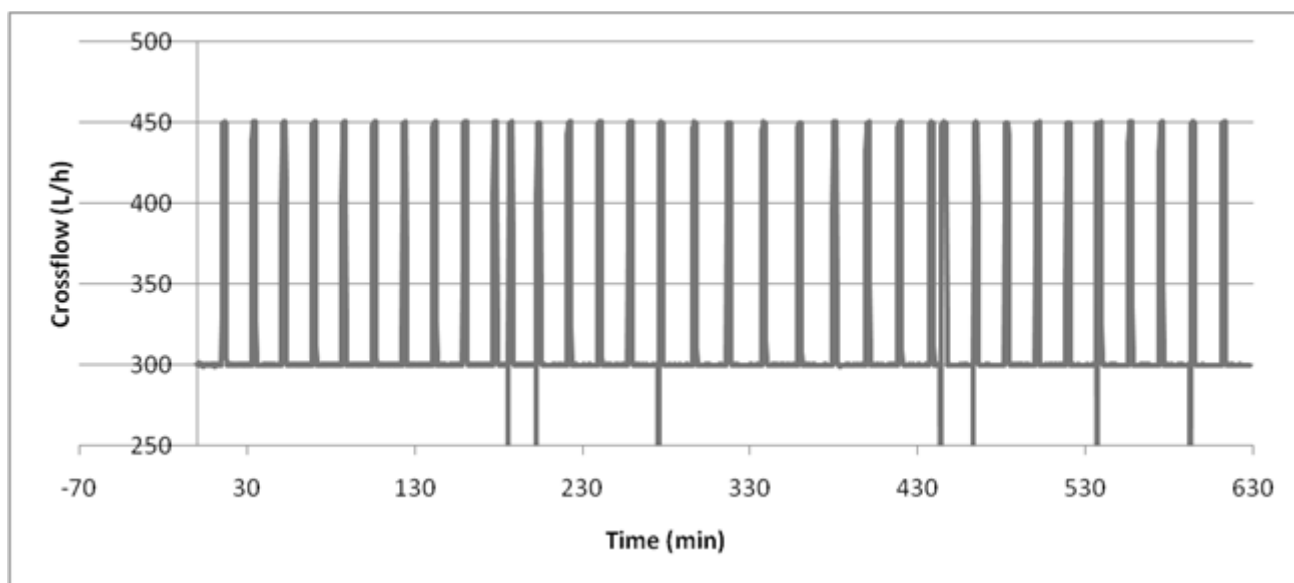
Appendix 2   Results experiment 2
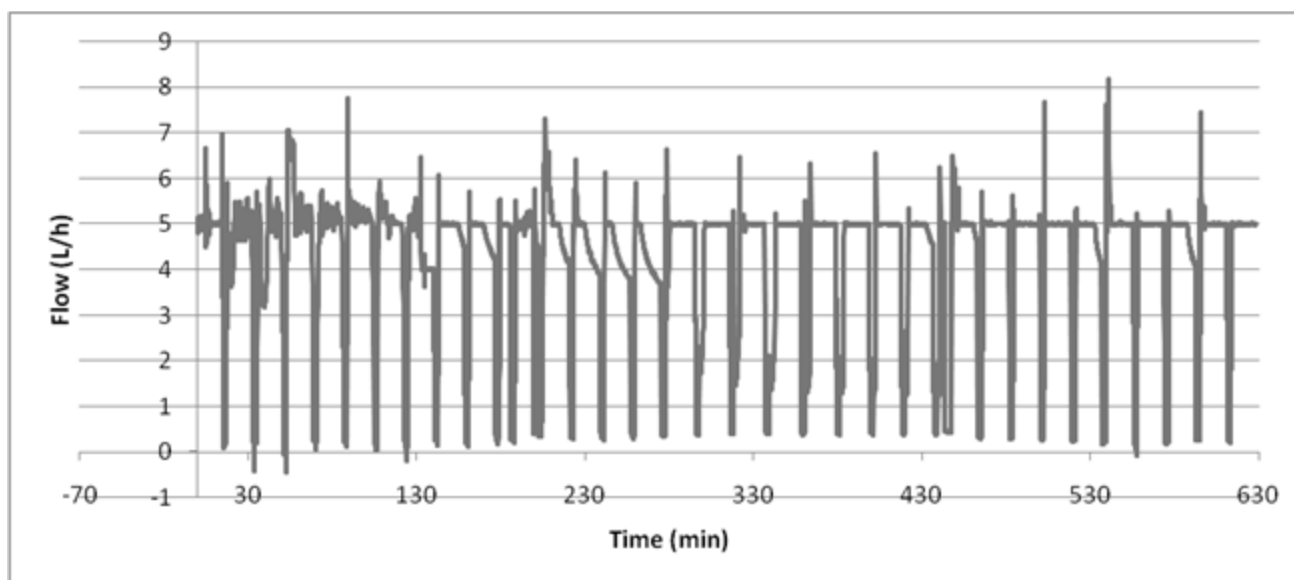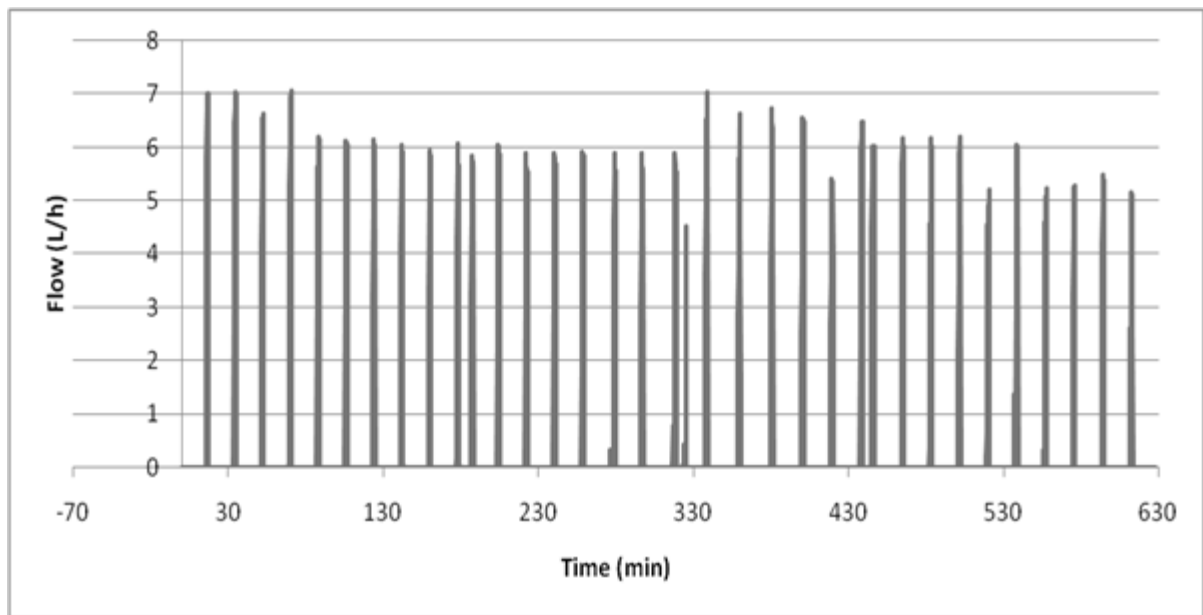

Figure 52 Crossflow
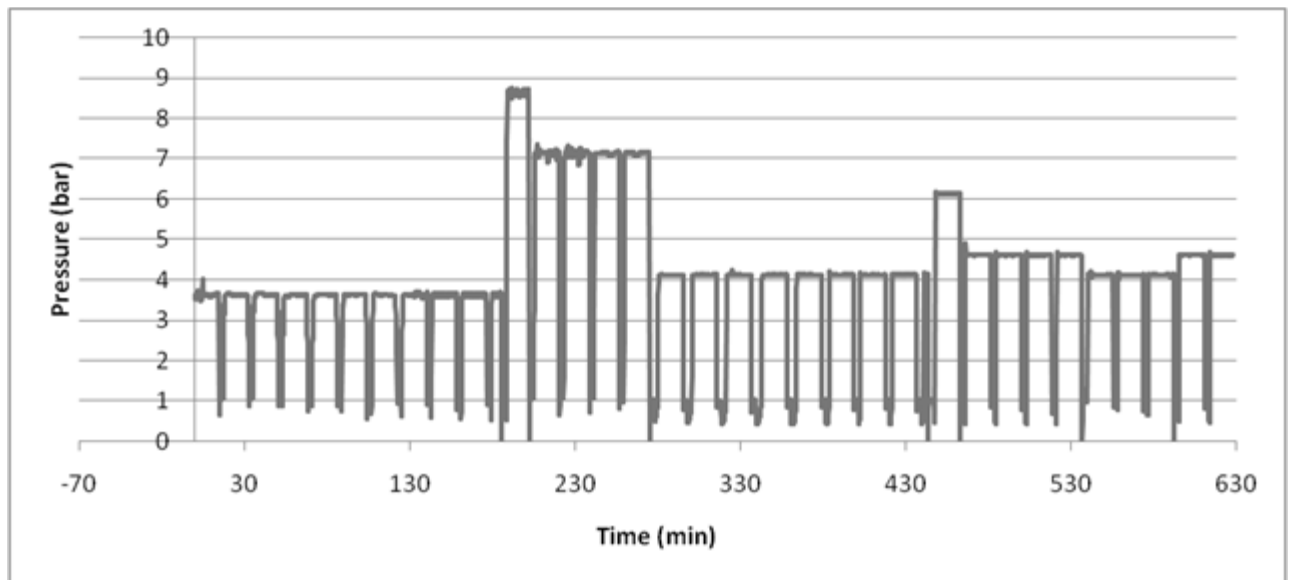

Figure 53 Permeate flow
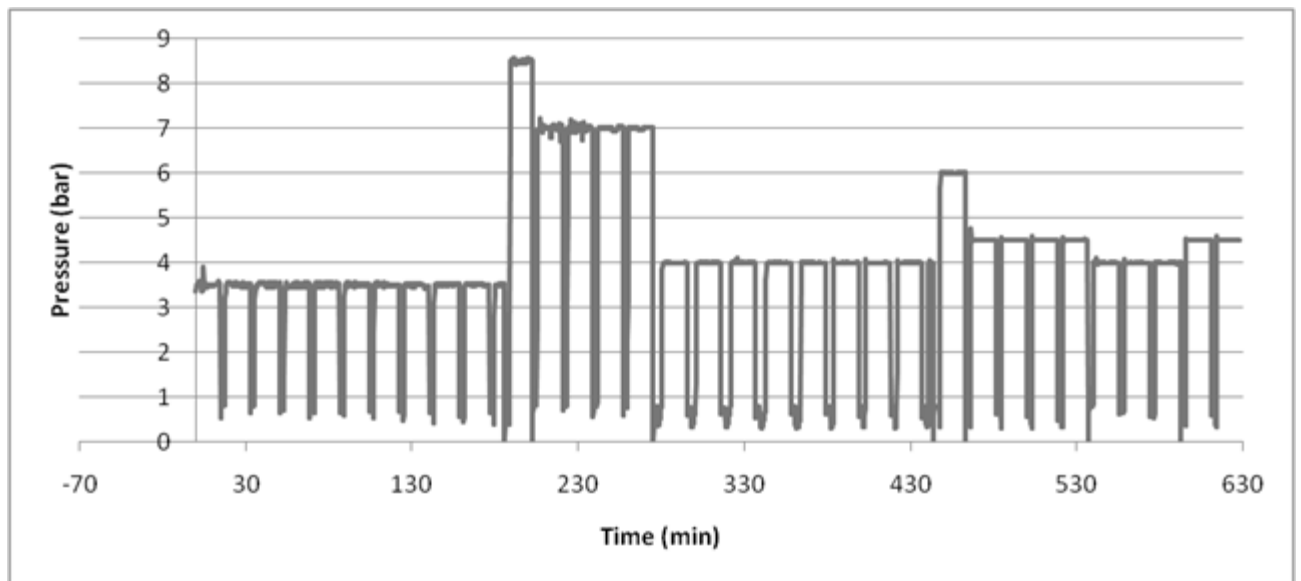
Figure 54 Backwash flow

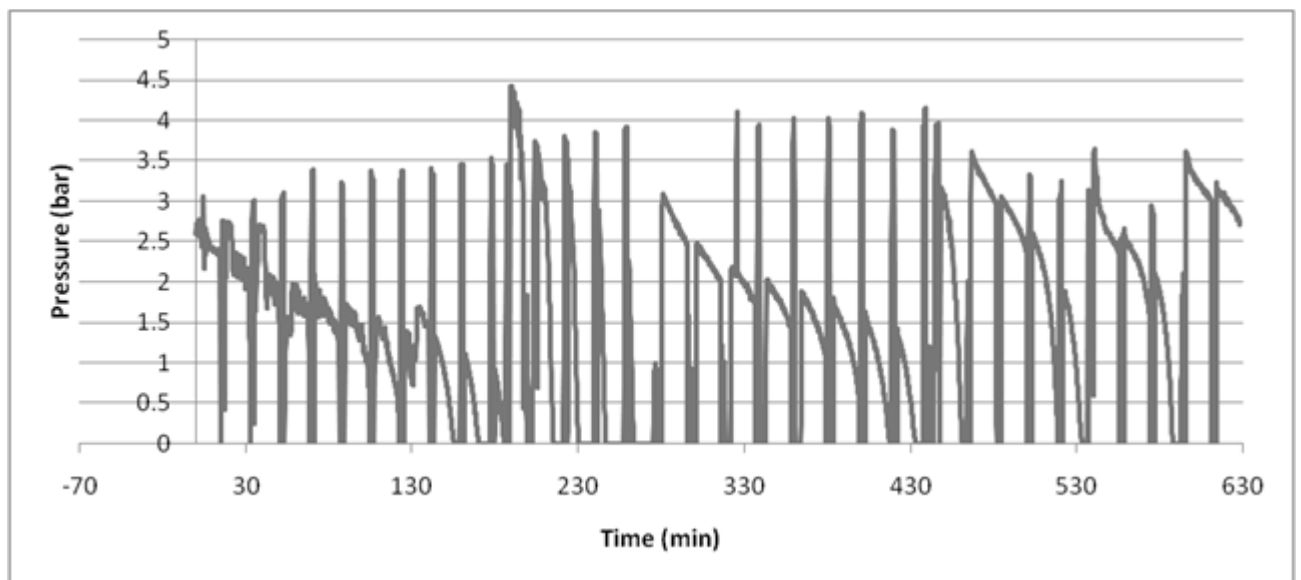*Figure 55 Feed pressure*

Figure 56 Crossflow pressure

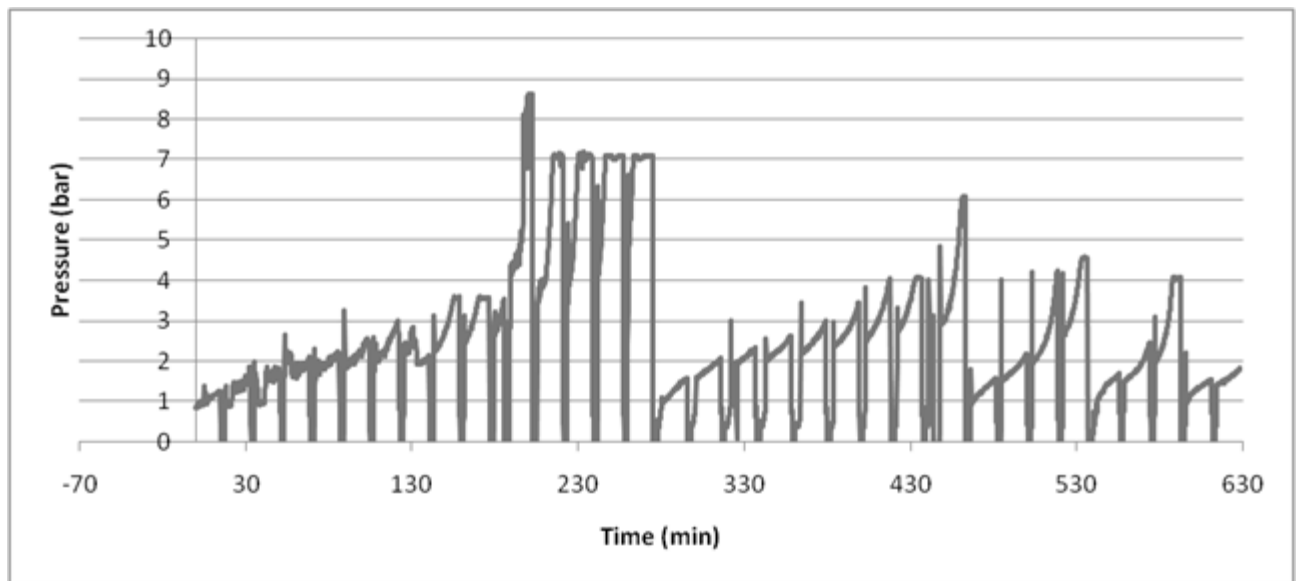*Figure 57 Permeate pressure*
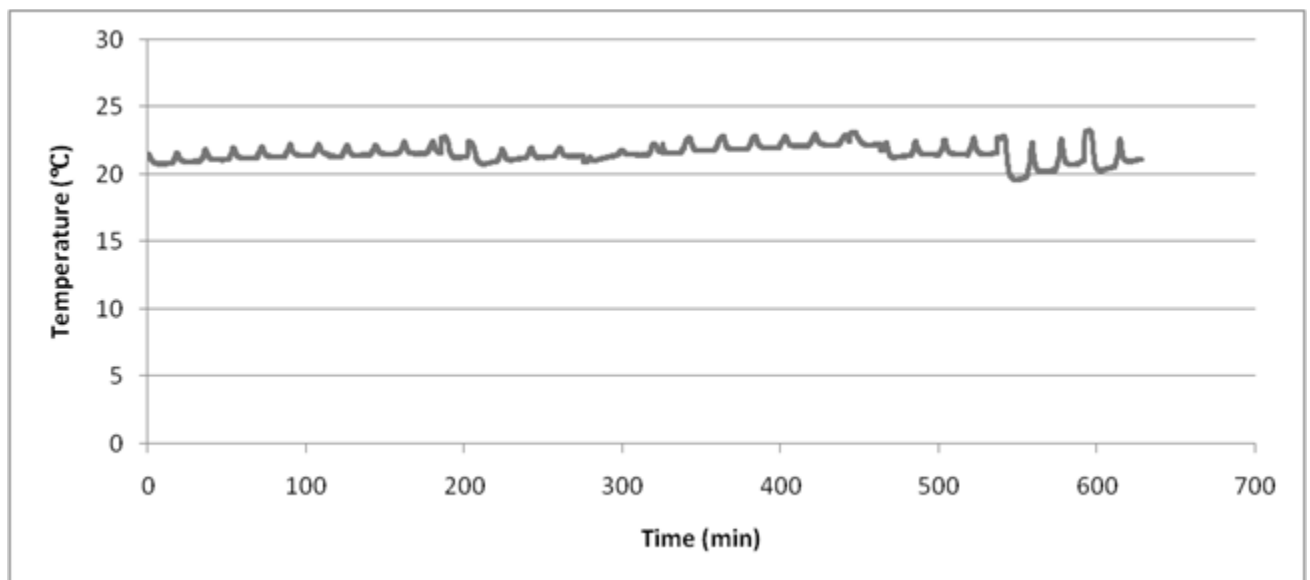
Figure 58 TMP

Figure 59 Feed water temperature
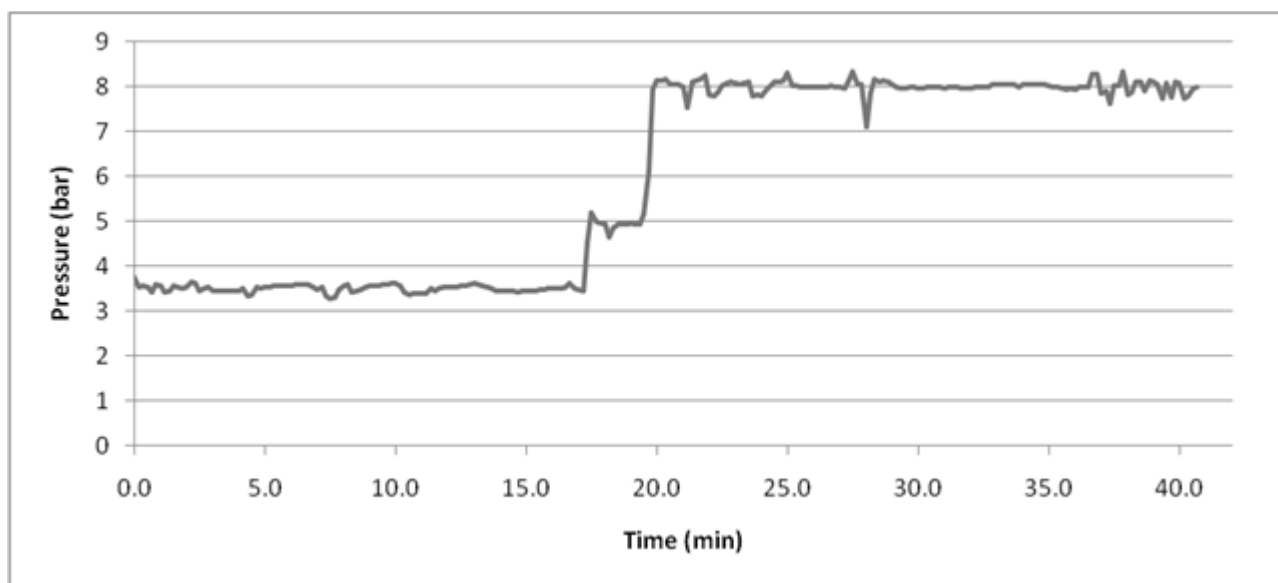
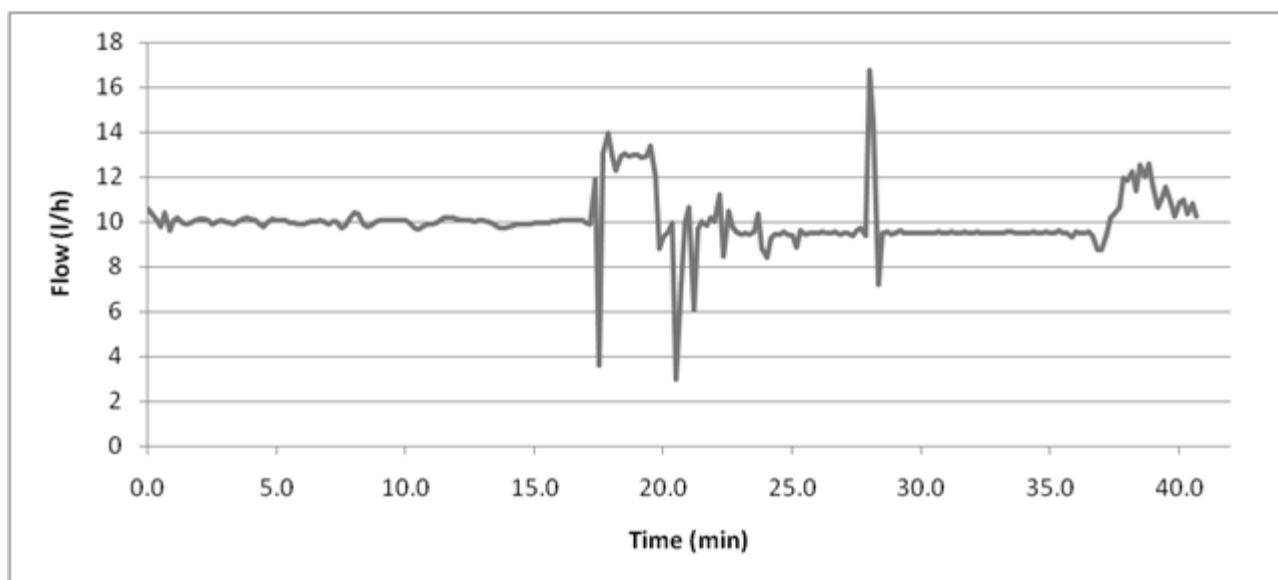# Appendix 3　Results experiment 3

Figure 60 Crossflow pressure
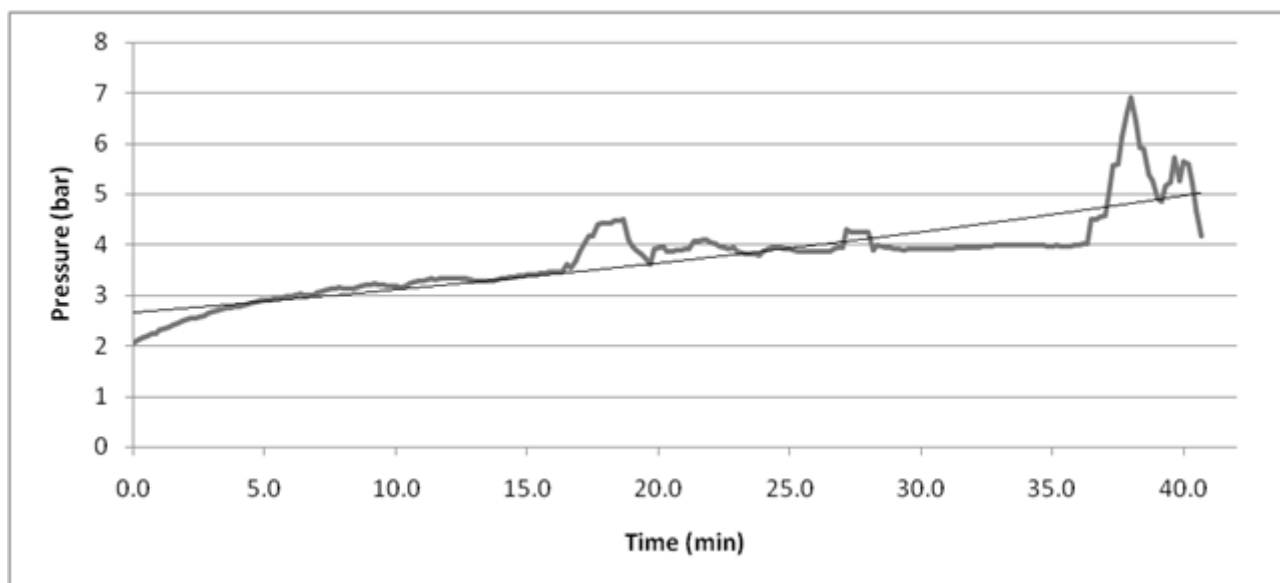

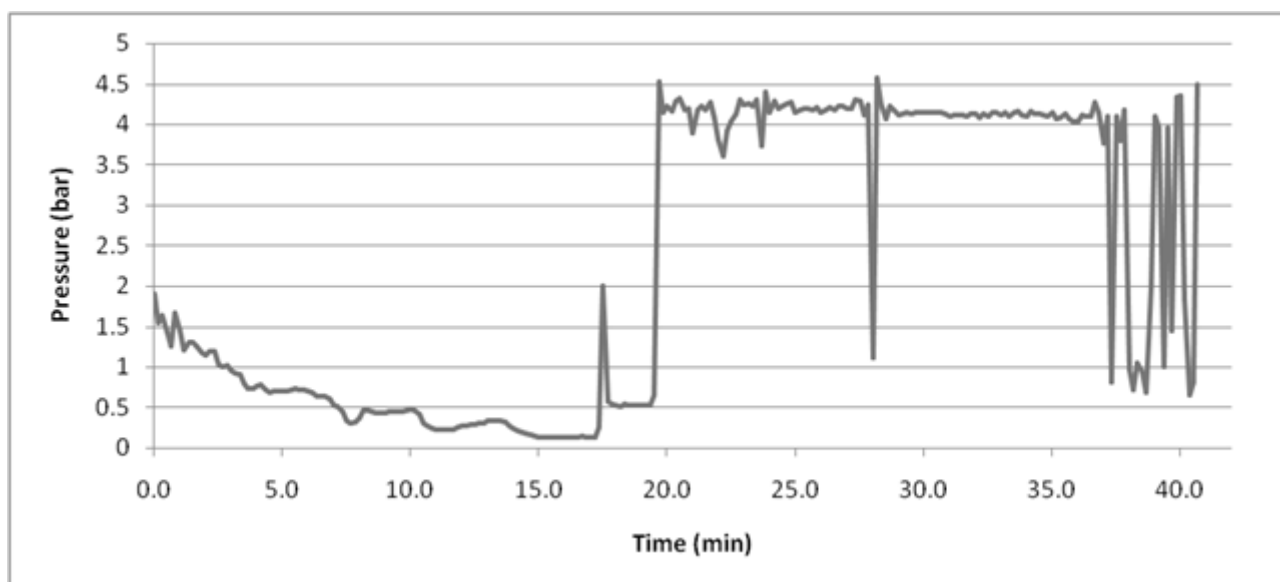
Figure 61 Permeate flow

Figure 62 TMP

Figure 63 Permeate pressure

Appendix 4 Results experiment 4
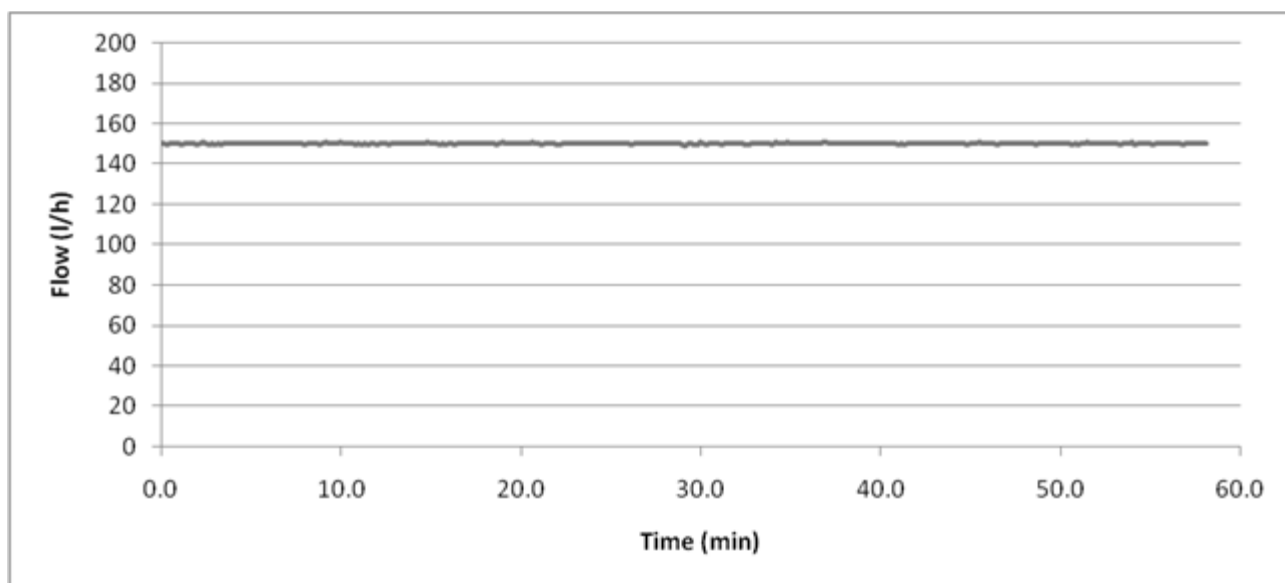

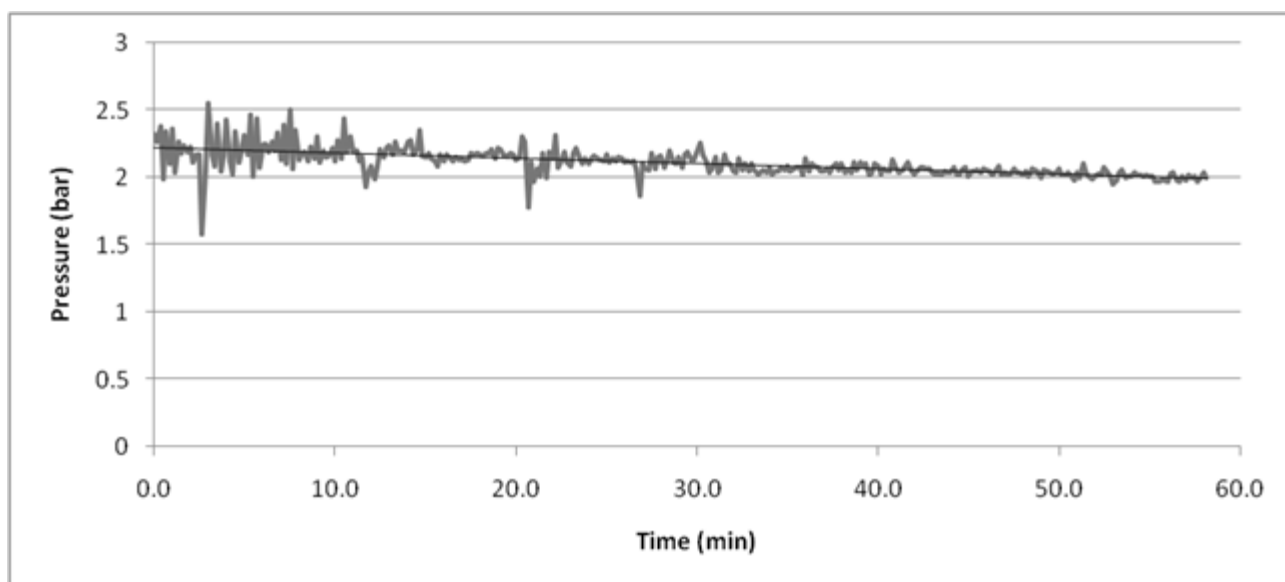*Figure 64 Crossflow*

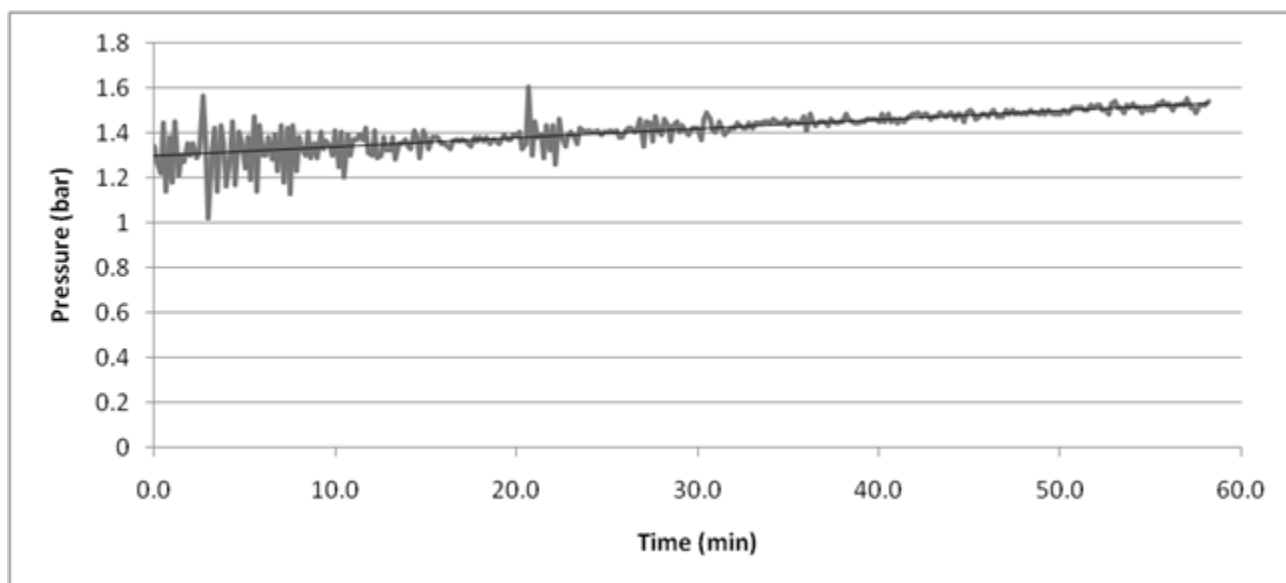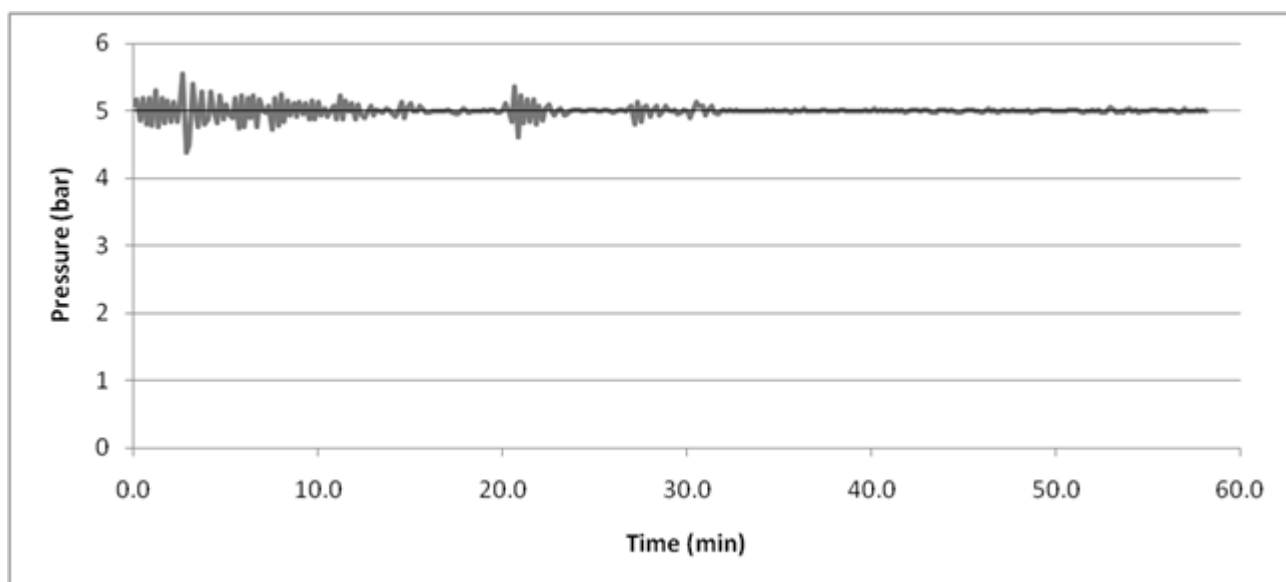*Figure 65 Permeate pressure*

Figure 66 TMP

Figure 67 Permeate flow

Appendix 5 Digestion biogas production energy balance

Table 19 Overview energy production and consumption during biogas production with CNF recovery of 80% and FO recovery of 70 %

	Sc1	Sc2	Sc3	Sc4
Needed temperature	30.00	30.00	30.00	30.00
Raw wastewater temperature	20.00	20.00	21.20	21.20
Biogas production per kg COD (m3/kg)	0.40	0.40	0.40	0.40
COD removed (mgCOD/L wastewater)	557.51	622.22	944.68	989.33
COD removed (gCOD/m3 wastewater)	557.51	622.22	944.68	989.33
COD removed (kgCOD/m3 wastewater)	0.56	0.62	0.94	0.99
Biogas produced (m3/m3 wastewater)	0.22	0.25	0.38	0.40
Total biogas + kitchen waste (m3/m3 wastewater)	0.22	0.25	0.76	0.79
Calorific value biogas (MJ/m3)	35.00	35.00	35.00	35.00
Energy from biogas (MJ/m3 wastewater)	7.81	8.71	26.45	27.70
Electricity production (MJ/m3 wastewater)	2.73	3.05	9.26	9.70
Heat production (MJ/m3 wastewater)	5.07	5.66	17.19	18.01
Electricity production (kWh/m3 wastewater)	0.76	0.84	2.56	2.69
Heat needed to heat 1 m3 of water 1 °C (MJ)	4.20	4.20	4.20	4.20
Heat needed (MJ/m3 wastewater)	8.40	12.60	8.90	10.84
kWh needed for heating (kWh/m3 wastewater)	1.20	2.24	-1.34	-0.99

Table 20 Overview energy production and consumption during biogas production with CNF and FO recovery of 90%

Parameter	Sc.1	Sc.2	Sc.3	Sc.4
Needed temperature (°C)	30.00	30.00	30.00	30.00
Raw wastewater temperature (°C)	20.00	20.00	21.20	21.20
Biogas production per kg COD (m3/kg)	0.40	0.40	0.40	0.40
COD removed (mgCOD/L wastewater)	549.42	622.22	939.10	989.33
COD removed (gCOD/m3 wastewater)	549.42	622.22	939.10	989.33
COD removed (kgCOD/m3 wastewater)	0.55	0.62	0.94	0.99
Biogas produced (m3/m3 wastewater)	0.22	0.25	0.38	0.40
Biogas produced + kitchen waste (m3/m3 wastewater)	0.22	0.25	0.75	0.79
Calorific value biogas (MJ/m3)	35.00	35.00	35.00	35.00
Energy from biogas (MJ/m3 wastewater)	7.69	8.71	26.29	27.70
Electricity production (MJ/m3 wastewater)	2.69	3.05	9.20	9.70
Heat production (MJ/m3 wastewater)	5.00	5.66	17.09	18.01
Electricity production (kWh/m3 wastewater)	0.75	0.84	2.55	2.69
Heat needed to heat 1 m3 of water 1 °C (MJ)	4.20	4.20	4.20	4.20
Heat needed (MJ/m3 wastewater)	4.20	4.20	6.97	6.97
kWh needed for heating (kWh/m3 wastewater)	0.06	-0.09	-1.86	-2.06

Appendix 6 Report visit Noorderhoek

Date: 03-07-2012
Time: 10:00
Location: Noorderhoek, Pieter Zeemanstraat 6, Sneek

Present: L. Wiersma, S.G.J. Heijman, J. Legierse

Subject: Visit pilot Noorderhoek

On the 3rd of July, a visit was made to the community of Noorderhoek in Sneek. Here, the company of Desah has a wastewater treatment pilot which treats separately collected wastewater (grey and black) from the community of Noorderhoek. The Noorderhoek community currently consists of 100 households with an estimated 250 inhabitants. In the houses, the wastewater is collected separately. The grey wastewater from the bathroom and kitchen is transported to the pilot installation through a conventional sewer, while the black wastewater from the toilet is transported through a vacuum sewer to the pilot installation. Also, all the kitchens of the houses contain a separate sink for the disposal of green kitchen waste, which is first grinded, and then transported together with the black wastewater. The setup of the treatment process of the pilot can be seen in Figure 68.
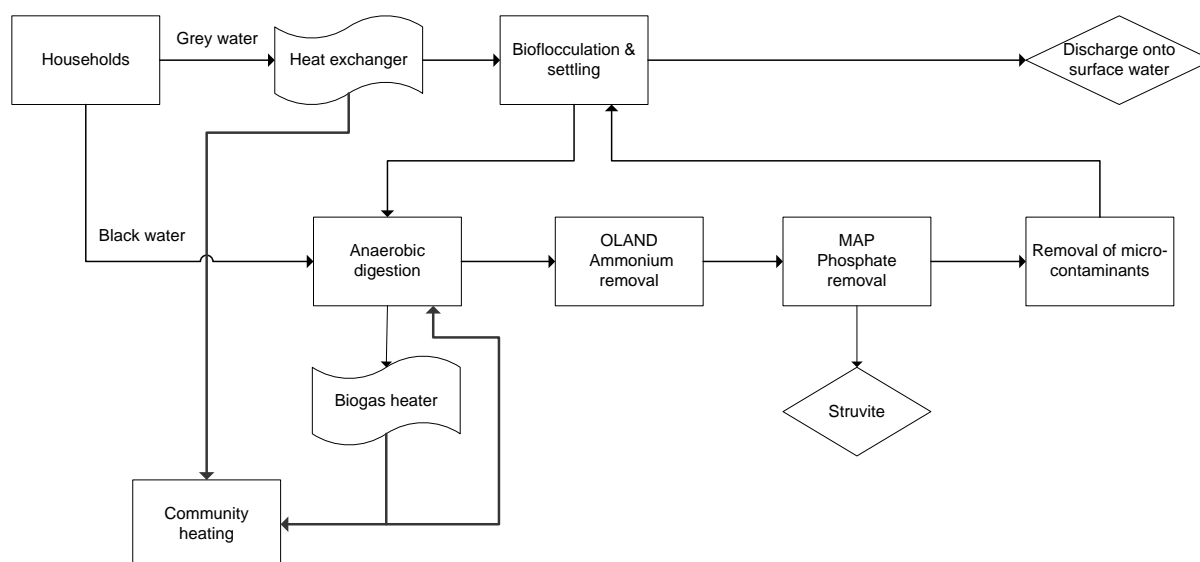

Figure 68 Process scheme of pilot at Noorderhoek

The grey water has an average temperature of 25 ℃, this energy can be recovered as heat with a heat exchanger to supply heating for the community. After the heat exchanger, the grey water is fed to a bioflocculator and settler after which it is discharged onto surface water. The settled particles are fed to the anaerobic digester where the black water is treated to supplement the amount of biological material in the digester. In the digester, biological material is converted into sludge and biogas. This biogas is also used to assist in the heating for the community. After the digestion process, ammonium is removed by the OLAND process, which is an Anammox® process on a biorotor. After the OLAND process, ortho-phosphate is precipitated into struvite by the MAP process. After micro/contaminants removal the water is fed back into the bioflocculator and settler. See also the figures on the next page.

By installing kitchen grinders and adding the green kitchen waste to the black water stream, the biogas production is doubled, compared to when the green kitchen waste would be thrown away. This biogas, together with the thermal energy of the grey wastewater, can contribute up to 22 % of the heat delivery to the community.
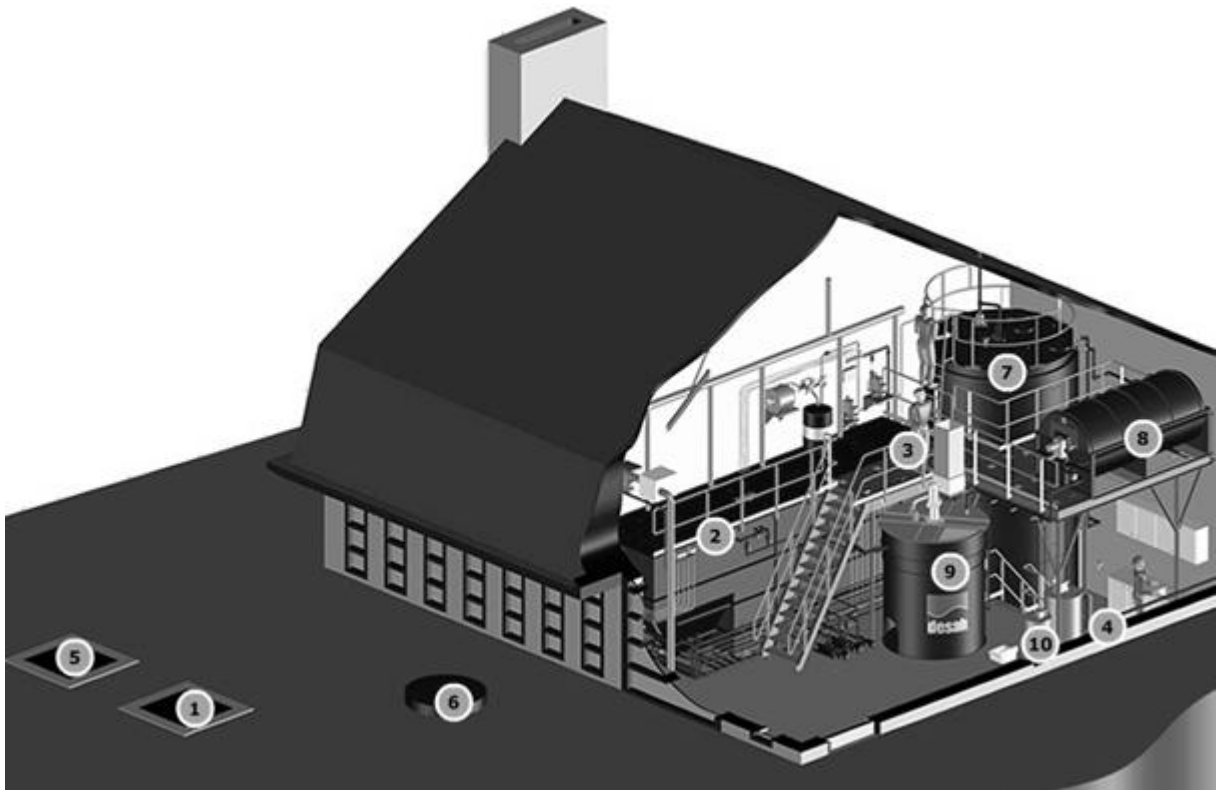
Figure 69 Overview pilot Noorderhoek

Figure 70 Bioflocullator and settler pilot Noorderhoek

Figure 71 MAP tank pilot Noorderhoek

Appendix 7 Report meeting Qua-Vac B.V.

Date: 06-09-2012
Time: 15:00
Location: Qua-Vac BV, Televisieweg 159, Almere

Present: Dhr. Quatfass, Joeri Legierse

Subject: Meeting energy use vacuum sewer

After the first introduction, Mr.Quatfass explains about the background of the company Qua-Fac. In the early 1970's he worked for the firm Electrolux, where he worked on the development of the vacuflow system. In 1985 this company sold its vacuum division to EVAC. In 1990 Mr. Quatfass decided to buy all of the Vacuflow patents and continue developing the system through the company of Qua-Vac BV.

This system is now widely used throughout the world, especially at vacation parks, on ships and in harbours. In the 1980's it was subsidized by the Dutch government, and was also used in the Netherlands, this subsidy was however stopped in 1989 from where on the Vacuflow system was mainly exported to other countries.

Mr. Quatfass explains that the vacuflow system piping only needs about 80 – 120 cm of coverage since it will be mostly empty during use. The piping diameters vary between 10 to 16 cm. The piping is laid out in a sawtooth configuration (see Figure 72). In this manner when water is transported through the pipe, pockets of water form at each jump. With every flush, these pockets of water are progressively moved through the pipe to the next jump.
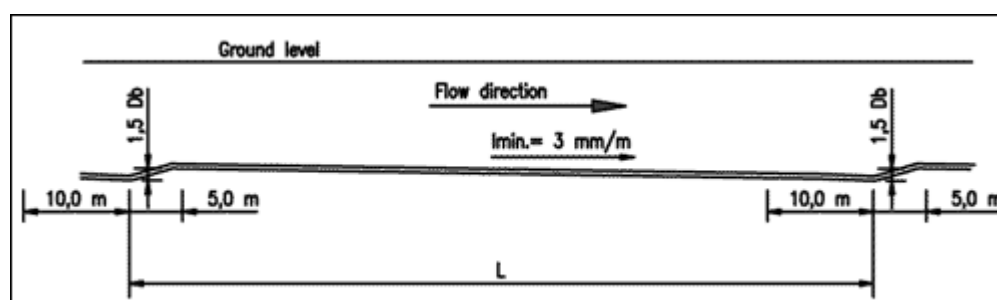

Figure 72 Sawtooth piping configuration (www.quavac.com)

The pipes in the system come together in a vacuum station. This station contains vacuum pumps which maintain the pressure in the piping, and a reservoir which is emptied when it reaches a certain predetermined level.

When asked for the energy requirements of the vacuflow system, Mr. Quatfass explains that under normal conditions (no use of vacuum toilets, only vacuum transport), with every litre of water, 3 litre of air is added to the system. By pumping this air out, the water is transported to the vacuum station. However, when using vacuum toilets, 66 litres of air is added for each litre of water transported. When considering that most pumps used nowadays use approximately 5 kWh/m^3 of air removed, he determines the energy use of the system in this configuration needs 1.4 kWh/m^3 transported wastewater. Considering we only transport 10% of the water through a vacuum sewer, the overall energy use would be 0.14 kWh/m^3.

When asked after the possibility for use of this system on floating houses, he responds that this is perfectly possible by connecting the houses using flexible piping.

Appendix 8   Report visit Triqua B.V.

Date: 26-09-2012
Time: 15:00
Location: Triqua, Vadaring 7, Wageningen

Present: H. Ramaekers, J. Legierse

Subject: Meeting forward osmosis

On the 26th of September a visit was made to the company of Triqua, located in Wageningen. This company is specialized in the design and testing of various biological (MBBR, MBR) and membrane treatment processes (from MF to FO and RO). The subject of this visit was to collect information on the working and application of forward osmosis.

Forward osmosis is a membrane filtration technique that has some similarities with reversed osmosis. Both processes use osmotic membranes that allow water to pass, but stop other substances, while striving to keep an equal concentration of substances on both sides of the membrane. Because a solution containing salt has a higher osmotic pressure, it would attract water from the other side of the membrane if the salt concentration on that side would be lower. This process is the basis of both FO and RO. With reversed osmosis, pressure is applied to the salt side to overcome the osmotic pressure and reverse the flow from fresh to salt, to salt to fresh. This way, a pure water stream is produced. However, this process requires a lot of energy.
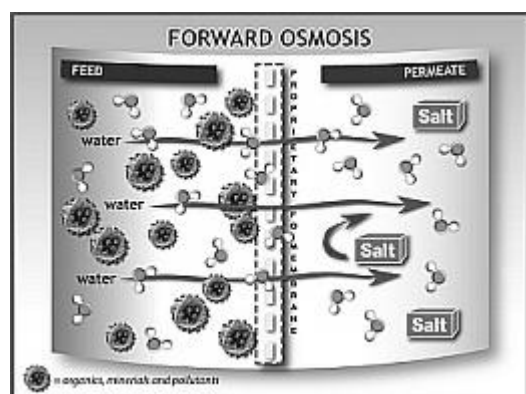

Figure 73 Process scheme of forward osmosis

Forward osmosis on the other hand, uses no pressure (see Figure 73). It uses a salt draw solution, to draw water from a wastewater stream through the membrane. This creates a thickened wastewater stream, and a salt stream. This configuration is often used to thicken wastewater streams with little energy consumption. However, if fresh water is to be produced, fresh water needs to be extracted from the salt stream produced by the forward osmosis process. By doing this, the concentration of salt in the draw solution is kept constant (and thus the osmotic pressure executed by the draw solution is kept constant as well) while the water that is drawn through the forward osmosis membrane is pumped through the RO membranes to produce process water.

In the Netherlands, the practical applications of FO are (at present) small, but internationally it is already used for various applications like the treatment of wastewater streams from waste dumps, or treatment of water used in oil and gas production and extraction.