FACULTY OF
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SIMULATION OF HOUSEHOLD-BASED WATER SYSTEMS FOR AN EFFICIENT WATER MANAGEMENT

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CONTENTS

SAMENVATTING

Een stijgende industriële en agrarische productie, in combinatie met het vooruitzicht van klimaatverandering heeft tot een toenemende behoefte aan efficiënt waterbeheer en waterhergebruik in de watersector geleid. Bovendien wordt waterbeheer momenteel op een sterk gecentraliseerde wijze beoefend, met uitgebreide leidingnetwerken voor drinkwatervoorziening en gescheiden afvalwaterlozing via riolen, wat vele nadelen met zich meebrengt. Kleinschalige, gedecentraliseerde watersystem hebben hierdoor recent aan interesse gewonnen. In dit proefschrift hebben we het functioneren van gedecentraliseerde waterhergebruiksystemen bestudeerd, met bijzondere aandacht op het functioneren op huishoudniveau en onafhankelijk van het drinkwaternet of rioleringsnetwerk.

Er werd een theoretisch waterbeheersysteem uitgewerkt, waarvoor een technologische configuratie geconstrueerd werd op basis van biologische en physico-chemische waterzuiveringstechnologieën met additionele disinfectie. Centraal staat het gecombineerd hergebruik van grijswater en regenwater in het systeem, waarbij het systeem regenwater als enige instroom gebruikt en zwartwater als uitstroom. Dit laatste wordt samen met overtollig grijswater naar een zwartwatertank geleid. Door middel van simulatie van de verschillende waterstromen in het systeem, hebben we aangetoond dat onafhankelijkheid van het net haalbaar is. Verder hebben we de invloed van huishoudgrootte, neerslagpatronen en verschillende hergebruikschema's op systeemprestaties beoordeeld. Ook werden de effecten van klimaatverandering en het wegleiden van de eerste, meer gecontamineerde regenwaterstroom geëvalueerd. Bovendien werd aangetoond dat accumulatie van contaminanten, met name zouten een belangrijk obstakel voor de implementatie van circulaire waterhergebruiksystemen is. Tot slot werd zwartwater hergebruik geanalyseerd, en werd gedemonstreerd dat dit een interessante manier is om de circulariteit van het systeem te verhogen.

Deze studie toont in zijn geheel aan dat de implementatie van gedecentraliseerde waterbeheersysteem, onafhankelijk van het net en op huishoudniveau, realiseerbaar is en welke factoren van belang zijn om in rekening gebracht te worden.

SUMMARY

Agricultural intensification and increasing industrial production in combination with the effects of climate change, has led to an increasing need for efficient water management and water reuse in the water sector. Moreover, our water system is currently operated in a highly centralized manner, relying on extensive piping networks for potable water supply and for separated wastewater discharge through sewers, bringing about many disadvantages. Therefore, small-scale decentralized water management systems are gaining interest. In this dissertation, we have analyzed the performance of decentralized water reuse systems, specifically on a household level and independent from any sewer system or potable water grid.

A theoretical water management system was constructed, for which a technological configuration based on a combination of biological and physico-chemical treatment technologies with subsequent disinfection was assumed. We considered the combined reuse of greywater and rainwater in the system, whereby rainwater is the only external input of water into the system and blackwater and excess greywater the output, discarded towards a blackwater tank. Through simulation of the water flows in the system, we demonstrated that independence from the grid is feasible. Furthermore, we assessed the influence of household size, precipitation patterns and different reuse schemes on system performance. Also the influence of first-flush diversion and climate change on the system was briefly evaluated. Accumulation of contaminants, with salts in particular, was shown to be a major obstacle to the application of circular water reuse systems. Finally, blackwater reuse as way of increasing system circularity was also shown to be an interesting addition to the system configuration.

Overall, this study demonstrated the feasibility of decentralized water management systems to operate independent from the potable water grid and sewer system and presents an in depth discussion of several important considerations when implementing such systems.

CHAPTER 1

INTRODUCTION

1.1 Background

By 2050 the world population is expected to approximate 10 billion people, of which 68% will reside in urban areas (United Nations, 2017, 2018b). The effects of climate change along with agricultural intensification and increasing industrial production to accommodate the increasing world population will impede the realization of many Sustainable Development Goals, in particular goal 6: Ensuring availability and sustainable management of water and sanitation. Water scarcity and an absence of proper wastewater management are large obstacles to social and economic development, as they obstruct advancements in food security and poverty reduction (United Nations, 2018a; Brears, 2016).
Water scarcity does not only arise from a physical shortage of water. Exploitation of water resources can also form a bottleneck, whereby technological, economical and social challenges must be overcome. These include infrastructural difficulties, high costs, wasteful water use and imbalanced social and power relations (Falkenmark et al., 2007; Ohlsson and Turton, 1999).

Due to its high population density and the high dependency on imported water from neighbouring regions, Flanders is considered a water scarce region. It ranks among the most water scarce regions in the OECD and has, together with the Brussels region, a lower water availability than most Southern European states (*e.g.*, Spain, Portugal, Greece) (Brouwers et al., 2015; De Nocker et al., 2017). A recent study performed by Wolfs et al. (2018) concluded that intensification of precipitation patterns due to climate change, intertwined with longer periods of drought in summer times can lead to more frequent sewer overflows into surface waters and severe groundwater depletion. Hence, there is an increased need for efficient water management in Flanders, with a focus on sustainable water resource management.

The American Society of Civil Engineers defined sustainable water resources systems as follows: "Sustainable water resources systems are those designed and managed to fully contribute to the objectives of society, now and in the future, while maintain-ing their ecological, environmental, and hydrological integrity." (ASCE, 1998). Some main criteria to assess the sustainability of such systems which are applicable to this thesis, were presented by Asano et al. (2007): One must a) meet basic human needs for water, b) maintain long-term renewability, c) promote efficient use of resources, d) encourage water conservation and e) design for resilience and adaptability.

1.2 Current centralized approach in water management

Water management in most of the developed world is currently practiced in a highly centralized manner, relying on extensive piping networks for potable water supply and for separated wastewater evacuation through sewers (Asano et al., 2007). The main cause for the implementation of centralized sewage systems as we know it today was the deteriorated public health in European cities in the nineteenth century. For instance, in London a series of severe cholera epidemics had occurred and the arrival of an extensive sewage system led to enormous improvements in public health and the quality of life. The growing sewage systems simultaneously led to large centralized potable water supply networks (Burian et al., 2000; Larsen et al., 2013). Water distribution in industrialized countries evolved to be supply-driven, leading to sufficient quantities of high-quality water being available. Consequently, reuse of (municipal) wastewater streams was often undervalued (Jabornig, 2014).

Water reuse can be strictly defined as the treatment of wastewater to a sufficient quality with the aim of applying it for beneficial uses, *e.g.*, in agriculture or industry. It is often considered when existing water resources do not suffice and single use of water is not favoured (Asano et al., 2007). A growing water demand along with salinization and a decreased quality of water bodies, as well as the threat of climate change, all led to an increased interest in water reuse options (Falkenmark et al., 2007; Jeffrey et al., 2018).

1.3 Greywater and rainwater as alternative water resources

Greywater and rainwater are two considerable water flows that are often left unused in modern water management. In Flanders, 2.22% of the precipitation should be harvested in order to satisfy all domestic water needs, which shows the potential for reuse. Greywater, defined as wastewater from most domestic sources excluding toilet water, is free from large quantities of faecal contamination and hence more easily considered for reuse. Another advantage is the generally constant relation between its availability and the potential for greywater reuse (Larsen et al., 2013; Leong et al., 2017). Rainwater is defined as rooftop runoff from rainfall precipitation and is generally considered directly applicable for non-potable uses. However, rainwater harvesting is subject to temporal and seasonal differences in precipitation (Li et al., 2010). In the last few decades, a substantial amount of research on in-house reuse options for both rain- and greywater separately has been conducted with many pilot projects (Jabornig, 2014; Larsen et al., 2013; Leong et al., 2017). So-called hybrid rainwater–greywater systems, benefiting from the properties of both water flows, are also gaining interest as will be discussed in section 2.5 (Leong et al., 2018).

CHAPTER 2

DECENTRALIZED WATER MANAGEMENT

2.1 Motives for decentralization

Decentralized wastewater installations are standalone systems used for the treatment of small wastewater flows. Here, collection, treatment, discharge and potential reuse of wastewater takes place near the point of formation (Larsen et al., 2013). As stated in the previous chapter, most wastewater infrastructure is currently operated in a highly centralized manner with wastewater being transported over long distances towards a central treatment plant. These plants are capable of handling large amounts of influent per area occupied and are ideal for supporting large, densely-populated areas (Siegrist, 2017). However, (partial) decentralization of wastewater management might be an option worth considering in the context of increased urbanization, water scarcity and necessity for water reuse (Libralato et al., 2012). Due to past investments into centralized infrastructure and difficulties in implementing decentralized technology, decentralization is not always practicable, even when it is the desired option (Larsen et al., 2013). A summary of advantages and disadvantages of decentralized compared to centralized wastewater treatment is given in table 2.1.

The economic factor forms an important consideration in the choice between centralized and decentralized wastewater treatment. For instance, sewer infrastructure alone is responsible for approximately 25% of the operational costs and more than 80% of capital and replacement costs within centralized wastewater treatment systems. In addition, long-term investments in sewer systems limit flexibility and adaptability, decreasing the resilience of centralized systems to disturbances. In decentralized treatment systems, the treatment unit requires the highest investment (Maurer et al., 2005; Leigh and Lee, 2019). Furthermore, centralized systems tend to benefit from economies of scale, as treatment costs per unit wastewater decrease as the capacity increases. This leads to decentralized systems often only being considered when the cost of connection to the sewer system is too high. However, decentralized systems are increasingly able to compete with centralized systems due to technological advancements, *e.g.* in membrane technology, and an "economy of numbers". The latter implies the decreasing cost of small-scale treatment systems due to mass-production (Larsen et al., 2009; Libralato et al., 2012).

In Flanders, a considerable concern is ribbon development ("*lintbebouwing*"), for which more sewer infrastructure is needed per building compared to, *e.g.*, the amount needed in the city centre. The cost of installing sewer infrastructure in such outlying areas is estimated at 3.6 billion euro over the 15-year period of 2012-2027, with a cost of up to 16 500 euro per household. As such, considering decentralized management of wastewater at the household-level can be economically beneficial (Schauvliege, 2015).

When implementing decentralized wastewater reuse systems in an area where a centralized wastewater collection system is already in place, effects on the characteristics of the wastewater flowing towards the wastewater treatment plant are expected to occur (Friedler and Hadari, 2006). An important concern is an increased likelihood of piping blockage, odour and sedimentation in the sewer network due to more concentrated wastewater streams, as the less concentrated streams are reused (Marleni et al., 2012). However, Penn et al. (2013) argue through simulation that there is no evidence of an increased risk of piping blockage due to greywater recycling, as the lower flow rate of more concentrated wastewater remains sufficiently high to avoid clogging. Furthermore, more wastewater contributors could be connected to an existing sewage network without enlargement of piping size being necessary, generating large savings.

Table 2.1: Advantages and disadvantages of decentralized compared to centralized wastewater treatment. Compiled from: Larsen et al. (2013); Siegrist (2017); Leong et al. (2018); Maurer et al. (2005); Libralato et al. (2012); Liberman et al. (2016); Leigh and Lee (2019). *Sunk costs here imply the large past investments typical for centralized wastewater infrastructure. These should not influence decision-making, although, in reality, they do guide investments away from new technologies if these do not build further upon the set foundation.

Advantages
• Infrastructure can be designed water-tight and resistant against corrosion. This is not evident for large sewer networks. • Recharge of water bodies at point of extraction is possible • Potential for faster innovation due to more competition on the market, connection with the user and less sunk costs* • Increased user awareness of own consumption and wastewater production • Less susceptible to natural disasters or terrorist attacks • Increased resilience and possibility to adapt to changing conditions, in contrast to mere robustness • Prevention of environmental pollution due to easier handling of leakage • Events such as heavy rainfall can more easily be handled. Events such as combined sewer overflow, where wastewater is not treated and released in the environment, do not take place • Reuse of water is facilitated, infrastructure for source separation is more easily implemented • Less impact on near environment • Increased water buffering capacity • Potential reduction of high-quality water used for applications with a low-quality need such as toilet flushing
Disadvantages
• Does not benefit from economies of scale • Lack of legal framework or standardization, obstructing large initiatives • More risk of accumulation of contaminants such as micropollutants in smaller, circular systems • Difficult flow equalization throughout the system • Generally a higher energy input per amount of water treated and a higher physical footprint • Less public acceptance

2.2 Current status of decentralization

The use of decentralized wastewater systems can be traced back to the late nineteenth century, where remote residences or small communities were provided with basic wastewater treatment, most often a septic tank followed by infiltration or disposal into a body of water. For a long time, decentralized wastewater treatment systems were seen as temporary installations, awaiting connection with a centralized facility. In the United States, where more than 25% of the population uses decentralized solutions for wastewater management, decentralized treatment systems have only recently been installed for permanent operation (Siegrist, 2017; UNEP, 2000). Also in Australia, as a response to years of increasing pressure on water reserves as a result of accelerating climate change, a rapid evolution towards decentralization and increased circularity has been ongoing (Larsen et al., 2013; Tjandraatmadja et al., 2009).

Developments in wastewater treatment technologies and an increased insight into wastewater composition have led to a growing market in decentralized systems for (waste)water management (Gikas and Tchobanoglous, 2009; Larsen et al., 2013). For example, in India, a heavy increase in reverse osmosis (RO) based treatment systems for in-house potable water creation from tap water and other less contaminated sources has taken place. Here, the domestic sector makes up 85% of the total small-scale water purifiers market (O'Connor et al., 2016).

2.3 Water flow characterization

In order to understand the operation of decentralized wastewater treatment systems, information on the water flows going through these systems is essential. Rainwater and greywater are the most important flows considered in the scope of this thesis.

2.3.1 Rainwater

Rainwater is considered a relatively clean water source which can be used directly for non-potable purposes or reliably turned into potable water after sufficient treatment (Li et al., 2010). It is collected as rooftop runoff and a significant part of the contamination is therefore caused by leaching of contaminants from the roofing surface. Here, pollutants accumulate either due to atmospheric deposition or due to deterioration of roofing materials (Campisano et al., 2017). A significant fraction of these pollutants is washed off by an initial amount of precipitation, the so-called "first-flush". First-flush diversion can consequently be applied, during which the initial volume of rainwater coming from the rooftop is separated from the bulk, avoiding entrance of this more heavily-contaminated rainwater into the system (De Buyck, tion; Mendez et al., 2011).

A summary of the physico-chemical quality of harvested rainwater is given in table 2.2. Harvested rainwater can contain elevated concentrations of heavy metals, mainly due to leaching of roofing material and piping. Elevated lead, copper and zinc concentrations in particular could form a threat to public health when using rainwater for potable use. Furthermore, as metal leaching potentially increases on aging roof materials, this could form a significant long-term risk (Leong et al., 2017; Campisano et al., 2017; Clark et al., 2008; Meera and Ahammed, 2006). Rainwater is also characterized by a low pH, resulting from acid rain. The acidity of rainwater can be compensated by using a concrete storage tank, which releases calcium, increasing the pH (Abbasi and Abbasi, 2011).

Significant microbial contamination of harvested rainwater can occur through animal droppings and decaying organic matter on roofing material (Lee et al., 2017). However, instances of illness caused by drinking rainwater are few and, although strongly dependant on the location and climate, rainwater is often considered microbially and physico-chemically safe for consumption (Australian Government Department of Health, 2011; Campisano et al., 2017; Meera and Ahammed, 2006). In the contrary, other studies reported significant levels of pathogenic bacteria and protozoans, arguing that rainwater should be sufficiently treated before potable use (Ahmed et al., 2010; Gikas and Tsihrintzis, 2012). Furthermore, high concentrations of trace organic compounds such as pesticides may occur in rainwater, possibly exceding drinking water standards (Meera and Ahammed, 2006). Low quality rainwater can particularly occur after long periods of dry weather, when accumulated contaminants are not washed off. This can be counteracted by first-flush diversion (further discussed in section 2.5.1) (Sazakli et al., 2007; Meera and Ahammed, 2006).

2.3.2 Greywater

Household wastewater can be split up into two main fractions: blackwater and greywater. Blackwater is wastewater coming from the toilet, and therefore a highly contaminated waste stream. Greywater is defined as wastewater originating from all conventional household applications excluding the toilet. This significantly reduces microbial contamination relative to combined household wastewater and as such makes greywater more applicable for reuse (Larsen et al., 2013). Another advantage of greywater is the reliability of production: when water is being used in a household, greywater is produced. Furthermore, greywater is produced in large quantities, as it

Table 2.2: Quality characteristics of rainwater. Median data compiled from: De Buyck (tion); Sazakli et al. (2007); Göbel et al. (2007). Ranges adopted from Leong et al. (2017). DWG = Drinking Water Guidelines, adopted from Flemish drinking water standards, (Vlaamse Regering, 2017). All microbial data adopted from Sazakli et al. (2007).

Parameter	**Unit**	**Median**	**Range**	**DWG**
pH	-	6.6	3.10-11.40	6.5-9.2
Total Suspended Solids (TSS)	mg/l	43	1.00-153.00	-
Hardness	mg/l	40	0.00-270.00	-
Turbidity	NTU	1.2	0.20-303.50	Not visible
Chemical Oxygen Demand (COD)	mg/l	23	8.74-23.83	-
Nitrate	mg/l	7.04	5.28-13.02	50
Nitrite	mg/l	0.013	0.003-0.043	0.1
Ammonium	mg/l	0.01	0.01-0.05	0.5
Total Phosphorus	mg/l	0.22	0.21-50.00	-
Chloride	μg/l	7740	0.00-164 000	250 000
Copper	μg/l	153	1.10-4500	2000
Iron	μg/l	11	0.00-1390	200
Lead	μg/l	<2.0	2.00-271.00	10
Sodium	μg/l	6000	0.00-32 320	200 000
Zinc	μg/l	370	0.50-3200	-
Total Coliforms	CFU/100ml	11	0-570	0
E. coli	CFU/100ml	0	0-250	0

accounts for 50–80% (78% in Flanders) of the total domestic wastewater production (Vlaamse Milieumaatschappij, 2018; Winward, 2007).

Greywater can also be subdivided according to the origin: Shower (SH), bathtub (BT) and bathroom washing basin (WB) generate relatively less contaminated wastewater, termed 'light greywater'. Kitchen sink (KS), dishwasher (DW) and washing machine (WM) lead to relatively heavily polluted greywater, *i.e.* 'dark greywater' (Winward, 2007). Greywater quality and quantity are highly variable, depending on household habits and the usage of chemicals. Also regional and temporal differences in greywater properties occur. For instance, significant peaks in greywater production usually take place in the morning and evening, with greywater flow coming to a complete halt late at night. This variability brings the need for resilient treatment systems, which can handle unstable loading rates (Fountoulakis et al., 2016; Larsen et al., 2013).

Greywater makes up the majority of the total domestic wastewater production but contains only 30% of the total organics and 9–20% of the nutrients (Fountoulakis et al., 2016). Greywater also consists of relatively low concentrations of nitrogen, phosphorus and potassium, as most is found in urine and therefore separated (Larsen et al., 2013). Furthermore, it often contains elevated concentrations of personal care products, detergents and up to 900 species of trace compounds (Eriksson et al., 2002). Also microbial contamination can be significant, with faecal coliform concentrations up to 10^8 CFU/100ml (Friedler, 2004). Certain greywater streams may also contain elevated salt concentrations such as high concentrations of primarily sodium, chloride and bromide. These mostly originate from detergents and washing powders (Gross et al., 2015). A summary of greywater quality characteristics is given in table 2.3.

Greywater from the kitchen sink is sometimes excluded due to the amount of fat, detergents and microbial contamination from food-handling being introduced in the wastewater. In addition, it is responsible for 40–60% of the total greywater COD, sodium, VSS and BOD load (Liberman et al., 2016; Friedler, 2004; Ottoson and Stenström, 2003).

Table 2.3: Quality characteristics of greywater. Data adapted from Larsen et al. (2013); Fountoulakis et al. (2016).

Parameter	**Unit**	**Mean**	**Range**
pH	-	7.2	6.4-10
Chemical Oxygen Demand	mg/l	620	7-2570
Total Suspend Solids	mg/l	216	2.0-1070
Turbidity	NTU	95	20-280
Total Nitrogen	mg/l	23	0.1-128
Total Phosphorus	mg/l	8.5	0.1-42
Chloride	mg/l	181	9-227
Sodium	mg/l	148	7.4-480
Bromide	mg/l	0.6	/
Total Coliforms	log10/100ml	5.7	4-7
E. Coli	log10/100ml	5.5	4-6

2.4 Legal framework

Worldwide, lacking legal framework obstructs the implementation of wastewater reuse systems. Especially for urban reuse purposes or when considering specific household applications, personal judgment of regulations is often required. This gives rise to a variety of interpretations on the necessary effluent standards to be obtained (Jabornig, 2014). Guidelines for reuse often differ significantly between nations. Also, they apply to different needs and applications and subjected to different social factors such as the public perception of water reuse (Li et al., 2009; Pidou, 2006; Asano et al., 2007). Most reuse regulations only define microbial contamination limits, as they have the most impact on human health. Also aesthetic parameters such as turbidity are often defined, as they influence human perception of wastewater reuse (Pidou et al., 2007).

The WHO's guideline on greywater reuse for agricultural irrigation is one of the few international guidelines on greywater reuse, though it exclusively focuses on microbial parameters (WHO, 2006). In the European Union, the Urban Waste Water Treatment Directive of 1991 is the main regulation concerning collection of wastewater and protection of water bodies (EC, 1991). The EU Water Framework Directive, implemented in 2000, and its revisions contain guidelines for integrating water reuse into water management and specifies the aim of transitioning towards a more circular water management (Jeffrey et al., 2018). Furthermore, a proposal for EU quality guidelines on water reuse has been submitted in 2018 and is yet to be implemented (Alcalde Sanz and Gawlik, 2017; EC, 2018). Another, often used, European guideline is the European Bathing Water Directive, used as a non-potable water quality standard for microbial contamination (EC, 2006). Proposed standards for unrestricted non-potable urban reuse (*e.g.*, for toilet flushing, laundry, cleaning, landscape irrigation) by Li et al. (2009) are given in table 2.4.

Table 2.4: Proposed quality standards for unrestricted non-potable urban reuse of greywater, by Li et al. (2009). BOD_5 is 5-day Biological Oxygen Demand. CFU is Colony-Forming Unit.

Parameter	**Standard**
pH	6-9
BOD_5	$\leq$ 10 mg/l
Turbidity	$\leq$ 2 NTU
Faecal coliforms	$\leq$ 10 CFU/ml
Total coliforms	$\leq$ 100 CFU/ml
Residual chlorine	$\leq$ 1 mg/l

The American National Standard NSF/ANSI Standard 350 for on-site residential and commercial water reuse provides a water quality guideline for non-potable urban applications (toilet and irrigation) (NSF, 2011). Other national reuse guidelines are shown in appendix A.

The environmental legislation in Flanders, collected in VLAREM II (Vlaams Reglement betreffende de Milieuvergunning), does not contain any guidelines on reuse of wastewater. However, it does state the necessary quality of domestic wastewater for discharge into the environment in areas which are not connected to a sewer network

(Vlaamse Regering, 2016). Water quality standards for potable reuse in Flanders are defined in the Flemish Drinking Water Decree (Drinkwaterdecreet) (Vlaamse Regering, 2017). Since 2013, all new construction projects and renovations larger than 40 m^2 in Flanders must incorporate separated drainage of rainwater and wastewater and a rainwater tank. Also, buildings larger than 250 m^2 must add a rainwater infiltration or buffering basin (Vlaamse Regering, 2013). Rainwater is preferably handled according to the following order: Reuse, infiltration, buffering, disposal in separate rainwater sewer and lastly, disposal in a combined sewer system (CIW, 2016).

Regarding infiltration of rainwater and domestic wastewater into the soil, guidelines do exist in Flanders. Infiltration of domestic wastewater into the soil is allowed in areas without connection to a sewer network, as long as a) it is first directed into a sump pit, b) located in a safe distance from other water bodies or groundwater extraction points and c) meets the water quality standards given in table 2.5. A sump pit can be defined as an infiltration unit wherein wastewater is directed after treatment in a septic tank or treatment unit. Rainwater is encouraged to be infiltrated when it can not be reused (VLARIO, 2018).

Table 2.5: Water quality standards for infiltration of domestic wastewater, obtained from article 5BIS.19.8.4.5.6. from Vlaamse Regering (2016).

Parameter	**Standard**
pH	6.5-9
BOD	25 mg/l
SS	60 mg/l
Safe level of pathogenic contamination	
No oils or fats floating on the surface	

2.5 State of the art practices

As this thesis is dedicated to combined decentralized reuse of greywater and rainwater, the technical aspect of the concept must be assessed. In the past, a wide range of technologies have already been created and applied for small-scale treatment of water. Reuse of rainwater and greywater has been an upcoming field of research in last few decades, with combined reuse of rainwater and greywater recently emerging (Asano et al., 2007; Leong et al., 2017).

2.5.1 Rainwater reuse

Rainwater harvesting is an age-old practice used in water scarce areas. Today, advanced rainwater reuse systems are a well-established technology, addressing increased freshwater needs and avoiding urban runoff and flooding (Gikas and Tsihrintzis, 2012; Campisano et al., 2017; Sojka et al., 2016). For these reasons, many countries today support implementation of rainwater harvesting and reuse technologies. For instance in Flanders, it is in most cases mandatory to equip newly built houses with a rainwater collection system (Vlaamse Milieumaatschappij, 2014). Rainwater is most often reused for applications necessitating little treatment, such as toilet flushing and garden irrigation (De Kwaadsteniet et al., 2013). Although subjected to the variability of rainfall patterns, its total use is mostly dependant on variables such as consumption patterns, tank size and catchment area (Hall, 2013).

Rainwater harvesting and storage

Rainwater can be harvested from terraces, paved areas and from other surfaces. However, rooftops are now most often considered as they represent 30–40% of the impenetrable urban surfaces and are relatively easy to use (Sojka et al., 2016). Still, the rooftop also contributes significantly to the pollution present in harvested rainwater, depending on a) roof size and geometry, b) roofing material, c) proximity of roof to sources of pollution and d) maintenance of roof (Abbasi and Abbasi, 2011). Theoretically, 1 millimeter of rainfall entails harvesting 1 liter of rainwater per square meter of roof surface. In practice however, correctional factors are applied. One such factor is the runoff coefficient (RC), adjusting the total amount of caught rainwater for spillage, leakage, surface wetting and evaporation. Depending on the roofing material, the RC lies between 0.7 and 0.95 (Farreny et al., 2011). The Flemish Environmental Agency (2000) also recommends adding correctional factors for roof inclination and filter losses.
Multiplying the correctional factors with the surface area (m^2) and the amount of precipitation (mm/month) equals the total harvested rainwater per month. As was mentioned in section 2.3, first-flush diversion can be applied in order to improve harvested rain water quality. Martinson and Thomas (2005) derived the following rule of thumb: "For each mm of first flush diverted, the contaminate load will halve". In practice, 1-2mm of precipitation is most often diverted (Förster, 1999).

After rainwater is collected on the roof, it is directed towards the collection tank through the piping system. A pre-filtering step before entering the collection tank is often necessary, as debris is easily transported by the rainwater (De Kwaadsteniet et al., 2013; Sojka et al., 2016). A pre-filtering step often means using a coarse filter or screen. This avoids accumulation of organics, clogging of the pumping system and reduces the amount of cleaning necessary (Leong et al., 2017). After collection, storage of rainwater in a storage tank follows. Storage tanks are most often made out of concrete or plastic, depending on location and desired tank properties, and are often installed underground (De Kwaadsteniet et al., 2013). Installing a large storage volume is often reasonable when aiming at increasing reuse and reducing overflow. However, larger tank volumes logically imply higher capital costs and can moreover lead to excessive stand-still of the harvested rainwater. The latter can cause accumulation of contaminants. Dimensioning tank sizes in pursuance of regular overflow is thus advised (Vlaamse Milieumaatschappij, 2014; GhaffarianHoseini et al., 2016; Australian Government Department of Health, 2011).

Rainwater treatment and use

Rainwater can be directly used for applications where no treatment is required, such as garden irrigation and toilet flushing. However, in order to obtain water of sufficient quality for unrestricted non-potable reuse, treatment is necessary. Here, combined membrane filtration and disinfection of screened rainwater is often considered sufficient (Leong et al., 2017; Jordan et al., 2008; Li et al., 2010). Household scale technologies tested in literature include: disinfection using solar radiation, slow sand filtration and activated carbon filtration, often followed by some kind of additional disinfection (Naddeo et al., 2013; Li et al., 2010). Furthermore, recent advances in small-scale treatment units for rainwater have made it feasible for decentralized conversion of harvested rainwater to potable water. These units can be sized to easily fit in a small room (Sojka et al., 2016; Peter-Varbanets et al., 2009).

Ultrafiltration (UF) and reverse osmosis (RO) technologies are also gaining interest in both industrialized and developing countries for treatment of tap water or rainwater. Both technologies can provide complete disinfection, producing potable water. UF and RO also have a relatively small treatment footprint, as they often do not necessitate chemicals (excluding chemical in situ cleaning (CIP)) or much maintenance. Also, both operational and capital costs have gone down remarkably in the past decades (O'Connor et al., 2016; Peter-Varbanets et al., 2009). Furthermore, as metals form a concern for rainwater reuse, RO is reported to achieve significant metal removal rates (Qdais and Moussa, 2004).

Rainwater harvesting and treatment systems can be optimized using the following criteria (Melville-Shreeve et al., 2016):

- Minimize capital cost of the system

- Maximize water efficiency
- Minimize system energy requirements
- Minimize rainwater discharge into sewer network

Besides treatment infrastructure, a rainwater reuse system consists of a wide range of different components, *e.g.* pumps, pressure vessels, piping, valves and control systems. The implemented configurations, and by consequence costs, differ strongly between installations (Melville-Shreeve et al., 2016).

Infiltration

Infiltration of rainwater is an important aspect of rainwater control. It is crucial in preventing rainwater runoff and flooding in an urban environment and when less rainwater flow into the sewer system is desired or in order to replenish groundwater supplies. In rainwater reuse systems, the storage tank is therefore often connected to an infiltration system (Herrmann and Schmida, 2000). In some cases, excess rainwater can be led to a canal or other surface water through an overflow instead of infiltration. Otherwise, the infiltration basin must be dimensioned large enough so no overflow is needed (VLARIO, 2018). When calculating necessary tank dimensions for infiltration, the rate of infiltration must be considered, since this is not an instantaneous process. Regional differences play an important role here, as different soils have different infiltration capacities and as the height of the water table varies (CIW, 2018).

Infiltration basins are available in different forms, such as perforated tanks, pipe systems and hollow crate structures. The choice depends partly on the height of the water table, as tanks often lay deeper than pipe systems (Vlaamse Milieumaatschappij, 2014). Some rainwater harvesting systems are designed with real-time control systems that use weather forecast data. These can direct rainwater towards an infiltration unit or another point of disposal, for instance right before heavy rain would occur, therefore saving tank capacity (Melville-Shreeve et al., 2016; Han and Mun, 2011).

2.5.2 Greywater reuse

Greywater lends itself better for reuse than combined household wastewater (grey- and blackwater), as it makes up the majority of the domestic wastewater stream and is characterized by low levels of faecal contamination. However, depending on the reuse application and the amount of human contact, a varying degree of treatment is necessary. As awareness and acceptance towards greywater reuse is rising, the

Table 2.6: The relative performance and cost of greywater treatment systems. Reference indicates where technologies were reviewed for greywater reuse in literature. +++ = can reach sufficient effluent quality for non-potable reuse; ++ = can not always obtain sufficient effluent quality; + = poor performance. ***/**/* = high/medium/low cost. Cost is defined as the combination of capital, operational and maintenance costs, adopted from Ghunmi et al. (2011). Other information compiled from Jabornig (2014); Li et al. (2009); Ghunmi et al. (2011).

Treatment type	**Performance**	**Cost**	*Reference*
Sand filtration	++	**	*Itayama et al. (2006); Prathapar et al. (2006)*
Constructed wetland	++	*	*Dallas and Ho (2005); Nolde (2000)*
Microfiltration	++	***	*Shin et al. (1998)*
Ultrafiltration	+++	***	*Šostar-Turk et al. (2005)*
Rotating biological contactor	++	**	*Nolde (2000); Friedler et al. (2005); Friedler and Gilboa (2010)*
Upflow anaerobic sludge blanket reactor	++	*	*Elmitwalli and Otterpohl (2007); Hernández Leal et al. (2010)*
Biological aerated filter	++	***	*Jefferson et al. (2000)*
Sequencing batch reactor	++	**	*Hernández Leal et al. (2010); Shin et al. (1998)*
Membrane bioreactor	+++	***	*Friedler and Gilboa (2010); Jabornig (2014)*

number of greywater reuse systems in practice is growing (Jabornig, 2014; Asano et al., 2007).

Almost no legal standards for reuse exist, bringing about a wide variety of treatment objectives that are being pursued. Also a wide range of treatment technologies have been investigated in literature, among which physical, chemical and biological treatment systems (Li et al., 2009). A summary of different treatment technologies for non-potable reuse of greywater can be seen in table 2.6. Much of these treatment schemes can not persistently achieve minimal effluent standards for non-potable reuse, in particular when considering more stringent standards for urban reuse. Requirements on removal of microbial contamination are an especially important bottleneck. Overall, the implementation of membrane bioreactors (MBR) performed notably well, being the only technology to consistently achieve adequate removal of microbial contamination without additional disinfection requirements (Pidou et al., 2007; Jefferson et al., 2000). However, disinfection before urban reuse is still advised (Larsen et al., 2013).

Membrane bioreactors for greywater reuse

A membrane bioreactor can be defined as combined biological treatment and membrane filtration, the former mostly being activated sludge treatment (Jabornig, 2014). Wastewater is fed continuously or in batch, after which aerobic treatment occurs. Finally, the water is separated from the suspended biomass through a membrane filtration step. Both ultrafiltration (UF) and microfiltration (MF) membranes are used, the former being most prevalent due to superior contaminant removal (Judd, 2010; Arévalo et al., 2012). Membranes are mostly placed inside the bioreactor (submerged or immersed) but can also be located externally (sidestream) (Stephenson et al., 2000). Furthermore, aeration is needed in order to provide dissolved oxygen for the aerobic degradation of contaminants, for the suspension of sludge flocs and for scouring of the membrane. This accounts for about half of the total MBR energy consumption (Meng et al., 2017; Sun et al., 2016).

The MBR is a compact, modular system and produces a steady, high-quality effluent, making it an ideal treatment technology for decentralized applications (Asano et al., 2007; Stephenson et al., 2000). A membrane is used for biomass separation instead of conventional clarification, leading to several advantages: a) smaller infrastructure, b) higher quality effluent, c) more mixed liquor suspended solids (MLSS) leading to a more stable sludge, d) no need for sludge recirculation, e) easier operation (Asano et al., 2007). Besides MBR treatment bringing forth a functionally stable sludge, being relatively resistant to shocks, it also has a lower total sludge production relative to conventional treatment (Wagner and Rosenwinkel, 2000). The main obstacles when considering MBR are the high energy demand for operation, high capital costs of the membrane units and membrane fouling and replacement (Judd, 2010). Much research is being performed on the topic of membrane fouling in particular, as the impact on the performance of the MBR is considerable. As was mentioned earlier, membrane scouring through aeration is currently the most practiced solution (Meng et al., 2017).

MBR treatment of greywater is characterized by its high removal rate and removal efficiency of solids, organics, pathogens and nutrients (Stephenson et al., 2000). It also produces a clear and odourless effluent, making urban reuse more attractive (Merz et al., 2007). Average contaminant removal efficiencies are given in table 2.7. Pathogen removal is most often complete, providing effluent pathogen concentrations below the detection limit (Larsen et al., 2013). However, bypassing of the membrane unit can take place due to the "hopping phenomenon", where transport of microorganisms occurs through aerosols (Friedler and Gilboa, 2010). During greywater treatment, the contaminant load entering the MBR is highly variable. Therefore, an equalization basin is often implemented (Shin et al., 1998; Friedler and Hadari, 2006).

Table 2.7: Average pollutant removal efficiencies for greywater treatment by MBR. *: Microbial removal efficiencies depicted as log10 removal. Based on data from: Atasoy et al. (2007); Merz et al. (2007); Liberman et al. (2016); Lesjean and Gnirss (2006); Jabornig (2014); Bani-Melhem et al. (2015).

Parameter	**Average removal (%)**	**Range (%)**
Chemical Oxygen Demand	85.7	64–97
Biological Oxygen Demand (5 days)	95.8	94–98
Total Suspend Solids	99.0	98–100
Total Nitrogen	71.5	52–92
NH4+-N	84.4	72–96
Total Phosphorus	46.2	17–91
Total Coliforms	4.5*	4–5
Faecal Coliforms	3.7	3–4

Disinfection

All water intended for potable reuse must be free of any microbial contamination that poses a threat to human health. Caution regarding pathogen presence is also required for non-potable applications, as aerosol inhalation or accidental water ingestion form a substantial risk (Winward, 2007). In this light, adequate disinfection of reused wastewater streams is of great importance.

A wide range of disinfection technologies that are applicable on a small scale already exist. The most prominent disinfectants are chlorine, ozone, UV radiation and advanced oxidation processes (AOP's) (Larsen et al., 2013). These cause microbial inactivation through damaging of the cell wall, cell permeability or DNA/RNA and/or through inhibition of enzymatic activity (Asano et al., 2007). AOP's are a set of techniques that combine H_2O_2, ozone and/or UV light to form hydroxyl radicals, which are used as a strong oxidizing agent for both disinfection and trace organics removal (Dominguez et al., 2018; Barazesh et al., 2015). Ozone is a strong disinfectant, injected as fine bubbles in the pre-treated wastewater. The actual disinfection occurs through free radical formation (HO_2 and OH) followed by what is generally accepted to be cell lysis of the bacterial cell. Ozone generation occurs onsite as it is unstable and decomposes into oxygen in a short amount of time (Asano et al., 2007; EPA, 1999).

Chlorine is the most widely used disinfectant and can be applied either in dry or wet form. The most commonly applied chlorine compounds are chlorine (Cl_2), sodium hypochlorite (NaOCl), chlorine dioxide (ClO_2). For small-scale treatment systems, calcium hypochlorite ($Ca(OCl)_2$) is often applied due to the ease of handling. Disinfection often occurs in a specially designed contact chamber. Chlorination as a disinfection method has been a subject to debate in recent years as it is a highly toxic substance and as it potentially reacts with organic constituents, forming carcinogenic disinfection by-products (DBP's) (Asano et al., 2007; Winward, 2007). An important consideration for all mentioned disinfection methods is that it must be safe to handle and apply, as it takes place in close proximity to the consumer (Asano et al., 2007).

UV radiation disinfection, with wavelengths between 220-320 nm being most common, is a physical disinfection method leading to damage of pathogen DNA and RNA. It is considered well suited for small scale disinfection as there is no storage or dosing unit necessary, contrary to the other mentioned disinfection options (Friedler and Gilboa, 2010; Asano et al., 2007). UV is often used as a polishing step after treatment with MBR, as combined treatment results in excellent disinfection of water (Jabornig, 2014; Radjenović et al., 2008). Key advantages and disadvantages of the mentioned disinfection technologies are summarized in table 2.8.

2.5.3 Combined rainwater–greywater systems

Despite the lack of research on combined reuse of rain- and greywater, it is a promising idea when pursuing water conservation or independence of a water reuse system. Combined usage of both rainwater and greywater can overcome several drawbacks of separate use. For instance, the amount of rainwater harvested is highly variable, while greywater production conveniently matches water usage (Loux et al., 2012). The acidity of rainwater can also be mitigated by mixing with greywater (Leong et al., 2018). Furthermore, Ghisi and Ferreira (2007) performed an economic analysis and determined that the installation of a combined greywater and rainwater reuse system leads to a decrease in total cost of more than 20% compared to separate installation, in addition to higher potable water savings (36.7-42.0% for combined reuse compared to 28.7-34.8% for greywater reuse alone).

Few pilot installations of combined rainwater–greywater reuse systems exist. Leong et al. (2018) recently carried out a pilot-scale study using a granular activated carbon filter (GAC) with ozone disinfection. Two main conclusions were (1) the possibility of achieving urban water reuse quality standards by dilution of greywater with rainwater or tap water and (2) that during combined rainwater–greywater reuse, treatment must be able to withstand both high levels of organics from greywater and high metal concentrations from rainwater.

2.5.4 Monitoring of reuse quality

Decentralized water management systems are consistently increasing in complexity, making adequate monitoring of the system a necessity. Monitoring is defined

Table 2.8: Key advantages and disadvantages of different disinfection techniques. Compiled from: Winward (2007); Asano et al. (2007); Friedler and Gilboa (2010); Friedler et al. (2011); Barazesh et al. (2015).

	Key advantages	Key disadvantages
UV radiation		
	+No formation of taste or odour	-No residual disinfection
	+Short treatment time	-Energy intensive
	+Safety	-Frequent lamp replacement
	+Cost-effective	-Turbidity heavily influences result
	+Does not increase TDS of effluent	-Relatively high-maintenance due to fouling, scaling
	+No need for dosage and storage units	-Less effective in inactivating some viruses, spores and cysts
	+No disinfection by-products	
Chlorine		
	+Additional oxidation of organics	-Formation of carcinogenic by-products
	+Residual disinfection	-Relatively long contact time necessary
	+Well-established technology	-Monitoring of chlorine residual might be necessary
		-Toxicity (in high concentrations)
Ozone		
	+Additional oxidation of organics	-Highly corrosive
	+High deodorizing ability	-Usually not cost-effective for small systems
	+More effective than chlorine in inactivating most viruses, spores, cysts, oocysts	-Requires a feed gas preparation unit and pump
	+Contributes to dissolved oxygen	-Difficult monitoring of ozone residual
	+Shorter contact time than chlorine	-Little residual disinfection
		-Toxicity (in high concentrations)
Advanced oxidation processes		
	+Higher efficiency than solely ozone, UV or hydrogen peroxide	-Often special reactor required
	+Good removal of trace organic compounds	-Difficult transport and storage
	+Possibility of electrochemical production	-Expensive
		-Potential production of bromated byproducts

as the tracking of certain water or process characteristics over time through online instrumentation. This allows for an operator to analyze performance and detect malfunctioning of the system (Larsen et al., 2013). Monitoring can be implemented into a control system, controlling pumps, valves and the general treatment operation. The system must be able to function autonomously, which includes being resistant to a variable influent quantity and quality and against treatment failure, thus requiring monitoring (Asano et al., 2007).

When dealing with a highly variable influent flow and quality, which is the case for decentralized systems, a faster response to system malfunctioning is required than for larger systems. The complexity of treatment systems involving biological processes, considering the many strong non-linear relationships between variables, requires much attention towards process control (Kazor et al., 2016).

2.6 Current pilot projects and experiences

As reuse of wastewater streams has been becoming increasingly interesting over the last few decades, many pilot and full-scale reuse installations have been installed. Here some projects involving rainwater, greywater or combined greywater–rainwater reuse are shown:

Case 1 The Star City development project in Seoul, South Korea aims at collecting rainwater from their buildings counting 1300 apartments in order to prevent flooding in the area, store water for irrigation and fire emergencies and to allow water conservation. The rainwater is harvested from 6200 m^2 roof surface and 45 000 m^2 of terrace surface, after which the rainwater is stored in 3000 m^3 of storage tanks (three tanks of 1000 m^3) underneath the buildings (Han and Mun, 2011).

Case 2 In Cu khe, near Hanoi in Vietnam, two schools obtain potable water through harvesting rainwater from the building roof. In order to obtain safe drinking water, the water is filtrated and disinfected with a UV-unit. The filtration step consists of cartridge filter, two carbon filters and a membrane filter. The obtained purified rainwater is of potable water quality (Lee et al., 2017).

Case 3 Participating in the international Solar Decathlon competition of 2019, Ghent University commenced the project "The Mobble", a modular building component where several innovative building concepts are displayed (`https://www.themobble.be`). In cooperation with the Centre for Advanced Process Technology for Urban Resource Recovery (CAPTURE), sustainable water management is also investigated in this project. Here, combined rainwater and greywater reuse is accomplished through the use of an

MBR unit for greywater treatment and an activated carbon filter for rainwater purification. This could lead to water savings of up to 48%.

Case 4 In the residential area of Preganziol, Italy a greywater reuse system is in place for greywater treatment of 250 people. Here, two constructed wetlands (horizontal subsurface flow) are implemented, creating purified greywater of sufficient quality for toilet flushing. The constructed wetlands offer a relatively efficient and odourless treatment (De Gisi et al., 2016).

Case 5 An apartment building in Berlin-Kreuzberg collects greywater from the shower, bathtub and washing basin from 70 people. This greywater is treated by Rotating Biological Contactor (RBC) after which it is disinfected by UV. The purified greywater is then stored in a storage tank where water from the potable water grid can be added if needed and is subsequently used as toilet flushing water (De Gisi et al., 2016).

Case 6 In Switzerland, the "Self" project (`https://www.empa.ch/web/self/water`) aims at demonstrating the possibility of independence from the potable water grid for a household. Potable water is acquired through ultrafiltration of rainwater, harvested on the roof. It is subsequently stored in a storage tank with an internal UV disinfection unit. The potable water is then used in the kitchen sink and washing machine, after which the greywater is sent to a membrane bioreactor with UV disinfection. Hereafter, the purified greywater is reused in the dishwasher, shower and toilet. Blackwater is stored in a 400 l storage tank which is frequently emptied. For this installation, the pumps are energy-efficient and require little maintenance.

Case 7 Gust'eaux, a restaurant in Flanders, Belgium, is planning on treating the restaurant's wastewater to drinking water that will be served to customers (`https://www.i-qua.eu/2018/12/05/premium-water-van-eigen-bodem/`). The primary treatment exists of a vertical bed halophyte filter, after which the water can already be reused as toilet water. The water is then further treated by reverse osmosis and remineralized, creating potable water.

CHAPTER 3

OBJECTIVES

As described in the previous chapter, an alternative to the current paradigm of centralized water management is the introduction of decentralized water systems. Therefore, it is crucial to obtain a thorough understanding of the functioning of such systems and to determine the difficulties that must be considered during system design.

The principal aim of this thesis is to simulate the operation of decentralized water reuse systems, specifically on a household level and independent from any sewer system or potable water grid. Various technological configurations will be assessed and practical considerations when aiming for the implementation of these systems will be discussed. The focus will lie on combined greywater–rainwater reuse with MBR and RO-based treatment technologies. The following research questions (RQ) will be investigated:

- **RQ 1**: Can a household water management system provide independence from the potable water grid or sewer system?
- **RQ 2**: What are the main obstacles to implementation of a water management system at household level?
- **RQ 3**: What is the effect of first-flush diversion on the system?
- **RQ 4**: What will the effect of climate change be on the system?
- **RQ 5**: What should be considered for blackwater reuse?

CHAPTER 4

SIMULATIONS

4.1 Scenario 1: Off-grid residential building

A principal aim of this thesis was to assess water management systems for a single household, disconnected from the potable water grid and sewer system, in which combined reuse of rainwater and greywater was performed. This concept analysis was carried out in detail by scenario simulations. A specific technological configuration was considered and the water flows within the system were calculated (Microsoft Excel 2016). Furthermore, the feasibility of complete disconnection from the grid was assessed by determining the necessity of external water supply or wastewater removal by means of a purge.

The scenario described in this section was used as the foundation for further scenario analyses of small-scale water management systems. Different water flows into, out of and within the system were defined, all within the physical boundaries of a single house. Furthermore, assumptions for water storage and the technological configuration were made. The purpose was to develop a comprehensive concept analysis of a home-based water management system.

Table of Abbreviations

BOD	Biological Oxygen Demand
BS	Bathroom sink
BTSH	Bath and shower
BW	Blackwater tank
CL	Cleaning
COD	Chemical Oxygen Demand
$[COD]_A$	COD concentration of (flow) A
DW	Dishwasher
GW	Greywater
GWinc	Increased greywater reuse
INF	Infiltration
KS	Kitchen sink
MBR	Membrane bioreactor
MTBF	Mean time between failures
PGW	Purified greywater tank
PW	Potable water tank
$Q_{A,B}$	Flow of water from A to B
$Q_{GW,A}$	Flow of greywater from or to A
rec	Recovery
RO	Reverse osmosis
RW	Rainwater tank
RWinc	Increased rainwater reuse
TL	Toilet
WM	Washing machine and hand wash of clothes

4.1.1 General assumptions

In order to construct a viable scenario simulation, certain general assumptions were made. All of the assumptions lay within the context of an average household in the Flanders region in Belgium and within the boundaries of a single free-standing house. The household size was assumed to be 2.3 individuals in size, being the Flemish average (Statbel, 2017). The roof of the house was considered to have a surface area of 100 m^2, which is in line with the average roof sizes of stand-alone houses in Ghent, Flanders (Stad Gent, nd).

The average individual consumption of water in Flanders is 3.3 m^3/month, not considering losses through leakage and consumption of bottled water (Vlaamse Milieumaatschappij, 2018). The average water consumption per household application is given in table 4.1. The following water flows were distinguished: Greywater, purified greywater, rainwater, potable water and blackwater (see section 2.3 for the flow characteristics). Purified or treated greywater was defined as water applicable for direct non-potable reuse. Potable water, originating from rainwater, was considered suitable for human consumption. Blackwater, or toilet wastewater, was stored separately and was not considered for reuse in this scenario. Moreover, blackwater was assumed to flow into a septic tank (blackwater tank) which is emptied when full. Rainwater was harvested from the roof surface and collected in a storage tank. Here, rainwater was used for a) direct use for gardening, without treatment, b) mixing with greywater in the greywater treatment component, c) treatment to potable water and d) infiltration of surplus rainwater.

Table 4.1: Average household water use in Flanders, data obtained from Vlaamse Milieumaatschappij (2018).

Application	**Individual use** (m^3/p/y)	**Share of total use** (%)
Toilet	7.8	19.5
Bath & Shower	10.6	26.6
Bathroom sink	3.4	8.6
Washing machine & Hand wash clothes	6.1	15.4
Dishwasher	0.8	2.1
Kitchen sink	6.2	15.7
Cleaning	2.1	5.3
Plants and garden	2.7	6.8
TOTAL	39.7	100

All forms of water consumption were assumed to take place uniformly over the year, excluding water use for plants and garden as this use would not be evenly distributed throughout the year in a temperate climate. Due to a lack of data on the exact distribution of garden and plant watering throughout the year a standard normal distribution was chosen, balanced around the warmest month, July.

The system layout was given in figure 4.1. Four storage tanks were considered in the system: A potable water tank (tank **PW**), a rainwater tank (tank **RW**), a purified greywater tank (tank **PGW**), and a blackwater tank (tank **BW**). Between the storage tanks, different water flows took place.

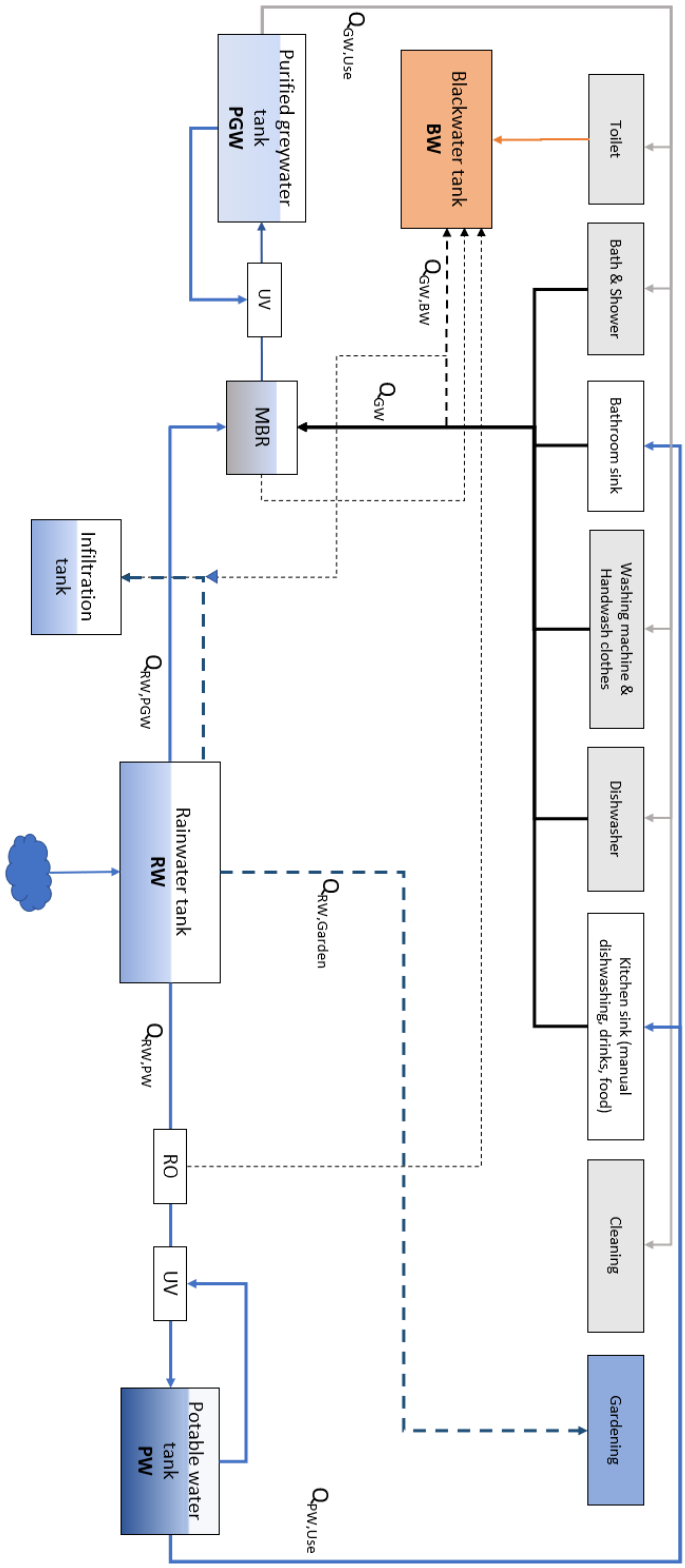

Figure 4.1: The system configuration. Black, dotted flows leaving treatment units are treatment waste flows.

4.1.2 Scenario development

Every scenario was conducted in a discrete manner, per month (a month defined as 1/12th of a year). This means that no day-to-day analysis was made, but that every mass-balance was resolved on a monthly basis. This way, daily fluctuations could be neglected. Every flow was considered to happen evenly distributed across the month. The volume of water in a tank was calculated as the sum of all the flows during that month, and consequently represented the volume of water present in a tank at the last day of the month. Every scenario was ran over a period of 5 years (60 months).

Two sub-scenarios were distinguished: (a) increased rainwater reuse and (b) increased greywater reuse:

(a) In the increased rainwater reuse scheme, an increased amount of rainwater was used as a mixing agent along with greywater in the MBR unit. This would lead to a better influent quality into the treatment step, thus requiring less pollutant removal. The aim of increased rainwater reuse was therefore to take advantage of the superior quality of rainwater. An unfavorable result of this approach was an increased flow of greywater to the blackwater tank, increasing the amount of emptying necessary for the blackwater tank. Furthermore, rainwater was used for toilet flushing instead of greywater.

(b) The increased greywater reuse scheme aimed at making the entire system more circular, by wasting less greywater to the blackwater tank. This would increase the amount of contaminants entering the treatment setup, decreasing the amount of emptying of the blackwater tank that was needed and increasing the amount of rainwater infiltration, which would be beneficial for groundwater levels.

To ensure a continuous water supply, minimal available water volumes were considered per tank (see table 4.2). These were defined as PW_{min}, RW_{min} and PGW_{min} for tank PW, RW and PGW respectively. These minimal available water volumes were determined based on a minimal period of normal water consumption without any input of rainwater into the system (*i.e.*, during a drought). The minimum water volume should be enough for at least one month of normal water use without input of rainwater. When water levels decreased below the minimum allowed volume, a lower outflow and/or higher inflow was put in place as a means to lower the risk of water depletion. For example, when the minimal available water in tank RW was reached, there would be less flow from tank RW to tank PW and PGW. In this scenario, a failure was defined as a completely empty tank during a month. Total tank volumes were chosen to be larger than this minimum volume. The rainwater tank volume was set at 10 m^3 as this is already frequently the case in real installations. The blackwater tank was considered to be 20 m^3, which is relatively large.

Table 4.2: Parameters used in Scenario 1. Minimal tank volumes were based on the volume of water necessary to endure a dry period (defined by the Days of Drought Margin). RW_{min} and PGW_{min} were different for the two sub-scenarios as the water use also differs. a) PW_{min} was not used in calculations in this scenario analysis.

Parameter	**Value**	*Unit*
Roof area	100	m^2
Inhabitants	2.3	*p*
Days of Drought Margin	30.41 (=365/12)	*d*
Volume Tank PW	2	m^3
Volume Tank RW	10	m^3
Volume Tank PGW	6	m^3
Volume Tank BW	20	m^3
RO recovery	0.8	-
MBR recovery	0.9	-
Increased greywater reuse		
PW_{min} a)	1.85	m^3
RW_{min}	4.33	m^3
PGW_{min}	5.24	m^3
Increased rainwater reuse		
PW_{min} a)	1.85	m^3
RW_{min}	5.82	m^3
PGW_{min}	3.75	m^3

4.1.3 Rainwater harvesting, storage and treatment

Rainwater supply

Precipitation in Flanders is rather homogeneous across the region, which allows data from one measuring station to be representative for the rest of Flanders (CIW, 2012). Rainfall data was obtained for Vinderhoute, near Ghent, Belgium (Waterinfo.be, nd). Average monthly precipitation data was used from the period 2012-2017, containing the relatively wet year 2012 (983.1 mm) and the relatively dry year 2015 (700.85 mm), when compared to the average Flemish yearly rainfall of 858 mm (KMI, 2014). This data allows for a realistic depiction of the variation in Flemish precipitation.

According to a statistical analysis on Flemish meteorological data for the period 1835-2014 performed by for the Flemish Environmental Agency, there is no evidence that the length or magnitude of droughts in Flanders will increase due to climate change. The average duration of dry periods in Flanders, with a daily precipitation below 0.5 mm, is 20 days. Wet periods on the other hand are expected to be less frequent but will rise in precipitation intensity, both in winter and summer (Brouwers et al., 2015). However, Wolfs et al. (2018) argue that decision-making in the water sector in Flanders requires using the high-impact climate change models constructed by the IPCC (Intergovernmental Panel on Climate Change). These do predict an increased duration and intensity of droughts. Therefore, the water management system must be able to withstand:

- Prolonged periods of drought or periods with a small total amount of rainfall, when water input into the system is minimal.
- Periods with high rainfall intensity or an abundance of rainfall, potentially challenging infiltration of surplus rainwater into the soil.

In order to endure these changes, the different tank sizes and water flows must be designed adequately and water infiltration must be practiced appropriately. This will be further investigated in the following sections.

Rainwater harvesting

Rainfall (expressed in mm or l/m^2) was assumed to be harvested from a roof with surface S_{roof}. Several correction factors were included to calculate the total amount of harvested rainwater from the roof (see table 4.3). These correction factors were based on the type of roofing material and the type of pre-filter that removed debris from the harvested rainwater before entering the rainwater tank. Assuming a sloping roof, a correction factor of 0.9 was taken and assuming a downpipe or self-cleaning filter is in place, an additional correction factor of 0.9 was used (Vlaamse Milieumaatschappij, 2000). This means that the total amount of rainwater harvested was 0.81 times the total rainfall on the roof (eq. 4.1).

$$\text{Harvested rainwater} = \mathrm{Rainfall} \cdot S_{\text{roof}} \cdot 0.81 \cdot 0.001 \tag{4.1}$$

Here, harvested rainwater was expressed in m^3 of rainwater. Any effects of the slope and orientation of the roof were neglected, considering that the roof is used in its entirety and is oriented equally to the opposite wind directions. No first-flush diversion was implemented in this scenario, this will be further discussed in section 4.5. See figure 4.2 for the total amount of rainwater harvested from the roof per month.

Table 4.3: Correction factors in rainwater harvesting, obtained from Vlaamse Milieumaatschappij (2000).

Parameter	Correction factor
Roof shape	
Flat roof	0.6-0.9
Sloping roof	0.8-0.95
Filter type	
Downpipe filter	0.9
Self-cleaning filter	0.9
Cyclone filter	0.95

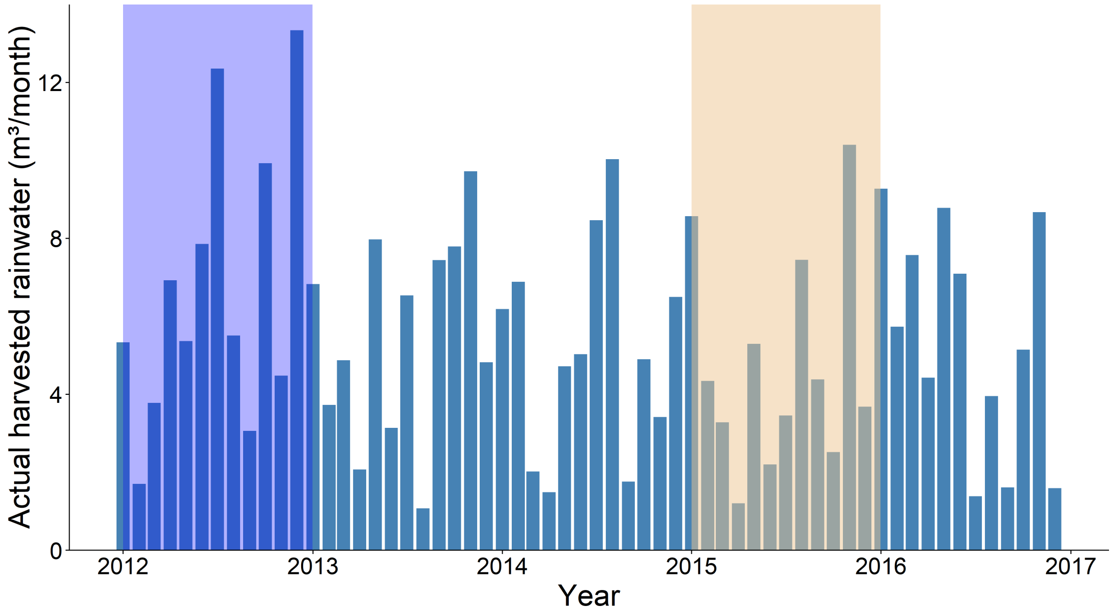

Figure 4.2: Total harvested rainwater per month. Highlighted in blue: The relatively wet year 2012. Highlighted in beige: The relatively dry year 2015. Data obtained from Waterinfo.be (nd).

Rainwater used for potable water production was assumed to be treated by reverse osmosis (RO), ultraviolet light (UV) disinfection and a remineralisation step. After treatment, the water was assumed to be safe for potable reuse and to be stored in the potable water tank (tank PW). Potable water was considered to be recirculated over a UV disinfection unit to avoid regrowth of bacterial contamination. The water recovery in the RO treatment step was assumed to be 80% (O'Connor et al., 2016).

4.1.4 Greywater collection, storage and treatment

All wastewater production in the household, with exception of the heavily polluted toilet wastewater, was defined as greywater. All flows were characterized according to their application (see table 4.4). No losses of water were assumed to occur when a household application was used. This means that the amount of inflow equalled the outflow of an application. Cleaning and gardening both formed an exception to this rule, as there was no outflow or return to the system.

Table 4.4: Water flows defined according to the application in l/d/p. Q indicates the presence of a water flow. These flows were used as an input to an application or as an output (waste flow) of the application.

Application	**Variable**
Bath & Shower	Q_{BTSH}
Bathroom sink	Q_{BS}
Washing machine & Hand wash clothes	Q_{WM}
Dishwasher	Q_{DW}
Kitchen sink	Q_{KS}
Toilet	Q_{TL}
Cleaning	Q_{CL}

Greywater was assumed to be collected in an unaerated buffer tank and afterwards led to the membrane bioreactor (MBR), where it was treated to purified greywater. Within the unaerated buffering tank denitrification was assumed to ensure sufficient nitrogen removal. Furthermore, the greywater production of a household often exceeds its potential for reuse in the same household. This implies that in those cases where only a fraction of the produced greywater can be reused, only the less polluted greywater streams should be considered for reuse (Friedler and Hadari, 2006). Therefore, a part of the collected greywater was sent to the blackwater tank for disposal. A fraction of this greywater flow to the blackwater tank was separated and was led into the infiltration tank, as to reduce the amount of emptying of the blackwater tank necessary and increase groundwater replenishment (see section 4.1.6). Depending on whether increased rainwater reuse or increased greywater reuse was chosen, rainwater was added to the MBR in order to decrease the total contamination load. After treatment in the MBR, the purified greywater was assumed to go through a UV unit, where sufficient disinfection for reuse was assumed to occur, after which it was stored in the purified greywater tank (Friedler and Gilboa, 2010).

In both the increased greywater and rainwater reuse sub-scenarios, greywater reuse always occurred, although in different quantities (see figure 4.3). Each sub-scenario had a maximal and a minimal amount of greywater reuse, depending on how much greywater reuse was needed in the system. For the increased rainwater reuse scenario, all greywater origins except the relatively heavily contaminated kitchen sink were considered during maximal greywater reuse ($Q_{GW,max}$). During minimal greywater reuse ($Q_{GW,min}$) only the less contaminated bath, shower and bathroom sink were chosen. In the increased greywater reuse scenario, all greywater origins were continuously used with no difference between minimal or maximal greywater flow to tank PGW (see section 4.1.5).

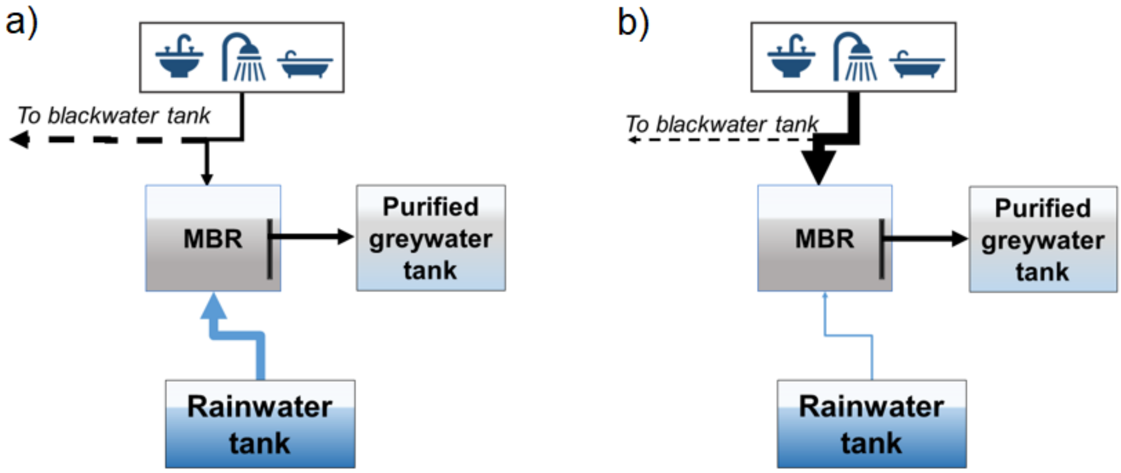

Figure 4.3: Two considered reuse sub-scenarios. In the increased rainwater reuse scenario (a), more greywater was diverted to the blackwater tank in order to benefit from the cleaner rainwater flow. In the increased greywater reuse scenario (b), a minimal amount of greywater was diverted to the blackwater tank to enable maximum greywater reuse, maximizing circularity. Here, rainwater was only supplemented to the MBR in case the greywater flow rate was too small to accommodate the necessary water supply.

4.1.5 Flows in the system

Four different types of water flows were distinguished inside the system: (i.) Greywater production by household applications, (ii.) the flow of water towards an application, (iii.) the flow of water between tanks and (iv.) the blackwater flow, which will be discussed in section 4.1.7.

Greywater production

The total greywater production ($Q_{GW,Prod}$) by household applications was given by equation 4.2. The amount of greywater reuse ($Q_{GW,Use}$) differed between increased greywater reuse (GWinc, eq. 4.3) and increased rainwater reuse (RWinc, eq. 4.4). All flows in both mentioned equations were given in l/d/p.

$$Q_{GW,Prod} = Q_{BTSH} + Q_{BS} + Q_{WM} + Q_{DW} + Q_{KS} \tag{4.2}$$

$$Q_{GW,Use} = Q_{BTSH} + Q_{WM} + Q_{DW} + Q_{CL} + Q_{TL} \quad \text{(GWinc)} \tag{4.3}$$

$$Q_{\text{GW,Use}} = Q_{\text{BTSH}} + Q_{\text{WM}} + Q_{\text{DW}} + Q_{\text{CL}} \quad \text{(RWinc)} \tag{4.4}$$

As was stated earlier, the amount of greywater treated to purified greywater (tank PGW) had a maximal and minimal flow rate. For increased greywater reuse, $Q_{GW,max}$ and $Q_{GW,min}$ were both set equal to the greywater entering the system (eq. 4.5). During increased rainwater reuse, $Q_{GW,max}$ and $Q_{GW,min}$ differed, with $Q_{GW,min}$ being equal to the relatively cleaner water flows (eq. 4.6). All flows were converted from l/d/p to m^3/month in order to be implemented.

$$\text{GWinc}: \begin{cases} Q_{\text{GW,max}} = \text{Inhabitants} \cdot \frac{365}{12 \cdot 1000} \cdot Q_{\text{GW,Prod}} \\ Q_{\text{GW,min}} = Q_{\text{GW,max}} \end{cases} \tag{4.5}$$

$$\text{RWinc}: \begin{cases} Q_{\text{GW,max}} = \text{Inhabitants} \cdot \frac{365}{12 \cdot 1000} \cdot (Q_{\text{BTSH}} + Q_{\text{BS}} + Q_{\text{WM}} + Q_{\text{DW}}) \\ Q_{\text{GW,min}} = \text{Inhabitants} \cdot \frac{365}{12 \cdot 1000} \cdot (Q_{\text{BTSH}} + Q_{\text{BS}}) \end{cases} \tag{4.6}$$

Both sub-scenarios for increased rainwater and greywater reuse led to different amounts of inflow into the blackwater tank.

Water (re)use

The water available in the different tanks (excluding the blackwater tank, BW) was used for different applications, with $Q_{PW,Use}$ being the potable water use (eq. 4.7), $Q_{RW,Use}$ the direct rainwater use (*i.e.*, not converted to potable water) (eq. 4.8) and $Q_{PGW,Use}$ the purified greywater use (eq. 4.9). Note that rainwater and greywater use varied depending on whether the toilet is flushed with rainwater or greywater.
First, assume $C = \text{Inhabitants} \cdot \frac{365}{12 \cdot 1000}$:

$$Q_{\text{PW,Use}} = C \cdot (Q_{\text{BS}} + Q_{\text{KS}}) \tag{4.7}$$

$$Q_{\text{RW,Use}} = \begin{cases} Q_{\text{RW,Garden}} + C \cdot Q_{\text{TL}} & \text{(RWinc)} \\ Q_{\text{RW,Garden}} & \text{(GWinc)} \end{cases} \tag{4.8}$$

$$Q_{\text{PGW,Use}} = \begin{cases} C \cdot (Q_{\text{BTSH}} + Q_{\text{WM}} + Q_{\text{DW}} + Q_{\text{CL}}) & \text{(RWinc)} \\ C \cdot (Q_{\text{BTSH}} + Q_{\text{WM}} + Q_{\text{DW}} + Q_{\text{CL}} + Q_{\text{TL}}) & \text{(GWinc)} \end{cases} \tag{4.9}$$

Greywater to purified greywater tank and blackwater tank

An important flow to consider was the greywater flow to the treatment unit, continuing to the purified greywater tank (tank PGW). This flow, defined as Q_{GW}, had a maximal ($Q_{GW,max}$) or a minimal ($Q_{GW,min}$) flow rate, depending on whether the increased rainwater or increased greywater reuse scenario was chosen. Both these flows have been defined earlier in this section. The selection of the maximal greywater flow versus the minimal flow happened according to eq. 4.10 in the increased rainwater reuse scenario. Here, the amount of rainwater available in tank RW was taken into account. Again, in the increased greywater reuse scenario, $Q_{GW,max}$ equals $Q_{GW,min}$.

$$Q_{\mathrm{GW}} = \begin{cases} Q_{\mathrm{GW,min}}, & \text{if RW} < \text{RW}_{\text{min}} \text{ and PGW} > \text{PGW}_{\text{min}} \\ Q_{\mathrm{GW,max}}, & \text{if RW} < \text{RW}_{\text{min}} \text{ and PGW} \leq \text{PGW}_{\text{min}} \\ Q_{\mathrm{GW,min}}, & \text{if RW} \geq \text{RW}_{\text{min}} \end{cases} \tag{4.10}$$

The remaining greywater flow ($Q_{GW,BW}$) was directed towards the blackwater tank in accordance to eq. 4.11. When possible, a part of the greywater flow ($Q_{GW,INF}$) was directed towards the infiltration facility along with excess rainwater.

$$Q_{\mathrm{GW,BW}} = Q_{\mathrm{GW,Prod}} - Q_{\mathrm{GW}} - Q_{\mathrm{GW,INF}} \tag{4.11}$$

Rainwater tank to potable water tank

Another flow considered was the flow of rainwater from the rainwater tank (RW) towards the potable water tank (PW). This water passed through a treatment step, bringing about treatment losses depending on the treatment recovery (RO_{rec}, eq. 4.12) (O'Connor et al., 2016). $Q_{RW,PW}$ is defined in eq. 4.13 and 4.14. As this flow was responsible for providing drinking water to the consumer, the supply must remain assured until the tank was empty. This was why the minimal tank volume for resisting a drought was not considered in the calculation of $Q_{RW,PW}$. Furthermore, the RO recovery was considered in the calculation of the actual flow, in order to decrease the risk of having an insufficient input in the potable water tank. Here, $Q_{RW,PW,max}$ equalled 2.5 m^3/month and $Q_{RW,PW,min}$ equalled $Q_{PW,Use}$ (see eq. 4.7).

$$\mathrm{RO_{rec}} = \frac{\text{Permeate flow rate through RO}}{\text{Feed flow rate in RO}} \tag{4.12}$$

Assume $a = \frac{Q_{\mathrm{RW,PW,min}}}{\mathrm{RO_{rec}}}$ and $b = \frac{Q_{\mathrm{RW,PW,max}}}{\mathrm{RO_{rec}}}$. If PW < $\mathrm{PW_{max}}$; then,

$$Q_{\mathrm{RW,PW}} = \begin{cases} b, & \text{if RW} > b \\ a, & \text{if RW} \leq b \text{ and RW} > a \\ \mathrm{RW}, & \text{if RW} \leq b \text{ and RW} \leq a \end{cases} \tag{4.13}$$

If PW = $\mathrm{PW_{max}}$; then,

$$Q_{\mathrm{RW,PW}} = \begin{cases} a, & \text{if RW} > a \\ \mathrm{RW}, & \text{if RW} \leq a \end{cases} \tag{4.14}$$

Rainwater tank to purified greywater tank

In this scenario, rainwater was used as a supplementary water source for greywater reuse applications next to potable water production. Therefore, rainwater was also sent from the rainwater tank (RW) to the MBR (and thus to the purified greywater tank, PGW) for combined treatment, defined as $Q_{\mathrm{RW,PGW}}$. $Q_{\mathrm{RW,PGW,max}}$ and $Q_{\mathrm{RW,PGW,min}}$ were the two flow options for $Q_{\mathrm{RW,PGW}}$, see eq. 4.15. These two flows were equal for both increased greywater reuse and increased rainwater reuse. Note that in order to obtain the relevant $Q_{\mathrm{RW,PGW}}$ flow, the minimal tank volumes for both tank RW and PGW were taken into account. Here, $Q_{\mathrm{RW,PGW,max}}$ equals 1.5 m^3/month and $Q_{\mathrm{RW,PGW,min}}$ equals 0 m^3/month.

$$Q_{\mathrm{RW,PGW}} = \begin{cases} Q_{\mathrm{RW,PGW,min}}, & \text{if RW} < \mathrm{RW_{min}} \\ Q_{\mathrm{RW,PGW,min}}, & \text{if RW} \geq \mathrm{RW_{min}} \text{ and PGW} > \mathrm{PGW_{min}} \\ Q_{\mathrm{RW,PGW,max}}, & \text{if RW} \geq \mathrm{RW_{min}} \text{ and PGW} \leq \mathrm{PGW_{min}} \end{cases} \tag{4.15}$$

4.1.6 Infiltration

According to Flemish environmental regulations, rainwater should first be considered for reuse and then for infiltration (Vlaamse Regering, 2016). Therefore, only surplus rainwater was sent to the infiltration tank along with a fraction of greywater, diverted from the greywater flow heading to the blackwater tank. The assumption here was that greywater can be infiltrated when diluted with enough rainwater, such that the mix remains below legal discharge limits (see section 2.4). Here, the fraction of greywater allowed to infiltrate was determined on the basis of the Chemical Oxygen Demand (COD) concentration that was tolerated during infiltration. This COD concentration boundary was set at 60 mg/l and was based on the relatively stringent French irrigation standards (with respect to other European irrigation standards) due to an absence of infiltration quality standards for the COD concentration (Drewes et al., 2017). The volume of greywater allowed to be diverted to the infiltration tank ($Q_{\mathrm{GW,INF}}$) was calculated according to equation 4.16 (with $Q_{\mathrm{TOT,INF}} = Q_{\mathrm{RW,INF}} + Q_{\mathrm{GW,INF}}$),

$$[\mathrm{COD}]_{\mathrm{TOT}} = \frac{Q_{\mathrm{RW,INF}}}{Q_{\mathrm{TOT,INF}}} \cdot [\mathrm{COD}]_{\mathrm{RW}} + \frac{Q_{\mathrm{GW,INF}}}{Q_{\mathrm{TOT,INF}}} \cdot [\mathrm{COD}]_{\mathrm{GW}} ,$$

$$\Leftrightarrow Q_{\mathrm{GW,INF}} = Q_{\mathrm{RW,INF}} \cdot \frac{[\mathrm{COD}]_{\mathrm{TOT}} - [\mathrm{COD}]_{\mathrm{RW}}}{[\mathrm{COD}]_{\mathrm{GW}} - [\mathrm{COD}]_{\mathrm{TOT}}} . \tag{4.16}$$

Here, $[\mathrm{COD}]_{\mathrm{TOT}}$ and $Q_{\mathrm{TOT,INF}}$ were the allowed COD concentration for infiltration and the total flow being infiltrated respectively. $Q_{\mathrm{RW,INF}}$ was the flow of rainwater being infiltrated and $[\mathrm{COD}]_{\mathrm{GW}}$ and $[\mathrm{COD}]_{\mathrm{RW}}$ were the COD concentrations of greywater and rainwater respectively. Depending on the height of the groundwater table, an infiltration tube (for a high groundwater table) or well (for a low groundwater table) are often considered (CIW, 2018). The infiltration tube, placed near the ground surface, has a relatively small radius (r_{tube}), unlike the much larger infiltration well. It is custom to only consider half of the tube cross-section and the entire sidewall of an infiltration well (VLARIO, 2017). The necessary infiltration surface was calculated with equation 4.17, using $Q_{\mathrm{TOT,INF}}$ in l/h and a safety factor (Sf) of 2.

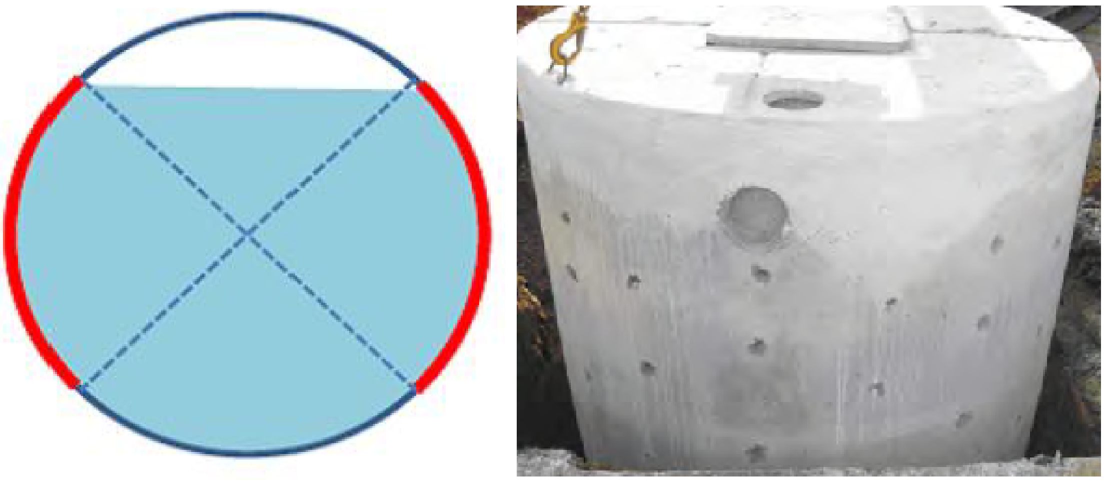

Figure 4.4: Left: The infiltration surface (in red) of an infiltration tube, cross-section. Right: An infiltration well, made of perforated concrete (VLARIO, 2017)

$$S_{\mathrm{INF}} = \frac{Q_{\mathrm{TOT,INF}}}{K_{\mathrm{sat}}/Sf} \tag{4.17}$$

It is important to note that climate change, bringing about increased precipitation intensity, will have a large effect on the infiltration capacity. A larger precipitation intensity will lead to more rainwater diversion to infiltration at the same time. The infiltration capacity must thus be well designed to prevent flooding (Wolfs et al., 2018).

4.1.7 Change in tank water volume

Four tanks were considered in this scenario: The potable water tank (PW), the rainwater tank (RW), the purified greywater tank (PGW) and the blackwater tank (BW). Per month *i*, the tank volume was calculated as the sum of the previous tank volume at *i-1* with all flows entering and leaving the tank during that month. In these calculations, the volume of water in the tank might not exceed the maximal tank volume or go below zero. All tanks were assumed to be completely filled in the onset of the simulation (at i = 0). Flows were designed in such a way that the maximal tank volumes were not exceeded.

Potable water tank

Assume $\theta(\mathrm{PW}_{i-1}) = \mathrm{PW}_{i-1} - Q_{\mathrm{PW,Use}} + \mathrm{RO}_{\mathrm{rec}} \cdot Q_{\mathrm{RW,PW}}$; then,

$$\mathrm{PW}_i = \begin{cases} \mathrm{PW}_{\mathrm{max}} & \text{if } \theta(\mathrm{PW}_{i-1}) \geq \mathrm{PW}_{\mathrm{max}}, \\ 0 & \text{if } \theta(\mathrm{PW}_{i-1}) \leq 0, \\ \theta(\mathrm{PW}_{i-1}) & \text{otherwise.} \end{cases} \tag{4.18}$$

Rainwater tank

Assume $\theta(\mathrm{RW}_{i-1}) = \mathrm{RW}_{i-1} - Q_{\mathrm{RW,Use}} + \mathrm{RAIN} - Q_{\mathrm{RW,PW}} - Q_{\mathrm{RW,PGW}}$; then,

$$\mathrm{RW}_i = \begin{cases} \mathrm{RW}_{\mathrm{max}} & \text{if } \theta(\mathrm{RW}_{i-1}) \geq \mathrm{RW}_{\mathrm{max}}, \\ 0 & \text{if } \theta(\mathrm{RW}_{i-1}) \leq 0, \\ \theta(\mathrm{RW}_{i-1}) & \text{otherwise.} \end{cases} \tag{4.19}$$

Purified greywater tank

$$\mathrm{MBR}_{\mathrm{rec}} = \frac{\text{Permeate flow rate through MBR}}{\text{Feed flow rate in MBR}} \tag{4.20}$$

Assume $\theta(\mathrm{PGW}_{i-1}) = \mathrm{PGW}_{i-1} - Q_{\mathrm{PGW,Use}} + \mathrm{MBR}_{\mathrm{rec}} \cdot (Q_{\mathrm{GW}} + Q_{\mathrm{RW,PGW}})$; then,

$$\mathrm{PGW}_i = \begin{cases} \mathrm{PGW}_{max} & \text{if } \theta(\mathrm{PGW}_{i-1}) \geq \mathrm{PGW}_{max}, \\ 0 & \text{if } \theta(\mathrm{PGW}_{i-1}) \leq 0, \\ \theta(\mathrm{PGW}_{i-1}) & \text{otherwise.} \end{cases} \tag{4.21}$$

Blackwater tank

The input into the blackwater tank consisted of toilet water, treatment residues and excess greywater which was not infiltrated (eq. 4.22). As no blackwater treatment was considered in this scenario, the blackwater tank must be emptied when filled to its maximum. The maximal volume for the blackwater tank was set at 20 m^3. Whenever the blackwater tank would become full in month *i*, the tank was assumed to be emptied in month *i-1*. This was implemented as follows:

Assume $\theta(\mathrm{BW}_{i-1}) = \mathrm{BW}_{i-1} + Q_{\mathrm{GW,BW}} + \mathrm{Inhabitants} \cdot \frac{365}{12 \cdot 1000} \cdot Q_{\mathrm{TL}} + (1 - \mathrm{MBR}_{\mathrm{rec}}) \cdot (Q_{\mathrm{GW}} + Q_{\mathrm{RW,PGW}}) + (1 - \mathrm{RO}_{\mathrm{rec}}) \cdot Q_{\mathrm{RW,PW}}$; then,

$$\mathrm{BW}_i = \begin{cases} \theta(\mathrm{BW}_{i-1}), & \text{if } \theta(\mathrm{BW}_{i-1}) \leq \mathrm{BW}_{max}, \\ \theta(\mathrm{BW}_{i-1}) - \mathrm{BW}_{i-1}, & \text{otherwise.} \end{cases} \tag{4.22}$$

4.1.8 Results and discussion

Tank volume variation

During simulation, the variation in tank volumes was calculated and can be seen in figure 4.5 for the increased greywater reuse scenario (GWinc) and in figure 4.6 for the increased rainwater reuse scenario (RWinc). In GWinc, the tank volumes fluctuated less compared to RWinc. This could be attributed to the smaller volume of rainwater being used and therefore to a larger amount of rainwater available to replenish the other tanks. A lack of rainfall resulted in an empty rainwater tank can lead to water shortage. The blackwater tank volume is discussed further in this section.

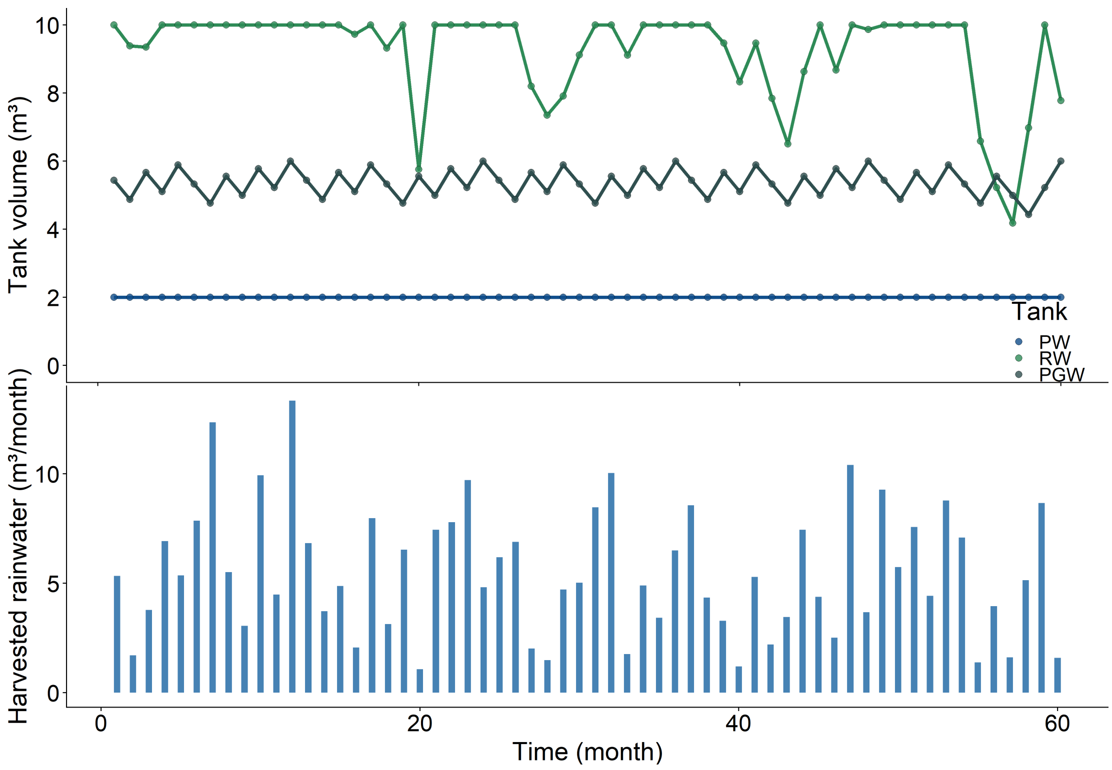

Figure 4.5: Variation in tank volume (above) and precipitation (below) over the 5 year period for the increased greywater reuse scenario.

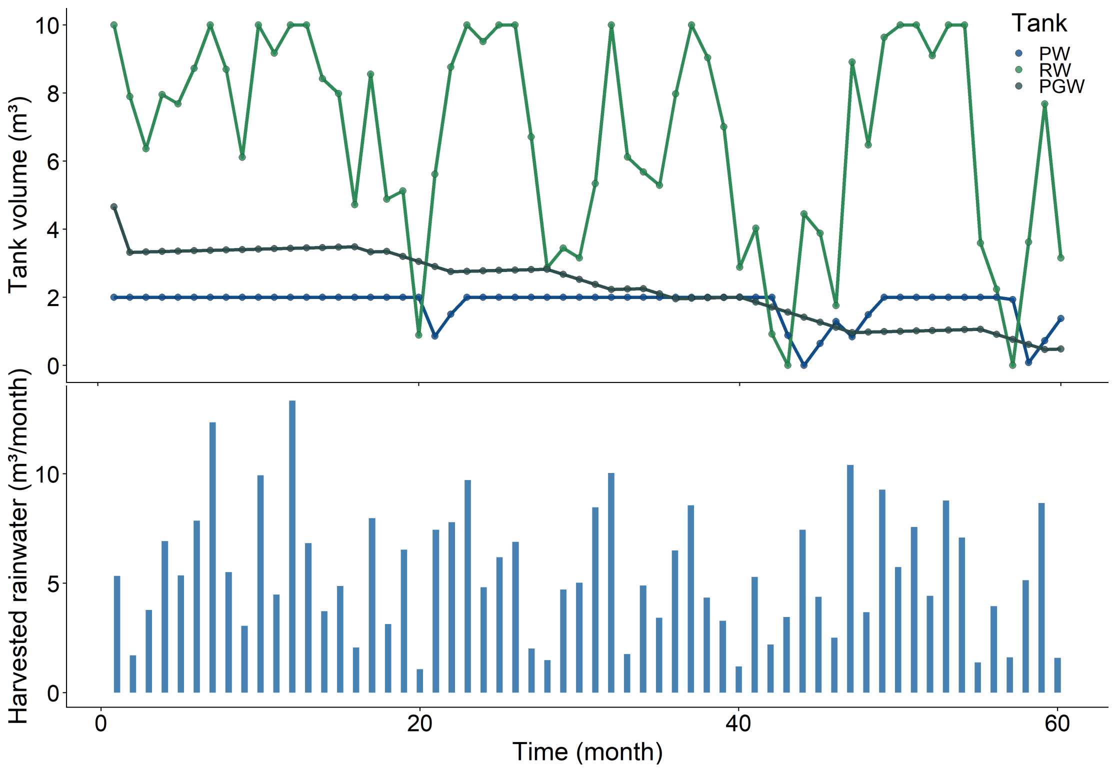

Figure 4.6: Variation in tank volume (above) and precipitation (below) over the 5 year period for the increased rainwater reuse scenario.

Mean time between failures

In order to obtain realistic tank sizes and flow rates, a certain failure rate must be considered. Failure here implied that a tank ran dry. The Mean Time Between Failures (MTBF) is a measure for the average time span between two occurrences of tank failure. One can work towards a certain MTBF-goal (*e.g.*, aiming for a MTBF of 60 months). For the current set of parameter values, the MTBF-values for both RWinc and GWinc can be found in table 4.5. As the supply of greywater towards tank PGW was relatively constant, tank PGW did not empty completely. For tank PW and RW, failure only occurred in RWinc and with a relatively long interval of 2.5 years for tank RW and 5 years for tank PW. In practice, tank failure can be prevented by importing water into the system through a refill. Emptying of tank PW and PGW could be considered unacceptable, as household applications using these water streams would no longer be able to function. Allowing the rainwater tank to be refilled when empty could be a solution to this problem, as this would avoid any tank from running dry (see section 4.2.1).

Table 4.5: Mean time between failures for operational parameters as defined in previous sections. -: No tank failure occurred.

Parameter	**GWinc**	**RWinc**
MTBF, Tank PW (month)	-	60
MTBF, Tank RW (month)	-	30
MTBF, Tank PGW (month)	-	-

Infiltration

As shown in figure 4.7, a strong difference in infiltration necessity could be noticed between GWinc and RWinc. Depending on the soil type, the infiltration capacity differed, resulting in different infiltration tank sizes or infiltration tube lengths (table 4.6, calculated in accordance to section 4.1.6). For GWinc, where the most rainwater required infiltration, combining the surplus rainwater and diverged greywater to be infiltrated, a maximal infiltration need ($Q_{TOT,INF}$) of 9.5 m^3/month was found. First, the infiltration capacity (K_{sat}, mm/h) of the soil was assessed. As this is very much dependent on the location, three soil types were considered: (i) Sand, (ii) loamy sand (typical for the Ghent region) and (iii) loam (Databank Ondergrond Vlaanderen, nd).

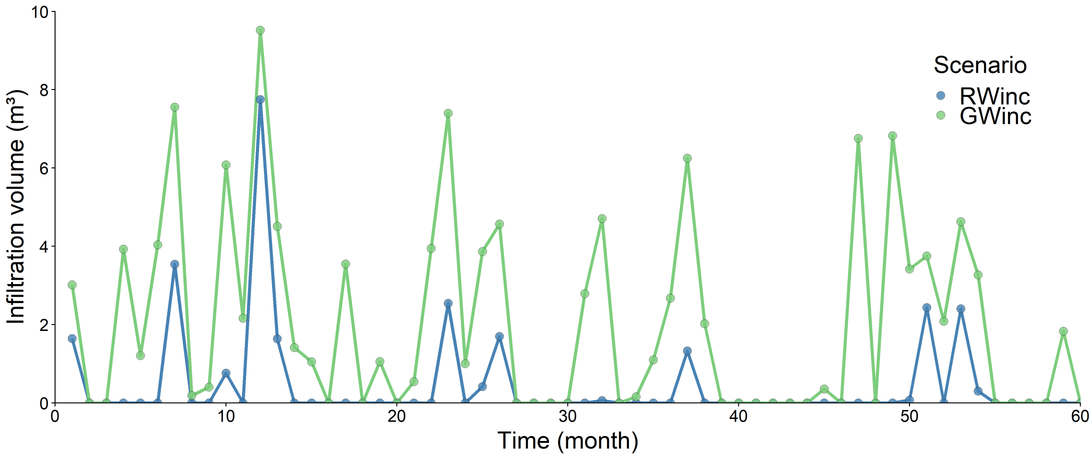

Figure 4.7: Volume of water being infiltrated for both GWinc and RWinc. Majority exists of rainwater with a fraction of greywater being added.

Table 4.6: Necessary length of tube and depth of well for infiltration per ground type, depending on the necessary infiltration surface. Infiltration capacity data and calculation method adopted from CIW (2018).

Parameter	**Sand**	**Loamy sand**	**Loam**
K_{sat} (mm/h)	74	13.64	5.69
$Q_{TOT,INF}$ (m^3/month)	9.5	9.5	9.5
$Q_{TOT,INF}$ (l/h)	26.03	26.03	26.03
Safety factor	2	2	2
S_{INF} (m^2)	0.703	3.816	9.148
Infiltration tube			
(High groundwater table)			
r_{tube} (m)	0.15	0.15	0.15
V_{INF} (m^3)	0.106	0.572	1.372
Length (m)	**1.493**	**8.099**	**19.414**
Infiltration well			
(Low groundwater table)			
r_{well} (m)	0.5	0.5	0.5
V_{INF} (m^3)	0.176	0.954	2.287
Depth (m)	**0.224**	**1.215**	**2.912**

Blackwater tank volume

The blackwater tank received a relatively high amount of wastewater, as defined in section 4.1.7. Figure 4.8 shows how many times the blackwater tank must be emptied in the 5 year period. In this scenario analysis, the increased greywater reuse scenario required the tank to be emptied 8 times, the increased rainwater reuse scenario 14 times. Emptying could therefore form a significant share of the total costs, apart from the infrastructural cost of the relatively large blackwater tank volume that is assumed.

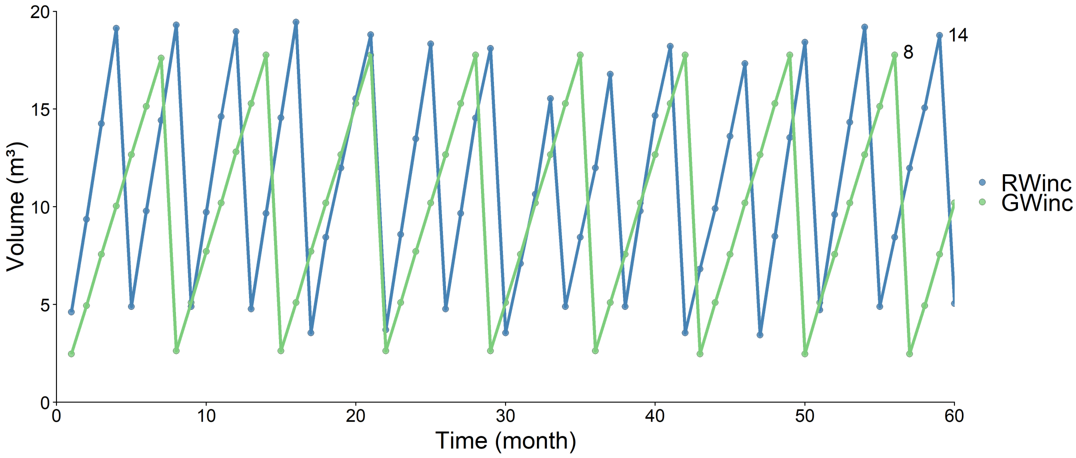

Figure 4.8: Change in blackwater volume in the blackwater tank (BW) over time. Note the difference in the amount of emptying that occurred between GWinc and RWinc (14 and 8 emptying moments respectively).

4.2 Scenario 2: Adding refill and sensitivity analysis

4.2.1 Refill of tank RW

The scenario analysis in this section was conducted with the same technological configuration as the basic scenario. An important modification to the basic scenario was that the rainwater tank (RW) would be completely refilled once empty, *e.g.* by a water delivery truck bringing water of potable water quality. This is defined by equation 4.23. Furthermore, the flows that initially were dependant on tank RW did no longer take the volume of rainwater in tank RW into account in this scenario as tank RW was refilled when empty. Flows $Q_{RW,PGW}$ and $Q_{RW,PW}$ were defined by equations 4.24 and 4.25. An important consequence of this scenario was that the volume of water in tank PGW and tank PW fluctuated much less, with no emptying of the tanks occurring. The number of refills necessary over the five year simulation period could be used as a measure for the performance of the water management system.

Assume $\theta(\mathrm{RW}_{i-1}) = \mathrm{RW}_{i-1} - Q_{\mathrm{RW,Use}} + \mathrm{RAIN} - Q_{\mathrm{RW,PW}} - Q_{\mathrm{RW,PGW}}$; then,

$$\mathrm{RW}_i = \begin{cases} \mathrm{RW}_{\mathrm{max}}, & \text{if } \theta(\mathrm{RW}_{i-1}) \geq \mathrm{RW}_{\mathrm{max}}, \\ \mathrm{RW}_{\mathrm{max}}, & \text{if } \theta(\mathrm{RW}_{i-1}) \leq 0, \\ \theta(\mathrm{RW}_{i-1}), & \text{otherwise.} \end{cases} \tag{4.23}$$

$$Q_{\mathrm{RW,PGW}} = \begin{cases} Q_{\mathrm{RW,PGW,min}}, & \text{if PGW} > \mathrm{PGW}_{\mathrm{min}}, \\ Q_{\mathrm{RW,PGW,max}}, & \text{if PGW} \leq \mathrm{PGW}_{\mathrm{min}}. \end{cases} \tag{4.24}$$

$$Q_{\mathrm{RW,PW}} = \begin{cases} \dfrac{Q_{\mathrm{RW,PW,min}}}{\mathrm{RO}_{\mathrm{rec}}}, & \text{if PW} \geq \mathrm{PW}_{\mathrm{min}}, \\ \dfrac{Q_{\mathrm{RW,PW,max}}}{\mathrm{RO}_{\mathrm{rec}}}, & \text{if PW} < \mathrm{PW}_{\mathrm{min}}. \end{cases} \tag{4.25}$$

4.2.2 Sensitivity analysis for different parameters

Building further on the conceptual modifications to the reuse system made in the previous section, the influence of different parameters on the water management system was assessed. In order to be able to compare the different simulations, a set of fixed parameter values were chosen for all simulations. The only difference with the parameter values used in scenario 1 was the enlargement of tank RW to 20 m^3.

Influence of household size

A simulation of the effect of the household size on the water management system was carried out for a five year period. As the tank volumes were not changed along with household size, the minimal tank values (PW_{min},RW_{min} and PGW_{min}) were fixed to the minimal values for 2.3 inhabitants (see section 4.1.2). $Q_{RW,PW,min}$ still equalled $Q_{PW,Use}$ but $Q_{RW,PW,max}$ was adjusted to always remain larger than the minimal value, with the same ratio $Q_{RW,PW,max}/Q_{RW,PW,min}$ as for 2.3 inhabitants. Water for garden use ($Q_{RW,Garden}$) was also fixed at that of an average household size. The scenario was simulated according to the increased greywater reuse scenario. The effect of the household size on the number of necessary refills of tank RW was given in table 4.7. Note that the minimal roof surface necessary to avoid any refills increased by 25-28 m^2 per added individual.

Table 4.7: Number of refills of the rainwater tank necessary over a 5 year period for different household sizes and the minimal roof surface necessary to avoid any refills of the rainwater tank.

Household size *(p)*	**Number of refills** *(refills)*	**Min. roof surface** *(m^2)*
1	0	28
2	0	53
3	0	78
4	1	106
5	4	133
6	7	161

Influence of the precipitation pattern

The precipitation in the Ghent area in Flanders, as used in the previous simulations, is a) relatively high throughout the year and b) has little seasonality (seasonal differences in precipitation). Precipitation patterns from three other regions were implemented in this scenario analysis with increased greywater reuse, being:

(a) Cape Town, South Africa: Figure 4.9. Here warm, dry summers and mild, wet winters occur. Cape Town is classified as a Mediterranean warm/cool summer climate (Csb) according to the Köppen climate classification. Note: water use in the garden was changed to center around January, as Cape Town lies in the southern hemisphere.

(b) New Delhi, India: Figure 4.10. Here, the climate is described as a monsoon-influenced humid subtropical climate (Cwa) bordering a hot semi-arid climate (Bsh). Large weather variations between summer and winter occur, both in precipitation and temperature.

(c) Hyderabad, Pakistan: Figure 4.11. Hyderabad experiences an arid climate (Bwh) with monsoon rains in the summer, although with much less precipitation than the monsoon rains in New Delhi.

Climate information and precipitation data obtained from Köppen (1936); TU Dresden (nd). The other parameters were equal to those used in the previous section. The rainwater use for garden applications was assumed equal for all locations.

A strong seasonal variation in precipitation occurs in Cape Town, New Delhi and Hyderabad. This leads to an increased stress on the rainwater tank in the dry period, when compared to Ghent. For both Cape Town and New Delhi, one refill of the rainwater tank over the five year period was necessary. For Hyderabad, 7 refills were necessary. This showed that rainwater harvesting in arid climates is not sufficient to sustain independence from the potable water grid.

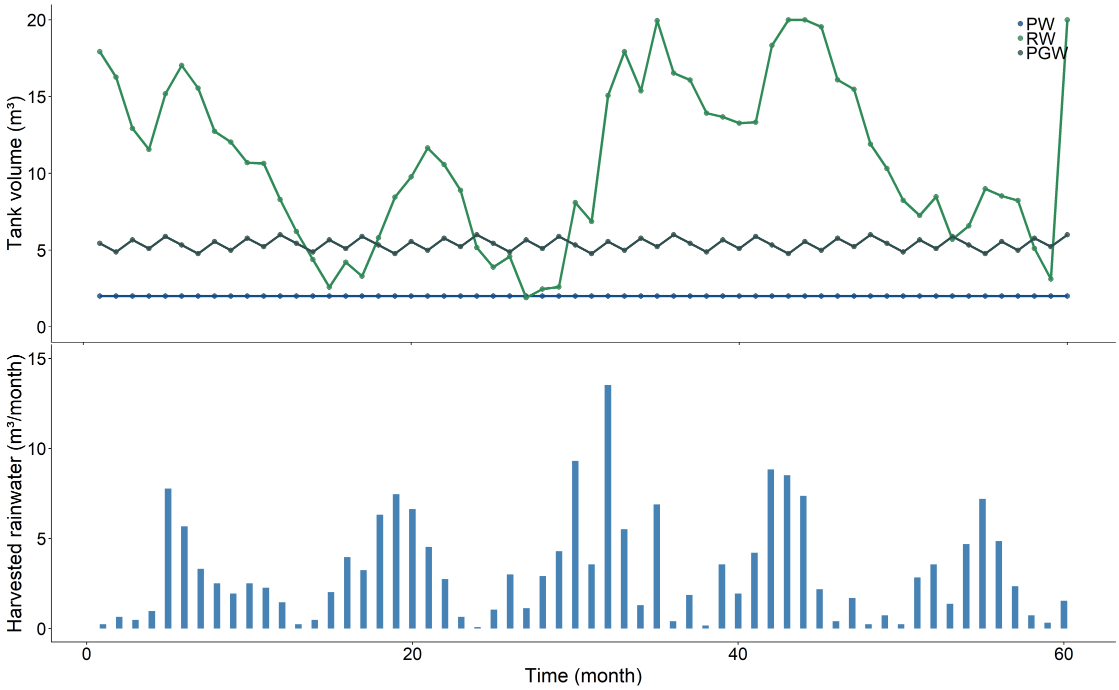

Figure 4.9: **Cape Town, South Africa**. Harvested rainwater and variation in tank volumes in a water management system placed in Cape Town. One refill occurred at the end. Harvested rainwater obtained through rainfall data for 2010,2012-2014,2016 (some years were missing as a result of incomplete data sets).

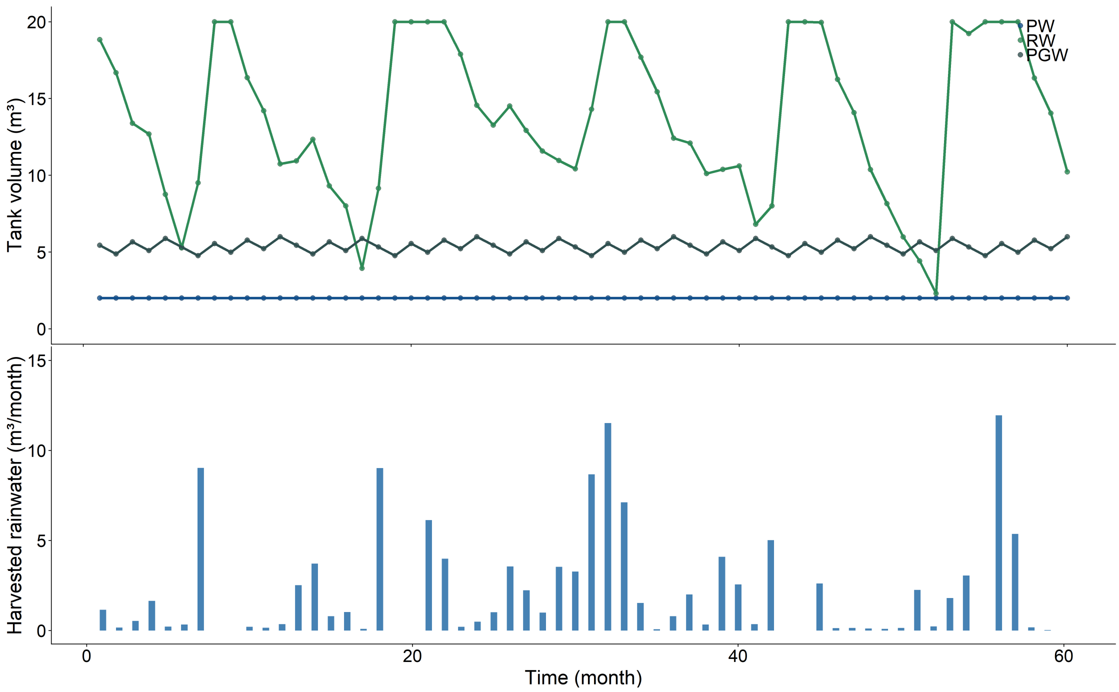

Figure 4.10: **New Delhi, India**. Harvested rainwater and variation in tank volumes in a water management system placed in New Delhi. One refill occurred towards the end. Harvested rainwater obtained through rainfall data for 2012-2017.

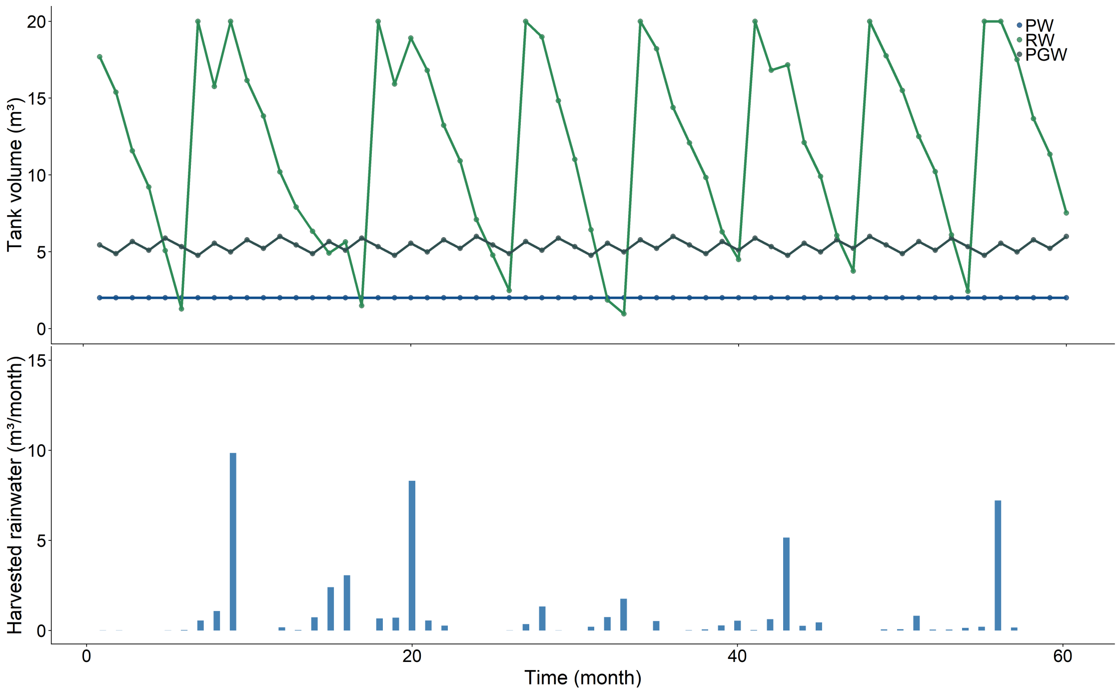

Figure 4.11: **Hyderabad, Pakistan**. Harvested rainwater and variation in tank volumes in a water management system placed in Hyderabad. 7 refills were needed. Harvested rainwater obtained through rainfall data for 2012-2017.

4.3 Scenario 3: Introducing a purge in the system

In the previous scenarios, rainwater was added to the MBR in order to decrease the contaminant load entering the treatment step towards purified greywater. However, the resulting fluctuations of the incoming load might decrease the efficiency of the biological treatment occurring in the MBR (Larsen et al., 2013). Another technological option could be to add rainwater that has passed the RO treatment (potable water) directly to the purified greywater tank (see figure 4.12). This was practically implemented by adding a flow from the potable water tank to the treated greywater tank. In the simulation, the only modification necessary was that the amount of rainwater entering the purified greywater tank now depended on the recovery of the reverse osmosis treatment ($\mathrm{RO}_{\mathrm{rec}}$) instead of the recovery of the MBR treatment step ($\mathrm{MBR}_{\mathrm{rec}}$) (eq. 4.26). Only the increased greywater reuse scenario was still considered here.

Assume $\theta(\mathrm{PGW}_{i-1}) = \mathrm{PGW}_{i-1} - Q_{\mathrm{PGW,Use}} + \mathrm{MBR}_{\mathrm{rec}} \cdot Q_{\mathrm{GW}} + \mathrm{RO}_{\mathrm{rec}} \cdot Q_{\mathrm{RW,PGW}}$; then,

$$\mathrm{PGW}_i = \begin{cases} \mathrm{PGW}_{max} & \text{if } \theta(\mathrm{PGW}_{i-1}) \geq \mathrm{PGW}_{max}, \\ 0 & \text{if } \theta(\mathrm{PGW}_{i-1}) \leq 0, \\ \theta(\mathrm{PGW}_{i-1}) & \text{otherwise.} \end{cases} \tag{4.26}$$

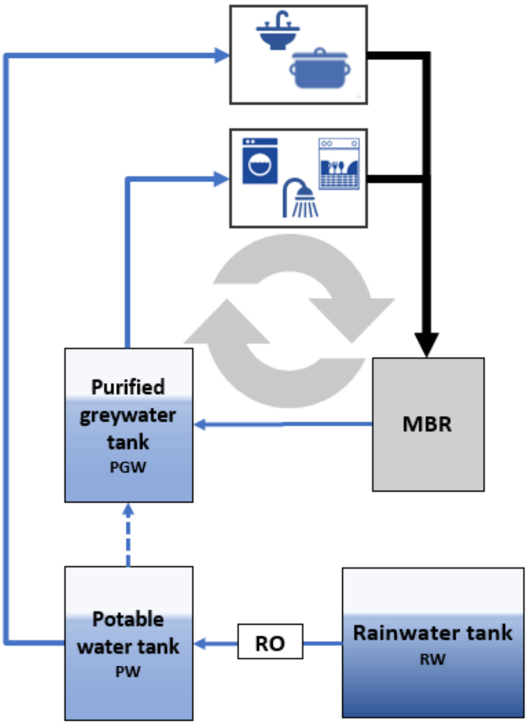

Figure 4.12: Here, potable water was added directly from tank PW to tank PGW instead of sending rainwater from tank RW to the MBR. This would lead to less variation of the incoming load into the MBR. The circularity that exists within the greywater reuse compartment is also illustrated here (this will be discussed further in this section).

As the recovery of RO treatment was assumed to be lower than that of the MBR, a slightly larger treatment residual flow entered the blackwater tank. However, no large differences regarding the flows in the system occurred compared to the previous configuration.

4.3.1 Contaminant accumulation in circular water management systems

As illustrated in figure 4.12, a circular flow existed within the system boundaries. The MBR could not fully remove all contamination from the greywater flow, which could possibly lead to accumulation of contaminants within the system. In order to investigate this, the water flows and added contamination per application and over time were implemented into MATLAB and Simulink (MATLAB 9.5, Simulink 9.2, The MathWorks, Inc., Natick, Massachusetts, United States), along with contaminant removal efficiencies per treatment step. This software was used to simulate the accumulation of contaminants taking place over the five year period. The objective was to simulate this accumulation of contaminants in the purified greywater tank (PGW), which was equal to the concentration of contaminants in the water flows used in the different purified greywater reuse applications. The MATLAB code of the first simulation and all Simulink block models are given in appendix B.

First, COD accumulation in the system was assessed. The COD concentration in the effluent of every household application, obtained from Friedler (2004) (table 4.8), was implemented in the simulation. Two flows entered tank PGW: The flows originating from applications that used potable water (Q_{BS} and Q_{KS}) and the flows originating from applications that used purified greywater (Q_{BTSH}, Q_{WM} and Q_{DW}). Only the latter contributed to the accumulation of contaminants. The following constants were introduced here:

- $Q_{PGW,Use}$, defined in section 4.1.5.
- fractionUse, which was the fraction of $Q_{PGW,Use}$ that was used in applications which produced greywater, that would again be reused (eq. 4.27). Hence, the fraction of applications where accumulation of contaminants could occur.
- $[COD]_{RW}$, the COD in rainwater in the rainwater tank. COD concentrations in the different greywater flows are shown in the same manner.
- COD1, the COD concentration in the waste flow of applications which used purified greywater as input (eq. 4.28).
- COD2, the COD concentration in the waste flow of applications which used potable water as input (eq. 4.29).

$$\text{fractionUse} = \frac{Q_{BTSH} + Q_{WM} + Q_{DW}}{Q_{BTSH} + Q_{WM} + Q_{DW} + Q_{TL} + Q_{CL}} \tag{4.27}$$

$$COD1 = \frac{Q_{BTSH} \cdot [COD]_{BTSH}}{Q_{BTSH} + Q_{WM} + Q_{DW}} + \frac{Q_{WM} \cdot [COD]_{WM}}{Q_{BTSH} + Q_{WM} + Q_{DW}} + \frac{Q_{DW} \cdot [COD]_{DW}}{Q_{BTSH} + Q_{WM} + Q_{DW}} \tag{4.28}$$

$$COD2 = \frac{Q_{BS} \cdot [COD]_{BS}}{Q_{BS} + Q_{KS}} + \frac{Q_{KS} \cdot [COD]_{KS}}{Q_{BS} + Q_{KS}} \tag{4.29}$$

Table 4.8: Average total COD concentration in the effluent of household applications, obtained from Friedler (2004).

Household application	COD concentration (mg/l)
Bath & Shower (Q_{BTSH})	572
Bathroom sink (Q_{BS})	386
Washing machine	1339
& Hand wash clothes (Q_{WM})	
Dishwasher (Q_{DW})	1296
Kitchen sink (Q_{KS})	1340

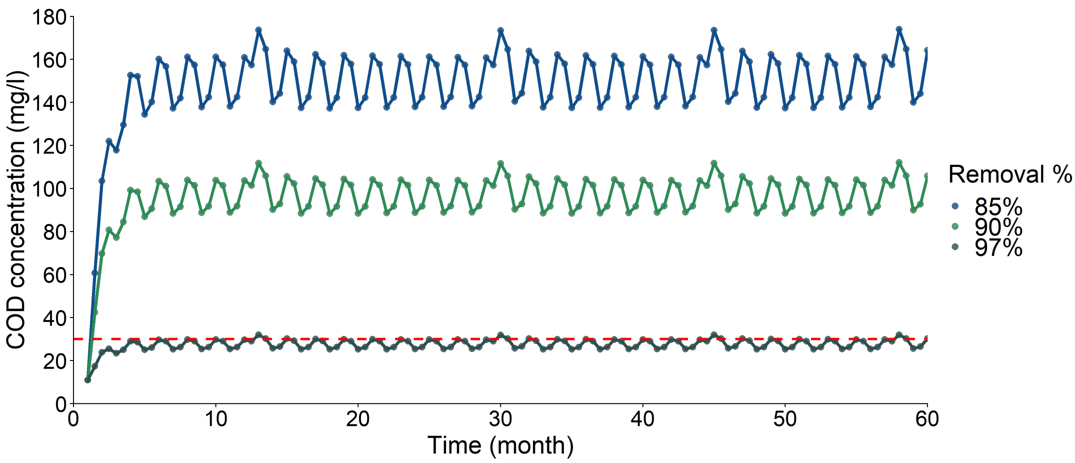

Figure 4.13: COD accumulation in the purified greywater tank (PGW) for an MBR removal efficiency of COD of 85%, 90% and 97%. The reuse standard for purified greywater was set at 30 mg/l (indicated by the red dotted line).

The obtained accumulation of COD in tank PGW for 85, 90 or 97% COD removal by the MBR can be seen in figure 4.13. An acceptable concentration of COD for non-potable reuse was assumed to be 30 mg/l (see section 2.4). The outcome of the simulation demonstrated that if 97% or more of the COD is removed during treatment of greywater to purified greywater, accumulation of COD above acceptable limits could be avoided. Such high removal efficiencies with the chosen treatment technology have been achieved in the past, although a 97% removal efficiency being described as an upper limit (Drews and Kraume, 2005; Liberman et al., 2016).

In the context of treatment with an MBR, salt accumulation in the circular system could form a considerable hazard, as practically no removal of salts occurred. Sodium, chloride and boron salts are the most common salts present in greywater (section 2.3). Figure 4.14 shows the accumulation of sodium, chloride and boron in the closed system (implemented analogously to the COD accumulation). Concentrations were implemented according to table 4.9. As can be seen, sodium, chloride and boron accumulated to concentrations exceeding the standards of respectively 200 mg/l, 250 mg/l and 1 mg/l respectively, as set in the Flemish drinking water standards (Vlaamse Regering, 2017). A possible solution to this accumulation could be the introduction of a purge (exiting flow) in the system, whereby the circularity is partly eliminated.

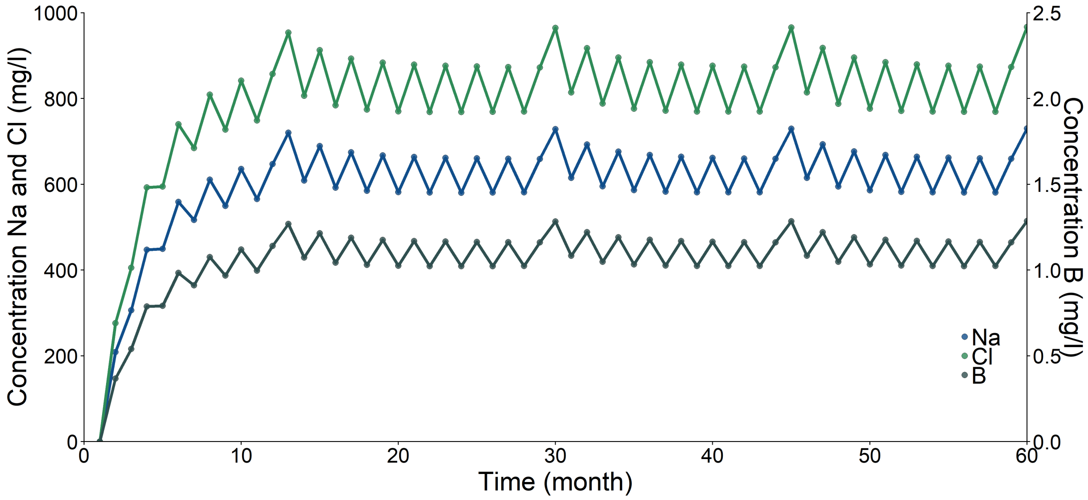

Figure 4.14: Sodium (Na), chlorine (Cl) and boron (B) accumulation in the purified greywater tank. Legal limits for these salts are 200 mg/l for sodium, 250 mg/l for chlorine and 1 mg/l for boron according to Flemish potable water standards (Vlaamse Regering, 2017). All salts exceeded the chosen standard.

4.3.2 Introducing a purge in the system

A purge of contaminated water could be implemented in the system by introducing an outgoing flow from the purified greywater tank and replacing it by water coming from the potable water tank (figure 4.15). This way, salts were removed from the purified greywater tank, preventing excessive accumulation. The outgoing water flow was led towards infiltration. Untreated greywater also no longer entered the infiltration tank ($Q_{GW,INF} = 0$). The effect of this purge on the accumulation of sodium, chlorine and boron in the system was assessed and shown in figures 4.16, 4.17 and 4.18 respectively. As can be seen, a purge of 4 m^3/month could result in tolerable sodium and chloride concentrations if the greywater contained a lower than average concentration of salts. A purge of 1 m^3/month could already lead to an acceptable boron concentration. However, compensating the purge out of tank PGW with potable water led to a considerable increase in rainwater use. When no purge was implemented, no refills of the rainwater tank were necessary. However, a purge of 1, 2, 3 or 4 m^3/month required 2, 5, 9 and 15 refills over the 5 year period respectively.

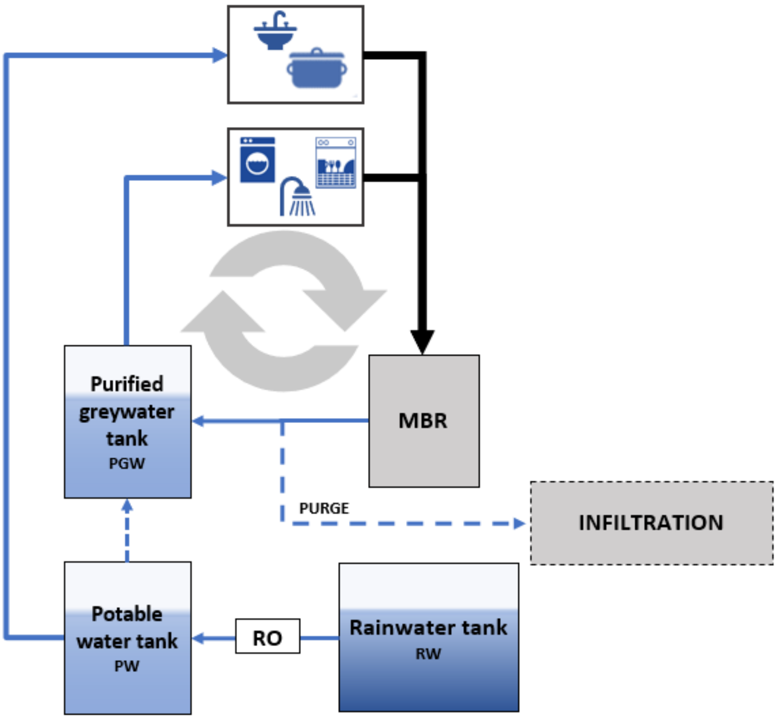

Figure 4.15: A purge was introduced in order to avoid excessive accumulation of contaminants (principally salts). A flow of purified greywater was then headed towards the infiltration tank. This efflux was fully compensated by an inflow of potable water from tank PW.

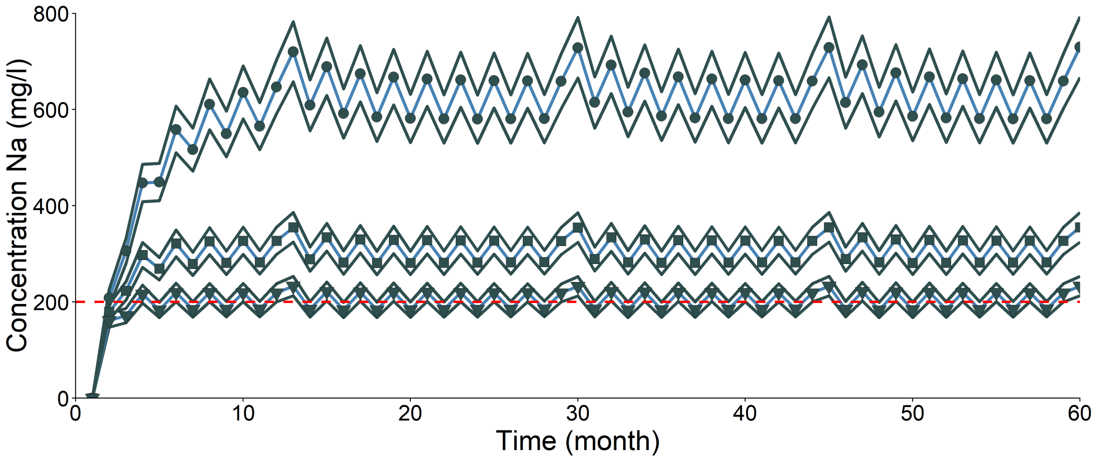

Figure 4.16: Accumulation of sodium in tank PGW over time, for a purge of 0 (•), 3 (■) and 4 (▼) m^3/month out of tank PGW. The dark green lines indicate the upper and lower boundary of Na concentrations in greywater. Red dotted line indicates the reuse standard of 200 mg/l.

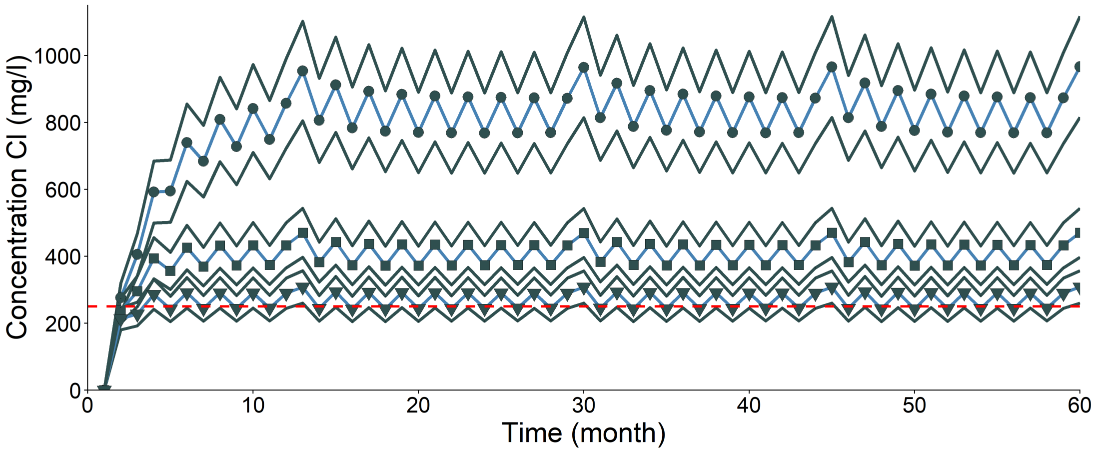

Figure 4.17: Accumulation of chloride in tank PGW over time, for a purge of 0 (•), 3 (■) and 4 (▼) m^3/month out of tank PGW. The dark green lines indicate the upper and lower boundary of Cl concentrations in greywater. Red dotted line indicates the reuse standard of 250 mg/l.

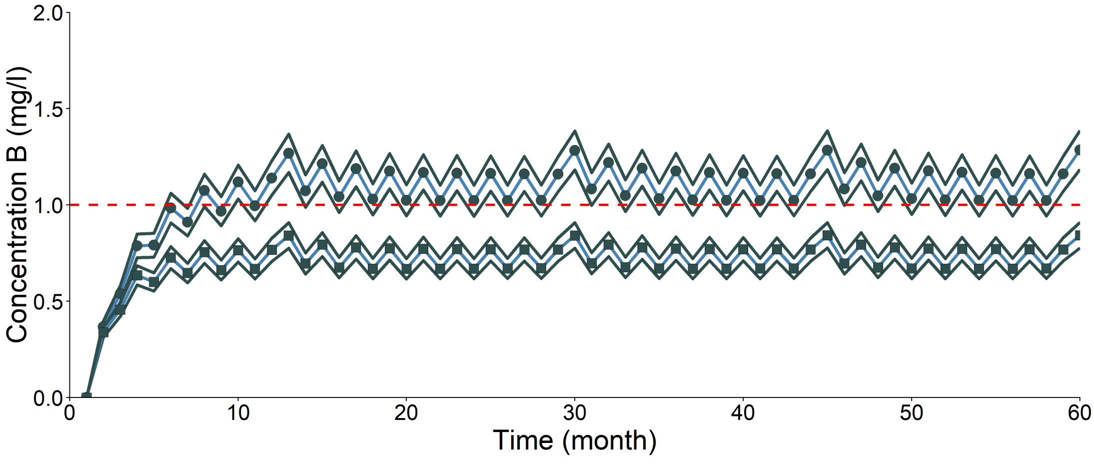

Figure 4.18: Accumulation of boron in tank PGW over time, for a purge of 0 (•) and 1 (■) m^3/month out of tank PGW. The dark green lines indicate the upper and lower boundary of B concentrations in greywater. Red dotted line indicates the reuse standard of 1 mg/l.

4.3.3 No reuse of washing machine and dishwasher wastewater

The introduction of a large purge led to an increase in rainwater use and a considerable increase in the necessary external water supply (*i.e.*, refills of the rainwater tank). Another possibility to avoid accumulation of salts was to avoid entrance of the salts in the circular system in the first place. This could be done by excluding the incoming greywater flows which contributed the most to sodium, chloride and boron concentrations to the total greywater flow. Greywater originating from the washing machine (WM) and dishwasher (DW) contain a considerable fraction of both sodium and chloride, with the dishwasher adding much to the boron concentration of greywater (see table 4.9). Therefore, flows $Q_{GW,min}$ and $Q_{GW,max}$ were modified to no longer include the flows WM and DW, which were now included in $Q_{GW,BW}$. Regarding the simulation in MATLAB, washing machine and dishwasher wastewater were removed from the fraction of purified greywater returning to the purified greywater tank after use (fractionUse) (see eq. 4.30).

$$\mathrm{fractionUse} = \frac{Q_{BTSH}}{Q_{BTSH} + Q_{WM} + Q_{DW} + Q_{TL} + Q_{CL}} \tag{4.30}$$

Table 4.9: Average concentrations and standard deviation of sodium, chloride and boron in the wastewater of household applications, obtained from Friedler (2004). In some applications, boron concentrations were below the detection limit. No standard deviation is given for WM and DW, as these applications operate in different functional stages, leading to a temporal variation in pollutant concentrations.

Household application	Sodium (mg/l)	Chloride (mg/l)	Boron (mg/l)
Bath (Q_{BT})	112±44.2	166±128	0.41±0.09
Shower (Q_{SH})	115±82.9	284±167	0.35±0.12
Bathroom sink (Q_{BS})	131±56.8	237±118	0.44±0.2
Washing machine & Hand wash clothes (Q_{WM})	530	450	0.4
Dishwasher (Q_{DW})	641	716	3.8
Kitchen sink (Q_{KS})	89±43	223±152	0.02±0.025

Figures 4.19, 4.20 and 4.21 show the effect of this system modification on the accumulation of sodium, chloride and boron respectively as they now remained under the set thresholds. Only the average chloride accumulation concentration exceeded the threshold. For this reason, ion exchange could be implemented or less products containing high concentrations of salts could be consumed. The removal of wash-

ing machine and dishwasher wastewater was compensated by an increased inflow of potable water of 1.3 m^3/month in tank PGW. This brought the number of necessary rainwater tank refills to 2 refills over the 5 year period (compare to 15 refills necessary when implementing a purge of 4 m^3/month). However, as the water quality of the greywater originating at the washing machine and dishwasher did not suffice for infiltration, the wastewater was sent to the blackwater tank. Consequently, the blackwater tank required more emptying (14, compared to 8 without purge and with reuse of washing machine and dishwasher wastewater). Instead of emptying BW this often, blackwater overflow could be sent to a different treatment step where it is treated to sufficient quality for infiltration.

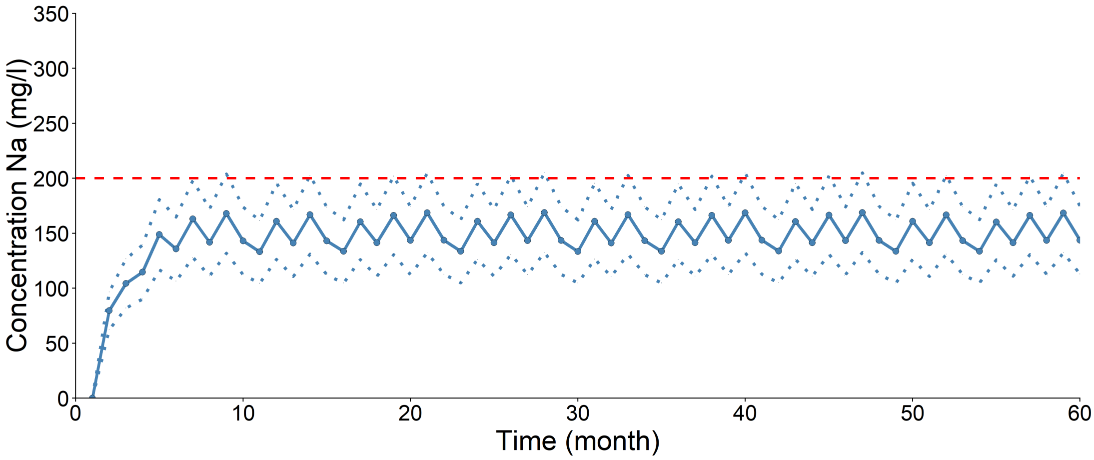

Figure 4.19: Accumulation of sodium when washing machine and dishwasher wastewater was not reused. Dotted lines show accumulation of sodium using upper and lower boundaries of sodium present in the wastewater.

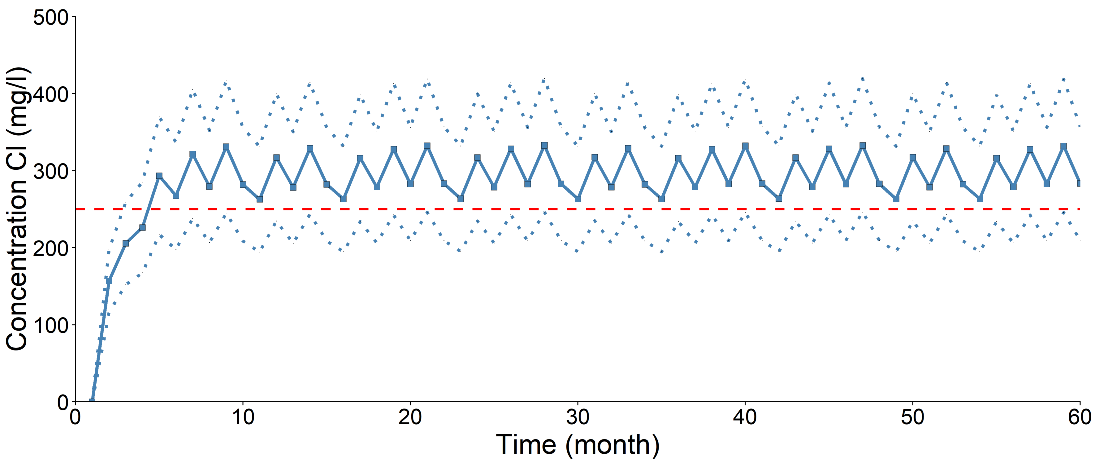

Figure 4.20: Accumulation of chloride when washing machine and dishwasher wastewater was not reused. Dotted lines show accumulation of chloride using the upper and lower boundaries of chloride present in the wastewater.

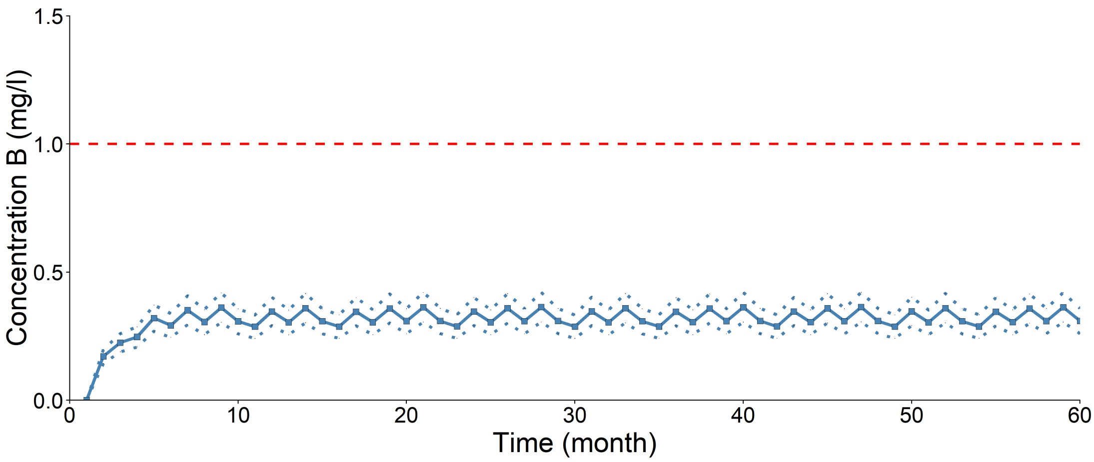

Figure 4.21: Accumulation of boron when washing machine and dishwasher wastewater was not reused. Dotted lines show accumulation of boron using the upper and lower boundaries of boron present in the wastewater.

4.4 Blackwater treatment

As can be seen in the previous scenarios, two important obstacles to implementation of water reuse systems were the necessary refills of the rainwater tank and emptying of the blackwater tank. Therefore, this scenario aimed at a) reusing blackwater, by sending the overflow of the blackwater tank towards the membrane bioreactor along with greywater and b) subsequently decreasing the amount of rainwater use necessary for refilling the greywater tank.

4.4.1 Blackwater recycling with an implemented purge

A new scenario was constructed, where tank BW remained full and was no longer considered to be emptied. The overflow was sent towards the MBR in its entirety. The volume of water flowing into tank BW equalled the outflow. Therefore, the flow $Q_{BW,PGW}$ was introduced (eq. 4.31), which equalled the flow of wastewater entering the blackwater tank. Excess purified greywater (now with addition of purified blackwater) was infiltrated together with excess rainwater. Pollutant concentrations in blackwater were adopted from Knerr et al. (2011) and Brandes (1978).

$$Q_{\mathrm{BW,PGW}} = \mathrm{TL} + (1 - \mathrm{MBR}_{\mathrm{rec}}) \cdot Q_{\mathrm{GW}} + (1 - \mathrm{RO}_{\mathrm{rec}}) \cdot (Q_{\mathrm{RW,PW}} + Q_{\mathrm{RW,PGW}}) \qquad (4.31)$$

The flow from tank RW to PGW ($Q_{RW,PGW}$), having undergone treatment by RO, was set at 0. Only when the water volume in tank PGW fell below its minimum value due to losses by the purge, would $Q_{RW,PGW}$ compensate for this and ensure the availability of purified greywater for use (eq. 4.32). The change in water volume in tank PGW was given by equation 4.33.

$$Q_{\mathrm{RW,PGW}} = \begin{cases} 0, & \text{if } \mathrm{PGW} > \mathrm{PGW}_{\mathrm{min}}, \\ \dfrac{\mathrm{PURGE}}{\mathrm{RO}_{\mathrm{rec}}}, & \text{if } \mathrm{PGW} \leq \mathrm{PGW}_{\mathrm{min}}. \end{cases} \qquad (4.32)$$

Assume $\theta(\mathrm{PGW}_{i-1}) = \mathrm{PGW}_{i-1} - \mathrm{PURGE} - Q_{\mathrm{PGW,Use}} + \mathrm{MBR}_{\mathrm{rec}} \cdot (Q_{\mathrm{GW}} + Q_{\mathrm{BW,PGW}}) + \mathrm{RO}_{\mathrm{rec}} \cdot Q_{\mathrm{RW,PGW}}$; then,

$$\mathrm{PGW}_i = \begin{cases} \mathrm{PGW}_{\mathrm{max}} & \text{if } \theta(\mathrm{PGW}_{i-1}) \geq \mathrm{PGW}_{\mathrm{max}}, \\ 0 & \text{if } \theta(\mathrm{PGW}_{i-1}) \leq 0, \\ \theta(\mathrm{PGW}_{i-1}) & \text{otherwise.} \end{cases} \qquad (4.33)$$

Furthermore, an important assumption was that a relatively long residence time of the wastewater in tank BW ensured a certain pollutant removal efficiency. This removal efficiency was considered constant over time. A COD removal in the blackwater tank of 55% and a salt removal (implying salt accumulation in the sludge) of 5% was assumed (Rahman et al., 1999; Brandes, 1978). No appropriate literature data was found to substantiate an estimate of salt removal efficiencies in the blackwater tank. Again, pollutant accumulation in the purified greywater tank was assessed.

As shown in figure 4.23, accumulation of COD in tank PGW above the set standard could only be avoided with 99% COD removal by the MBR. This removal efficiency must also be obtained when a purge (of 1-4 m^3/month) was implemented. For a purge of 1 to 4 m^3/month, respectively 0, 0, 1 and 3 refills of tank RW were needed, which was considerably less than before the introduction of blackwater reuse.

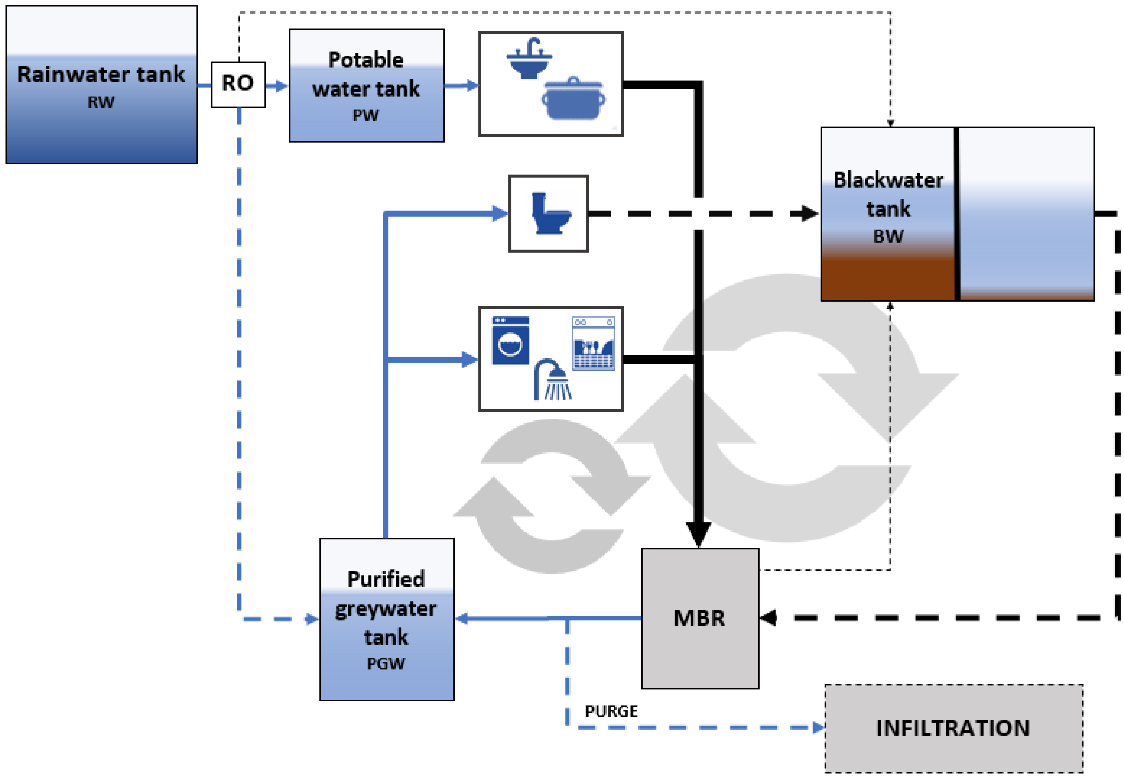

Figure 4.22: System configuration for reuse of blackwater. Flows going into tank BW equalled flow going out. Two circular configurations now existed: the circular greywater flow and the circular blackwater recirculation. The latter was simplified by considering it as a filter where a constant efficiency of pollutant removal (or fixation of contaminants in the sludge) occurred.

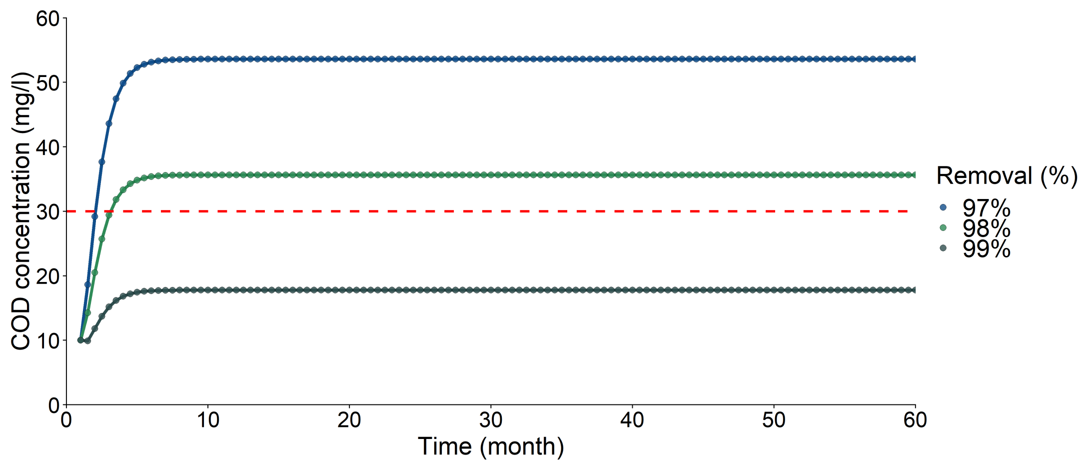

Figure 4.23: Accumulation of COD in the purified greywater tank for different COD removal efficiencies in the treatment step (MBR). Both blackwater and greywater now entered the MBR. No purge was implemented. Only a 99% removal efficiency could avoid accumulation of COD above the set standard.

It must be noted that a 99% COD removal efficiency is unlikely to be achieved, as literature does not show any instances where such high removal efficiencies have been consistently reached with MBR treatment of municipal (blackwater) wastewater flows (Drews and Kraume, 2005; Larsen et al., 2013; Knerr et al., 2011). Here, an additional treatment step or a larger purge could form the solution. When considering chlorine and sodium accumulation, no purge was sufficient to avoid accumulation without leading to a high amount of refills of the rainwater tank. The next section will therefore introduce a new configuration for purging the system of contaminants.

4.4.2 Introducing water recovery from the purge

A new configuration was simulated with water recovery from the purge by RO treatment (see figure 4.24). The configuration from the previous section was modified by adding the recovered water from the purge into tank PGW (eq. 4.34). Water still lost through the purge was compensated by purified rainwater by equation 4.35.

Assume $\theta(\mathrm{PGW}_{i-1}) = \mathrm{PGW}_{i-1} - \mathrm{PURGE} + \mathrm{RO}_{\mathrm{rec}} \cdot \mathrm{PURGE} - Q_{\mathrm{PGW,Use}} + \mathrm{MBR}_{\mathrm{rec}} \cdot (Q_{\mathrm{GW}} + Q_{\mathrm{BW,PGW}}) + \mathrm{RO}_{\mathrm{rec}} \cdot Q_{\mathrm{RW,PGW}}$; then,

$$\mathrm{PGW}_i = \begin{cases} \mathrm{PGW}_{\max} & \text{if } \theta(\mathrm{PGW}_{i-1}) \geq \mathrm{PGW}_{\max}, \\ 0 & \text{if } \theta(\mathrm{PGW}_{i-1}) \leq 0, \\ \theta(\mathrm{PGW}_{i-1}) & \text{otherwise.} \end{cases} \tag{4.34}$$

$$Q_{\mathrm{RW,PGW}} = \begin{cases} 0, & \text{if } \mathrm{PGW} > \mathrm{PGW}_{\min}, \\ \dfrac{(1 - \mathrm{RO}_{\mathrm{rec}}) \cdot \mathrm{PURGE}}{\mathrm{RO}_{\mathrm{rec}}}, & \text{if } \mathrm{PGW} \leq \mathrm{PGW}_{\min}. \end{cases} \tag{4.35}$$

As much less water from the purified greywater tank was now sent towards infiltration, less rainwater was used for compensation and a larger purge of contaminants could be implemented. For instance, for a purge of 10 m^3/month, 8 m^3/month was sent back to the purified greywater tank (assuming a RO recovery of 80%). The larger purge allowed avoiding excessive salt accumulation in the purified greywater tank, as illustrated by figures 4.25 and 4.26. Here, a purge of 6 m^3/month was sufficient to keep sodium and chloride concentrations below the set standards. COD accumulation above the reuse standard of 30 g/m^3 could be avoided by implementing a purge of 7 m^3/month, assuming a 97% COD removal rate in the MBR.

No refill of the rainwater tank was necessary up to a purge of 13 m^3/month. This absence of refills of the rainwater tank, combined with no need to empty the blackwater tank, could entail a large cost reduction for system operation. However, a second RO unit would result in additional operational and infrastructural costs. The septic

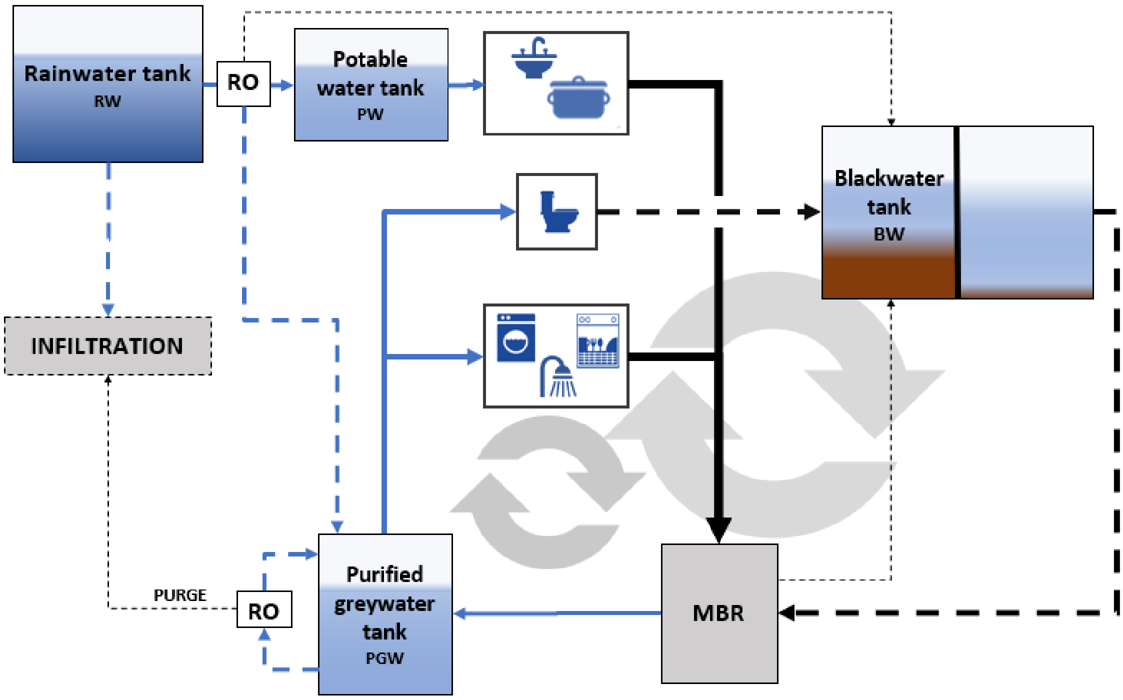

Figure 4.24: System configuration with water recuperation from the purge through a reverse osmosis unit. Less purified rainwater was now necessary to compensate for water losses through the purge. Blackwater reuse was still implemented.

pit would also require periodic cleaning to remove accumulated sludge, although this would be in terms of years rather than months (Brandes, 1978).

The contamination of the water that was infiltrated must also be considered. As no untreated wastewater flows (other than untreated rainwater) were infiltrated, the contamination entering the infiltration basin was limited in terms of BOD and suspended solids. However, due to a more concentrated purge flow, the concentration of salts entering the infiltration basin had increased. In the case of chloride with an implemented purge of 7 m^3/month, simulation showed that up to 1000 g/m^3 of chloride was being directed towards infiltration through the purge flow. As no rainwater was infiltrated during certain dry periods, chloride concentrations in the infiltration basin of up to 1000 g/m^3 were reached. This could be considered too high for infiltration, although no Flemish infiltration standards for salts exist. Therefore, the flow could be considered for further treatment as to remove pollutants from the system.

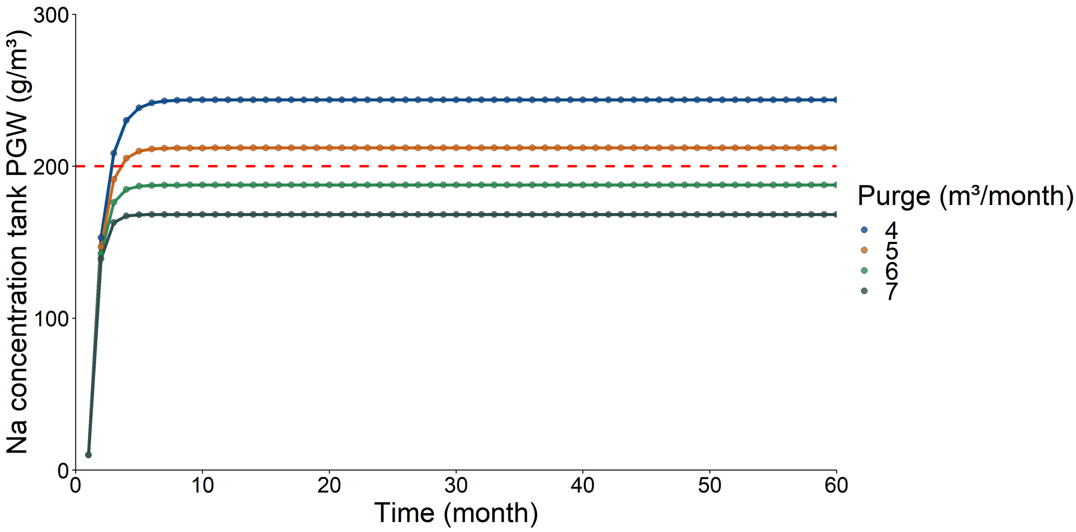

Figure 4.25: Sodium accumulation for different purge flows. As water from the purge flow was recovered, higher monthly purge flows were applicable.

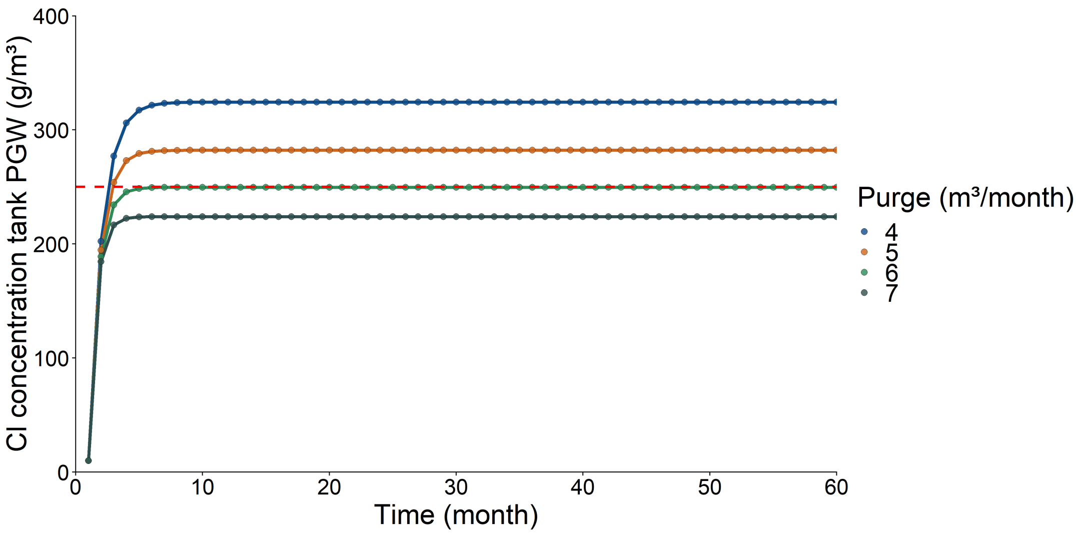

Figure 4.26: Chlorine accumulation for different purge flows. As water from the purge flow was recovered, higher monthly purge flows were applicable.

4.5 Effect of first-flush diversion

First-flush diversion can be defined as the separation of the first few millimeters of rainfall from the rest of the bulk volume. As that first volume of rainwater carries much of the contamination that accumulated on the roofing surface, first-flush diversion results in less contaminated harvested rainwater (see section 2.5.1). Therefore, first-flush diversion can be considered most interesting when high rainwater quality is desired, *e.g.* when several household applications are directly fed with rainwater or in regions with long dry periods leading to much contaminant accumulation on the roof. In order to simulate the effect of first-flush diversion on a water reuse system, the amount of rainfall being diverted must be calculated and subtracted from total harvested rainwater. Here, it was assumed that the amount of rainfall in one day equalled one rain shower on which first-flush diversion could be applied. This was considered more accurate than applying a first-flush diversion on monthly rainfall data. Daily rainfall data for the period 2012-2017 from Vinderhoute in the Ghent region of Flanders was obtained from Waterinfo.be (nd).

A first-flush diversion of 2 mm was applied, which is considered a standard amount of first-flush diversion (Abbasi and Abbasi, 2011). This was then converted from daily to monthly precipitation data and implemented in the basic scenario with refill of the rainwater tank and increased rainwater reuse. After simulation, it could be noted that the rainwater tank experienced more depletion, as the rainwater input had been reduced. One refill was now needed, compared to zero refills being necessary without first-flush diversion. This could be compensated by having a larger catchment surface (no refills necessary with 120 m^2, compared to 100 m^2 now) or with a larger rainwater tank volume (no refills necessary from RW = 13 m^3, compared to 10 m^3 now).

4.6 Climate change in Flanders

4.6.1 Effect of climate change on rainwater harvesting in Flanders

In Flanders, a recent study performed by Wolfs et al. (2018) concluded that intensification of precipitation patterns due to climate change, intertwined with longer periods of drought (in summer times) can lead to more frequent sewer overflows into surface waters and severe groundwater depletion. For decentralized water management systems, rainwater harvesting and infiltration in particular will be affected by climate change. Here, an assessment of the effect of climate change on system behaviour was performed by determining the influence of intensification of precipitation on rainwater harvesting and infiltration in Flanders.

This assessment was performed by converting the basic scenario (section 4.1) from a monthly scale to a daily scale. The simulation was ran over 31 days with an assumed average monthly precipitation of 68 mm/month. Here, precipitation intensity was defined as the spread of precipitation over a number of days, meaning that higher intensity precipitation would occur over fewer days than lower intensity precipitation. This was simulated by taking the average monthly precipitation and fitting it into a normal distribution over the total number of days of the month. Furthermore, the standard deviation of this distribution could be varied, resembling the effect of varying precipitation intensity.

All flows that were previously defined on a monthly basis were scaled to function on a daily basis. All tanks were assumed to be completely filled in the beginning of the month. No greywater was infiltrated in this scenario. Figure 4.27 shows the daily evolution of the volume of water in tank RW and the flow of rainwater towards infiltration over the period of one month. Higher precipitation intensity increased the amount of rainwater redirected towards infiltration at once, essentially leading to more water losses from the system. This could be seen in the difference in final volume of tank RW at the end of the month for the different precipitation intensities. A standard deviation of the precipitation distribution of 4.2, 1.15 and 0.66 was implemented, respectively leading to 10%, 35% and 60% of the monthly precipitation occurring in one day.

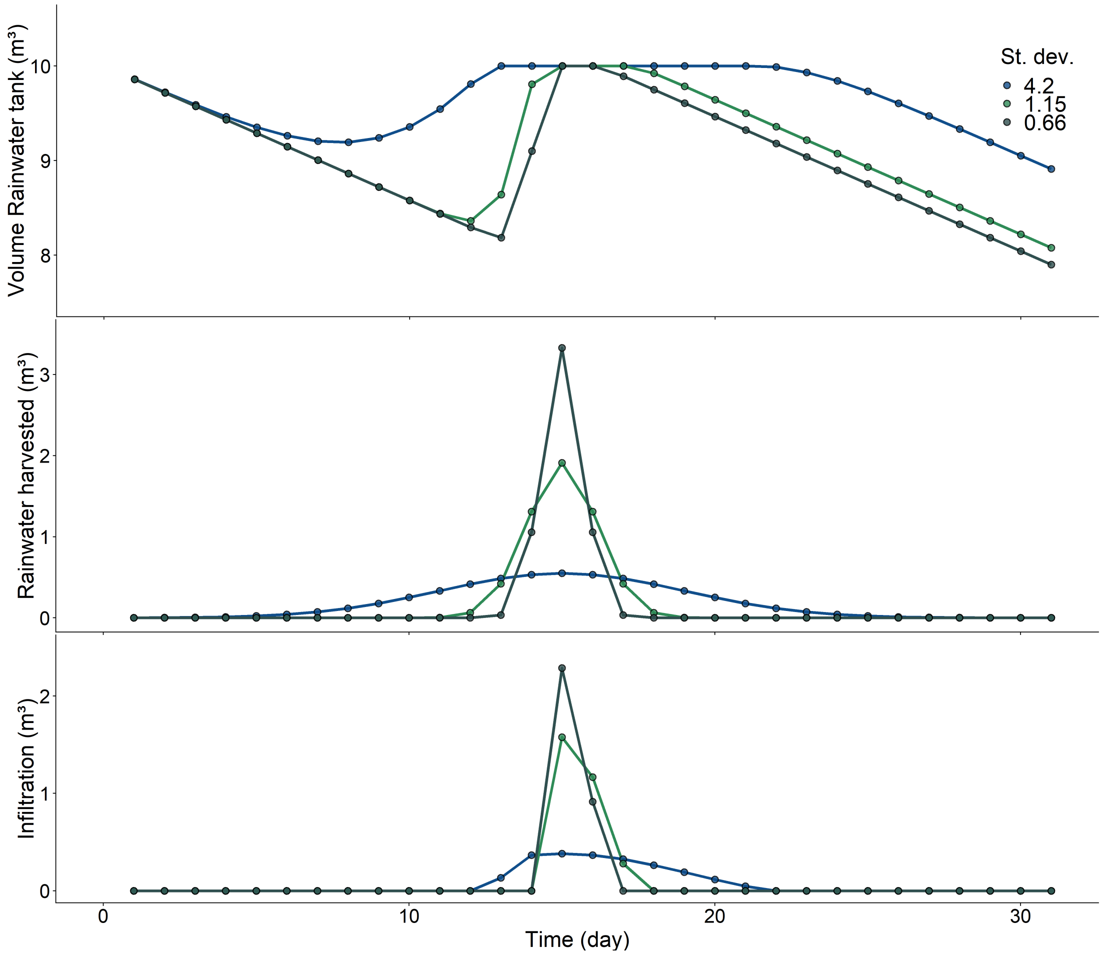

Figure 4.27: Effect of increased precipitation on rainwater in tank RW and on infiltration of excess rainwater. St. dev = The standard deviation of the distribution of rainfall over the entire month. A low standard deviation implies more rainfall at one moment.

4.6.2 The summer drought of 2018 in Flanders

The region of Flanders, Belgium experienced a heavy drought in the summer of 2018, leading to governmental action with regards to mandatory water rationing and much media attention. Figure 4.28 shows the change in tank water volumes with rainfall data from 2018 implemented into the basic scenario (see section 4.1). A rainwater tank of 20 m^3 was used as to clearly show the effect without the tank running dry. Also note the slow recovery after the drought.

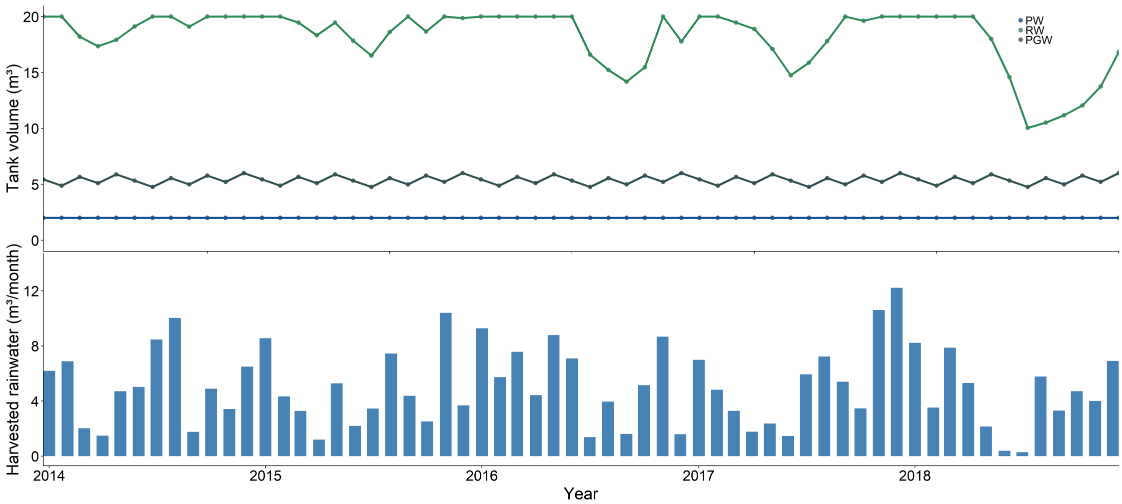

Figure 4.28: Tank water volume variation for 2014-2018 for the basic scenario. Rainfall data obtained from Waterinfo.be (nd)

CHAPTER 5

DISCUSSION

5.1 Implementing a water reuse system on household level

The fundamental water system analyzed in this dissertation is based on rainwater as a system input, blackwater (along with treatment residues) as an output and a high degree of circularity implemented within the system. The diversity of water types used in the system can be considered an addition to the resilience of the system to external disturbances, as flows can be switched between applications relatively easily. A primary objective of this dissertation was to understand to what degree such a decentralized water management system could gain independence from the grid, and what the main obstacles are with regards to a successful implementation. Simulation of the basic scenario has shown that disconnection from the potable water grid or sewer system is feasible in terms of water flows. However, independence is relative as the blackwater tank often requires emptying and as certain scenarios require the rainwater tank to be refilled. Nonetheless, the amount of emptying and refilling arguably remains within acceptable limits. This is especially the case when greywater is increasingly reused, adding to the circularity of the system.

Furthermore, the simulations of the different scenarios show that contaminant accumulation forms an important obstacle to the implementation of circular reuse systems. Here, salts form a primary concern. This goes to show that introducing increased circularity in decentralized water systems requires increased contaminant removal efficiencies when compared to linear systems, or a smart implementation of a purge out of the system. In such circular systems, one must minimize the outgoing water flow, while maximizing the outgoing contaminant flow.

Another objective of this thesis was to investigate the implementation of blackwater reuse in the system. We found that this can form a solution to rainwater depletion and excessive blackwater tank emptying. Disregarding issues with acceptance of blackwater reuse, implementation of blackwater reuse seems worth consideration in terms of aiming for independence from the grid. Furthermore, difficulties with the

high level of contamination in blackwater can again be avoided by implementing a sufficiently efficient treatment step and by a calculated purge of contaminants out of the system (which can then be infiltrated along with excess rainwater).
However, in these scenario analyses, ambiguous assumptions are made in terms of pollutant removal efficiencies (in particular salt removal efficiencies) in the blackwater tank. Despite the need for more research, adding blackwater reuse to the water system remains worth considering.

With regards to the expected climate change in Flanders, increased duration of droughts and precipitation intensity will increase stress on Flemish water management. Rainwater reuse systems will also endure additional stress due to a decreased rainwater input, depending on the amount of buffering capacity.

As has been said before, technological advancements in recent decades already allow for the development of advanced decentralized water management systems. However, before implementation of such systems on a household level can be considered, integration of the existing technologies into one functional system still requires further research. For instance, there is a particular need for developments in online monitoring tools, increasing the system safety, and preventive and predictive maintenance via machine learning in order to minimize maintenance necessities (Kazor et al., 2016).

It can be expected that decentralized water management systems will be introduced to the household market through an increasing demand for personalized water provision, whereby water can be tailored to the personal preferences of the household. At first, these systems will only be available to wealthy households. However, this is expected to change with an increasing number of early adopters: The introduction to the consumer market would cause rapid innovation of water treatment technologies, leading to increased functionality, sustainability and decreasing costs (*cfr.*, the market evolution of photovoltaic cells).

5.2 A water treatment system brought into practice

In the course of 2019, Ecopuur, a Belgian construction company, will initiate the sale and installation of commercial greywater reuse systems to the Belgian household market. These systems, with a treatment system from the brand Hydraloop, perform reuse of certain greywater flows for non-potable applications. This is one of the first instances of such technology being available for Belgian households. The Hydraloop system is comparable to the reuse systems provided by Intewa (Germany, also installed in the Netherlands by Mijn Waterfabriek), which also allow greywater reuse. The principal technology in the Hydraloop system is an aerated bioreactor including skimmer technology and bottom sludge removal. Treated water is further disinfected by UV light. As no membranes are used, the risk of clogging and necessity for chemical cleaning is minimized. The system can be operated with a smartphone app.

Ecopuur aims at applying the Hydraloop system to treat wastewater originating from bath, shower and washing machine and reuse it as feed for the toilet and washing machine. This would lead to 45% reduction of mains water usage, entailing significant cost savings: With an initial investment (including installation) of 5000 euro, the pay-back time would be around 5-7 years. Besides being relatively affordable, the low maintenance needs, attractive design and good contaminant removal efficiencies make the Hydraloop system a promising new technology.

However, some concerns may exist. The calculations for water savings do not include any present use of rainwater in the household. As an increasing amount of households in Flanders use rainwater for toilet flushing and washing machine (and even for bathing and showering) purposes, the question arises whether the Hydraloop system would remain as financially appealing as is the case now if this is taken into account. Ecopuur does aim at adding rainwater to the Hydraloop system for new construction projects, however this could entail significantly less savings. Eliminating UV disinfection of treated greywater intended for toilet use could also decrease energy costs.

A final remark could be made that in no case should the effluent from this reuse system be communicated to the general public as being potable water. For instance, a complete absence of *Escherichia coli* must be guaranteed in the effluent of the treatment system according to Flemish potable water standards. This is not the case now as there is a reported concentration of <1 colony-forming unit/ml. Therefore, the contaminant removal by biological and UV treatment is not sufficient for the water to be considered completely safe for human consumption.

5.3 Applying pressure to decentralized water systems

An important consideration when designing decentralized water systems is how to pressurize this system. No water tower is used to gravitationally provide pressure, hence pumps ought to be used. Current water systems often make use of pumps with buffer tanks, and it could be expected that the system configurations discussed in this dissertation would require many. As the system includes several membrane units (UF, RO), considerable pressure losses (up to 60 kPa per UF membranes, according to Asano et al. (2007)) must be taken into account.

When aiming for sustainability and a low energy consumption by the system, other sources of pressure may be considered as well. For one, pressure losses throughout the system may be minimized by designing airtight water tanks and treatment units. Fermentation of certain waste flows, creating biogas, could possibly also pressurize (certain parts of) the reuse system (for example through fermentation of blackwater in a Chinese dome digester).

5.4 Social acceptance of the implementation of water reuse systems

The implementation of water reuse systems can not be fully understood if the social context in which it the transition will occur is not considered (Pahl-Wostl, 2007). As rainwater harvesting and reuse has gained social acceptance over the past years, the same might occur for other wastewater flows (Campisano et al., 2017; Domènech and Saurí, 2010).

5.4.1 Acceptance of wastewater reuse

Direct wastewater reuse mostly occurs for non-potable applications such as irrigation, cleaning and industrial cooling. The extent to which water reuse is implemented depends on many factors such as necessity and opportunity, with water scarcity being an important driver (Asano et al., 2007). However, one of the largest obstacles for paradigm shifts in the water sector has always been social acceptance. For instance, the community behind source-separation of wastewater flows has been looking into ways to deal with acceptance for some decades now. Here, various projects have already shown that when dealing with sanitation, not only the quality of the treatment but also comfort and status play an important part (Larsen et al., 2013). Therefore, behavioural psychology and marketing research ought to play a crucial part in the implementation of new water management strategies. A crucial factor in gaining social acceptance of water reuse is the perceived risk for human health. This perception is very vulnerable to system failure occurrence (Domènech and Saurí, 2010).

According to a survey on water use among Flemish households in 2018, Flemish households find the idea of producing potable water themselves appealing, as long as it is technologically and financially feasible (VLAKWA, 2018).

5.4.2 Value driven innovation in the water sector

The installation and operation of decentralized wastewater treatment may cause a relatively high impact on everyday life, both on individual level and societal level. This requires acceptance by society, for which a VIN-Canvas (value-driven innovation, dutch: Waardengedreven innovatie, WIN-Canvas) may be used. The VIN-Canvas functions as a guideline for value-driven entrepreneurship, to be used especially when indirect stakeholders and indirect impacts are expected to be significant. It essentially acts as a tool for accommodating innovation to the moral values of society early on in the design process (Eynickel, 2017).

First of all, the stakeholder group must be determined, *i.e.* identifying who is directly or indirectly affected by the innovation. This stakeholder analysis is followed by an impact analysis in which different possible societal effects are evaluated and the associated impacts are determined. Finally, an examination of the moral values of the innovation is expected, for which an investigation of contributions and challenges to moral values must occur.

Identifying the stakeholders

The principal stakeholders are given in table 5.1, along with the motivation of the stakeholders to interact with the system or the impact the system has on the stakeholders.

Societal impact analysis

A crucial step in integrating morality in innovation in the water sector is by examining the impact of such an innovation on society. For a large part, these impacts are already described in section 2.1. Furthermore, it is important to also assess potential negative impacts on society and to differentiate between hard (*i.e.* measurable, direct) and soft (subtle, indirect) impacts. Table 5.2 gives a summary of potential societal impacts and the values involved.

Moral values of the innovation

The third part of the VIN-canvas involves looking for moral conflicts arising through the planned innovation. What aspects of decentralized water management could embrace or defy moral values? Table 5.3 shows an overview of values which are beneficially affected by the system and values which might challenge the implementation of the system.

Table 5.1: Direct and indirect stakeholders for water management system implementation. Also given is the motivation of the stakeholders to interact with the system. Partly adapted from Dixon (2000).

Stakeholder	Motivation
Direct	
Consumer	User of the system.
System supplier	Sells the system and might need to provide maintenance.
Installation operative (plumber)	Must be informed on basic system operation and the system installation (and/or maintenance).
Indirect	
Centralized water supplier	Provision of additional water if decentralized systems run dry. May play a part in water quality monitoring.
Sewer company	Possible collection (and treatment) of system waste flows, if any are present. Might have to deal with altered wastewater flow rates.
Household visitors	Must be informed on the origin and/or quality of the different water flows.
Regulating body	Provision of regulations on water quality, monitoring requirements and environmental impacts.

Table 5.2: Possible societal impacts of introducing decentralized water management systems on a household level, divided between hard en soft impacts. Qualification implies positive/neutral/negative (+/0/-) impacts. Also given are the values involved with the impact.

	Qualification	**Values involved**
Hard impacts		
Decreased stress on water bodies	+	Sustainability
Hacking of monitoring software by criminals	-	Safety, privacy
Changes in water infrastructure	0	
Increased resilience to climate change	+	Sustainability, safety
Increased energy use	-	Sustainability
Increased difficulty of water quality monitoring	-	Health, safety
Soft impacts		
More innovation and faster turnover of technology generations	+	Progress
Increased user awareness of consumption	+	Sustainability
Possibility to customize water quality	+	Health, quality of life
Wide range of small-scale treatment technologies lead to difficult standardization of best techniques	-	Safety
Water technologies more available to the upper class (in the beginning)	-	Social justice
Consumer in control of entire water cycle	+	Independence

Table 5.3: The impact the new water management system might have on different values, either explicitly or implicitly.

Values positively impacted by system	**Values that challenge system implementation**
Sustainability	Safety of consumer
Independence	Social justice
Health	Privacy
	Accountability

5.5 District level approach to decentralized water management

It is relevant to consider wastewater management for a collection of buildings or on a district level, as water reuse technologies are often first implemented at this scale. This is especially the case for communities living in ribbon developments, small village centers or suburban communities. Cluster- or community type decentralized water management can benefit from certain advantages of scale (with regards to, *e.g.*, the used treatment technologies), yet avoiding the necessity of placing a full-scale, expensive sewer network (Larsen et al., 2013). Furthermore, a district level approach could require only one monitoring system to monitor water quality (easier to control) and much less pumping material, leading to decreased system costs. Disadvantages of connecting these households include that underground piping networks are required and that there must be sufficient social acceptance of shared water reuse. The district could operate its reuse system(s) with shared water basins, *e.g.* placed below a shared communal area such as a park. This could function as a storage volume for rainwater or blackwater.

Recently, an increasing amount of projects are emerging in Flanders, whereby decentralization of water management and an increased sustainability is a key objective. One such ambitious project in Ghent is "De Nieuwe Dokken" (`https://denieuwedokken.be/`), a construction project of about 400 apartments with a focus on decentralizing water and energy cycles. For example, blackwater is collected through a vacuum toilet and digested. The produced energy, in combination with the waste heat available in greywater, is used for the heating of the buildings and water. Greywater is treated and reused as industrial water source for a nearby company and as cleaning water in the buildings. Furthermore, rainwater is reused for certain applications that do not require water of potable water quality.

CHAPTER 6

CONCLUSION

The main objective of this dissertation was to analyze the operation of decentralized water reuse systems, specifically on a household level and independent from any sewer system or potable water grid.

In a first part, the basic technological configuration and scenario was constructed for a system located in the Ghent region of Flanders, Belgium and the water flows in the system were simulated. The water treatment configuration was based on MBR and RO treatment technology, with separate treatment of rainwater for potable reuse applications and combined greywater–rainwater treatment for non-potable applications. Increasing greywater reuse led to less waste flow towards the blackwater tank and less emptying of the rainwater tank. It was shown that, in terms of a water mass balance, independence from the potable water grid or sewer system is feasible. Increasing the household size logically led to a larger water input in the system being necessary, which could be solved by an increased rainwater harvesting surface. It was also shown that precipitation patterns such as those in Flanders facilitate the use of rainwater as an input to water management systems, as precipitation is fairly uniformly spread over the year.

In a next section, the accumulation of contaminants in the reuse system was assessed. Here, it was concluded that adding a high degree of circularity in reuse systems increases the risk of COD and salt accumulation if the removal efficiency is too low. This can be resolved by introducing a purge in the system, where treated water is led towards infiltration, or by excluding certain waste flows of being reused. This either causes an increased depletion of the rainwater tank or an increased amount of emptying of the blackwater tank being necessary. Subsequently, blackwater reuse was investigated as a solution to both these problems. When blackwater reuse was added, a large purge was found necessary to avoid accumulation of COD and salts, which would again lead to depletion of the rainwater tank as the purge must be compensated. A final technological configuration was therefore to add an additional RO unit to recover water from this purge, leading to a more concentrated pollutant flow towards infiltration. Here, much less rainwater was necessary for purge compensation and therefore a much larger purge was possible.

Finally, the influence of first-flush diversion and intensification of precipitation due to climate change on system performance was assessed. First-flush diversion led to a decrease in available rainwater. Increased precipitation intensity combined with longer periods of drought due to climate change can lead to a rapid filling of the rainwater tank and subsequent rainwater losses due to diversion towards infiltration.

This preliminary research should be considered as a framework for technological choices and system configuration decisions for decentralized water management systems. As the technologies discussed in this dissertation all exist, these are to be integrated in a system in practice and tested in order to validate the conclusions made here. Also, further research on increasing social acceptance towards wastewater reuse, monitoring technologies and the removal of certain waste flows is still necessary. However, in the context of climate change, agricultural intensification and increasing industrial production, adding circularity on a household level to water management appears very promising.

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APPENDIX A

WATER REUSE GUIDELINES

An important obstacle in the implementation of water reuse systems is the lacking availability of, and the disparity between international water reuse guidelines (Li et al., 2009).

Table A.1: International COD and TN guidelines for water reuse applications. COD = Chemical Oxygen Demand, TN = Total Nitrogen. Adapted from Drewes et al. (2017); Bastian and Murray (2012); Alcalde Sanz and Gawlik (2017).

	COD	**TN**
Cyprus	70	15
France	60	-
Greece	-	30
Italy	100	15
Spain	-	10
Japan	30	-

Table A.2: International non-potable reuse guidelines. BOD_5 = 5-day Biological Oxygen Demand. CFU = Colony-Forming Unit, NTU = Nephelometric Turbidity Unit. *(g) = guideline, (m) = mandatory. Adapted from Leong et al. (2017).

	Faecal coliforms (CFU/100 ml)	**Total coliforms** (CFU/100 ml)	**BOD_5** (mg/l)	**Turbidity** (NTU)	**TSS** (mg/l)	**DO** (% sat.)	**pH** (-)
California		2.2		2			
China (toilet flushing) (m)*		3	10	5		1	6.0-9.0
Cyprus	50		10		10		
European Commission bathing water (m)	2000	10000		1		80-120	6.0-9.0
France	1000						
Germany (g)	100	500	20	1 - 2 (m)	30	80-120	6.0-9.0
Japan (m)	10	10	10	5			6.0-9.0
Israel		12	15		15	0.5	
Malaysia (Class IIB - recreational water) (m)	400	5000	3	50	50	5-7	6.0-9.0
Oman	200		15		15		6.0-9.0
Spain (Canary Islands)		2.2	10	2	3		6.5-8.4
Tunisia			30		30	7	6.5-8.5
UAE		100	10		10		
UK Bathing Water Criteria	2000 (m)	10,000 (m)		1		80-120	6.0-9.0
US EPA (g)	14		10	2			6.0-9.0
WHO (lawn irrigation) (m)	1000						

APPENDIX B

MATLAB SIMULATIONS

The following sections show MATLAB Simulink (by MathWorks) implementations for the different scenarios. Parameters are described in chapter 4.

B.1 Simulating COD accumulation in tank PGW

For this simulation, the MATLAB code is given. Other simulations are operated with different inputs but according to the same principles.

```
clc
close all
clear all

%%%%Defining constants
timestop = 60; %month
volPGW = 6; %m3, initial condition
NrPeople=2.3; %i.e.
conv=NrPeople*365/12/1000; %l/p/d ->m3/month
    TL=21.3*conv; %m3/month
    BTSH=28.9*conv; %m3/month
    BTSHCOD=571.765; %gCOD/m3
    BS=9.4*conv; %m3/month
    BSCOD=386; %gCOD/m3
    WM=16.6*conv; %m3/month
    WMCOD=1339; %gCOD/m3
    DW=2.3*conv; %m3/month
    DWCOD=1296; %gCOD/m3
    KS=17.1*conv; %m3/month
    KSCOD=1340; %gCOD/m3
    CL=5.8*conv; %m3/month
QPGWUse = TL+BTSH+WM+DW+CL; %m3/month (assumed constant)
fractionUse = (BTSH+WM+DW)/(BTSH+WM+DW+TL+CL); % Application fed by C, wastewater
    back to C
BSKS = BS+KS; %m3/month
COD1 = BTSH*BTSHCOD/(BTSH+WM+DW)+WM*WMCOD/(BTSH+WM+DW)+DW*DWCOD/(BTSH+WM+DW);%gCOD/
    m3BTSHWMDW
COD2 = BS*BSCOD/(BS+KS)+KS*KSCOD/(BS+KS);%gCOD/m3 BSKS
```

```
RWCOD = 23; %gCOD/m3 %Source rainwater COD: Leong, 2017
initCODtankPGW = 10; %gCOD/m3

DataGWOPT = xlsread("DataGWOPTRWnaRO.xlsx"); %Results from excel simulation
QRWPGW(:,2) = DataGWOPT(:,2);
QRWPGW(:,1)=1:60; %om input in simulink te doen
PGW(:,2)=DataGWOPT(:,1);
PGW(:,1)=1:60;
MBRrec = 0.9; %Water recovery
MBRCODremvec = [0.85,0.9,0.97]; %COD removal
ROrec = 0.8;
ROrem = 1;
norm = 30; %mgCOD/l

hold on
for i = 1:3
    MBRCODrem = MBRCODremvec(i);
    sim("Sim_OptGWRWnaRO")
    time = simout(:,1);
    concTankPGW = simout(:,4);
    plot(time,concTankPGW,'-.','LineWidth',1.8)
    DATA(:,i)=simout(:,4);
end

%PLOTTING

line([0,60],[norm,norm])
ylabel('COD concentration tank PGW [g/m3]')%,'interpreter','latex','FontSize',12)
xlabel('Time [months]')%,'interpreter','latex','FontSize',12)
hAx = gca;              % handle to current axes
set(gca,'fontsize',18)
hAx.XAxis.Exponent=0;  % don't use exponent
hAx.YAxis.Exponent=0;
hAx.XAxis.LineWidth=1.5;
hAx.YAxis.LineWidth=1.5;
hAx.XAxis.TickDirection='both';
hAx.YAxis.TickDirection='both';
hAx.XAxis.TickValues=0:10:60;
ylim([0,180])
hAx.YAxis.TickValues=0:20:160;
hAx.YAxis.FontWeight='bold';
hAx.XAxis.FontWeight='bold';

box off
legend('MBRCODrem = 0.85','MBRCODrem = 0.9','MBRCODrem = 0.97','Standard','Location'
    , 'northeast',...
    'FontSize',14)%'interpreter','latex'
hold off
```

```
T=table(time,DATA);
writetable(T,'DatasimulatieRWnaRO.xlsx');
```

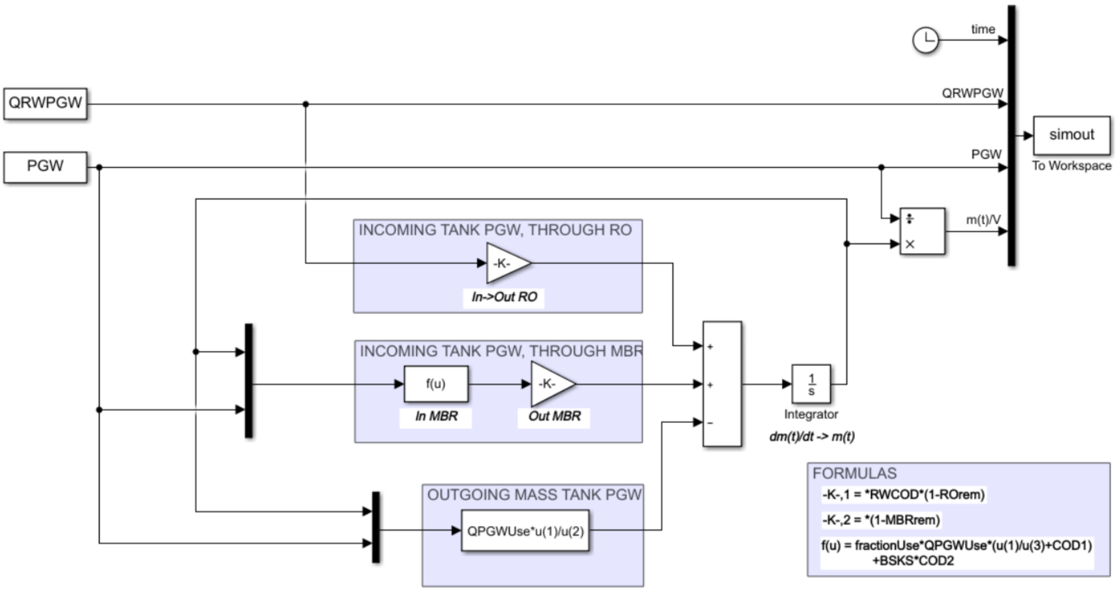

Figure B.1

B.2 Simulating salt accumulation with purge

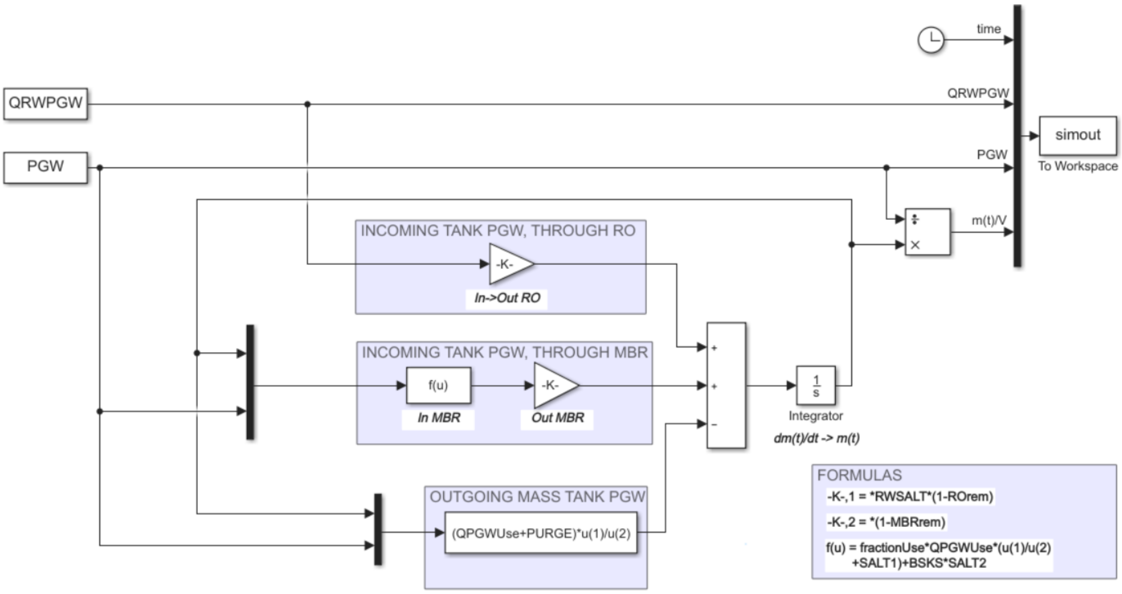

Figure B.2

B.3 Simulating salt accumulation without WM and DW

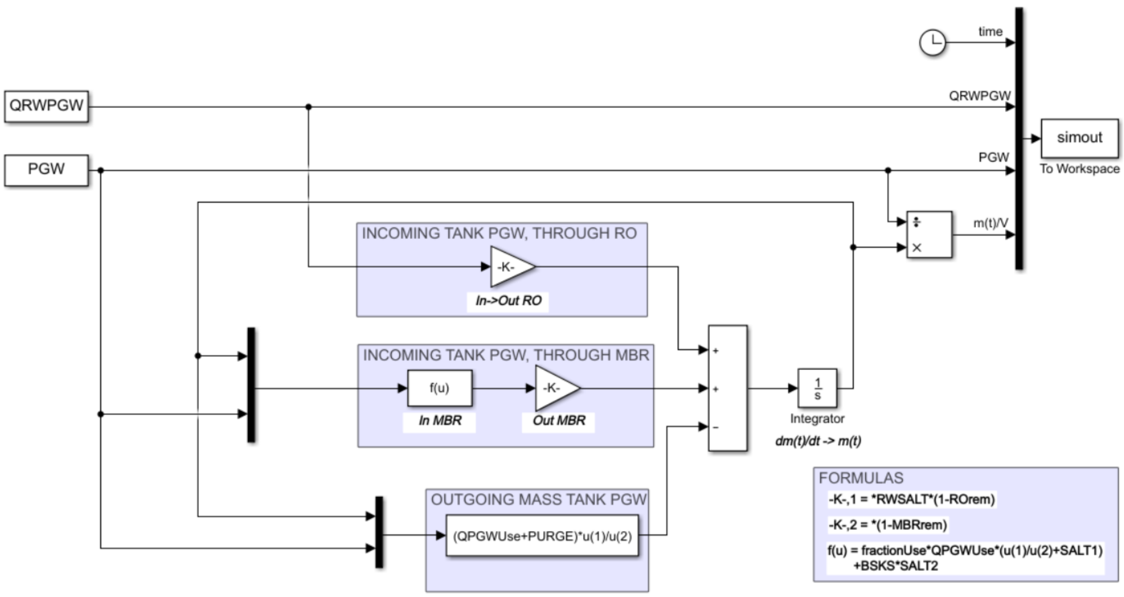

Figure B.3

B.4 Simulating COD accumulation with BW recirculation

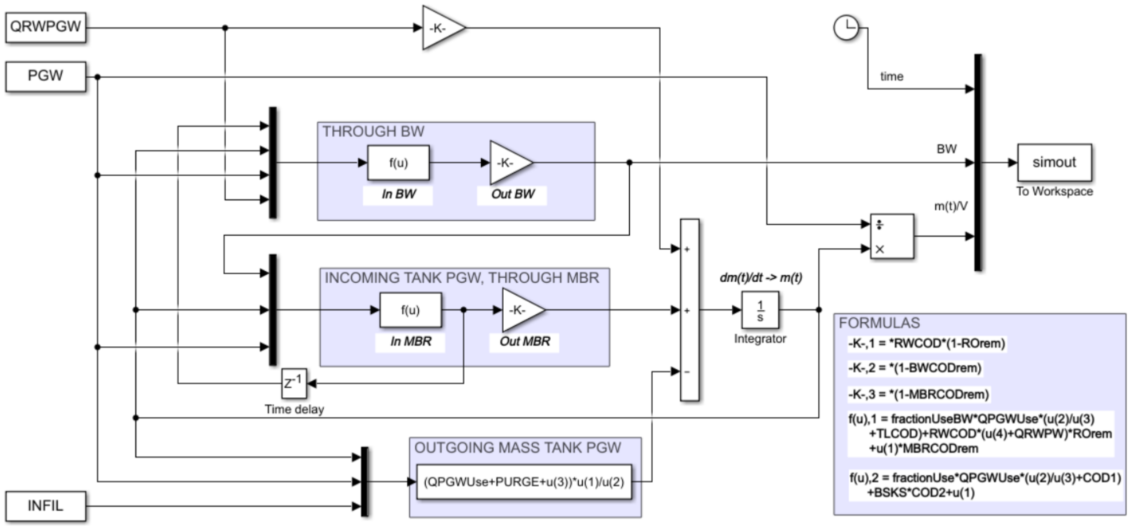

Figure B.4

B.5 Simulating salt accumulation with BW recirculation

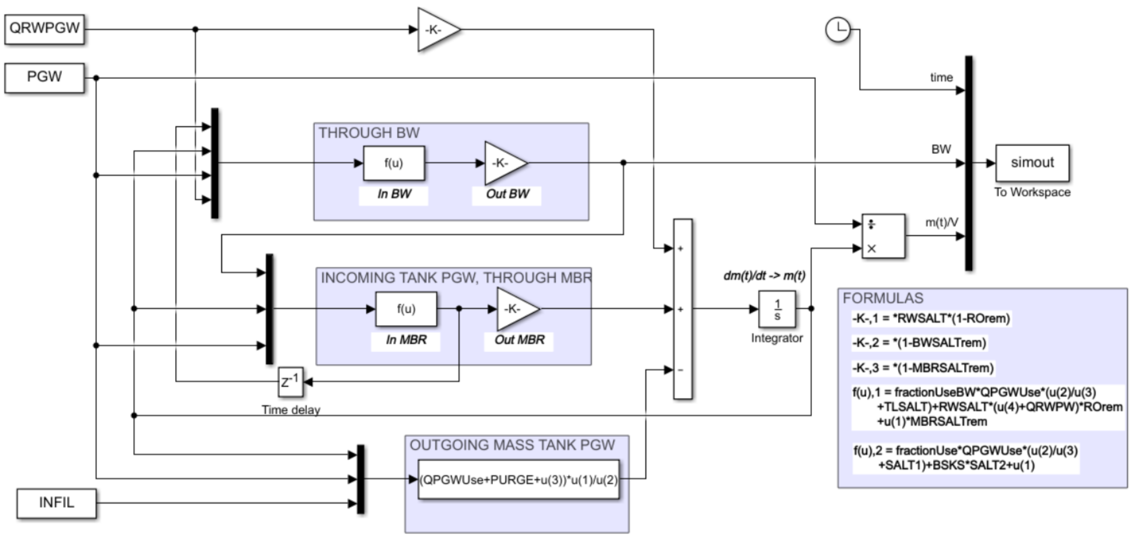

Figure B.5

B.6 Simulating COD accumulation with BW recirculation and purge recovery

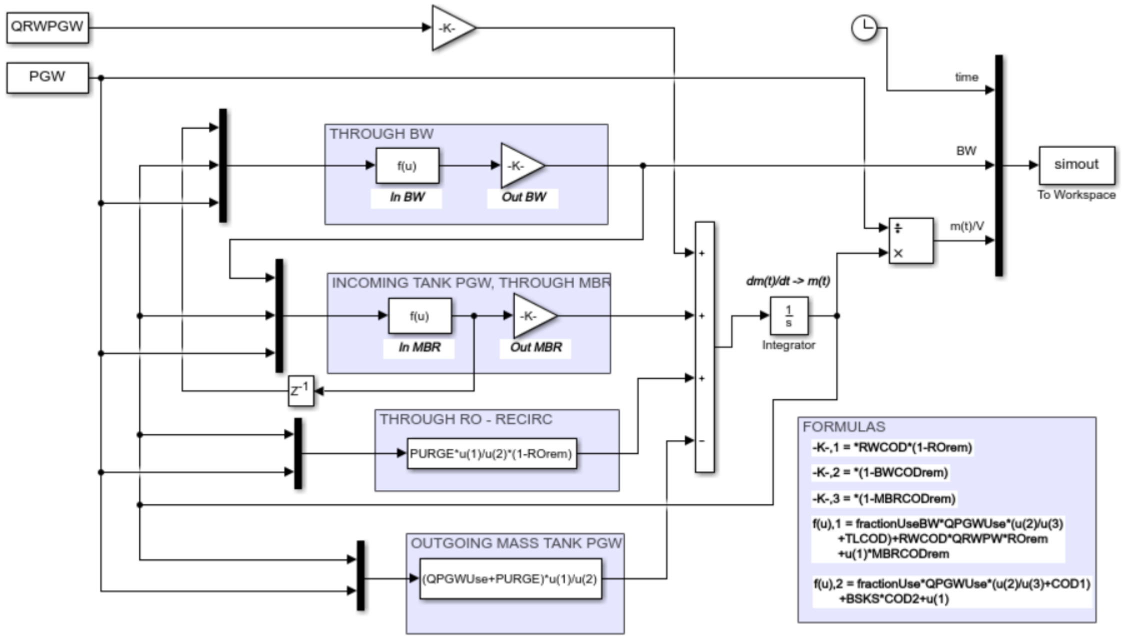

Figure B.6

B.7 Simulating salt accumulation with BW recirculation and purge recovery

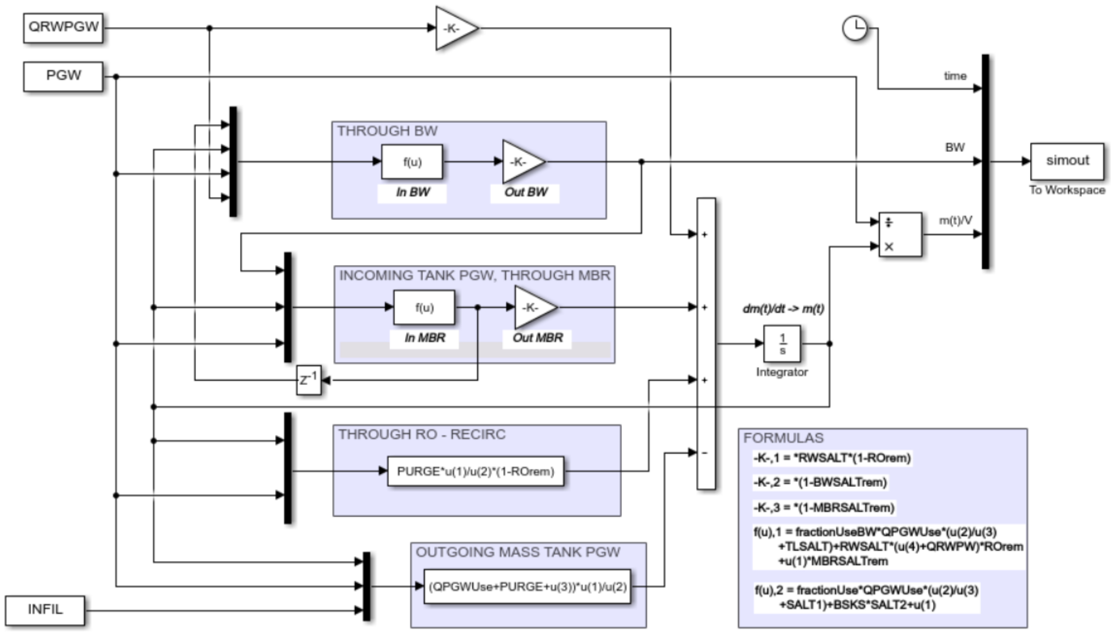

Figure B.7

B.8 Simulating salt flowing towards the infiltration tank during BW recirculation and purge recovery

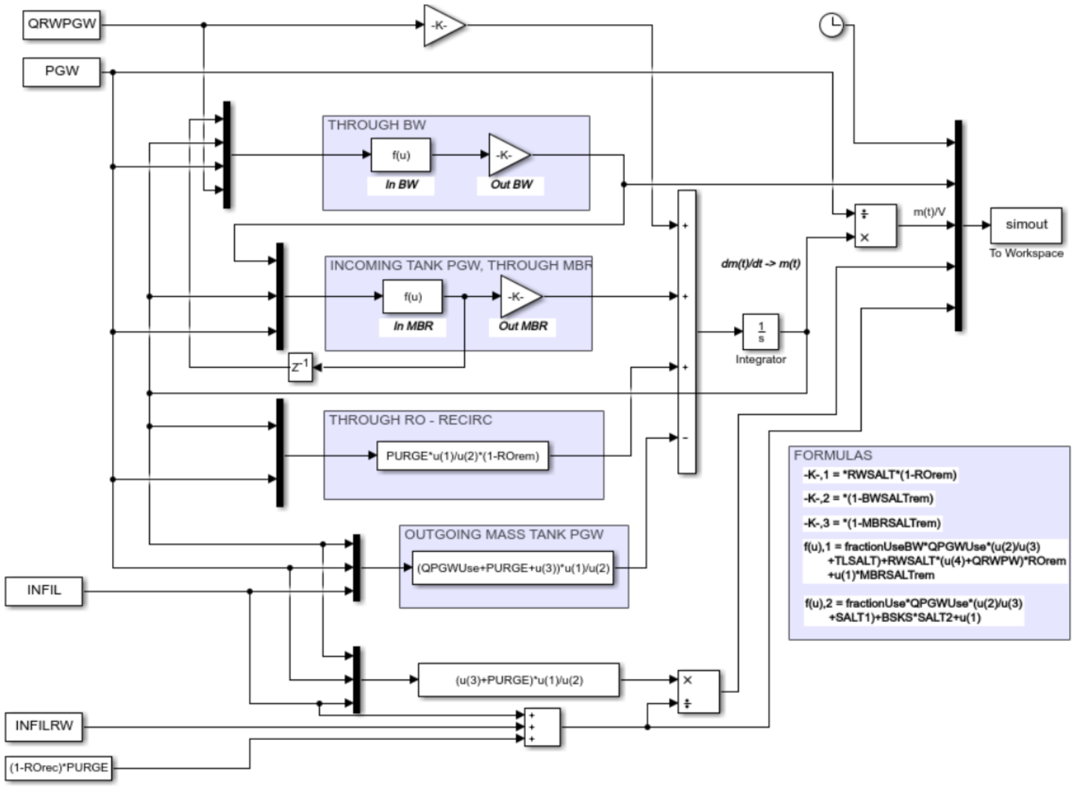

Figure B.8