Characterization and treatment of grey water; options for (re)use

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Characterization and treatment of grey water; options for (re)use

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Abstract- Abu Ghunmi, L. (2009) Characterization and treatment of grey water; options for (re)use. PhD thesis, Wageningen University, The Netherlands.

Addressing the issues of water shortage and appropriate sanitation in Jordan, domestic grey water treatment receives growing interest. Grey water comprises the domestic wastewater flows excluding waters associated with the toilet. The topics of concern for grey water are its characteristics, treatment and potentials for use after treatment. The target of this thesis is to develop a concept for treating grey water on-site for agricultural usage, thus sustaining a recycling process of grey water in Jordan. A review was made regarding the currently available grey water treatment technologies. In addition, grey water was quantitatively and qualitatively characterized, and then grey water reuse requirements including treatment, were analyzed. Biodegradability and biodegradation rates of the grey water were investigated for selecting appropriate design and operation criteria of the treatment technology to be developed. A low-tech semi-technical scale treatment system was tested to treat grey water discharges from a dormitory at the Jordan University campus. The treatment system was evaluated on obtained removal efficiencies and conformity of the effluent to the guidelines for the use of reclaimed water for irrigation in Jordan. Finally, the objectives, approaches and the results of each chapter are summarized, and then both the results and the potential of applying decentralised sanitation and reuse (DeSaR) concepts in Jordan are discussed.

Results show that storage and treatment are prerequisites for any type of grey water use. Grey water is aerobically and anaerobically biodegradable but the conversion rates are low. The core of the treatment concept consists of an integrated storage and anaerobic treatment unit, fed with a natural influent flow pattern, in a down-flow mode, up to a one day operational cycle, i.e. a variable HRT ≤ 24 hours. The second step consists of an aerobic post-treatment, mechanically aerated in a down-flow mode and a one day operational cycle, i.e. 24 hours HRT. Both units need insulation in the winter period. The final effluent, stable in winter and summer, meets the Jordanian standard, except for the pathogens, for usage in restricted irrigation. The achievable treatment efficiency for the COD_{tot} is 44% in the anaerobic unit and 70% in the combined anaerobic-aerobic, unlike the high anaerobic and aerobic biodegradability in batch experiments, viz. 70 and 86%, respectively. The highest removal efficiency achieved was for the COD_{ss} fraction, viz. 71% in the anaerobic and 85% in the combined system. Therefore, it is expected that the COD_{tot} removal efficiency of the system can be improved, by enhancing the COD_{col} and COD_{dis}, removal, i.e. applying filtration and/or adding chemicals such as adsorbents, coagulants and/or flocculants to the treatment units.

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Chapter One

General Introduction and Outline

1.1 Introduction

Water is an essential element for life therefore it is a valuable resource. This resource however, has been badly mismanaged by overconsumption and pollution. This resulted in water resources pollution and/or water shortage (Falkenmark, 1990; Arnell, 1999; Bouwer, 2000). The amount of people currently living under water stress is 700 million and it is projected to be 3 billion people in 2035 (World Bank, 2005). Therefore, the need for sustainable water use has emerged, which is defined as the use of water at undefined periods of time, keeping the same level and quality standards (Oxford wordpower dictionary, 1999).

A water sector issue of prime importance is the domestic wastewater and its sanitation. Poor sanitation causes yearly 1.8 millions of deaths, mostly children, by diarrheal disease (World Bank, 2005). The World Bank (2005) estimated that 40% of the world population in 2005 does not have access to basic sanitation, i.e. facilities that hygienically separate human excreta from human, animal, and insect contact. The currently adopted urban sanitation concept targets centralization of public networks such as water and sewerage. Centralisation, however, is costly in terms of construction, treatment and maintenance (Lettinga et al., 2001). In this concept the domestic wastewater is transported in an extensive sewerage system to a centralised treatment plant (Lettinga et al., 2001). Centralised urban sanitation focuses on the quality of the effluent without considering the value of domestic wastewater components; i.e. organic matter, nitrogen, and phosphorous. In response to that, various new concepts have been developed: i.e. Decentralized Sanitation and Reuse (DeSaR) and Ecological Sanitation (EcoSan) (e.g. Otterpohl et al., 1999; Zeeman and Lettinga, 1999; Werner et al., 2003).

DeSaR and EcoSan concepts view domestic wastewater streams as a resource of water, nutrients and organic matter (Werner et al., 2003; Lettinga et al., 2001). These resources could be recovered by on-site source separation, treatment and reclamation. Source separation of wastewater streams separates black water, i.e. toilet wastewater, from grey water, i.e. shower, laundry and washbasin wastewaters. Kitchen wastewater sometimes is gathered with black water and other times with grey water. The largest fraction of nitrogen, potassium and phosphorus in domestic wastewater is included in the black water (Kujawa-Roeleveld and Zeeman, 2006). The organic matter content is dependent on the added fraction of kitchen wastewater (Kujawa-Roeleveld and Zeeman, 2006). Black water can be considered as part of the food cycle and after treatment, as fertilizer for food production (Otterpohl *et al*., 1999). Its share of domestic wastewater components, in mass, is 38% of the COD, 82% of nitrogen (N), 68% of phosphorous (P), and 78% of potassium (K). Kitchen wastewater components are 33% of the COD, 9% of N, 12% of P, and 4% K (Kujawa-Roeleveld and Zeeman, 2006). The rest of the domestic wastewater comprises grey water (Otterpohl *et al*., 1999). Grey water research issues are the quantity and quality characteristics, the treatment and the use options as well as the standards.

Jordan annual requirement for fresh water is about 1.0 billion cubic meters, while its renewable internal freshwater resources are just 0.7 billion cubic meter (World Bank, 2005). The fresh water distribution is 20.8% for domestic, 75.2% for agriculture and 4% for industrial purposes (World Bank, 2005). The population growth rate is 3.5% (MWI, 2001). In Jordan, by law, domestic wastewater is considered a resource, making part of the national water resources. Reclaimed domestic wastewater forms 6% of the water budget in 1999 (WAJ, 1999). Furthermore, the treated sludge allows use in agriculture (MWI, 2001) taking into account that both reclaimed domestic wastewater and the treated sludge should comply with the Jordanian standards for reuse. Of the population of Jordan 85% has access to sanitation, of which 71% in rural areas and 88% in the urban areas (World Bank, 2005).

1.2 The objectives of this thesis

The objective of this thesis is the development of a concept for on-site grey water treatment and use as irrigation water, thus providing a recycling process of grey water in Jordan. The approach included reviewing grey water treatment technologies, thorough characterization of grey water quantity and quality, analysing the reuse requirements including the treatment, and assessing biodegradability and biodegradation rates. A low tech semi-technical scale treatment system was designed and tested to treat grey water discharges from a dormitory at the Jordan University campus.

1.3 Outline of the thesis

Chapter 2 reviews the literature of grey water treatment technologies regarding operation, affordability and efficiency.

Chapter 3 characterizes grey water in Jordan to determine the potential for reuse and the reuse requirements.

Chapter 4 deals with grey water anaerobic and aerobic biodegradability, i.e. factors affecting the determination process, biodegradability and conversion rates.

Chapter 5 gives the results of a pilot treatment plant at Jordan University campus treating on-site dormitory grey water for irrigation.

Chapter 6 summarizes the objectives, applied approaches and results of the accomplished research. Both the results and the potentials of applying DeSaR concepts in urban and rural situations in Jordan are discussed.

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Chapter Two

Treatment system for grey water: Review

Abstract:

This review aims to discern a treatment for grey water by examining grey water characteristics, reuse standards, technology performance and costs. The review reveals that the systems for treating grey water, whatever its quality, should consist of processes that are able to trap pollutants with a small particle size and convert organic matter to mineralized compounds. For efficient, simple and affordable treatment of grey water with safe effluent reuse, a combined anaerobic-aerobic process is recommended, with disinfection being an optional step. The removal and subsequent conversion of suspended and colloidal particles in the anaerobic process need further improvement. Furthermore, the reuse standards should be revised and classified considering the reuse options and requirements.

Keywords: grey water; treatment technologies; physical; chemical; biological; reuse; standards.

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Nomenclatures

AnBF	Anaerobic Bio-Filter
AP	Artificial Pond
BAF	Biological Aerated Filter
BOD	Biochemical Oxygen Demand
BSIRA	British Scientific Instrument Research Association
CF	Coarse Filter
CFU	Colony forming unit
Cl_2	Chlorine
COD	Chemical Oxygen Demand
col	Colloidal
CW	Constructed Wetlands
dis	dissolved
DHS	Down-flow Hanging Sponge
EB	Equalization Basin
E. coli	*Escherichia coli*
EPA	Environmental Protection Agency
FBR	Fluidized Bed Reactor
FC	*Faecal Coliform*
GRWRS	Green Roof Water
MF	Membrane Filter
M(F)BR	Membrane (Fibrous) Biological Reactor
M(F)CR	Membrane Chemical Reactor
MRT	Maximum Retention Time
MWCO	Molecular-Weight Cut-Off
N	Nitrogen
NTU	Nepthelometric Turbidity Unit
P	Phosphorous
PCOR	Photo-Catalytic Oxidation Reactor
PFS	Poly-Ferric Sulfate
PFST	Pre-Filtration Storage Tank
pH	-log [hydrogen ion concentration]
RBCs	Rotating Biological Contactors
RVFB	Recycled Vertical Flow Bioreactor
SAF	Submerged Aerated Filter
SB	Sedimentation Basin
SBF	Submerged BioFilters
SF	Sand Filter
SRT	Sludge Retention Time
SS	Suspended Solids

	Recycling System	SSr	Sub-Surface
GW	Grey Water	SSrF	Sub-Surface Filters
H	Horizontal	ST	Settling Tank
HPSF	Horizontal-Flow Planted Soil Filer	tot	Total
		TC	Total Coliform
HRT	Hydraulic Retention Time	TF	Trickling Filter
IVPSF	Intermittent Vertical-Flow Planted Soil Filter	TiO_2	Titanium Dioxide
		Tkj	Total Kjeldahl
IVSF	Intermittent Vertical-Flow Soil Filter	TOC	Total Organic Carbon
		TON	Threshold Odor Number
K	Potassium	UF	Ultra-Filtration
LAS	Linear Alkyl Benzene Sulfonates	UV	Ultra-Violet
		V	Vertical
MBR	Membrane Biological Reactor	WHO	Word Health Organization

2.1 Introduction

Water shortage and water pollution have become global issues; related issues are scarcity of water resources, mismanagement, population growth, and climate change (Falkenmark, 1990; Arnell, 1999; Bouwer, 2000). Industrial and domestic wastewaters' constituents contribute to water resource and soil pollution (Metcalf and Eddy, 2003). Wastewater treatment and recycling of useful products, i.e. water, nutrients and organic matter, mitigates both water shortages and environmental pollution. To maximize the possibility of recycling and minimize the energy required for treatment, industrial and domestic wastewaters have been separately treated (Metcalf and Eddy, 2003), and source-separation of domestic wastewaters into grey and black waters has been promoted recently (Otterpohl *et al*., 1999; 2003;

Zeeman and Lettinga, 1999). Excluding toilet (black water) and sometimes kitchen streams, grey water combines one or more of less polluted domestic wastewater streams (Christova-Boal *et al*., 1995; Jefferson *et al*., 1999; Otterpohl *et al*., 1999; Eriksson *et al*., 2002).

Grey water contribution to domestic wastewater is 60-75% of the water volume (Gulyas *et al*., 2004), and includes 9-14%, 20-32%, 18-22% and 29-62 % of N, P, K and organic matter respectively (Kujawa-Roeleveld and Zeeman, 2006). Several issues emerge with grey water, namely, (1) Reusing with or without simple treatment (Christova-Boal *et al*., 1995; Al-Jayyousi, 2002). (2) Recycling for indoor use such as flushing toilets, washing clothes and/or bathing (e.g. Christova-Boal *et al*., 1995; Bingley, 1996; Nolde, 1999; Jefferson *et al*., 1999; 2001; Shrestha *et al*., 2001a and b; Li *et al*., 2003; Cui and Ren, 2005) and for outdoor use such as irrigating domestic gardens, lawns on college campuses, athletic fields, cemeteries, parks and golf courses, washing vehicles and windows, extinguishing fires, feeding boilers, developing and preserving wetlands and recharging ground water (e.g. Christova-Boal *et al*., 1995; Bingley, 1996; Fittschen and Niemczynowicz, 1997; Nolde, 1999; Otterpohl, 1999; Okun, 2000; Jefferson *et al*., 2001; Shrestha *et al*., 2001a and b; Eriksson *et al*., 2002; Al-Jayyousi, 2002; 2003). (3) Standards are mainly related to health and social aspects in order to improve the control of the recycling process (e.g. Nolde, 1999; Jefferson *et al*., 1999; 2000; 2001; Li *et al*., 2003; Cui and Ren, 2005). (4) Obtaining affordable treatment technologies to cope with the quantity and quality variation of grey water sources (Imura *et al*., 1995; Eriksson *et al*., 2002), and the recycling requirements (e.g. Nolde, 1999; Jefferson *et al*., 1999; 2000; 2001; Li *et al*., 2003; Cui and Ren, 2005).

A wide range of treatment technologies have been applied and examined for grey water considering one or more of the grey water issues, and producing effluents with different qualities. This review, therefore, examines various grey water treatment technologies with the aim of coming up with an efficient, simple and affordable treatment system with safe effluent for use. A treatment system is considered efficient if it produces the required effluent quality, is simple in operation with minimum maintenance, and affordable due to its low energy consumption and low operational and maintenance costs. Safe effluent refers to a situation where the possibility of pathogens re-growth is minimal. The issues considered in the selection of grey water treatment systems are grey water characteristics, used standards, technology performance and costs.

2.2 Grey water characteristics and use standards

Raw grey water treatment is a prerequisite for storage and use. The aim of treatment is to overcome esthetic, health and technical problems, which are caused by organic matter, pathogens and solids, and to meet reuse standards. Raw grey water pollutants, measured as COD, have an anaerobic and aerobic biodegradability of respectively 72-74% (Elmitwalli and Otterpohl, 2007; Zeeman *et al*., 2008) and 84±5% (Zeeman *et al*., 2008). Furthermore, 27-54% is dissolved, 16-23% colloidal, and 28-50% suspended (Elmitwalli and Otterpohl, 2007; Zeeman *et al*., 2008). Grey water can contain recalcitrant organic matter (Friedler *et al*., 2006; Hernandez *et al*., 2007). For example anionic and cationic surfactants are slowly or non-biodegradable under anaerobic conditions (Garcia *et al*., 1999; Matthew *et al*., 2000). Storing grey water for 48 hours at 19 to 26 ºC deteriorates its quality, (Dixon *et al*., 1999); biological degradation produces malodorous compounds, causing an "aesthetic problem" (Kourik, 1991; van der Ryn, 1995; Christova-Boal *et al*., 1995; Dixon *et al*., 1999), pathogen growth (Christova-Boal *et al*., 1995; Rose *et al*., 1991; Dixon *et al*., 1999) and mosquito breeding (Christova-Boal *et al*., 1995), which are a "health threat". Use of raw grey water clogs the recycling system due to build-up of suspended material and/or the biological growth in the systems (Christova-Boal *et al*., 1995). Raw grey water quality characteristics (Tables 2.2 to 2.7) do not comply with the standards (Table 2.1). Treatment is therefore required (Eriksson *et al*., 2002) and the treatment level depends on the reuse options (Pidou *et al*., 2007). A biological treatment system is appropriate for stabilizing the organic matter (Nolde, 1999; Jefferson *et al*., 1999; 2004).

Grey water treatment does not aim at providing water of drinking water quality but at water for toilet flushing, laundry, lawn irrigation, windows and car washing, ground water recharge, or fire extinguishing (e.g. Jefferson *et al*., 1999; Eriksson *et al*., 2002). The adopted standards (Table 2.1) for use of grey water are originally for reclaimed domestic (grey + black) wastewater. The adopted standards almost resemble drinking water quality and do not consider significant variation in the qualities required for different use options. The standards also ignore the presence of resources such as nutrients. For instance, the standards for turbidity and nitrogen content of respectively <2 NTU and 30 mg N L^{-1} are lower than the World Health Organization (WHO) guidelines for drinking water quality; non-detectable Faecal Coliform (FC) and/or Total Coliform (TC) are lower than bathing water standards in the United Kingdom (UK). Furthermore, China created differentiated standards e.g. for toilet flushing, car cleaning and lawn irrigation, but the variation of standards for the different uses is only minor. Moreover the standards for domestic water recycling prevailing in various countries (Table 2.1) are neither uniform nor globally standardised. Development of multi-category standards is required for an optimal use of grey water. The standards should include different aspects such as health, aesthetic and environment. For instance the WHO (2006) guidelines for use of grey water have two categories, viz. restricted and unrestricted irrigation. Furthermore, it is recommended to combine grey water use standards with guidelines for safe practice e.g. the maximum retention time in the toilet cistern. The WHO (2006) guidelines for reuse of grey water for irrigation are combined with guidelines for safe practice; e.g. applying drip irrigation techniques, covering the soil with mulch, avoiding contact with wet soil.

Table 2.1. Water quality standards and criteria for domestic water recycling in different countries

Standards	Turbidity NTU	BOD_5 mg L^{-1}	COD mg L^{-1}	SS mg L^{-1}	N mg L^{-1}	P mg L^{-1}	pH	TC Cfu/100ml	FC cfu/100ml	EC cfu/100ml	References
USA-EPA											
Unrestricted Use[1]	2	≤ 10	-	-			6-9	-	ND		U.S. EPA, Guidelines for Water Reuse, EPA/625/R-92/2004
Restricted Use[2]		≤ 30		≤ 30			6-9		≤ 200		
WHO											WHO, Water Quality: Guidelines for the safe use of grey water (2006); Drinking Water Quality 1996, 2001
Restricted irrigation	-	-	-	-			-	≤ 1E5	-	-	
Unrestricted irrigation[3]								≤ 1E3			
Drinking Quality[4]	≤ 5				50		6.5-8.5				
UK-Bathing water								$5E2^{g}$-$1E4^{m}$	$1E2^{g}$-$2E3^{m}$		Parker and Frost (2000)
China[5]											The reuse of urban recycled water-water quality standard for urban miscellaneous water consumption. GB/T 18920--2002
Toilet flushing	5	10			10		6-9			0.3	
Cleaning Car	10	15			10		6-9			0.3	
Lawn irrigation	10	20			20		6-9			0.3	
Japan											
Toilet flushing							5.8-8.6	≤ 1000			Ogoshi *et al.* (2001)
Landscape irrigation							5.8-8.6	ND			
Jordan											Jordanian standards for use of reclaimed domestic wastewater for recharge of aquifers (JS893/2002)
Recharge aquifer	2	15	50	50	30	15	6-9	-	<2.2	<2.3	
Unrestricted irrigation[6]	10	30	100	50	45		6-9	-	100	101	

1; urban uses, crops eaten raw, recreational impoundments
2; restricted access area irrigation, processed food crops, non-food crops, aesthetic impoundments, construction uses, industrial cooling, and environmental reuse
3; crops eaten raw
4; drinking water quality, 1993
5; Nitrogen are for ammonia measurements
6; irrigation of vegetables (to be cooked before consumption), parks, playgrounds and roadsides or roads within city limits

2.3 Grey water treatment systems

A grey water treatment system consists of different treatment steps that might be considered, depending on the required quality of the effluent (Figure 6.1). Several treatment technologies can be used in each step. Technologies examined for treating grey water are classified based on the treatment principle: physical, biological, chemical or a combination of these. Furthermore, the technologies are reviewed in terms of performance, operation, and the encountered problems.

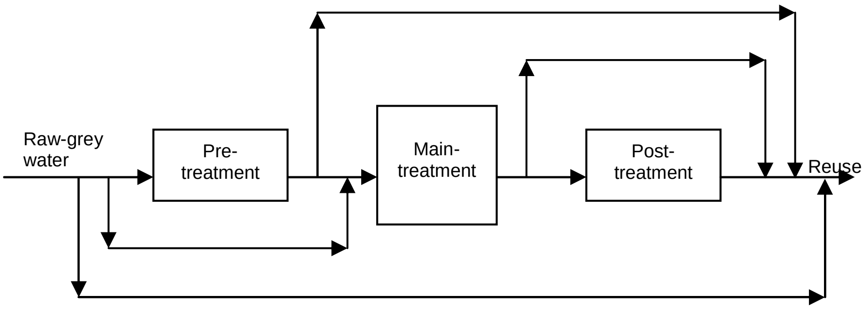

Figure 1. Grey water recycling and treatment, possible steps and tracks

2.3.1 Filtration and physiochemical processes

Several types of macro- and membrane-filtration units for grey water treatment have been tested. The tested macro-filtration units include a strainer series with pore size ≥ 0.17mm, nylon sock type filters, geotextile (filter sock) filters, fibrous (cloth) filters, coarse filters (CF), and sand filters (SF) (Christova-Boal *et al*., 1995; Jefferson *et al*., 1999; Al-Jayyousi, 2003; Friedler *et al*., 2006). The tested membrane-filtration units, sheet or tubular were (a) micro-filter 0.1 µm Membrane Fibrous Filters (M(F)F) (Ahn *et al*., 1998) and ≤ 0.2 µm M(F)F (Shine *et al*., 1998); (b) 300 kDa Ultra Fibrous Filter (U(F)F) (Ahn *et al*., 1998), 4, 6, and 200 kDa MWCO U(F)F (Hills *et al*., 2001), 30, 200, and 400 kDa MWCO UF (Ramon *et al*., 2004). The pore size of the UF of Cui and Ren

(2005) was not reported, and Nghiem *et al*. (2006) tested 0.045 µm submerged U(F)F. (c) 75, 80 and 90% $CaCl_2$ rejection Nano Fibrous filter (N(F)F) (Hills *et al*., 2001) and 200 Da MWCO ≅ 75% $CaCl_2$ rejection (Ramon *et al*., 2004).

The efficiency of the filtration techniques depends on the particle size distribution of grey water pollutants and the filters' porosity; in general the smaller the filters' porosity the better the effluent quality (Table 2.2). Ahn *et al*. (1998) reported that the pore size of the tested membrane filters has marginal effect on the treatment efficiency of grey water; the reason being that the average particle size of tested grey water was 2.18 µm while membranes with a pore size of 0.1 µm, 300 and 15 kDa were tested. In contrast, Ramon *et al*. (2004) reported better effluent qualities produced by N(F)F as compared to UF; the underlying reason is the presence of organic matter with low molecular weight in grey water that cannot be rejected by UF. Also Table 2.2 shows that the U(F)F effluent quality (BOD) reported by Hills *et al*. (2001) is better than the quality of the MF effluent as reported by Jefferson *et al*. (1999). This is also in agreement with Nolde (1999), who reported replacing ultra filtration and reverse osmosis by 0.2 µm membrane eliminates the microorganism but hardly reduces the BOD. None of the examined-filters, presented in Table 2.2, have been tested for nutrients removal viz. nitrogen (N) and phosphorous (P).

Filtering raw grey water, whatever its quality, through macrofilters reduces blockages in the recycling system (Christova-Boal *et al*., 1995; Jefferson *et al*., 1999). However, macrofiltration units, except sand filters**,** show no absolute barrier for the suspended pollutants, and the chemical nature of grey water in terms of organic load and turbidity, remains almost unaltered, thereby promoting biological growth (Christova-Boal *et al*., 1995; Jefferson *et al*., 1999). In addition to the filter effluent quality problems, filters produce unstable primary sludge which needs further treatment. Also, the primary sludge residence time in the filter affects the filter effluent qualities. Thus, the smaller the pore size and the shorter the primary sludge residence time, the better and the more stable the effluent quality. Meanwhile the previously mentioned small pore size and shorter sludge residence time increase fouling and operational costs and cleaning frequency. Treating grey water's BOD, COD and pathogens by filters as main treatment units is not recommended.

Filters face a number of operational problems, such as the cleaning frequency of macrofiltration units, which may vary from once after each use to once per week (Christova-Boal *et al*., 1995; Jefferson *et al*., 1999; Friedler *et al*., 2006). Effluent qualities in terms of organic content and turbidity cause periodical failures of disinfection by halogen compounds (Jefferson *et al*., 1999), which have the affinity to react with the organic matter. Operation over extended time periods (no time value given) of membrane filters in microfiltration units can result in anaerobic conditions of the grey water (Jefferson *et al*., 1999) and generate organic components that are less readily rejected by the membrane (Holden and Ward, 1999). Nghiem *et al*. (2006), using a "< 0.04 µm" U(F)F membrane unit, reported an increased thickness of the cake layer at increased particulate organic matter concentrations. The hydraulic resistance and fouling was worsened by the humic acids content, and the presence of calcium may even increase that effect (Nghiem *et al*., 2006). Increased hydraulic resistance leads to more energy consumption for the membrane permeation (Jefferson *et al*., 1999). A general aspect of ultrafiltration is a very high energy demand (Nolde, 1999), and MWCO needs optimization for economics and permeate quality (Ramon *et al*., 2004).

The pre-treatment of raw grey water in storage/settling tanks mitigates partially the clogging problems of sand filters and could replace the coarse filter. However, the same amount of unstable primary sludge is still produced in addition to the increase in the total volume of the treatment system. Moreover, the hydraulic and sludge residence time of the pre-treatment tank should be optimized to prevent deterioration of its effluent quality (Imura *et al*., 1995; Shrestha *et al*., 2001a and b). Adding coagulants, like $AL_2(SO_4)_3$, $FeCl_3$, poly-aluminium chloride, and PFS in combination with mixing, enhances the performance of the pre-treatment tank (Cui and Ren, 2005). Application of physical-chemical processes, as shown by Pidou *et al*. (2007), is promising for grey water treatment, certainly when considering the short HRT, <1 hour, that can be applied. However, more primary sludge is produced resulting in an increase in operational costs. Different types of post-treatment units have been used to enhance the filters' effluent turbidity, suspended solids, organic matter and/or pathogens qualities. The tested units (Table 2.2) are Ultra Membrane Filtration (Hills *et al*., 2001; Cui and Ren, 2005), Activated Carbon Absorber, and Ultraviolet radiation (Cui and Ren, 2005), and disinfection by halogens (Christova-Boal *et al*., 1995; Al-Jayyousi, 2002). From the latter unit the effluent quality was not reported while the rest produced effluents that complied with the most conservative turbidity, SS and pathogens standards (Table 2.1). Therefore, membrane filtration i.e. micro-, ultra- and nano-filters, could be an option for post treating grey water to achieve the most conservative standards.

Table 2.2. Filters and/or physical-chemical units treating grey water. Influent and effluent qualities and operation conditions

Treatments	BOD_5 mg L^{-1}		COD mg L^{-1}		SS mg L^{-1}	Turbidity NTU	N mg L^{-1}	P mg L^{-1}	FC cfu/100 ml	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}	COD_{tot}	COD_{dis}									
Macro-Filters													
CF +Disinfection												Moderate installation cost 500 to 1000 £, 8 years pay back period for four-person house hold	Jefferson *et al.*, 1999
GW-Source	Single house grey water; Bath/shower, washbasin, laundry												
CF	Metal strainer, usually main feature a short HRT (no specific time given)												
Disinfection	Chlorine or bromine dispensed in a small release block or dosed in liquid solution									Chlorine or bromine dispensed			
Influent													
Effluent	>50						<3	<1					
SF													Jefferson *et al.*, 1999
Influent	33.3		143			44.5							
Effluent	12.3		35.7			32.3							
ST+ Backwashing SF	Collection tank followed by Automatic backwashing sand filter, followed by storage tank												Al-Jayyousi *et al.*, 2003
GW-Source	Grey water generated by rural houses in Ain Al-Badia, Tafileh governorate, Jordan.												
Influent	1500[a]				316								
Effluent	392 [a]				189								
Equalization basin+ SF	1 mm fine screen followed by equalization basin followed by sand filter.												Friedler *et al.*, 2006a
GW-Source	Bath, shower and washbasin streams discharge from seven student apartments at Technion campus; accommodate married students												
Equalization basin	Preceding Equalization Basin there was 1 mm square shaped screen to remove gross solids. EB volume 330 L maximum residence time is 10 hr												
Effluent	69 (33)	36 (20)	108 (47)	211 (141)	92 (115)	65 (68)			3.4E5 (4.2E5)				
SF	Gravity filter, diameter 10 cm and 70 cm media depth; the medium consists of quartz and sand, porosity 36 %, and supported by 5 cm gravel, the filter operated intermittently 11 time a day, 15 minutes each time, the filtration velocity is 8.33 m hr^{-1}, back washed weekly after filtration of 1.26 m^3.												
Effluent	62 (21)	40 (2)	87 (28)	130 (37)	32 (13)	35 (25)			1.3E5 (1.4E5)				
Micro-Filters													
M(F)F	Volume 20 m^3, membrane hollow fibre polyproplyne with 0.2 μm nominal pore size. Air automatic backwashing and chemical for cleaning after long operation												Shine *et al.*, 1998
GW-Source	Japanese Office building; Cooking, bathing, washing												
Influent					19-113								
Effluent					around 1								
MF or UF	Fibrous membrane, ultra filters, applied pressure up to 2 bar.									Chemical for cleaning the fouling	Substantially higher than depth filters	low organic load reduces energy demand but increases overall cost	Jefferson *et al.*, 1999
GW-Source													
Influent	33.3		143			44.5							
Effluent	4.7		22.2			0.34			0 [b]				
GW-Source													
Influent	25-185		86-410			12-100			2E0-3.1E5 [c]				
Effluent	1-19		21-112			<1			ND-2.4E3 [c]				

a; reported BOD mg L^{-1}: b; E.Coli: c; TC= Total Coliform; ND; Not Detectable: d; TOC (Total Organic Carbon)

Table 2.2. Filters and/or Ppysical-chemical treated grey water. Influent and effluent qualities and operation conditions (continued)

Treatments	BOD[a] mg L^{-1}		COD mg L^{-1}	pH	Odor TON	Turbidity NTU	N mg L^{-1}	P mg L^{-1}	FC cfu/100 ml	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}											
U(F)F (different types)	Membrane rig consisted of six single tubes with a total membrane area 0.22 m^2, and operated on the batch mode.												Hills *et al*., 2001
GW-Source	Artificial Grey water; mimicking grey water discharge from hand basins and treated in BAF												
MWCO rejects 200 kDa	Polyvinyslidenofluoride												
Influent	20-25	6-10											
Effluent	10.6	8.3											
MWCO rejects 6 kDa	Modified polyethersulphone												
Influent	20-25	6-10											
Effluent	5.1	4.7											
MWCO rejects 4kDa	Tight polyethersulphone												
Influent	20-25	6-10											
Effluent	6.3	5.8											
75 % $CaCl_2$ rejection	Polyamine film												
Influent	20-25	6-10											
Effluent	3	3.1											
80 % $CaCl_2$ rejection	Polyamine film												
Influent	20-25	6-10											
Effluent	2.3	2.4											
90 % $CaCl_2$ rejection	Cellulose acetate												
Influent	20-25	6-10											
Effluent	4.8	4.7											
Grey water source	from public shower of the sport centre												Ramon *et al*., 2004
U(F)F	Polyacrilonitrile (PAN) membrane sheet, applied pressure 1-2 bar , the module dimension 100 and 60 mm, the outer and inner diameter												
MWCO rejects400 kDa													
Influent			80 (21.5)			1.4 (0.4)							
% removal			45			92.3							
MWCO rejects 200 kDa													
Influent			74 (28.6)			1 (0.5)							
% removal			49.1			94.2							
MWCO rejects 30 kDa													
Influent			50.6 (6.6)			0.8 (0.2)							
% removal			69.3			96.6							

a; reported BOD mg L^{-1}: b; E.Coli: c; TC= Total Coliform; ND; Not Detectable: d; TOC (Total Organic Carbon)

Table 2.2. Filters and/or physical-chemical treated grey water. Influent and effluent qualities and operation conditions (continued)

Treatments	BOD mg L^{-1}		COD mg L^{-1}	pH	Odor TON	Turbidity NTU	N mg L^{-1}	P mg L^{-1}	FC cfu/100 ml	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}											
N(F)F	Tubular nanofilterm 30CM length, 1.25 inner diameter, and 0.014 m^2 filtration area, applied pressure 6-10 bar, cross flow filtration unit, 150 l/h flow rate												Ramon *et al*., 2004
MWCO rejects 200 Da													
Influent			226			29.5 (0.6)							
% removal			93.3			98.1							
Physical-chemical Processes													
Coagulation	Coagulant dosage is 30 mg L^{-1} of $FeCl_3$												Jefferson *et al*., 1999
Influent	100					29.4							
Effluent	30					2.41							
Oxidation	Oxidant dosage is 2 g L^{-1} of TiO_2 activated by UV radiation												
Influent			41 (TOC)[d]						9E5 [c]				
Effluent			25 (TOC) [d]						ND [c]				
Multiple Physiochemical Units	Grey water treated subsequently in Coagulation unit, sand filter, adsorber, UF and UV disinfection.												Cui and Ren, 2005
Coagulation	Coagulant tested $AL_2(SO_4)_3$, $FeCl_3$, PAC and PFS were tested using 20 and 40 mg L^{-1} dosage; the optimum coagulant and dosage were 20 mg L^{-1} PFS												
Adsorber	Activated carbon and with optimum flow velocity 10 m hr^{-1}												
UF	Optimum operating pressure 0.1 bar												
UV	Dosage 250 mj cm^{-2}												
GW-Source	Shower												
Influent			63	7.2	39	35							
Effluent			1.2	6.8	0	0.15							
GW-Source	Bath												
Influent			137	7.3	81	75							
Effluent			1.6	6.7	0	0.15							
GW-Source	Mixed												
Influent			86	7.3	55	61							
Effluent			1.3	6.8	0	0.14							

a; reported BOD mg L^{-1}: b; E.Coli: c; TC= Total Coliform; ND; Not Detectable: d; TOC (Total Organic Carbon)

2.3.2 Modified filters

Filters' performances have been improved by modifying the operational conditions, such as flow direction, hydraulic retention time (HRT), and planting the filter media i.e. constructed wetlands. Also filters are developed that combine two types of treatment in the same unit, namely, biofilters combining physical and biological processes, and chemfilters combining physical and chemical processes.

2.3.2.1 Soil filters and constructed wetlands (CW)

The tested filters can be classified into two categories: unplanted and planted filters. Each category is sub-classified according to the tested flow-directions. Unplanted filters are Intermittent Vertical-Flow Soil Filter (IVSF) (Nolde and Dott, 1992), Sub-Surface Flow Filters (SSrF) (Dallas et al., 2005), Slanted Soil system (SSo) (Itayama et al., 2006) and Recycled Vertical Flow Bioreactor (RVFB) (Gross et al., 2007a). Planted filters are Intermittent Vertical-Flow Planted Soil Filter (IVPSF), Horizontal-Flow Planted Soil Filter (HPSF) (Hegemann, 1993), and planted Sub-Surface Filter (SSrF) (Dallas et al., 2005), Recycled Vertical Flow Constructed Wetlands (RVFCW) (Gross et al., 2007b), and Green Roof Water Recycling System (GRWRS) (Windward et al., 2008). The first four planted filters are also called root-zone facilities or respectively Vertical, Horizontal and Sub-Surface Constructed Wetlands (V-CW, H-CW, SSr-CW) (Fittschen and Niemczynowicz, 1997; Otterpohl et al., 1999; Li et al., 2003), or (V-, H- and SSr-) Reedbed (Dallas et al., 2005; Winward et al., 2008).

Performance tests (Table 2.3) for IVSF, SSrF, RVFB, H-CW, V-CW, SSrF-CW, and GRWRS show that the tested units' effluent qualities, BOD, COD, SS, turbidity and pathogens, are better than that of macro-filters (Table 2.2). In addition, CWs and RVFCW show capacity in treating nitrogen and phosphorous. The overall treatment performance could be improved by applying less porosity, longer HRT and introducing plants and/or applying vertical flow (Table 2.3). Moreover, as shown in Table 2.3, H-CW (Fittschen and Niemczynowicz, 1997), V-CWs (Shrestha et al., 2001a and b; Windward et al., 2008), SSrF-CW (Dallas et al., 2005) and GRWRS (Windward et al., 2008) produce effluent qualities in terms of BOD that comply with all standards (Table 2.1). The main feature of the well performing CW reported by of Fittschen and Niemczynowicz (1997), and Dallas et al. (2005) is the long HRT, viz. 14, and 5.1 - 8.5 days respectively, compared with other tested systems (Tables 2.2, 2.4-2.7). The tested systems (Table 2.3) show different capacities in treating pathogens according to differences in their internal structure and/or the applied HRT. However, none of the effluents comply with all standards with regard to pathogens (Table 2.1).

Constructed wetlands face a number of problems such as uneven distribution of the wastewater over the bed surface, and inappropriate selection of bed media grain size (Shrestha et al., 2001a and b; Dallas et al., 2005). Local conditions must be considered in the design, such as temperature, rain fall and wastewater composition (Fittschen and Niemczynowicz, 1997; Shrestha et al., 2001a and b; Dallas et al., 2005). Consequently, Shrestha et al. (2001a and b) recommended that development of appropriate design guidelines for constructed wetlands is imperative.

Pre-treatment of grey water in settling tanks for constructed wetlands have been tested (Fittschen and Niemczynowicz, 1997; Shrestha *et al*., 2001a and b; and Li *et al*., 2003). Shrestha *et al*. (2001a and b) noted that the irregular removal of sludge from the settling tank causes failure of constructed wetland systems. However, Dallas *et al*. (2005) did not report this problem. Post-treating CW effluent in a sand filter enhances the quality (Table 2.3) in terms of N, P and pathogens (Fittschen and Niemczynowicz, 1997). CW effluent treated, by photo-oxidation using TiO_2 and UV, shows improved quality in terms of TC and EC (Table 2.3) and complies easily with European bathing water standards (Table 2.1) (Li *et al*., 2003). The effluent treated with TiO_2 needs further treatment to remove the TiO_2, which takes a relatively long time to settle. Therefore, centrifugal separation may be needed, and this makes the disinfection process expensive (Li *et al*., 2003). Applications of photo-oxidation followed by separation of TiO_2 by MF are reported in the section chemfilters.

Table 2.3. Soil Filters and planted soil filters “CWs” combined with others units treating grey water. Influent and effluent qualities and operation conditions.

Treatments	BOD^a	COD	SS	Turbidity NTU	TN	TP	TC cfu/100 ml	Surface Areas $m^2\ Cap^{-1}$	Volume m^3	Hydraulic Load $m^3\ d^{-1}$	HRT d^{-1}	Chemical	Energy	Cost	References
	mg L^{-1}				mg L^{-1}										
Soil Filters															
IVSF								1.4		38-75					Nolde and Dott, 1992
GW-Source	Grey water without kitchen wastewater														
Effluent	< 3 [b]														
SSrF	Two types of media were separately tested: 20 mm crushed rock with porosity 40% and 100-150 mm PET plastic drinking water bottles porosity 94%; the bottom of the filters were lined with two lyres of plastic sheets. The dimensions of the filters’ bed are 1.5 m long, 0.25 m wide and 0.2 m depth. Also SSF of both media were tested further to be operated as reed bed and planted with *Coix-lacryma-jobi*.														Dallas, *et al.*, 2005
GW-Source	Monteverde Institute’s grey water.													Operation and maintenance costs are generally low	
Influent	216 ± 55.3	Dry-season					8.5 E7 ± 5.7E7 [c]			5					
Effluents															
SSrF (PET)	16 ± 3.6						1.1E6 ± 2.0E6 [c]		0.075		5.6				
SSrF (Rock)	9.0 ± 1.3						2.9 E5 ± 6.9E5 [c]		0.075		4.6				
Influent	155 ± 14.1	Dry-season					3.8E6 ± 1.0E6 [c]			10					
Effluents															
SSrF (PET)	18 ± 1.7						1.1E6 ± 2.3E6 [c]		0.075		3.3				
SSrF (Rock)	19 ± 3.8						1.3E5 ± 2.9E5 [c]		0.075		2.7				
Influent	290 ± 36.2	Wet-season					1.0E8±1.0E8 [c]			5					
Effluents															
SSrF (PET)	14 ± 5.1						2.7E5 ± 1.7E5 [c]		0.075		2.9				
SSrF (Rock)	14 ± 10.5						2.2E4 ± 3.3 E4 [c]		0.075		2.9				
Influent	285 ± 106.1	Wet-season								10					
Effluents															
SSrF (PET)	31 ± 2.3								0.075		2.1				
SSrF (Rock)	28 ± 0.0								0.075		1.7				
SSo	Slanted Soil system; soft particle with 1 cm of Kanuma soil that composes of alumina and hydrated silica, the setup; three stacks of 100x50x17.5 cm slanted plastic foam trays; each trey has three 6 cm ridges to prevent clogging. 12.5 cm layer thickness in each tray. (Small footprint, meaning limited land use). The variation in temperature 5-28°C has no influence on the performance.													Compact, low cost , can work 3 years with no maintenance	Itayama *et al.*, 2006
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus														
Influent	41 [d]	23.3 [d,e]	9			1.78	0.323			0.1					
Effluent	6.8	3.6	2			0.38	0.046								

a; BOD: b; BOD_7:c; faecal coliform: d; the units of the influent and effluent is g $m^{-2}\ day^{-1}$: e; COD based on Mn measurement: f; ammonia: g; thermostable coliform bacteria: h; hours: i; TOC=Total Organic Carbon: j; E. Coli.

Table 2.3. Soil Filters and planted soil filters "CWs" combined with others units treating grey water. Influent and effluent qualities and operation conditions (continued).

Treatments	BOD_5	COD	SS	Turbidity NTU	TN	TP	TC cfu/100 ml	Surface Areas $m^2\ Cap^{-1}$	Volume m^3	Hydraulic Load $m^3\ d^{-1}$	HRT d^{-1}	Chemical	Energy	Cost	References
	$mg\ L^{-1}$				$mg\ L^{-1}$										
RVFB	The system consists of two plastic tanks: the upper 'treatment tank' the lower "reservoir tank". Treatment tank : punctuate at the bottom in an even interval, the drain holes were covered by a 2 cm think layer of pebbles; 2.5 cm crushed limes-stones and dolomite followed by 12 cm of plastic filter media with 800 $m^2\ m^{-3}$ surface area, then topped with 4 cm think layer of peat. Grey water percolated through these layers to the reservoir tank then recirculated back to the upper tank at 60 L hr^{-1}. The volume of each tank is 0.2x0.35x0.5 m													soil-based systems for wastewater treatment have low construction and operational costs, easy maintenance	Gross *et al.*, 2007a
GW-Source	Synthetic grey waters prepared using shower and laundry detergents, cooking oil in addition to raw kitchen effluent in different proportions.														
Influent	339 (30.6)	46 (3.0)		3.5 (0.1)	3.5 (0.1) [f]	1.9 (0.2)	5E4 (1.3E0)		0.2x0.35 x0.5 m		2-3				
Effluent	46.6 (5.5)	3.0 (0.0)		1.8 (0.3)	1.0 (0.06) [f]	0.5 (0.1)	1.3E0 (1.1E 0)								
Planted soil filter															
HPSF															Hegemann, 1993
Influent								3.25							
Effluent	10-40														
H-CW Multi-Stage system	Consisting of settling tank, CW, SF, artificial pond									7.9-14.5					Fittschen and Niemczynowicz, 1997
GW-Source	Grey water of Ecovillage Toarp in Sweden														
Three-chamber ST									5.6						
Influent															
Effluent	164.6 [b]	361			18.1	3.9	5.4E5- 3.3E6 [g]								
H-CW	Water flows horizontally, a reed bed planted with *Phragmites communis*								$600m^2$x0.6m		14				
Influent	164.6 [b]	361			18.1	3.9	5.4E5- 3.3E6 [g]								
Effluent	< 4.9 [b]	46.4			7.4	1.4	1.E2-3.3E4 [g]								
VFSF									$300m^2$ x 0.8m		2-4 [h]				
Influent	< 4.9 [b]	46.4			7.4	1.4	1.E2-3.3E4 [g]								
Effluent	< 4.3 [b]	43.3			1.3	0.79	0- 2E1 [g]								
AP	Collected Storm Water								$130m^2$x1.0 m						
Influent	< 4.3 [b]	43.3			1.3	0.79									
Effluent	< 4.3 [b]	56.3			< 0.43	0.23									

a; BOD: b; BOD_7:c; faecal coliform: d; the units of the influent and effluent is g $m^{-2}\ day^{-1}$: e; COD based on Mn measurement: f; ammonia: g; thermostable coliform bacteria: h; hours: i; TOC=Total Organic Carbon: j; E. Coli.

Table 2.3. Soil Filters and planted soil filters "CWs" combined with others units treating grey water. Influent and effluent qualities and operation conditions (continued).

Treatments	BOD[a]	COD	SS	Turbidity	TN	TP	TC	Surface Areas	Volume	Hydraulic	HRT	Chemical	Energy	Cost	References
	mg L^{-1}			NTU	mg L^{-1}		cfu/100 ml	m^2 Cap^{-1}	m^3	Load m^3 d^{-1}	d^{-1}				
H-CW	Horizontal Flow Reed Bed (**HFRB**); water flows continuously to Sand/soil/compost mix media (≤1 mm diameter), planted with *Phragmites australis*														
GW-Source	Bathroom sinks, baths and showers of 18-student flat on the Grandfield University Campus														
Influent	20 (11)	87 (38)	29 (32)	19.6 (14)			2.51E+05± 6.31E+00		6 m^2 x 0.7 m	0.48	2.1				
Effluent	2 (1)	29 (9)	9 (8)	16.9 (16)			3.981E+02								Winward *et al*., 2008
GW-Source	Bathroom sinks, baths and showers of 18-student flat on the Grandfield University Campus (real)+ 10 % (v/v) mixtures of Tesco Value Shampoo in tap water (synthetic). The ratio of real:synthetic is 1:55.														
Influent	164 (39)	495 (192)	93 (66)	67.4 (92.3)			2.00E+07± 3.16E+00		6 m^2 x 0.7 m	0.48	2.1				
Effluent	57 (32)	124 (50)	34 (15)	12.3 (13.4)			2.51E+04								
Planted SSrF ≅CW	The Panted SSrF means SSrF-CW of both media were tested further to be operated as reed bed and planted with *Coix-lacryma-jobi.*														
GW-Source	Monteverde Institute's grey water.														
Influent	216 ± 55.3	Dry-season					8.5 E7 ± 5.7E7 [c]								
Effluents										5					
SSrF -CW (PET)	4.0 ± 3.6						2.2 E3 ± 2.4E3 [c]		0.075		8.5				
SSrF-CW (Rock)	7.0 ± 3.6						2.4 E3 ± 4.7 E3 [c]		0.075		5.1				
Influent	155 ± 14.1	Dry-season					3.8E6 ± 1.0E6 [c]							Operation and maintenance costs are generally low	Dallas, *et al*., 2005
Effluents										10					
SSrF -CW (PET)	13 ± 1.9						2.1E3 ± 1.7E3 [c]		0.075		4.2				
SSrF-CW (Rock)	18 ± 0.6						2.6E5 ±5.1E5 [c]		0.075		2.5				
Influent	290 ± 36.2	Wet-season					1.0E8±1.0E8 [c]								
Effluents										5					
SSrF -CW (PET)	10 ± 6.4						1.5E3 ± 1.7E3 [c]		0.075		4.8				
SSrF-CW (Rock)	18 ± 4.0								0.075		2.4				
Influent	285 ± 106.1	Wet-season													
Effluents										10					
SSrF -CW (PET)	26 ± 5.6								0.075		2.8				
SSrF-CW (Rock)	26 ± 1.6								0.075		1.7				

a; BOD: b; h; hours: i; TOC=Total Organic Carbon: j; E. BOD$_7$:c; faecal coliform: d; the units of the influent and effluent is g m^{-2} day^{-1}: e; COD based on Mn measurement: f; ammonia: g; thermostable coliform bacteria: Coli.

Table 2.3. Soil Filters and planted soil filters "CWs" combined with others units treating grey water. Influent and effluent qualities and operation conditions (continued)

Treatments	BOD_5	COD	SS	Turbidity	TN	TP	TC	Surface Areas	Volume	Hydraulic	HRT	Chemical	Energy	Cost	References
	mg L^{-1}			NTU	mg L^{-1}		cfu/100 ml	m^2 Cap^{-1}	m^3	Load m^3 d^{-1}	d^{-1}				
V-CW Multi-Stage system	Consists consequently of a feeding tank, two-chambered polyethylene settling tank, V-CW and last storage tank									0.5				Total cost 63 USD/m^2. Depends also on available land, negligible operational cost. Minimizing the construction cost possible.	Shrestha *et al.*, 2001a;b
GW-Source	grey water of 7 person household; grey water is hydro-mechanically flushed														
Feeding Tank									0.2						
Two-chambered ST									0.5						
Influent	100-400	177-687	52-188	0.5- 6	3.66-25.7[f]										
Effluent															
V-CW	Intermittent vertical water flow, planted with reed; *Phragmites Karka*, filled with coarse sand and grey water is hydro-mechanically flushed 3 to 4 times per day							0.86							
Effluent															
Storage Tank									0.7						
Effluent	0-12	6.8-72			0.02-1.98 [f]										
V-CW Multi-Stage system	Consists of consequently of three settling tanks, V-CW, and storage tank , for pathogens removal; TiO_2 +UV radiation were used in lab experiment														Li *et al.*, 2003
GW-Source	Grey water of Luebeck settlement in Germany														
Three ST	Removes grits, solids and grease														
Influent															
Effluent		258-354 80-94 [i]			9.7-16.6	5.2-9.6	7.5E3-2.6E5 [j]								
V-CW	Intermittent vertical water flow, filled with gravels are between 4-8 mm							2							
Influent		80-94 [i]			9.7-16.6	5.2-9.6	7.5E3 -2.6E5 [j]								
Effluent															
Storage Tank															
Influent															
Effluent		< 5-28 [i]			1.18- 5	5.6-6.8	3.3 E2-2.6 E4 [j]								
Disinfection	The optimum TiO_2 dosage (among 1, 3, 5 and 10 g L^{-1}) and UV radiation time (among 1, 2, 3, 4, 5, 6 and 19 hr) were respectively 5 g L^{-1} and 3 hr; the result reported in this table is for 10 g L^{-1} TiO_2 with 3 hrs UV radiation.														
Influent							58 [j]								
Effluent							1[j]								

a; BOD: b; BOD_7:c; faecal coliform: d; the units of the influent and effluent is g m^{-2} day^{-1}: e; COD based on Mn measurement: f; ammonia: g; thermostable coliform bacteria: h; hours: i; TOC=Total Organic Carbon: j; E. Coli.

Table 2.3. Soil Filters and planted soil filters "CWs" combined with others units treating grey water. Influent and effluent qualities and operation conditions (continued).

Treatments	BOD_5	COD	SS	Turbidity	NO_3-N	TN	TP	TC	Volume	Hydraulic Load	HRT	Chemical	Energy	Cost	References
	mg L^{-1}			NTU	mg L^{-1}			cfu/100 ml	m^3	$m^3 d^{-1}$	d^{-1}				
V-CW	Vertical Flow Reed Bed (**VFRB**); water flows to Sand/soil/compost mix media (≤1 mm diameter); 10 batches 2 hr HRT per batch, planted with *Phragmites australis*.												Based on pilot scale 0.4kWh m^{-3} less than GRWRS, HFRB, MBR and MCR		Winward *et al*., 2008
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus														
Influent	20 (11)	87 (38)	29 (32)	19.6 (14)				2.51E+05 (6.31E+00)	6 m^2 x 0.7 m	0.48	20 [h]				
Effluent	1 (1)	21 (6)	2 (2)	8.1 (10)				5.01E+0							
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus (real)+10 % (v/v) mixtures of Tesco Value Shampoo in tap water (synthetic). The ratio of real: synthetic is 1: 55.														
Influent	164 (39)	495 (192)	93 (66)	67.4 (92.3)				2.00E+07 (3.16E+00)	6 m^2 x 0.7 m	0.48	20 [h]				
Effluent	5 (6)	31 (30)	10 (6)	2.2 (1.5)				1.26E+03							
RVFCW	GW flow to 40 L primary ST, and then to the root-zone of Recycled V-CW. Consists of two tanks the upper V-CW and the lower 500 L reservoir. CW depth 0.5 m and 1 m^2 perforated surface areas, the layers are 3 in the bottom 5 cm of lime stone pebble, followed by 30 cm of tuff or plastic media, and the upper layer 15 cm planted org soil. The trickled in the lower tank recycled continually to upper filter. Recycling rate 390 L hr^{-1}; the average time the water cycles in the system is 8-24 hr average retention time, i.e. 7 and 21 times the water penetrated the bed. The effluent discharge to 40 L secondary ST and then is used for irrigation.													US 600$ invest ment cost and 100 labor and mainte nance cost.	Gross *et al*., 2007b
GW-Source	Estimated 450 L d^{-1} of shower, laundry and sink wastewaters from a 5-member household family.														
Influent	466 (66)	839(47)	158 (30)		3 (1.3)	34.3(2.6)	22.8 (1.8)	5E7(2E7) [c]			8-24 [h]				
Effluent	0.7 (0.3)	157(62)	3 (1)		8.6 (4.3)	10.8 (3.4)	6.6 (1.1)	2E5 (1E5) [c]							
GRWRS	Green roof water recycling system (**GRWRS**): water flows to continuously five rows of shallow troughs (depth), Optiroc expanded clay media (10mm diameter) topped with gravel chippings (20mm diameter), planted with a variety of aquatic plants. Baffles and weirs create plug flow and additional aeration for 1 hr per day.														Winward *et al*., 2008
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus														
Influent	20 (11)	87 (38)	29 (32)	19.6 (14)				2.51E+05 ± 6.31E+00	1.2 m^2x 0.1 m	0.48	2.1				
Effluent	2 (1)	19 (8)	3 (3)	0.8 (2)				5.01E+03							
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus (real)+ 10 % (v/v) mixtures of Tesco Value Shampoo in tap water (synthetic). The ratio of real: synthetic is 1: 55.														
Influent	164 (39)	495 (192)	93 (66)	67.4 (92.3)				2.00E+07 ± 3.16E+00	1.2 m^2 x 0.1 m	0.48	2.1				
Effluent	80 (38)	159 (64)	20 (8)	28.8 (10.7)				3.16E+01							

a; BOD: b; BOD_7:c; faecal coli form: d; the units of the influent and effluent is g m^{-2} day^{-1}: e; COD based on Mn measurement: f; ammonia: g; thermostable coliform bacteria: h; hours: i; TOC=Total Organic Carbon: j; E. Coli.

2.3.2.2 Biofilters

The tested biofilters can be classified as macro- and membrane-biofilters. Macro-biofilters can be further classified into two sub-categories: attached and suspended. Membrane sub-categories are submerged and side-stream. Attached Macro-Biofilters have been tested, namely Biological Aerated Filters (BAF), which combine depth filtration through a porous media bed with a fixed film biological reactor (Jefferson *et al*., 1999; 2000 and 2001). Anaerobic Bio-Filters (AnBF) and Bed Submerged Biofilters (BSB) combine macro-filtration with an activated sludge system (Imura *et al*., 1995). Submerged or side-stream Membrane Biological Reactors (MBR) combine membrane-filtration with an activated sludge system. Jefferson *et al*. (1999; 2000; 2001) and Windward *et al*. (2008) tested a submerged MBR. A submerged Fibrous M(F)BR was tested by Jefferson *et al*. (2000) and Merz *et al*. (2007). A side-stream tubular U(T)BR was tested by Anderson *et al*. (2002) and an M(T)BR tested by Friedler *et al*. (2006a). All filters, except the anaerobic filter, are supplied with an external oxygen source.

The performance differences of micro and macro membrane biofilters are presented in Table 2.4. The micro systems produce better effluent qualities than a macro biofilter with an internal media structure of 2.36-4.75 mm and a 50% voidage (Jefferson *et al*., 2000; 2001). The performance tests of the biological filters show that the effluent qualities of biological filters are dependent on the porosity of the filtration media and/or the HRT (Table 2.4), which is similar to the conclusion for the physical filters and CWs. Biological Filters' nitrogen and phosphorous removal performance were not tested, except by Merz *et al*. (2007) who reported 63% and 19% removal, respectively (Table 2.4). Jefferson *et al*. (1999 and 2000) proved that the removal performance of MBR, M(F)BR and BAF are dependent on the internal system structure and not on the organic load. The MBR performance is not affected significantly by increasing the temperature and biomass concentration, i.e. 11℃ and 0.4 g VSS L^{-1} compared with 20℃ and 1.4 g VSS L^{-1} (Merz *et al*., 2007). Furthermore, the performance is not affected by the sludge age in the range of 4 to 20 days (Lesjean and Gnirss, 2006). Imura *et al*. (1995)

changed the volumes of the AnBF and BSB and other units in the system, consequently changing the HRT, which improved the performance of the total system. A disinfection stage is inevitable for BAF and MBR to guarantee risk free effluents (Jefferson *et al*., 1999; 2000; Friedler *et al*., 2006a; Merz *et al*., 2007).

Biological filters show problems with cleaning, membrane fouling and/or operational costs. Table 2.4 shows the operational conditions of a BAF operated by Jefferson *et al*. (2000), who reported that back washing to eliminate contamination accounted for over 20% of the total flow. This persistent contamination results from surface binding by macro-solids such as hair and precipitated soaps. Jefferson *et al*. (2000; 2001) shows that submerged MBR pilot plants treating artificial, low suspended solids, grey water suffer from fouling and need frequent cleaning. In contrast, the side-stream MBR pilot plants tested by Anderson *et al*. (2002) for treatment of laundry wastewater had limited fouling problems. Furthermore, Melin *et al*. (2006) reported for submerged MBRs, treating municipal wastewater at full scale, little fouling problems compared with a pilot plant. Applying sub-critical flux conditions allows a stable flux that reduces the typical operational and maintenance cost (Jefferson *et al*., 2000). Merz *et al*. (2007) reported that the MBR investment and operational costs are high and thus less affordable for developing countries. Fletcher and Judd (2007) compared the costs of MBRs with SAF, RBCs, SBR, TF and BAF systems. The capital costs, as well as the de-sludging and maintenance costs, are considered to be for the different systems similar. But the MBRs require four times the energy of the conventional systems. Fletcher and Judd (2007) justified their conclusions on a study of pre-fabricated units installed on-site, treating medium-strength municipal wastewaters of 6-20 persons.

Pre-treatment of biofilter's influent is advisable for MBR (Melin *et al*., 2006) and optional for other filters. Influent pre-treatment reduces blockage, fouling problems, and cleaning frequency. It also produces better effluent qualities in some cases (i.e. BAF). Applied pre-treatment techniques are a primary settling tank prior to biofilter (Imura *et al*., 1995), and screens (1x1 cm followed by 1x1 mm) prior to an MBR (Merz *et al*., 2007). Post-treatment of anaerobic filter effluent improves its qualities (Table 2.4), in terms of BOD, TN, TP, SS and/or pathogens (Imura *et al*., 1995).

Table 2.4. Modified Filters: Biofilters combined with others units treating grey water. Influent and effluent qualities, and operation conditions.

Treatments	BOD_5 mg L^{-1}		COD mg L^{-1}	SS mg L^{-1}	Turbidity NTU	TN	TP	TC cfu/100 ml	Volume m^{-3}	HRT Hr	Hydraulic Load L m^{-2} hr^{-1}	Organic Load Kg BOD_5 m^{-3} d^{-1}	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}				mg L^{-1}										
Macro-Biofilters																
Multi-Stage system	Consists consequently of settling tank, anaerobic filter, bed submerged biofilter, settling tank and disinfection. The water level was left variable to control the high fluctuation in the daily inflow.															Imura *et al.*, 1995
GW-Source	Japan Single house Cooking, bathing and washing six person single house								3.67-4.22	50-58						
ST	Separation bulky solids and oil								0.72-0.83							
Influent[a]	195			123		32.3	3.9									
Effluent	40-130			20 - 136		4-18										
AnBF	Spherical, reticulate, small mesh flat plastic filter media was placed to prevent short circuiting								1.28-1.48							
Effluent	17-74			23-34		3-11										
SBF	An 80 L min^{-1} blower was aerating and mixing (biological processes) the content; water + microorganisms								1.24-1.43							
Effluent	5.4-9.8			3-13		1.3-10	0.9-1.6									
ST									0.37-0.46							
Disinfection	solid-chlorine								0.019							
Effluent	5.4-9.8			3-13		1.3-10	0.9-1.6									
BAF	Small footprint. diameter 0.165 m, effective depth 1.64 m (used in 2000 article) and 1.75 m (used in 2001article), plastic media size range 2.36 to 4.75 mm, voidage 50% , supplied air 20 L min^{-1}. Back washing cycle: water 3 min, 20 L min^{-1} and air 15 min 20 L min^{-1}.															
GW-Source	Artificial Grey water; mimicking grey water discharge from multi-story building									2.7- 0.46		1.09±0.73				Jefferson *et al.*, 1999 and 2001
Influent	41.2±30			52±58				2E0-2E7								
Effluent	4.3±4.1			5.9-5.5	3.2±8.9			4E0-1E5								
GW-Source	Artificial Grey water; mimicking grey water discharge from multi-story building										400	0.45-7				Jefferson *et al.*, 2000
Influent	9-100			63±40				2E2-5E7								
Effluent	<32			7±9	<2-7.5			1E1-2E5								
GW-Source	Primary sewage + (Artificial Grey water; mimicking grey water discharge from multi-story building)									2.0- 0.4		1.2±1.1				Jefferson *et al.*, 2001
Influent								2E2-5E8								
Effluent					2.8±3.9			1E3±2E5								
GW-Source	Primary sewage									2.8- 0.4		2.45±1.0				
Influent			323±102	148±77				1.0E6-6E7								
Effluent			52.3±28	18±20.	12.6±27.3			1E4±1E7								

a; standards values, reported by "the institute of public health, ministry of health and welfare, 1999: b; faecal coliform: c; reported as BOD: ND; Not Detectable.

Table 2.4. Modified Filters: Biofilters combined with others units treating grey water. Influent and effluent qualities, and operation conditions (continued).

Treatments	BOD_5 mg L^{-1}		COD mg L^{-1}	SS mg L^{-1}	Turbidity NTU	TN	TP	TC cfu/100 ml	HRT hr	Hydraulic Load L m^{-2} hr^{-1}	Organic Load Kg BOD_5 m^{-3} d^{-1}	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}				mg L^{-1}									
Micro-Biofilters															
Submerged MBR															
MBR	Small footprint; submerged bioreactor. working volume 0.035 m^3(used in 2000 article) and 0.066 m^3(used in 2001 article), 2 membrane plates with surface area 0.24 m^2, pore size 0.4 µm and supplied air are 15 L min^{-1}, the immerse depth 0.6 m													Substantial capital cost	Jefferson *et al*., 1999; 2001
GW-Source	Artificial Grey water; mimicking grey water discharge from multi-story building								31.5- 3.4		0.14±0.07				
Influent	41.2±30		120±74.7					2E0±2E7							
Effluent	1.1±1.6		9.6±7.4		0.32±0.28			2E0-2E1							
GW-Source	Artificial Grey water; mimicking grey water discharge from multi-story building											acid cleaning	4.0E3 kWh kg $product^{-1}$	sub critical flux lowers membrane replacement cost, but significantly increases investment cost compares with BAF	Jefferson *et al*., 2000
Influent	9-100							2E2-5E7							
Effluent	<10				<2			≤1E1							
GW-Source	Primary sewage + (Artificial Grey water mimics grey water of multi-story building)								29.2- 5.5		0.16±0.08				Jefferson *et al*., 2001
Influent			144.±85.7	62.6±40				2E2±5E8							
Effluent			10.63±5.5	3.6±3.7	0.4±0.28			1E0-1E3							
GW-Source	Primary sewage														Jefferson *et al*., 2001
Influent			323(102)	148±77				1.0E6±6E7	34.2- 3.1		0.18±0.11				
Effluent			15.5(7.5)	18±20.2	12.6±27.3			1E0<2E2							
M(F)BR	Type-Submerged; working volume 0.0073 m^3, membrane surface area 0.04 m^2m^{-3}, fibre voidage membrane 0.04 µm, voidage 97% and supplied oxygen are 0.00973 L min^{-1}														Jefferson *et al*., 2000
GW-Source	Artificial Grey water; mimicking grey water discharge from multi-story building									1220	0.22-1.5				
Influent	9-100			52.±58				2E2-5E7							
Effluent	<18				<2-15			1E1-2E4							
M(F)BR	A small footprint submerged MBR, 3 Litre lab-scale, hollow UF fibre membrane, membrane area 400 cm^2, pore size 0.1 µm, trans-membrane pressure (73-402) average 249 mbar, average 1.3 (0.42-1.85) mg MLSS and 0.94 (0.26-1.32) mg MLVSS mg L^{-1}. Organic loading rate 0.16(0.09-0.21) kg COD m^{-3} d^{-1}. F/M 256(118-390) mg COD^{-1} g_{vss}^{-1}. Grey water first filtered through 1x1 cm followed by 1x1 mm screen. Air supply 0.32 m^3 hr^{-1}. Operational phase 45 min with permeate and 15 min relaxation phase. Temperature increased from 9-20°C.													Investment and operational cost high, expensive for developing countries	Merz *et al*., 2007
GW-source	Shower wastewater from sports and leisure club in Rabat-Morocco.								13 (9-18)	7-11 (8)					
Influent	59 (13)				29 (11)	15.2 (4.5)	1.6 (0.5)								
Effluent	4 (1.2)				0.5 (0.3)	5.7 (1.9)	1.3 (0.5)								

a; standards values, reported by "the institute of public health, ministry of health and welfare, 1999: b; faecal coliform: c; reported as BOD: ND; Not Detectable.

Table 2.4. Modified Filters: Biofilters combined with others units treating grey water. Influent and effluent qualities, and operation conditions (continued).

Treatments	BOD_5 mg L^{-1}		COD mg L^{-1}		SS mg L^{-1}	Turbidity NTU	TN	TP	TC cfu/100 ml	Volume m^3	HRT hr	Hydraulic Load L m^{-2} hr^{-1}	Organic Load Kg BOD_5 $m^{-3}d^{-1}$	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}	COD_{tot}	COD_{dis}			mgL^{-1}										
MBR	Two joint 34 L reactors, each fitted with two submerged A4 flat sheet Kubota membranes, 0.4 µm nominal pore size, seeded with activated sludge biomass, the aeration: 5 L min^{-1}, also 10 L min^{-1} air lift for generating recirculation loop, the HLR 168 L d^{-1} and solids retention time 68 d.																Winward *et al*., 2008
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus																
Influent	20 (11)		87(38)		29 (32)	19.6 (14)			2.5E5±6.3E0	2 m^2 x 0.034 m	9.7	15					
Effluent	1 (1)		47(13)		ND (ND)	0.2 (0.1)			≤4.0E+01								
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students flat on the Grandfield University Campus (real)+ 10 % (v/v) mixtures of Tesco Value Shampoo in tap water (synthetic). The ratio of real: synthetic is 1: 55.																
Influent	164 (39)		495(192)		93 (66)	67.4 (92.3)			2.0E7±3.2E0	2 m^2 x 0.034 m	9.7	15					
Effluent	1 (2)		53(24)		1 (2)	0.2 (0.1)			4.4±E+01								
Side-stream MBR																	
U(T)BR	**Side-stream** MBR consisted of 0.7 m^3 biological reactor and UF membrane unit; the membrane is polyacrylonitrile tubular membrane, 7 in one module, 0.95 m^2 the total membrane surface area, MWCO 500 kDa. Trans-membrane pressure 2 bar, cross-flow velocity 4 m s^{-1}. 1 day HRT is enough when sludge concentration is 10 g MLSS L^{-1}																Anderson *et al*., 2002
GW-source	Laundry wastewater resulted of washing garments coming from hotels and restaurants																
Influent	610-680		1700				5.3-12	20-48		0.7	24						
Effluent	< 2		50														
EB+M(F)BR	1 mm fine screen followed by an equalization basin followed by MF.													Hypochlorite solution to treat the membranes			Friedler *et al*., 2006a
GW-Source	Bath, shower and washbasin streams discharge from seven student apartments at Technion campus; accommodate married students																
Equalization basin	Preceding (Equalization Basin) there was 1mm square shaped screen to remove gross solids. EB volume 330L maximum residence time is < 1 to maximum 10 hr																
Influent																	
Effluent	69 (33)	36(20)	211(141)	108 (47)	92 (115)	65 (68)			3.4E5 (1.8E5)[b]								
M(T)BR	Aeration basin; with 0.2 m^3 and HRT 5-8 hr, total head of 3 atm creates cross flow velocity 4.0 m s^{-1}, an the (side-stream) membrane unit compromises of 4 parallel tubular polyethylene; MWCO 1000,000 Da, diameter 0.0155 m total surface area 0.34 m^2 , permeate flux varies from 0.0588 m^3 m^{-2} h^{-1} to 0.0382 equivalent to 20 L hr^{-1}. Hypochlorite is used to wash the membrane periodically, then by tap water several time; sludge age (15-20) day																
Influent	69 (33)	36(20)	211(141)	108 (47)	92 (115)	65 (68)			3.4E5 (1.8E5)[b]								
Effluent	1.1 (1.7)	0.5(0)	40(16)	37 (14)	12 (8)	0.2 (0.1)			27 (56)[b]								

a; standards values, reported by "the institute of public health, ministry of health and welfare, 1999: b; faecal coliform: c; reported as BOD: ND; Not Detectable.

2.3.2.3 Chemfilters

A Membrane Tubular Chemical Reactor (M(T)CR) combines a Photocatalysis Oxidation Reactor (PCOR) and a side-stream Membrane-Tubular -Filtration (M(T)F) unit. The PCOR oxidizes organic matter by means of TiO_2 in the presence of ultraviolet light and oxygen (Rivero *et al*., 2006; Windward *et al*., 2008); the TiO_2 is separated from the liquid phase in the subsequent M(T)F unit. The M(T)CR's performance depends on the membrane pore size, in addition to permeate flux, TiO_2-dose and mixing (Rivero *et al*., 2006). With proper optimization of the latter factors, an M(T)CR produces stable sludge and a stable effluent quality in terms of turbidity, BOD and Total Coliform, which can comply with the most conservative standards (Table 2.5). However, Windward *et al*. (2008) reported a high fluctuation of effluents' COD. Rivero *et al*. (2006) stated that a full recovery of the TiO_2 could be achieved and the process could run continuously. However, they noted that further studies are required to determine the efficiency under critical flux conditions.

Although an MCR overcomes the MF primary sludge production, the issues of concerns are high operational costs, membrane fouling, recovery of TiO_2 (Li *et al*., 2003) and effluents' COD high fluctuation (Windward *et al*., 2008). Pre-treatment of grey water with high SS is required before feeding the MCR. Influent pre-treatment reduces the blockage, the fouling problems and the cleaning frequency. Post-treatment of M(T)CR's effluent, i.e. recovery of the catalysts and reduction of the turbidity, is optional and depends on the PCOR treatment efficiency and the M(T)F porosity.

Table 2.5. Modified Filters: Chemfilters combined with others units treating grey water. Influent and effluent qualities, and operation conditions.

Treatments	BOD_5 mg L^{-1}		COD mg L^{-1}	SS mg L^{-1}	Turbidity NTU	TN	TP	TC cfu/100 ml	Volume m^3	HRT hr	Hydraulic Load L m^{-2} hr^{-1}	Chemical	Energy	Cost	References
	BOD_{tot}	BOD_{dis}				mg L^{-1}									
M(T)CR	Two units: Photo-catalytic Oxidation Reactor (PCOR) followed by M(T)F unit														
GW-Source	Shower wastewater														
PCOR	8 L stainless steel tank reactor with 25W UVC lamps (Philips), Hombikat UV-100 TiO_2 used and air source with a velocity 0.5-1.25 m s^{-1} to keep the slurry in suspension, TiO_2 UV-100 tested dosages 5 and 10 mg L^{-1}. The achieved minimum and maximum values are reported for he effluents														
Influent	114 – 135[c]		252 - 324		15.6-18.7										Rivero *et al.*, 2006
M(T)F	Membrane characteristics; 1 m length, 10 Lumen the internal diameter 5 mm, pore size 0.05 µm, total area 0.157 m^2, and cross-section area 200 mm^2, and the tested flux 15 and 55 L m^2 hr^{-1}.														
Effluent	2 - 17[c]		56 - 72		0.35-3.57			Meets WHO standards							
M(T)CR	9 L PCOR with four submerged 25W UV-C lamps and side stream air-lift M(T)F														
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus														
PCOR															
Influent	20 (11)		87 (38)	29 (32)	19.6 (14)			2.5E5±6.3E0	0.009	3.8	15				
M(T)F	0.05 µm nominal pore size, 5 g L^{-1} titanium dioxide, Aeration: 5 L min^{-1} and air left generates recirculation loop 10 L min^{-1}, HLR 57 L d^{-1}.														
Effluent	3 (2)		43 (14)	ND (ND)	0.1 (0.0)			ND				TiO_2			Winward *et al.*, 2008
GW-Source	Bathroom sinks, baths and showers of a flat for 18 students on the Grandfield University Campus (real)+ 10 % (v/v) mixtures of Tesco Value Shampoo in tap water (synthetic). The ratio of real: synthetic is 1: 55.														
PCOR															
Influent	164 (39)		495 (192)	93 (66)	67.4 (92.3)			2.0E7±3.2E0	0.009	3.8	15				
M(T)F															
Effluent	10 (8)		78 (18)	2 (1)	0.72 (1.1)			ND							

a; standards values, reported by "the institute of public health, ministry of health and welfare, 1999: b; faecal coliform: c; reported as BOD: ND; Not Detectable.

2.3.3 Biological treatment

Biological treatment of grey water followed by disinfection to guarantee risk-free effluent is recommended (Nolde, 1999). Such a system can be optimized for a minimal energy and maintenance (Nolde, 1999). Otterpohl *et al*. (1999) recommend application of attached biomass and avoiding activated sludge systems. Both systems have been examined; Nolde (1999) and Friedler *et al*. (2006a) examined attached systems, and Shine *et al*. (1998), Hills *et al*. (2001) and Hernandez *et al*. (2007) tested activated sludge systems. Elmitwalli and Otterpohl (2007) and Hernandez *et al*. (2007; 2008) showed the potential of UASB-systems for anaerobic pre-treatment of grey waters.

2.3.3.1 Aerobic attached growth processes

Aerobic attached growth processes such as the Fluidized-Bed Reactor (FBR) was examined by Nolde (1999) and the Rotating Biological Contactors (RBCs) was examined by Nolde (1999) and Friedler *et al*. (2005 and 2006a). Table 2.6 shows that a two-stage aerobic FBR and a multi-stage RBCs produce effluent qualities in terms of BOD similar to the MBR effluent quality. An RBC energy consumption and maintenance costs are less than that of an MBR (Friedler and Hadari, 2006b). However, both an FBR and an RBC are not successful in pathogen removal. Friedler *et al*. (2005 and 2006a) reported that an RBC removes BOD more efficiently than COD, which is attributed to the presence of non or slowly biodegradable organic matter in the grey water.

Nolde (1999) and Friedler *et al*. (2005 and 2006a) did not encounter any problems while treating grey water in an FBR and an RBC, except for the low removal of COD reported by Friedler *et al*. (2005 and 2006a). However, it can be expected that if a clarifying tank is not used, the SS in the RBC effluent may cause a failure in the disinfection process. Combining an RBC with primary and secondary sedimentation tanks (Table 2.6) leads to a reduction in the weekly maintenance time to 0.2 hour and an energy requirement for treatment, disinfection and service water of less than 1.5 $KWhm^{-3}$ **(**Nolde, 1999). Costs for the treatment of produced primary sludge are however not taken into consideration.

2.3.3.2 Aerobic suspended growth processes

The Sequencing Batch Reactor (SBR) operated by Shine *et al*. (1998) produced an unstable effluent quality in terms of SS, but stable regarding BOD. The BOD values were close to the effluent BOD of FBR, RBCs and MBR (Table 2.6). In agreement, Akunna and Shepherd (2001) reported RBCs and SBR, treating small communities' domestic wastewater, produced almost the same effluent quality in terms of BOD i.e. 1-15 and 2-22 mg L^{-1} respectively. Furthermore RBCs and SBR have the same capital and running costs in terms of energy consumption. Shine *et al*. (1998) optimized SBR operational modes to achieve the highest nitrogen removal through testing step-feeding mode, cyclic aeration mode and conventional mode. The results (Table 2.6) illustrate that an SBR operated at an HRT of 12 hours produces an effluent in terms of nitrogen comparable to that of CW operated at an HRT of 14 days (Table 2.3). An SBR treating a mixture of black and grey water reduces the nitrogen content from 20-59 mg NH_3-N L^{-1} to 5-25 mg NH_3-N L^{-1} (Akunna and Shepherd, 2001). However, an RBC produces a better effluent quality, viz. 0-5 mg NH_3-N L^{-1} (Akunna and Shepherd, 2001). Hernandez *et al*. (2007 and 2008) reported production of a low amount of sludge with good sedimentation characteristics when operating an SBR for the treatment of grey water at an HRT of 12 hours and 1 day. This contradicts Otterpohl *et al*. (1999) who recommended that an activated sludge processes should be avoided when treating grey water, due to risks posed by lack of nutrients. Akunna and Shepherd, (2001) applied both an SBR and an RBC preceded by a one and two primary settlers, respectively. An SBR, in comparison with an RBC, is more resistant to variation in the inflow quality and quantity (Akunna and Shepherd, 2001), which is an important characteristic of grey water (Butler *et al*., 1995 and Abu Ghunmi *et al*., 2008). Shine *et al*. (1998) stressed that the stable performance of the SBR, except for the SS, could not have been achieved without using an equalization basin. This is contradicted by Hernandez *et al*. (2007; 2008) who did not apply an equalization basin and still reported a stable performance of grey water treatment in an SBR. For post treatment, an MF coped with the variation in the SBR effluent and produced a stable effluent in terms of SS (Shine *et al*., 1998) (Tables 2.2 and 2.6).

Table 2.6. Aerobic biological processes: suspended and attached combined with others units treating grey water. Influent and effluent qualities, and operation conditions.

Treatments	BOD_5 mg L^{-1}	COD mg L^{-1}		Turbidity NTU	SS mgL^{-1}	TN	NH_4-N	NO_3-N	TP mg L^{-1}	TC	FC	HRT hr	Hydraulic Load $m^3 d^{-1}$	Chemical	Energy	cost	References
		COD_{tot}	COD_{dis}			mg L^{-1}				cfu/100 ml							
Attached Biological Treatment System																	
Multi-Stage System	Consists of consequently of Settling tank, followed RBC then secondary settling tank and UV disinfection												2.1-2.45			RBC low energy and maintenance costs are a challenge especially for small grey water plants	Nolde, 1999
GW-Source	Multi-Story Building; Showers, bath and hand-washing basin.																
ST																	
Effluent	50-125 [a]	100-430				5-10			0.2-0.6	1E2-1E6	1E1-1E6						
RBCs																	
Effluent	<5 [a]																
Disinfection	Ultra Violet																
Effluent										2E-2-2E0	2E-2-1E-1						
Multi-Stage System	Consists of equalization tank, RBC, sedimentation basin (SB), pre-filtration storage tank (PFST), sand filtration (SF) and disinfection											17.5			RBC consumes little energy		Friedler *et al.*, 2005 and 2006a [c]
GW-Source	Bath, shower and washbasin streams discharge from seven student apartments at Technion campus; accommodates married students																
Equalization Basin	1 mm square shaped screen removes gross solids followed by (EB); EB volume 330 L <1 hr minimum HRT and 10 hr the maximum											10					
Effluent	59 (29.6) [b]	158 (60)	110(54)	33 (23.3)	43 (25.1)						5.6E5 (6.5E5)						
RBCs+SB	RBC: Two basins each volume 15 L, the shaft carries the discs, total surface areas is 1 m^2, and is perpendicular to the flow rotates in a 1-1.5 rpm, means residence time (MRT) is 2 hr per basin. SB: Volume is 7.5 L, MRT 1 hr, sludge is removed manually, 40% of the disc surface areas is submersed											4+1					
Effluent	6.6 (9.45) [b]	46 (19.4)	47 (27)	1.9 (2.3)	16 (14.5)						9.7E3 (3.0E3)						
PFST	To regulates between SB (continues flow) and SF (batch flow), MRT 2.2											2.2					
SF	Gravity filter, diameter 10 cm and 70 cm media depth; the medium consists of quartz and sand, porosity 36%, and supported by 5 cm gravel, the filter operated intermittently 11 times a day, 15 minutes each time, the filtration velocity is 8.33 m hr^{-1},back washed weekly after filtration 1.26 m^3.											0.25					
Influent	6.6 (9.45) [b]	46 (19)	47(27)	1.9 (2.3)	16 (14.5)						9.7E3 ±3.0E4						
Effluent	2.3 (2.43) [b]	40 (14)	40(23)	0.61 (0.38)	7.9 (4.86)						5.1E4 ± 6.6E2						
Disinfection	By Chlorination, hypochlorite is 0.2-0.25%, carried out in a batch mode and calculated for 1mg L^{-1} residual after 0 min																
Influent											5.1E4 ± 6.6E2						
Effluent											1E-1 ± 3.2E1						

a; BOD_7: b; BOD: c; the data of Friedler *et al.*, 2005 and 2006a similar; data adopted from (2005): d; BOD (total): e; BOD (dissolved):

Table 2.6. Aerobic biological processes: suspended and attached combined with others units treating grey water. Influent and effluent qualities, and operation conditions (continued).

Treatments	BOD_5 mg L^{-1}	COD mg L^{-1}		Turbidity NTU	SS mgL^{-1}	TN	NH_4-N	NO_3-N	TP mg L^{-1}	TC	FC	HRT hr	Hydraulic Load m^3 d^{-1}	Chemical	Energy	cost	References
		COD_{tot}	COD_{dis}			mg L^{-1}				cfu/100 ml							
Attached Biological Treatment System																	
FBR + disinfection	**FBR** followed by disinfection												0.03-0.04			Total costs dependent on site conditions	Nolde, 1999
GW-Source	Bath and shower of two persons single house																
FBR																	
Influent	70-300[a]	113-633								1E4-1E6	1E4-1E3						
Effluent	<5 [a]									2E-2-1E6	2E-2-1E6						
disinfection	Ultra Violet the optimum UV dose 150 and 400 J m^{-2}																
Effluent										2E-2-4E2	2E-2-1E0						
BAF	Two-stage down flow fluidized bed, diameter 0.15 m, working volume 0.036 m^3, and the height 2 m, Bed media; Lytag pulverized flue ash media was used in both columns.																Hills et al., 2001
GW-Source	Artificial Grey water; mimicking grey water discharge from hand basins																
Influent	60 [b d] 30 [e]																
Effluent	20-25[b d] 6-10 [e]																
Multi-Stage System	Consists of consequently of Equalization tank, SBR followed by MF												2				Shin *et al.*, 1998
GW-Source	Japanese Office building; Cooking, bathing, washing																
Equalization Basin	volume 2.5 m^3																
SBR	SBR volume 1.0 m^3, liquid volume 0.6 m^3 and settled sludge volume 0.4 m^3 operated in three operational modes; cyclic aeration, conventional and step-feeding. SBR feed with mixing in 1 hr; settled in 1 hr, decant in 0.5 hr and idle phase was 0.5 hr. MLSS 3579 mg L^{-1} and SVI is 160 ml g^{-1}											12					
Cyclic Aeration Mode	Aeration phase 4 hrs then anoxic phase 5 hrs (DO 2.2-5.8 mg L^{-1})											9					
Influent			30 – 194		185	29 ±11	6-23		0.4-0.7								
Effluent			20		20		< 1	4-5									
Conventional Mode	Anoxic, Aerobic and post Anoxic																
Influent			30 – 130		185	29 ±11	6-12		0.4-0.7			9					
Effluent			20		20		< 1	12-14									

a; BOD_7: b; BOD: c; the data of Friedler *et al.*, 2005 and 2006a similar; data adopted from (2005): d; BOD (total): e; BOD (dissolved):

Table 2.6. Aerobic biological processes: suspended and attached combined with others units treating grey water. Influent and effluent qualities, and operation conditions (continued).

Treatments	BOD_5 mg L^{-1}	COD mg L^{-1}		Turbidity NTU	SS mgL^{-1}	TN	NH4-N	NO3-N	P mg L^{-1}	FC cfu/100ml	HRT hr	Organic Load Kg COD m^3 d^{-1}	Chemical	Energy	Cost	References
		COD_{tot}	COD_{dis}			mg L^{-1}										
Step-feeding Mode	The inflow divided into two parts: one is used for COD removal and nitrification, the other used for supplementary carbon source for denitrification															Shin *et al*., 1998
Influent			26 – 194		185	29 ±11	6-12		0.4-0.7		9					
Effluent			20		20		< 1	1-2								
SBR	SBR volume 3.6 L, sludge inoculums was activated sludge from wastewater treatment plant in Leeuwarden. The yield for the three experiments was 0.05 g VSS g^{-1} COD removed and sludge satiability was measured in terms of SVI and it was 51 ml g^{-1}.															Hernandez *et al*., 2007
GW-Source	Eco-village in Groningen, and DESAR project Sneek											0.15 – 8.0				
Influent		425(107)-1583 (382)									24					
% removal		90														
SBR	SBR volume 3.6 L, sludge inoculums was activated sludge from wastewater treatment plant in Leeuwarden. The yield for the three experiments 0.08 g VSS g^{-1} COD removed.															Hernandez *et al*., 2008
Influent		830 (211)				53.6 (50.7)	1.2 (1.3)		7.7 (5.6)		12					
% removal		88 (8)				24 (61)	24 (174)		8 (99)							

a; BOD_7: b; BOD: c; the data of Friedler *et al*., 2005 and 2006a similar; data adopted from (2005): d; BOD (total): e; BOD (dissolved):

2.3.3.3 Anaerobic biological processes

An UASB treating grey water produced a stable effluent quality and sludge (Table 2.7) compared with a primary settling tank (Table 2.4) (Elmitwalli and Otterpohl, 2007). However, Hernandez *et al.* (2008) reported that an UASB and SBR, when operated under the same conditions, produced the same amount of sludge. Apparently in their research the UASB excess sludge was not well stabilised. Table 2.7 reveals that the UASB capacity in the treatment of grey water is limited, even with increase of the temperature and HRT. Furthermore, according to the variation in grey water temperature (Eriksson *et al*., 2002; Abu-Ghunmi *et al*., 2008), operating the UASB at 30℃, as recommended by Elmitwalli and Otterpohl (2007), cannot be achieved throughout the whole operation period even with proper insulation of UASB and connection pipes and installing it in the building cellar. Nevertheless, anaerobic pre-treatment of grey water is recommended, particularly when grey water concentrations are high. The reasons are: (1) 74% of grey water pollutants are anaerobically biodegradable (Elmitwalli and Otterpohl, 2007; Zeeman *et al*., 2008). (2) The probable deficiency in the macro-nutrients, nitrogen and/or phosphorus, to sustain microorganism's growth in aerobic treatment (Abu Ghunmi *et al*., 2008). (3) Anaerobic treatment could produce less and stable sludge that is easily dewatered. (4) No energy is required for aeration. (5) Methane is produced that can be used as energy source (Lettinga *et al*., 1980). Thus pre-treating grey water in an anaerobic unit reduces maintenance and operation cost of the overall treatment system. For example, Tandukar *et al*. (2007) reported pre- and post-treatment of domestic wastewater in an UASB and aerobic down-flow Hanging Sponge (DHS). The tested system was as efficient as an activated sludge system, more efficient in pathogen removal, produced 15 times less sludge, and was cost-effective. The removal efficiency of the anaerobic processes could be improved by incorporating filtration, e.g. AnB (Imura *et al*., 1995), or physical-chemical processes, e.g. activated carbon (Cui and Ren, 2005).

Table 2.7. Anaerobic and aerobic biological processes: UASB and UASB followed by SBR treating grey water. Influent and effluent qualities and operation conditions

Treatments	COD mg L^{-1}				N mg L^{-1}			P mg L^{-1}			Temperature °C	SRT day	HRT hr	Chemical	Energy	Cost	References
	COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}	TN	NH4-N	Particulate - N	Total	Ortho	particulate							
UASB																	
GW-Source																	
Influent											23		12				Elmitwalli and Otterpohl,2007
% removal	41																
Effluent											18		20				
% removal	31																
UASB	Storage tank (with mixing) followed by 7 L UASB reactor; diameter 7 cm, height 200 cm. up flow velocity 0.33 m hr^{-1}																
GW-Source	Grey water was collected from 'Flintenbreite' settlement in Luebeck- Germany																
Influent[a]	618 (130)	308 (162)	177 (114)	133 (36)	27.1 (3.5)	5.5 (0.8)	21.6 (3.3)	9.9 (0.3)	6.6 (1.0)	3.3 (0.7)	30	93-481	16				
% removal	64 (5)	83.5 (5.4)	51.7 (19.0)	50.9 (8.9)	29.8 (4.8)	- 70.0 (44.0)	52.8 (10.5)	15.2 (3.6)	-5.5 (11.3)	53.0 (11.2)							
Influent	647 (137)	353(131)	177 (81)	117 (40)	27.3 (4.5)	3.9 (1.0)	23.4 (4.2)	9.7 (0.7)	8.7 (1.2)	1.0 (0.5)	30	64-377	10				Elmitwalli and Otterpohl, 2007
% removal	52.3 (4.8)	79.4 (7.6)	29.2 (19.8)	30.3 (7.6)	21.7 (5.2)	15.0 (35.5)	31.2 (13.1)	17.4 (5.1)	14.5 (9.3)	43.2 (33.3)							
Influent[a]	682 (106)	310 (86)	236 (90)	136 (33)		3.5 (1.6)		9.9 (0.8)	8.4 (0.1)	1.5 (0.3)	30	27-338	6				
% removal	52.0 (12)	67.6 (17.2)	37.1 (17.5)	34.8 (20.5)		47.2 (53.6)		20.6 (7.1)	18.7 (1.3)	30.2 (4.0)							
UASB	3.6 and 5.0 L																
GW-Source	Eco-village in Groningen, and DESAR project Sneek																Hernandez *et al.*, 2007
Influent	200-2700	50-2100	135-402	135-722							20-30		12-24				
% removal	40 (40)	56	33	25													
UASB	3.6 L																
Influent	827 (204)	385 (167)	246 (92)	196 (52)	29.9 (11.9)[a]	0.8 (0.6)		8.5 (9.2)			35	393	12				Hernandez *et al.*, 2008
% removal	47 (15)				3 (57)	-616 (642)		8 (36)									

a; Total Kjeldahl.nitrogen (TKN)

Table 2.7. Anaerobic and aerobic biological processes: UASB and UASB followed by SBR treating grey water. Influent and effluent qualities and operation conditions (continued).

Treatments	COD mg L^{-1}				N mg L^{-1}			P mg L^{-1}			Temperature °C	SRT day	HRT hr	Chemical	Energy	Cost	References
	COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}	TN	NH4-N	Particulate -N	Total	Ortho	particulate							
UASB+SBR	UASB followed by SBR																Hernandez *et al.*, 2008
UASB	5.0 L																
Influent	830 (211)	427 (181)	212 (81)	234 (70)	53.6 (50.7)	1.2 (1.3)		7.7 (5.6)			35	97	7				
% removal	36 (15)				-8 (56)	-856 (806)		5 (38)									
SBR	SBR volume 3.6 L, sludge inoculums was activated sludge from wastewater treatment plant in Leeuwarden. The yield for the three experiments 0.19 g VSS g^{-1} COD removed.																
Influent	528 (190)				31.9 (13.5)	5.44 (2.39)	5.7 (2.2)					378	6				
% removal	80 (9)				26 (27)	91 (6)	11 (31)										

a; Total Kjeldahl.nitrogen (TKN)

2.3.4 Toward optimal treatment systems for grey water

2.3.4.1 Selection criteria

Grey water treatment selection factors are the characteristics, the reuse requirements, the technology performance, the energy demand and costs, and the geographical location. These factors are inherently interrelated and influence each other.

2.3.4.1.1 The characteristics and reuse requirements

Tables 2.2 - 2.7 show that the best achievable effluent quality is < 10±5 mg BOD L^{-1}, < 30±10 mg COD L^{-1}, < 15±5 mg SS L^{-1}, turbidity < 2 NTU, TC < 1000 cfu/100ml, and/or FC or EC <200 cfu/100ml. This quality, complies with the most conservative standards except for the pathogens, and is achieved by three types of processes. The first process is biological treatment that applies a long HRT and therefore a long SRT, e.g. CW with 5-14 days HRT. The second process is the micro-filtration or biofiltration with a relatively short HRT of 9.7 hrs. The third process is physicochemical treatment, such as oxidation and coagulation. Grey water characteristics, viz. COD fractions, biodegradability, and biodegradation rate under aerobic and anaerobic conditions are key factors in selection, design and operation of treatment systems. These factors need detailed investigation. Tables 2.2 to 2.7 show N and P are not monitored in most of the studies but should be considered in the treatment and in the standards. The concentration of nitrogen in raw grey waters, except for that reported by Hernandez *et al*. (2008), is lower than the N standards for irrigation water in Jordan. The CW, operated at an HRT of 14 days (Table 2.3) and an SBR (Table 2.6) can reduce the N and/or the P to less than 1 mg L^{-1}, which is far below the irrigation standards in Jordan.

2.3.4.1.2 The technology

Performance. Tables 2.2 to 2.7 show that the detailed design criteria and/or the operational conditions of most of the tested systems are not reported. The performances of the technologies are not examined for seasonal variation nor under natural conditions such as the daily quality and quantity inflow variation. Furthermore, synthetic grey water and high process temperatures, e.g. 30℃, are used in some of the studies. It is concluded that the internal structure and the operational conditions, namely HRT and SRT, determine the performance of the physical and/or biological system.

Coarse, sand and membrane filters have limited capacity in treating grey water (Pidou *et al*., 2007). Furthermore, Pidou *et al*. (2007) reported biological and extensive treatment technologies (CW) are effective in organic matter removal. Table 2.8 of this study, presents grey water treatment units and processes, the possibility of improving their performances considering grey water characteristics, and proposes HRT and/or SRT that can result in good effluent quality. Table 2.8 shows that the performance of different presented technologies, except for the coarse filter, can be improved in terms of COD, BOD, SS and pathogens. All units and processes, except the UASB, primary settlers and anaerobic biofilter, could be optimized to produce effluents with a COD, BOD and SS content complying with the most conservative standards. It must be noted that only membrane, ultra-filtration, chem, and bio-filtration units produce effluents that meet the highest achievable quality concerning pathogens. Tables 2.2 to 2.7 give no information about sludge. Table 2.8 gives estimates of sludge type and handling, based on the applied technology, viz. physical, chemical or biological. Primary sludge production is higher than secondary sludge production, and anaerobic sludge production is considered to be lower than aerobic sludge production.

Energy demand, chemicals and costs. Reported data for energy demand, chemical requirements and costs are in general qualitative (Table 2.2-2.7). Based on basic requirements to operate and maintain systems and on reported data, estimations of technology demand and costs are made (Table 2.8). In general the tested technologies demand energy for aeration and mixing. Flux permeation in MBR systems has the highest energy demand. Therefore, the energy demand list is topped by Micro-, Nano- and Ultra- membrane filtration and biofiltration processes, followed by aerobic and physical-chemical processes, and then coarse and sand filter, CW and anaerobic processes. The chemicals are required as part of some treatments, such as M(F)CR, adsorption, coagulation, oxidation, and disinfection, or for maintenance such as cleaning the membranes of MF and MBR. The operational and maintenance costs in Table 2.8 are evaluated based on energy demand, chemical requirements, and sludge type and handling. The MCR and MBR are the most costly, followed by aerobic units and last are the CWs, the filters and the anaerobic units. All the units have a small footprint, except planted- or not planted-biological sand filters, which need in comparison more land which might increase its capital costs.

2.3.4.2 Optimizing the treatment systems for grey water

Table 2.8 shows that for a minimum or zero primary sludge production, biological treatment is the best option. For minimum energy consumption, an anaerobic step followed by an aerobic step is recommended. Accordingly, filters, physical-chemical and chemfilter units should be avoided as main units for treating grey water. To minimise energy consumption and operation and maintenance costs, the use of membrane biofilters is highly questionable in the biological process options. Table 2.8 shows the anaerobic options are anaerobic filter and UASB. The aerobic options are RBC, SBR, FB or CW. The shorter HRT and the smaller footprint of the biological processes like RBC, SBR and FB have an advantage compared with CW (Pidou *et al*., 2007). However, to choose between the suggested options, the system performance under variable inflow and temperature conditions should be investigated. Furthermore, the available space, detailed costs information is required and for the CW option one needs to consider in addition the geographical location and the climatic conditions. Table 2.8 shows that the selected processes treat COD, BOD and SS to the permissible standards. Thus the anaerobic-aerobic system is efficient, simple and affordable. Furthermore, to assure safe effluent viz. minimize possibility of pathogen re-growth in the treated effluent, a disinfection step is recommended. The disinfection techniques, according to the conducted studies are < 0.2 µm MF, ultra-filters, UV+TiO_2, or chlorine/bromine disinfection. Nevertheless, grey water standards should be revised in order to have multi-category standards for the different use options.

Table 2.8. Grey water treatment units and processes: compliancy with the standards, technology demands and costs.

Treatment Types	Effluent compliances with the reuse standards			Technology Demands							Cost		
							Extra-Units				Operational Investment		
	BOD, COD, and/ or SS	Pathogens	Sludge	Energy	Chemicals	Land-Availability	Pre-	Sludge-handling	Post	Disinfection	Investment	Operational	Maintenance
Equalization, settling, and storage tank	++	+											
Filters:													
Coarse	+	+	P	*	*	*	*	**	**	***		**	*
Sand	+++	++	P	*	*	*	*	**	*	***		**	*
MF	+++	+++	P	***	**	*	**	**	*	**		***	***
UF	+++	+++	P	***	**	*	**	**	*	*		***	***
Physicochemical Treatment:													
Coagulation	+++		P	**	***	*		**				***	*
Oxidation	+++			**	***	*			**				
Adsorption	+++		P			*		**				***	*
Modified Filters:													
VFSF	+++	++	P	*	*	**	**	**		***		**	*
IVSF	+++	++	P	*	*	**	**	**		***		**	*
SSrF	+++	++	P	*	*	**	**	**		***		**	*
HPSF	+++	++	S	*	*	**	**	*		***		*	*
H-CW	+++	++	S	*	*	**	**	*		***		*	*
V-CW	+++	++	S	*	*	**	**	*		***		*	*
Planted SSrF	+++	++	S	*	*	**	**	*		***		*	*
RVFB	+++	++	S	*	*	**	**	*		***		*	*
GRWRS	+++	++	S	*	*	**	**	*		***		*	*
Modified Filters:													
BAF	+++	+	S	**	*	*	*	*		***		**	*
MBR	+++	+++	S	***	**	*	**	*		**		***	***
M(F)BR	+++	+++	S	***	**	*	**	*		***		***	***
Anaerobic Filter	++	+	S	*	*	*	**	*		***			
Bed Submerged Bioreactor	+++	+	S	**	*	*	*	*		***		**	*
MCR	+++	+++	S	***	***	*	**	**	**	*		***	***

Table 2.8. Grey water treatment units and processes: compliancy with the standards, technology demands and costs (continued).

Treatment Types	Technology's effluent compliances with the reuse standards			Technology Demands							Cost Land		
							Extra-Units				Operational Investment		
	BOD, COD, and/ or SS	Pathogens	Sludge	Energy	Chemicals	Land-Availability	Pre-	Sludge-handling	Post	Disinfection	Investment	Operational	Maintenance
Aerobic Biological processes:													
Aerobic													
Attached processes:													
FBR	+++	+	S	**	*	*	*	*		***		**	*
RBC	+++	++	S	**	*	*	*	*		***		**	*
Suspended processes:													
SBR	+++	+	S	**	*	*	*	*		***		**	*
Anaerobic													
Suspended processes:													
UASB	++	+	S	*	*	*	*	*	**	***		*	*

Notifications:

Effluent compliances with the reuse standards in Table 2.1 are the most conservative standards reported in the literatures.

The BOD, COD, SS and/ pathogens values are not reported for all the tested-system, therefore, the results are anticipated as follows:-

+; Poor performance .

++; Performance can be improved but it can not reach a high effluent quality.

+++; Performance can be improved to reach a high effluent quality.

P and S; Primary and Secondary sludge respectively.

Technology demands/cost, and the operational and maintenance cost are evaluated based on the technology demands, namely:-.

*; No or Low demand/cost: **; Medium demand/cost: ***; high demand/cost

2.4 Conclusion

The tested grey water treatment processes are not optimized.

Some of the effluents are not complying with all reuse standards.

Reuse standards should be critically evaluated and likely revised and classified according to the different use options and requirements.

Considering sludge production, systems based only on physical removal should be avoided as they produce masses of non-stabilised sludge.

To save on energy requirements, an anaerobic plus aerobic process is recommended. For pathogen removal, a disinfection unit is required. Therefore, for efficient, simple, affordable treatment of grey water with safe effluent, a three step system, consisting of an anaerobic, aerobic and disinfection unit, is recommended.

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Chapter Three

Quantitative and qualitative characteristics of grey water for reuse requirements and treatment alternatives: the case of Jordan

Abstract:

The objective of this work is to assess the potentials and requirements for grey water reuse in Jordan. To achieve this objective the characteristics of grey waters were determined of a dormitory at Jordan university campus, and of shower, kitchen and laundry waters from houses in Amman and Al-Salt city. Moreover, grey water characteristics of Amman, Al-Tafileh and Al-Mafreqe city were reviewed. The results revealed that urban, rural and dormitory grey water production rate and

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concentration of TS, BOD_5, COD and pathogens varied between 18-66 L d^{-1} cap^{-1}, 848-1919, 200-1056, and 560-2568 mg L^{-1} and 6.9E2-2.7E5 CFU mL^{-1}, respectively. The variations were consequences of the differences in the water consumption, the frequency and volume of toilet, shower and laundry use, and the inhabitant's behavior between the studied areas. The study revealed that by applying grey water reuse 38 to 54 % of the water consumption can be saved. The grey water compromises 64 to 85 % of the total water flow in the rural and urban areas. Storing grey water is inevitable to meet reuse requirements in terms of volume and timing. All the studied grey waters need treatment, in terms of solids, BOD_5, COD and pathogens, before storage and reuse. Storage and physical treatment, as a pretreatment step should be avoided, since it produces unstable effluents and non-stabilized sludge, however, extensive biological treatment can combine storage and physical treatments. Furthermore, a biological treatment system combining anaerobic and aerobic processes copes with the fluctuations in the hydrographs and pollutographs, as well as the present nutrients. The inorganic content of grey water in Jordan is about drinking water quality and does not need treatment. Furthermore, the grey water SAR values were 3-7, revealing that the concentrations of monovalent and divalent cations comply with agriculture demand in Jordan. The observed patterns in the hydrographs and pollutographs showed that the hydraulic load could be used for the design of both physical and biological treatment units for dormitories and hotels. For family houses, the hydraulic load was identified as the key design parameter for physical treatment units and the organic load is the key design parameter for biological treatment units.

Keywords: grey water; storage; reuse; flow pattern; pollutograph; hydrograph; anaerobic; aerobic; batch.

3.1 Introduction

Source separation of domestic wastewaters into grey and black waters facilitates the recovery of resources, such as energy and nutrients from the concentrated black water fraction, while water can be more easily recovered from the lightly polluted grey water stream (Henze *et al*., 1997; Otterpohl *et al*., 1999). The recovered resources can subsequently be reused in agricultural production systems minimizing the need for non-renewable resources. To accomplish such recovery and reuse approach, socio-economical issues like public acceptance, health constraints, and economical feasibility should be resolved. Thus, source separation of domestic wastewaters into grey and black waters is a step forward towards a more sustainable approach in the sanitation sector.

In Jordan, which suffers from water scarcity, grey water recovery has gained increasing attention. At present, grey water forms up to 85% of the domestic wastewater flow in Jordanian urban areas (Jamrah *et al*., 2006), while reclaimed domestic wastewater forms 6% of the Jordanian water budget (WAJ, 1999), and is used for irrigation.

The variation in grey water characteristics at the same source and between the different sources (Nolde, 1999; Eriksson *et al*., 2002; Jefferson *et al*., 2004) provokes a variety of reuse options, which are classified by Jefferson *et al*. (2001) as external and internal reuse options, and created an abundance of grey water treatment systems (Jefferson *et al*., 1999). The reuse requirements on the one hand and the treatment alternatives on the other hand require the formulation of a sustainable grey water management plan. For actual reuse schemes, the reuse options, as well as the public acceptance, and quantitative and qualitative requirements should be considered. Possible treatment alternatives include physical, chemical and/or biological treatment, in both continuous flow and/or batch mode. Many different reuse options for grey water are described (e.g. Jefferson *et al*., 2001). External or off-site reuse of treated grey water refers to crop irrigation, wetland development and/or preservation, and aquifer recharge. With regard to internal or on-site reuse the water is retained within a local process loop. Internal use can be further classified as outdoor and indoor reuse.

Examples of outdoor reuse are irrigation of green spaces on college campuses, sport fields, cemeteries, parks and golf courses, private gardens and for washing of vehicles and windows, for fire protection, boiler feed water and concrete production. Examples of indoor reuse are toilet flushing, cloth washing and bathing.

The actually applied type of reuse depends on the local water availability problems, the public acceptance as well as the present practice of reuse. Jamrah *et al*. (2006) carried out a grey water reuse acceptance study in Amman, the capital city of Jordan, and revealed that inhabitants accept reuse of grey water for vegetable growing, irrigation of fruit trees, car washing, toilet flushing or home laundry provided that the reuse costs, health hazards, ground water pollution and environmental impact are meeting the regulations.

Al-Jayyousi (2003) presented the current practice of grey water reuse and discussed the reuse of kitchen and washbasin water after grease removal for garden farming in the rural city of Al-Tafileh in Jordan. Halalsheh *et al*. (2008) reported on the reuse of grey water, without treatment, for irrigating fruit trees in Al-Mafreq which is another rural city in Jordan. The Center Study Build Environment (CSBE, 2003) reported on the reuse of wastewaters of ritual washing from mosques that after filtering were used for garden irrigation. Furthermore, households in the urban area of Amman used bath, shower and bathroom sink wastewater directly for irrigation of plants in courtyards of the houses (CSBE, 2003). Grey water including the kitchen waste stream of a dormitory at Jordan University of Science and Technology was treated by reed beds and was subsequently used for irrigation of vegetables (CSBE, 2003). Finally, it was reported that washbasin and kitchen wastewater and grey water excluding laundry from households in rural areas were passed through stainless steel sieves and used for irrigations (CSBE, 2003).

This study aims to determine the potentials and the requirements for reuse, as well as treatment alternatives of grey waters from Amman, Al-Tafileh, Al-Mafreqe and students' dormitories. To achieve this objectives (1) Dormitory, shower, laundry and kitchen grey waters were collected, analyzed and characterized. (2) Amman, Al-Tafileh and Al-Mafreqe grey water characteristics were reviewed. (3) Urban and rural grey water flow patterns and pollutographs were determined and evaluated.

3.2 Methodology

3.2.1 Grey water sources and collection

Sources: Grey water was collected from 6 houses in urban areas in Amman and Al-Salt, and a dormitory at the Jordan university campus. The houses had 4-6 family members; laundry, kitchen and shower wastewaters were separately collected. The dormitory was occupied by 150 female students. The laundry and shower water was drained in one tube, from which samples were collected. Additionally, data reported by Jamrah *et al*., 2006; INWRDAM, 2000; Al-Jayyousi, 2002; and Both Suleiman *et al*., 2006; and Halalsheh *et al*., 2008, on grey water characteristics in Amman, Al-Tafileh and Al-Mafreqe, respectively, were used. Amman is the capital city of Jordan, an urban area, and Al-Tafileh and Al-Mafreqe are rural cities in south and east of Jordan, respectively.

Collections: Over a period of four months wastewater was collected from the six houses in the cities of Amman and Al-Salt. In order to evaluate the quality of the individual wastewater streams, the wastewater per use of laundry, shower or kitchen was collected separately in containers. A container was gently mixed and a sub-sample of 4 liters was used for immediate analysis or stored at 4℃ fo r a maximum of 24 hours. This procedure was done 8 to 13 times per waste stream.

At the students' dormitory at the Jordan University campus, a 5 liter pit was constructed to collect the grey water. Two devices were installed: a water recorder (Steven, 68) that plotted for two months the daily flow patterns and an auto-sampler (ISCO, 6712) consisting of 24 1 L bottles. The auto-sampler was programmed to withdraw every 15 minutes 250 ml wastewater from the pit. For hourly samples, the 24 bottles were analyzed separately. For composite analysis of a day sample, the 24 bottles were emptied in one container and subsequently analyzed. In total 96 day samples were collected in four months. Two flow meters were installed which measured for two months the dormitory's daily consumption of hot and cold water. In addition, twenty students at the dormitory determined the individual time periods, frequencies and the volumes of water required for use of the washbasin.

3.2.2 Analysis

Dormitory hourly samples were analyzed for chemical oxygen demand (COD) and total solids (TS) concentrations. With regard to the dormitory daily composite samples, 96 were analyzed for COD, 78 for TS, 26 for suspended solids (SS) and 20 samples were analyzed for biochemical oxygen demand (BOD_5), total nitrogen (TN), ammonia (NH_4) and total phosphorus (TP) concentrations. All shower, laundry and kitchen samples were analyzed for COD, TS, SS and 6 samples for TN and TP. Moreover, temperature (T), pH and electrical conductivity (EC) were measured of all grey water samples from the different sources and 13 drinking water samples from the dormitory. The alkalinity, hardness, Na, K, Cl, Ca and Mg were analyzed of 13 drinking water samples from the dormitory. The same analyses were performed for 13 of the dormitory grey water composite samples. Also the same analysis, except of alkalinity and hardness, were carried out for 5 samples of the separate streams of laundry, kitchen and shower. Analyses were conducted according to Standard Methods (APHA, 1995).

3.2.3 Calculations

Dormitory: Volume distribution of the dormitory grey water was calculated as follows: ArcGIS software (ESRI product, ArcGIS 8.1) digitized the daily water level recorder curves, and the areas under the curves represent the daily combined volume of shower and laundry water uses. The mean and standard deviation of the hand washbasin use characteristics were determined based on data collected from 20 students in the dormitory. Then the mean was used in equation 2.1 to calculate the daily production of washbasin volume.

Interviews of the students revealed that the use of kitchen sinks was very limited. Therefore it was assumed that the production of kitchen wastewater was negligible. The dormitory daily toilet flushing volume was the difference between drinking water consumption and the total daily water use in washbasin, shower and laundry. Daily water uses per capita (L cap^{-1} day^{-1}) were calculated by dividing each of the volumes by 150 students.

Urban and Rural areas: The daily wastewater volumes production per capita of shower, laundry and toilet in Al-Tafileh were calculated from equation 2.2 using the data of INWRDAM (2000). The latter reported for Al-Tafileh the daily frequency per capita and the volume per use of shower, laundry and toilet flushing. Moreover, almost half of the grey water in Al-Tafileh (Al-Jayyousi , 2003) and Al-Mafreqe (Suleiman *et al*., 2006; and Halalsheh *et al*., 2008) was kitchen wastewater. It was observed that the kitchen sinks in Al-Mafreqe village were used as washbasin sink and kitchen sink (Suleiman *et al*., 2006), however it was not clear what proportion of this wastewater source was indeed kitchen wastewater and what was the washbasin use. The kitchen wastewater volume in Al-Mafreqe is known and by assuming it is the same in Al-Tafileh, the washbasin volume contribution from the kitchen sink source in Al-Tafileh is calculated.

$$DV(m^3 d^{-1}) = F \times V \times N \tag{2.1}$$

$$DV(m^3 d^{-1} cap^{-1}) = F \times V \tag{2.2}$$

Where DV= Daily Volume; F= the mean daily frequency per capita; V= mean water volume per use; N= the number of students.

The quality parameters of shower and laundry wastewaters, found in this study, in terms of the volumes and the concentrations of COD, TS, SS, TN and TP, were used in equation 2.3 to calculate the quality parameters of Amman combined grey water. This calculation was also based on an assumption based on Jamrah *et al*. (2006) reported results, that the quality parameters of BOD_5, SS and COD shower and washbasin wastewaters are similar.

$$\text{CGP (mg L}^{-1}) = \frac{C_{\text{laundry}} \times V_{\text{laundry}} + C_{\text{shower}} \times V_{\text{shower}} + C_{\text{washbasin}} \times V_{\text{washbasin}}}{V_{\text{laundry}} + V_{\text{shower}} + V_{\text{washbasin}}} \quad (2.3)$$

Where CGP= the Parameter concentration in the Combined Grey water; C= the parameter Concentration in the individual stream (mass per volume); V= the water Volume of the individual stream.

The daily saving potentials in water consumption due to on-site recycling of grey water, in Amman and Al-Tafileh, were calculated by equation 2.4.

$$\text{SWC (\%)} = \frac{V_{\text{BlackWater}} + V_{\text{Laundry}} + V_{\text{GardiningIrrigation}}}{\text{TWC}} \times 100 \quad (2.4)$$

Where SWC= the Saving percentage in Water Consumption volume; V= Water Volume; TWC= Volume of the Total Water Consumption

The daily on-site recycling potentials of grey water of Amman and Al-Tafileh were calculated by equation 2.5.

$$\text{RGW (\%)} = \frac{V_{\text{BlackWater}} + V_{\text{Laundry}} + V_{\text{GardiningIrrigation}}}{\text{TGW}} \times 100 \quad (2.5)$$

Where RGW= the Recycling percentage of Grey Water generated on-site; V= the Water Volume; TGW= Volume of the Total Grey Water generated on-site by shower, laundry, washbasin and kitchen.

Rural and urban areas hydrographs were plotted from: water use, viz. frequencies and volumes, reported by INWRDAM, (2000) and Jamrah *et al*. (2006) for Al-Tafileh and Amman, respectively. Also the timing of water use was determined based on the assumption that the family size in both areas was 6 (4 to 6 family members occupied the houses studied by this study in Amman and Al-Salt, moreover, Al-Jayyousi (2002) used 5 family members for Al-Tafileh): three adults and three children of school age. In urban areas, all adults are working outdoors while in the rural areas just two are working outdoors, and the working hours in both areas are from 8 a.m. to 5 p.m.

Rural and urban areas sub-daily grey water concentration and organic load patterns, further referred to as pollutographs, were plotted using the hydrographs and the typical values of COD and TS concentrations for shower, laundry and kitchen grey waters, reported by Suleiman *et al*. (2006) and Halalsheh *et al*. (2008) for Al-Mafreqe, and the data found in this study for Amman.

One of the tools used for determining the suitability of grey water for irrigation purposes is the so-called sodium adsorption ratio (SAR) (equation 2.6). The SAR determines the relative concentrations of sodium to calcium and magnesium (Metcalf & Eddy, 2003), and is a measure indicating the salinity of the wastewater and predicting the possible adverse effects of monovalent cations (sodium) in soils.

$$\mathrm{SAR} = \frac{\mathrm{Na}}{\sqrt{\frac{\mathrm{Mg} + \mathrm{Ca}}{2}}} \qquad (2.6)$$

Where Na, Ca and Mg are expressed in milli-equivalents per liter (meq L^{-1}).

3.3 Results and Discussion

3.3.1 Quantity characteristics

3.3.1.1 Grey water volumes

Table 3.1 shows that the per capita water consumption in the students' dormitory and Amman was 3 times higher than in the rural areas. Grey water (GW) contribution to the total water consumption (TWC) in the dormitory place, the urban areas and rural areas were respectively 85%, 70 % and 64% (Table 3.1). Moreover, shower, laundry and washbasin contributions to grey waters varied from 22-36%, 17-28% and 11-47% respectively. All previous differences between urban and rural areas were the consequence of the variations in the family incomes and the inhabitant behaviors, which affected the water uses' frequencies and volumes. Shower and laundry activities are characterized by a low daily frequency and a high water volume consumption per use, in contrast to the washbasin and kitchen sink, which were characterized by a high daily frequency and low volumetric water consumption per use (Table 3.1). Washbasin and shower were the main grey water contributors in urban areas and kitchen and laundry were the main grey water contributors in rural areas (Table 3.1). In contrast, Butler *et al*. (1995) reported that washbasin comprised 5-13%, and bath and shower 28-31% of the grey water in England, USA and Malta, being the minimum and maximum contributors, respectively.

The results indicate that the reuse of grey water, including kitchen wastewater which is the main component of grey water in the rural areas (Table 3.1), for toilet flushing, laundry or garden irrigation and/or a combination of all, could save up to 38 to 54 % of water consumption in Amman and Al-Tafileh (Table 3.1). Meanwhile, the total amount of water required for on-site reuse is 54-83% of grey water production in the studied areas (Table 3.1). Thus 17-46% of the generated grey water is available for new reuse options.

3.3.1.2 Grey water flow patterns

Figures 3.1 and 3.2A and 3.2B show measured flows of grey water, for week days following a diurnal pattern: almost no flow between 12 to 6 a.m., the first peak occurs in the early morning between 7 and 9 a.m., and the second peak occurs late in the evening between 8 and 12 p.m. Peaks result from laundry and/or shower use. The volume and/or the number of uses per time unit determined volume of the peak. Figure 3.1 shows that at the dormitory no variation in the weekday flow rates; the variation occurs in the weekend (not shown here) when many students leave the dormitory.

For residential areas Metcalf and Eddy (2003) stated that a weekday flow rate is expected to change only on laundry day. The dormitory flow pattern showed an almost zero flow during the holiday months; February and September, the same is expected in recreational and institutional facilities.

Table 3.1. The mean values; Liter per capita per day (L cap^{-1} day^{-1}) of grey water streams generated at dormitory and residential places in urban and rural areas in Jordan.

Domestic Wastewaters Streams	Dormitory Jordan university Campus	Amman Urban Area	Al-Tafilah Rural Area (Southern Jordan)	Al-Mafreqe Rural Area (Eastern Jordan)
Total Water Consumption (TWC)	78 (8)[m]	84 (19)[e]	28[e]	NA
Black Water (BW)	12 (7)[c]	17 (7)[c]	10[c]	NA
Gardening Irrigation	NA	5 (2)[e]	NA	NA
Grey Water (GW)	66(9)[c]	59 (13)[c]	18[c]	14 (3)[m]
Shower	NA	21(16)[e]	4[c]	
Washbasin	21(5)[e]	28 (5)[e]	2[c]	1 (0)[m]
Laundry	NA	10 (4)[e]	5[c]	
Kitchen	NA	NA	7[e]	7 (2)[m]
Laundry & Shower	45 (6)[m]	NA	NA	NA
References	determined by this study	Jamrah *et al.* (2006)	INWRDAM (2000)	Suleiman *et al*. (2006); Halalsheh *et al*. (2008)

c; calculated values as described in the methodology
e; estimated values
m; measured values
( ); standard deviation
NA; Not Available

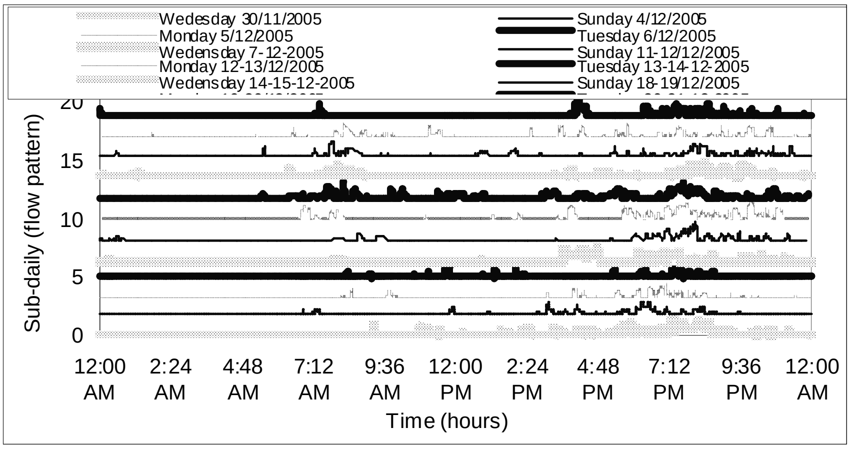
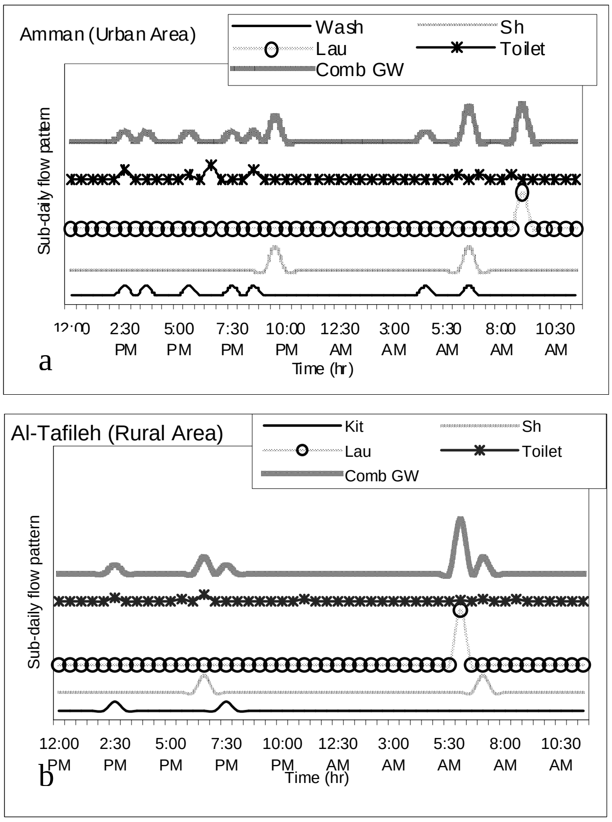


Figure 3.1. Recorded hydrograph sub-daily flow patterns of the dormitory at the Jordan University. The units of the Y-axis are calculative converted into liters after digitalization of the curves.

None of the grey water flow patterns, in both rural and urban areas, shown in Figure 3.2A and 3.2B were identical in volume, time and period of occurrence. Moreover, the flow patterns of both combined and separated grey water streams, don't comply with the water requirements for toilet flushing or laundry washing, with respect to volume (Table 3.1) and time of occurrence. The batch-wise production of grey water means that direct reuse, of grey water in Jordan, without storage is not possible. This is in agreement with Jefferson *et al*. (1999), Surendran and Wheatley (1998), who reported that a storage tank is required to meet the volume and timing of the toilet flushing in England.



Figures 3.2A and 3.2B. Sub-daily flow patterns for a weekday of grey waters generated in single houses in Amman and Al-Tafileh respectively, which were occupied each by six family members. The hydrographs were plotted based on the frequency and daily volume produced per capita per type of usage as described in the methodology. Kitchen (Kit), washbasin (Wash), shower (Sh) and/or laundry (Lau), combined grey water (Comb GW), domestic wastewater (DW).

3.3.2 Quality characteristics

3.3.2.1 TS, COD and BOD

Table 3.2 shows the concentration of pollutants in dormitory, Amman, Al-Tafileh and Al-Mafreqe grey waters. The difference in the grey water between the two rural cities Al-Tafileh and Al-Mafreqe was striking. The explanation can be found in the following: Halalsheh e*t al*. (2008) reported contamination of Al-Mafreqe's grey water with black water, mainly from rinsing diapers and washing babies in the sinks. From Table 3.2 this can also be concluded comparing Al-Mafreqe and Al-Tafileh regarding SS, NH_4 and TP. These components mainly originate from urine and faeces and were considerably higher in the Al-Mafreqe's grey water. To realize an easy treatment and safe reuse, black water contamination should be avoided. Therefore, increasing the people's awareness, regarding the reasons and targets of separating grey water form black water, should be a priority to produce better grey water in rural areas.

The dormitory grey water quality had almost shower water quality, which is the least polluted grey water stream. In the total grey water from the dormitory, the contribution of kitchen wastewater was zero. Therefore, the dormitory grey water consisted mainly of laundry and shower wastewaters. The shower wastewater was the predominant waste stream due to the large volume per use and the large amount of students. This explains the low concentration of polluting constituents found in the dormitory grey water.

The quality parameters SS, COD, and BOD of Amman grey water reported by Jamrah *et al*. (2006) were less than the values found in this study for Amman grey water. They were also less than the values found in this study for dormitory grey waters; the kitchen stream was excluded from Amman and dormitory grey water sources, furthermore, shower and washbasin wastewaters have almost the similar qualities, therefore, laundry wastewater contribution to the Amman grey water increased the value of the parameters over the dormitory grey waters.

Table 3.2. Grey water quality characteristics, those are determined by this study, of dormitory at Jordan University campus, laundry, shower, and kitchen, Amman and Al-Slat city, are determined by this study and reviewing for Amman, Al-Tafileh and Al-Mafreqe.

Parameters	Units	Dormitory Drinking water	Dormitory Grey water	Amman and Al-Salt Houses			Al-Tafileh	Al-Mafreqe	Capital Amman	Capital Amman	Jordanian standers for use reclaimed wastewaters	
				Kitchen	Shower	Laundry					(1)	(2)
Temperature	°C		24 (5)	25 (10) [8]	30 (10) [10]	35 (15) [10]						
pH		7.95 (.05) [13]	7.6 (0.2) [96]	6.83 (0.65) [8]	7.15 (0.87) [8]	9.6 (0.72) [10]	6.7	5.7-7 (6.4)	7.81	NA	6 - 9	6 - 9
EC	dS/m	0.87 (0.02) [13]	1..06 (0.185) [96]	1.244 (0.31) [8]	0.83 (0.19) [8]	4.542 (1.96) [10]	0.46-1.14	1.56 - 2.10 (1.89)	1.91	NA	> 1.9** > 1.2*	> 1.9** > 1.2*
Turbidity	NTU	NA	122 (78) [93]	NA	NA	NA	NA	NA	49	NA		
Alkalinity (caco3)	$mg\,L^{-1}$	397 (68) [13]	412 (106) [13]	NA	NA	NA	140	NA	225	NA		
HCO_3	$mg\,L^{-1}$	317 (34) [13]	542 (21) [13]	NA	NA	NA	NA	426	NA	NA		
TS	$mg\,L^{-1}$	500 (140) [13]	876 (201) [78]	4101 (419) [8]	750 (94) [8]	3940 (2885) [10]	NA	1840-1997 (1919)	1061	1291		
SS	$mg\,L^{-1}$	NA	122 (78) [26]	1180 (300) [8]	150 (109) [8]	760 (384) [10]	264	1007-1140 (1074)	168	253	50	50
COD	$mg\,L^{-1}$	NA	551 (202) [96]	8071 (3535) [8]	537 (165) [8]	2500 (1865) [10]	1460	2257-2878 (2568)	78	870	50	100
BOD_5	$mg\,L^{-1}$	NA	149 (46) [20]	1850 (890) [8]	120 (40) [8]	1266 (232) [10]	275-2287 (764)	977-1134 (1056)	41	314	15	30
T-N	$mg\,L^{-1}$	NA	10 (14) [20]	25.9 (33.8) [6]	2.4 (1.45) [6]	2.8 (0.45) [6]	25	94-161 (128)	9.2	2	45	45
NH4	$mg\,L^{-1}$	NA	8 (6) [20]	NA	NA	NA	1	75	NA	NA	5	
T-P	$mg\,L^{-1}$	NA	7 (7) [20]	4.3 (3.9) [6]	1.2 (1.1) [6]	9 (0.7) [6]	13.8	16-23 (20)	9.0	3	15	30

Table 3.2. Grey water quality characteristics, those are determined by this study, of dormitory at Jordan University campus, laundry, shower, and kitchen, Amman and Al-Slat city, are determined by this study and reviewing for Amman, Al-Tafileh and Al-Mafreqe / Continue.

Parameters	Units	Dormitory Drinking water	Dormitory Grey water	Amman and Al-Salt Houses			Al-Tafileh	Al-Mafreqe	Capital Amman	Capital Amman	Jordanian standers for use reclaimed wastewaters	
				Kitchen	Shower	Laundry					(1)	(2)
Mg	mg L^{-1}	28 (3) [13]	29 (9) [13]	21 (7) [5]	18 (5) [5]	31 (8) [5]	34		18	20		
Ca	mg L^{-1}	54 (10) [13]	58 (17) [13]	47 (10) [5]	45 (10) [5]	50 (15) [5]	50		36	46		
Na	mg L^{-1}	87 (5) [13]	143 (24) [13]	160 (20) [5]	130 (25) [5]	220 (30) [5]	136		120	145		
K	mg L^{-1}	5 (1) [13]	10 (2) [13]	7 (2) [5]	7 (2) [5]	10 (2) [5]	NA	21-27 (24)	NA	8		
Cl	mg L^{-1}	134 (15) [13]	141 (22) [13]	140 (25) [5]	170 (20) [5]	300 (25) [5]	NA		162	192		
TC	MPN / 100	NA	NA	NA	NA	NA	NA	1E7	4.20E+02	NA		
E.Coli.	MPN / 100	NA	NA	NA	NA	NA	NA	3.9E4-2.8E5 (2.0E5)	1.60E+02	NA	<2.2	<100
SAR		2	3	5	4	6	4		4	4	6 - 12** 3 - 6*	6 - 12** 3 - 6*
References		this study	this study	this study	this study	this study	INWRDAM (2000); Al-Jayyousi (2002)	Suleiman *et al.* (2006); Halalsheh *et al.* (2008)	Jamrah, *et al.* (2006)	Calculated in this study	Jordanian standards JS893/ 002	

( ): Typical values

1; for artificial recharge of ground water aquifer

2; for irrigation vegetables eaten cooked, parks, playgrounds and sides of roads within city limits* and ** ; the combination of SAR and EC values are the FAO guidelines for use reclaimed domestic water for irrigation (Papadopoulos, I., 1995).

Comparing households in urban Amman and rural areas the grey water was more dilute in Amman because the kitchen wastewater was not included in the Amman grey water whilst the kitchen wastewater is the main contributor of pollutants (Table 3.2). It might also be due to the higher water consumption in Amman diluting the pollutants (Table 3.1). Rural areas had the most concentrated grey waters in Jordan due to low water consumption, the inclusion of the kitchen water stream and in some cases the mixing with black water as observed in Al-Mafreqe.

Table 3.2 illustrated that in terms of COD, BOD, SS and pathogens the different grey water streams did not meet the Jordanian standards for reuse of reclaimed water for irrigation inside cities. In conclusion, all grey waters in Jordan need treatment before storage and reuse in terms of COD, BOD, SS and pathogens.

Physical treatment processes such as sand filters, coagulation and flocculation processes produce an effluent quality better than primary sedimentation or storage tanks (Jefferson *et al*., 1999). Nevertheless, in both approaches, primary sludge needs to be discharged periodically to be stabilized and to prevent deterioration of the effluent quality with time. Therefore, in order to simplify the treatment and to facilitate low-cost maintenance, it is proposed to avoid physical treatment and to apply extensive biological treatment, either aerobic or anaerobic treatment, or a combination of both. However, physical treatment could be still an option, when a complete decentralized concept is applied including black water digestion, in which grey water primary sludge is digested with black water.

The dormitory grey water pollutographs (Figure 3.3) show a negligible daily fluctuation in both COD and TS concentration patterns, owing to the fact that the shower is the dominant stream. The daily organic load and flow patterns of dormitory grey waters are identical, indicating that organic load peaks result from flow peaks rather than from concentrated discharges (Figure 3.3). As the grey water concentration is hardly changing over the day for dormitories and similar places that are occupied by large residence numbers at one time, the hydraulic load can be used to design the physical and/or biological treatment units.

For residential places, the daily grey water COD and TS concentration patterns largely fluctuate: the largest peak was three times the minimum one (Figures 3.4A and 3.4B). The daily concentrations and organic load patterns of TS are similar but are different from the hydrograph. Also the daily concentrations and organic load patterns of COD are similar but are different from the hydrograph. These results were found in both urban and rural areas (the latter not shown). The reciprocal effect of the concentrations and volume caused the similarity in the organic load and concentration patterns (Tables 3.1 and 3.2). The largest TS and COD load can be attributed to the contribution of laundry activities, then the kitchen and the least was the shower contribution. For residential locations the hydraulic and organic loads should be used respectively to design physical and biological treatment units.

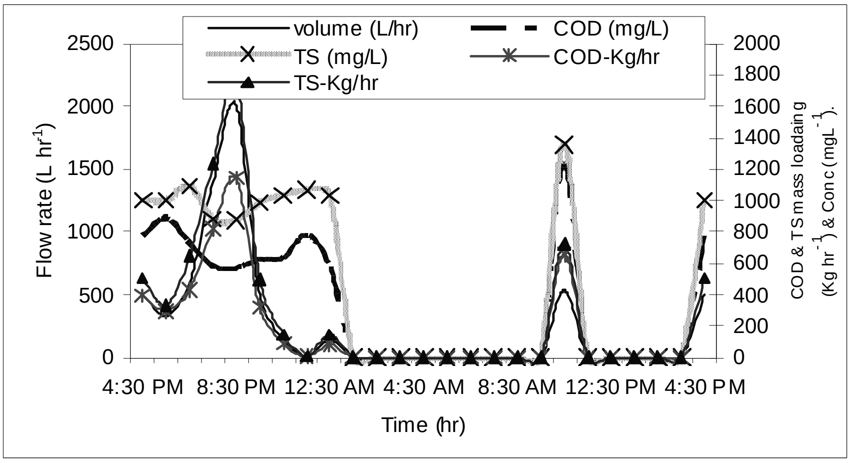
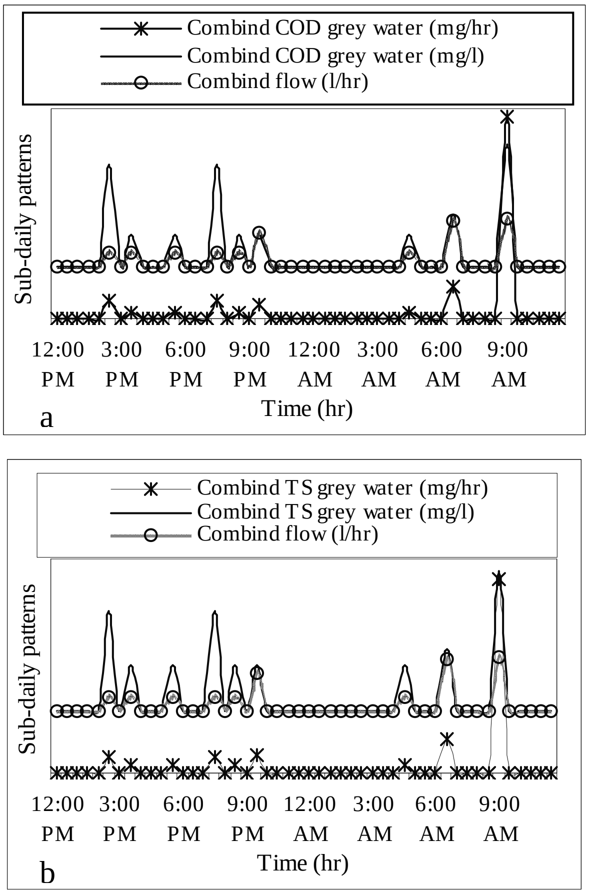


Figure 3.3. Dormitory sub-daily patterns in terms of flow (hydrograph), concentration and organic load (pollutograph), Concentration (Conc).

To meet the requirements of continuous treatment systems; namely steadily/or low variation in the daily-flow and/or -organic load (Metcalf and Eddy, 2003), and because of the intermittent nature of the grey water flow and organic load (Figure 3.3 and 3.4), pre-storage or an integrated storage-treatment unit is required. Therefore, a biological treatment unit that stores and treats grey water at the same time in a single unit is recommended.



Figures 3.4A and 3.4B. Prediction of sub-daily patterns of urban residential places, in terms of flow (hydrograph), COD and TS concentrations as well as organic load (pollutograph), based on the data given in table 3.1 and the assumptions made for Figures 3.2A and 3.2B.

3.3.2.2 Inorganic solids, phosphorus and nitrogen content

In Jordan, the grey water content of Ca, Mg, Na, K and Cl almost resembles that of drinking water quality, except for the grey water from laundry activities (Table 3.3). Removal of these compounds is not needed, when the treated grey water is used for irrigation purposes. The mentioned ions refer to the grey water salinity. The suitability of grey water salinity for irrigation is estimated based on both the measured values of electrical conductivity (EC) and the calculated sodium adsorption ratio (SAR) (Metcalf and Eddy, 2003). The results in Table 3.2 show that there is no restriction on reusing grey waters for irrigation in Jordan (Papadopoulos, 1995).

Table 3.3. Nitrogen and phosphorous requirement for aerobic and anaerobic biomass growth, based on BOD_5 content of different grey water streams.

Cell-Composition ($C_{60}H_{87}O_{23}N_{12}P$)*	Y** mgVSS $mgBOD_5^{-1}$	mgN mg BOD_5^{-1}	mg P mg BOD_5^{-1}
Anaerobic Cell requirement	0.0600	0.0073	0.0014
Aerobic Cell requirement	0.6000	0.0734	0.0140
Dormitory typical values		0.0350	0.0900
Al-Mafreqe typical values		0.1212	0.0189
Al-Tafileh typical values		0.0327	0.0181

*: After Benefield and Randall (1980) and Metcalf and Eddy (2003)
**: after Metcalf and Eddy (2003)

Food preparation, diapers change activities in the sink and detergents are the most important N and P sources in grey waters. Both N and P are essential nutrients to maintain bio-culture growth in a biological treatment system. The P content in grey waters is sufficiently high to maintain both an aerobic and anaerobic bio-culture (Table 3.3). The N in grey waters is only sufficient for anaerobic systems, except for Al-Mafreqe grey water which can support both anaerobic and aerobic systems (Table 3.3), due to cross contamination with black water. An N and P deficit and/or excess can be managed by a proper combination of anaerobic and aerobic units for treating grey waters. On the other hand, if irrigation is the reuse option of the grey water, the extent to which N, P and K need to be removed can be drastically reduced, if needed at all. The reason for this is that N, P and K are plant nutrients, reducing the need for synthetic fertilizer additions in the fields and/or city parks (Boom *et al*., 2007).

P, N and caustic soda, which are detergent constituents, increase the alkalinity of grey water compared to drinking water (Table 3.2). The alkalinity of grey water in Jordan is suitable for biological treatment and for direct irrigation (Table 3.2). The pH of combined grey water is almost neutral (Table 3.2). In kitchen wastewater and in grey waters of rural areas pH was slightly below neutral. The slightly decreased pH might originate from pollutants like natural acids. The high pH value of laundry wastewater is 8-10.5 which is due to the basic nature of the laundry detergents (Lanfax Laboratories, 2008). Nevertheless, the pH of Jordanian grey water is optimal for biological treatment and for irrigation.

3.4 Conclusion

The research showed that the quality of grey waters with respect to COD, BOD, TS and pathogens, in Jordanian urban, rural and dormitories places, requires adequate anaerobic and/or aerobic biological treatment prior to agricultural reuse. With regard to the Ca, Mg, Na, K and Cl, content no further treatment is required.

Quantity characterization of grey waters, in terms of volume and hydrographs, illustrates the possibility of the reuse for toilet flushing, laundry and/or garden irrigation when daily storage and treatment is provided. The reuse for toilet flushing, laundry and garden irrigation covers 54 to 83 % of the grey water production.

Pollutographs of grey waters illustrate that for the dormitory, as the concentration is hardly changing over time, the hydraulic load can be used for designing grey water treatment units, while for urban and rural residential places, both the hydraulic load and organic load should be used for designing grey water treatment units.

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Chapter Four

Grey Water Biodegradability

Abstract:

Knowing the biodegradability characteristics of grey water constituents is imperative for a proper design and operation of a biological treatment system of grey water. This study characterizes the different COD fractions of grey water and investigates the effect of applying different conditions in the biodegradation test. The maximum aerobic and anaerobic biodegradability and conversion rate for the different COD fractions is determined. The results show that, on average, grey water COD fractions are 28% suspended, 32% colloidal and 40% dissolved. The studied factors incubation time, inoculum addition and temperature are influencing the determined biodegradability. The maximum biodegradability and biodegradation rate differ between different COD fractions, viz. COD_{ss}, COD_{col} and COD_{diss}. The dissolved COD fraction is characterised by the lowest degradation rate, both for anaerobic and aerobic conditions. The maximum biodegradability for aerobic and anaerobic conditions is 86% and 70% respectively, whereas the first order conversion rate constant, k_{20} , is 0.119 and 0.005 day^{-1}, respectively. The anaerobic and aerobic conversion rates in relation to

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temperature can be described by the Arrhenius relation, with temperature coefficients of 1.069 and 1.099, respectively.

Keywords; anaerobic, aerobic, biodegradability, conversion rate, grey water, mineralisation, temperature.

Nomenclatures

APE	Alkyl-Phenol Ethoxylate	Ln	Natural logarithm
BMP	Biological Methane Potential	MBR	Membrane Biological Reactor
BOD	Biochemical Oxygen Demand	MCR	Membrane Chemical Reactor
C	Substrate concentration	mf	Membrane filtrate
C_0	Initial Substrate concentration	MW	Molecular Weight
℃	Celsius	MWCO	Molecular-Weight Cut-Off
CH_4	Methane	pf	Paper filtrate
COD	Chemical Oxygen Demand	SLS	Sodium Laurel Sulphate
Col	Colloidal	SRT	Sludge Retention Time
CSTR	Continuous Stirred Tank Reactor	ss	Suspended Solids
		t	Time
DEEDMAC	Di**e**thyl Ester Dimethyl Ammonium Chloride	T	Temperature
		tot	Total
dis	Dissolved	TVS	Total Volatile Solids
EXP	Exponential	UASB	Up-flow Anaerobic Sludge Blanket
fra	Fraction		
HRT	Hydraulic Retention Time	UBOD	Ultimate BOD
k	Kinetic rate constant	VSS	Volatiles Suspended Solids
LAS	Liner Alkyl Benzene Sulphonate	θ	Temperature coefficient

4.1 Introduction

Grey water is domestic wastewater without the input from toilets (Jefferson *et al*., 2004; Eriksson *et al.,* 2002), sometimes kitchen wastewater is excluded as well (Nolde *et al*., 1999). Grey water constituents form 38%, and up to 71% of the COD of domestic wastewater when excluding and including kitchen wastewater respectively (Kujawa-Roeleveld and Zeeman, 2006). Different biological treatment processes have been investigated to stabilise grey water (Nolde, 1999; Jefferson *et al*., 1999; 2000; 2001; Li *et al*., 2003; Friedler *et al*., 2005; 2006; Elmitwalli and Otterpohl, 2007; Shin *et al*., 1998; Hernandez *et al*., 2007; 2008). Evaluation of treatment performances, of which biological processes, reveal that grey water constituent are partly recalcitrant (Jefferson *et al*., 2001), slowly sometimes even non-biodegradable (Friedler *et al*., 2006). In addition grey water contains soluble organic matter with a low molecular weight; i.e. retained by a “MWCO 200 Dalton membrane” nano-filtraion unit, (Ramon *et al*., 2004). Many researches have been directed towards the use of Membrane Biological Reactor (MBR) (Jefferson *et al*., 2000; 2001; Friedler *et al*., 2006), and Membrane Chemical Reactor (MCR) systems (Rivero *et al*., 2006; Winward *et al*., 2008). The basic requirements to design and operate the biological processes are the amount of biodegradable organic matter and the rate of biodegradation at different temperatures. However, so far, little has been done concerning the biodegradability of grey water constituents, e.g. by Elmitwalli and Otterpohl (2007) and Zeeman *et al*. (2008).

Grey water biodegradation starts under anaerobic conditions within 2 days of storage (Dixon *et al*., 1999). The biodegradable amount, measured as COD, is 74±4% (Elmitwalli and Otterpohl, 2007) and 72±12 (Zeeaman *et al*., 2008) under anaerobic conditions and is about 84±5% under aerobic conditions (Zeeaman *et al*., 2008). Literature results show that the ratio of BOD_5 to COD is low and varies from 0.25 to 0.64 (Jefferson *et al*., 1999; and compiled from Friedler *et al*., 2005; 2006; Winward *et al*., 2008). Surfactants such as LAS, Alcohol sulfates, SLS, APE and DEEDMAC are common grey water constituents and their biodegradability differ between aerobic and/or anaerobic conditions (Berna *et al*., 1999). In general, biodegradability results are influenced by the test conditions such as the initial concentration, the incubation time, the used inoculum, and the temperature (Angelidaki and Sanders, 2004; Metcalf and Eddy, 2003). The effect of these test conditions on the biodegradation of grey water has not been investigated therefore the biodegradability results can not be evaluated and generalized.

This paper characterizes combined grey water from shower, hand washbasins and laundry in terms of COD of the different particle size fractions and studies the aerobic and anaerobic biodegradability of the grey water. To do this, experiments and tests were done to investigate the factors influencing the biodegradability percentage and conversion rate of the grey water under anaerobic and aerobic conditions. Maximum biodegradability and biodegradation rates of various grey water fractions were determined and the effect of temperature on the biodegradation rates of grey water was established.

4.2 Methodology

4.2.1 Grey water source and sampling

The source of the grey water used for this study was the waste streams of shower, laundry and washbasins discharged into a single outlet pipe of a 150-female dormitory at the Jordan university campus. The outlet pipe was retrofitted so that it discharged the wastewater, 7 m^3, directly into the pilot plant. An Auto-sampler collected daily composite samples from the inlet pipe of the pilot plant. The auto-sampler (ISCO, 6712), consisting of 24 1 L bottles, was programmed to withdraw every 15 minutes 250 ml of grey water. For analysis a daily composite sample was prepared on-site, by mixing the content of the 24 bottles in one container of which 3 litres was taken for analysis, performed on the same day. In this way 60 composite grey water samples were collected and analysed over a period of 2 years.

4.2.2 Preparation of grey water samples, inocula, nutrient and buffer solutions for batch tests

Grey water sample preparation entailed 3 litres per day of fresh grey water composite samples of which 2 litres were filtered through Whatman No. 40 filter paper (8 μm). One litre of the latter filtrate was filtered through 0.45 μm membrane filters. The aerobic inoculum consisted of the effluent of the activated sludge units of the Abu-Nussier domestic wastewater treatment plant. 1 litre fresh activated sludge effluent was collected and filtered through Whatman No. 40 filter paper and the filtered liquid was used as inoculum. Anaerobic inoculum was taken from a 1.4 m^3 UASB-pilot plant at the Abu-Nussier domestic wastewater treatment plant. 1 litre anaerobic flocculent sludge was collected from the bottom, middle and upper taps, and analyzed for VSS. The COD to VSS, feed/sludge, ratio applied in the biodegradability tests was 0.5.

Different solutions were prepared and used within the research. A 300 mg COD L^{-1} glucose solution was prepared with distilled water. A 100 mM phosphate buffer solution was prepared according to (after Sørensen, 1955). The supplied micronutrient solution (Alphenaar, 1994) was composed of 2 g L^{-1} $FeCl_3.4H_2O$, 2 g L^{-1} $CoCl_2.6H_2O$, 0.5 g L^{-1} $MnCl_2.4H_2O$, 30 mg L^{-1} $CuCl_2.2H_2O$, 50 mg L^{-1} $ZnCl_2$, 50 mg L^{-1} HBO_3, 90 mg L^{-1} $(NH_4)_6Mo_7O_{24}.4H_2O$ mg L^{-1}, 100 mg L^{-1} $Na_2SeO_3.5H_2O$, 50 mg L^{-1} $NiCl_2.6H_2O$, 1 g L^{-1} EDTA, 1 ml L^{-1} HCl 36% and 0.5 g L^{-1} resazurin in demineralized water. Macronutrient solution was composed of 170 g L^{-1} NH_4Cl, 8 g L^{-1} $CaCl_2.2H_2O$ and 9 g L^{-1} $MgSO_4.7H_2O$ in demineralised water.

4.2.3 Biodegradability experiments

The biodegradability experiments were conducted to investigate the following aspects: the percentage biodegradable matter, the biodegradation rate constant k, and the factors affecting the biodegradation process. A total of 54 fresh composite samples were used to perform 98 tests i.e. 54 aerobic and 44 anaerobic tests (Table 4.1). The investigated variables were inoculum addition, incubation time, temperature and the COD fractions of the grey water. The biodegradability was indentified by measuring BOD, CH_4-COD, or COD mineralized fraction. The biodegradation rate constant (k) was calculated.

4.2.3.1 Tests setups

Biodegradability tests were performed in 510 ml bottles, filled with respectively 250 ml and 300 ml test medium for aerobic and anaerobic conditions. The liquid volume of the aerobic test was chosen for sufficient headspace for oxygen availability according to APHA (1995).

The tested samples were 100% grey water, 66% and 50% grey water diluted with distilled water, paper filtered grey water, membrane filtered grey water, and glucose solution as a reference. Tests without inoculum were fed with 248 ml grey water sample for the aerobic tests and 288 ml for the anaerobic tests. Tests with inoculum were fed with 228 ml sample and 20 ml inoculum and 268 ml sample and 30 ml inoculum for the aerobic and anaerobic tests, respectively. Micronutrients and macronutrient, each 1 ml, were added to all bottles in each test, and for the anaerobic tests 10 ml of 100 mM phosphate buffer was added. The tests were conducted in duplicate according to the setup in Table 4.1. Additionally for each test a blank was included where distilled water replaced the grey water sample.

Table 4.1. Aerobic and anaerobic biodegradability test setups: tested grey water fractions and the variables.

Test	Grey water fraction	Variables			
		Inoculum (- or +)	Temperature (℃)	Incubation time (days)	Dilution (- or +)
Aerobic	Total (38)	- (30)	20 (30)	28 (30)	- (30)
				60 (6)	- (6)
		+ (8)	20 (2)	28 (2)	- (2)
			25 (4)	28 (2)	- (2)
					+ (2)
			35 (2)	28 (2)	- (2)
	Paper-Filtrate (8)	- (6)	20 (2)	60 (6)	- (6)
		+ (2)	35 (2)	28 (2)	- (2)
	Membrane-Filtrate(8)	- (6)	20 (2)	60 (6)	- (6)
		+ (2)	35 (2)	28 (2)	- (2)
	Glucose*	+ (1)	25 (1)	6 (1)	- (1)
Anaerobic	Total (32)	- (24)	35 (24)	90 (24)	- (24)
				160 (4)	- (4)
		+ (8)	20 (2)	30 (2)	- (2)
			30 (4)	30 (2)	- (2)
					+ (2)
			35 (2)	30 (2)	+ (2)
	Paper-Filtrate (6)	- (4)	35 (2)	160 (4)	- (4)
		+ (2)	35 (2)	98 (2)	- (2)
	Membrane-Filtrate(6)	- (4)	35 (2)	160 (4)	- (4)
		+ (2)	35 (2)	98 (2)	- (2)
	Glucose*	+ (1)	30 (1)	6 (1)	- (1)

( ); number of the batches

*;glucose used as a reference for comparing with grey water biodegradability tests. Performed under the same conditions as the grey water biodegradability tests but with glucose instead of grey water sample.

The gas phase of the anaerobic bottles was flushed for 5 min with nitrogen gas to create anaerobic conditions. Soda lime pellets were added in the gas phase to aerobic and anaerobic bottles to adsorb CO_2. Aerobic bottles were re-aerated for 5 min after 28 days and the soda lime was checked to be active. If the test was continued, re-aeration was applied every 10 days until the test finished. The anaerobic bottles were checked for the consumption of soda lime pellets after 30 days and re-checked every 30 days until the test finished. During the test period the pressure was released when the maximum pressure capacity of the barometric heads (Oxitop®, WTW GmbH, Weilheim, Germany) was reached. Both in the aerobic and anaerobic tests the soda lime pellets did not require replacement. The activity of anaerobic and aerobic inocula was tested with glucose as carbon source. The tests were performed in the same manner as the biodegradability tests with inoculum. Instead of grey water, 300 mg COD L^{-1} glucose solution was used.

4.2.3.2 Analysis

The COD was measured of each sample fraction according to the standard methods (APHA, 1995) at the beginning and also at the end of the experiments if inoculum was not added. Pressure monitoring was done versus time by barometric Oxitop® (WTW GmbH, Weilheim, Germany) measuring devices. The pH was monitored at the beginning and end of each biodegradability test, with a pH meter (HACH-HQ40Dc).

4.2.4 Calculations

4.2.4.1 Grey Water Characteristics

Grey water constituents are categorized into COD fractions ($COD_{fraction}$) being total (COD_{tot}), suspended solids (COD_{ss}), colloidal (COD_{col}) and dissolved (COD_{dis}). The COD_{ss} is the difference between the COD of the total and of paper filtered (No. 40 Whatman filter paper) grey water; the difference between the COD of the paper filtered and membrane filtered (0.45 µm membrane filter paper) grey water is COD_{col}. The COD of the filtrate of the membrane filtration is COD_{dis}. The same principle is applied for characterizing the grey water in terms of BOD.

4.2.4.2 Biodegradability Characteristics

The biodegradability is expressed in terms of the mineralized fraction (COD), Biochemical Oxygen Demand (BOD) and Biological Methane Potential (BMP). The biodegradation rate is expressed as oxygen consumption (BOD) rate and methane production rate.

Mineralized fraction: Equations 4.1 to 4.4 in Table 4.2 were used to calculate the mineralized-COD fraction of the total, suspended, colloidal and dissolved components. The particle size is expected to decrease during degradation. However, an increase in particle size may occur, owing to flocculation, cell yield and coagulation. In order to calculate the mineralization over time of a particle size fraction that was present at the start of the experiment, the calculated value needs to be corrected for the amount of produced particles over that time period. For instance, at time = 0 the amount of COD_{ss} is given by A_1-A_2 in equation 4.2, Table 4.2, whereas at time = t the remaining amount of COD originating from COD_{ss} (t=0) is given by B_1-C_1 in equation 4.2, Table 4.2. The result of equations 4.1 to 4.5 will thus be the percentage of COD <u>mineralization</u> only.

Table 4.2. Formulas to calculate mineralized % COD per particle size fraction. The COD is derived from measurements of the COD of the liquid phase and the tests are conducted without inoculum and soda lime pellets.

	Bottles filled with								
	Total (tot) sample*			Paper filtrate (pf) sample			Membrane filtrate (mf) sample		
COD fractions	tot	pf	mf	tot	Pf	mf	tot	pf	Mf
At time =0.0	A_1	A_2	A_3		A_2	A_3			A_3
At time = t	B_1	B_2	B_3	C_1	C_2	C_3	D_1	D_2	D_3
	COD fractions								
COD	COD_{tot}		COD_{ss}			COD_{col}			COD_{dis}
Mineralized-fraction (%) =	$\frac{A_1 - B_1}{A_1} 100$ (4.1)		$\frac{(A_1 - A_2) - (B_1 - C_1)}{A_1 - A_2} 100$ (4.2)			$\frac{(A_2 - A_3) - (C_1 - D_1)}{A_2 - A_3} 100$ (4.3)			$\frac{A_3 - D_1}{A_3} 100$ (4.4)

*; total is the non-filtrated sample

BOD and BMP calculations: The pressure decrease ΔP was used to calculate the moles per litre of sample of O_2 consumption (BOD) and the pressure increase ΔP to calculate the CH_4 production (BMP) in moles per litre of sample using the ideal gas law given in equation 4.5.

$$\text{BOD or BMP (mol L}^{-1}) = \frac{\Delta PV}{RT}\frac{1}{\text{v}} \qquad (4.5)$$

where ΔP the measured pressure change (kPa), V the bottle head-space volume (L), R the gas constant (8.314 J K^{-1} mol^{-1}), T the temperature (K) and v is the sample volume (L). The calculated moles per litre of sample were then converted into mg O_2 per litre of sample.

Biodegradation rate constants: Aerobic and anaerobic first order biodegradation rate constants (k) of the grey water fractions are calculated using equations 4.6 (Metcalf and Eddy, 2003) and 4.7, respectively.

$$BOD_{fra-t} = BOD_{U\text{-}fra}\left[1 - Exp(-k_{fra}\,t)\right] \quad (4.6)$$

where $BOD_{fra\text{-}t}$ is $BOD_{tot\text{-}t}$, $BOD_{ss\text{-}t}$, $BOD_{col\text{-}t}$ or $BOD_{dis\text{-}t}$ at time = t. $BOD_{U\text{-}far}$ is the ultimate BOD_{fra} (mg L^{-1}) and k (day^{-1}) the kinetic rate constant. Both $BOD_{U\text{-}fra}$ and k are derived from the curve calculated with the Microsoft Excel solver function.

$$CH_4 - COD_{fra-t} = BMP_{fra}\left[1 - Exp(-k_{fra}\,t)\right] \quad (4.7)$$

where CH_4-COD refers to methane production; CH_4-$COD_{fra\text{-}t}$ is CH_4-$COD_{tot\text{-}t}$, CH_4-$COD_{ss\text{-}t}$, CH_4-$COD_{col\text{-}t}$ or CH_4-$COD_{dis\text{-}t,}$ at time = t. BMP_{fra} Biological Methane Potential refers to the maximum CH_4-COD_{fra} (mg L^{-1}) and k the kinetic rate constant (day^{-1}). Both BMP_{fra} and k are derived from the curve calculated with the Microsoft Excel solver function.

Temperature impact on rate constant (k) measured by (θ): Equation 4.8 is derived from the Van't Hoff-Arrhenius relationship (Metcalf and Eddy, 2003), and θ is the exponential of the slope ln(θ).

$$\ln\left(\frac{k_{T_1}}{k_{T_2}}\right) = \left(T_1 - T_2\right)\ln\left(\theta_{20-35}\right) \quad (4.8)$$

where k_{T1} and k_{T2} are the kinetic rate constants (day^{-1}) at T_1 and T_2 respectively, T the temperature, in the range 20 to 35℃, and θ the temperature coefficient. Values of k are calculated from equations 4.6 and 4.7.

4.2.4.3 Reactor design

Equation 4.9; derived based on mass balance, assuming CSTR, steady state conditions and first order kinetics, is applied to calculate the digestion time (SRT) required in a CSTR, assuming steady state, to digest grey water total pollutants for aerobic and grey water suspended pollutants for anaerobic treatment.

$$SRT = \frac{C_0 - C_t}{k\, C_t} \tag{4.9}$$

where SRT is the Sludge Retention Time (days), k the kinetic rate constant (day^{-1}) of the total or suspended samples (k_{tot}), C_0 the initial and C_t the concentration of digestible substrate at SRT= t (mg COD_{tot} L^{-1}).

4.3 Results and Discussion

Grey water from a dormitory at Jordan university campus was characterized for its concentration of different COD fractions, maximum biodegradability and conversions rate of the different fractions. The concentration of the different fractions is presented in Table 4.3. The effect of applying different conditions in the biodegradation test is shown in Figure 4.1. The maximum biodegradability is reported in Table 4.4, and the bioconversion rate constant and the temperature impact on the rate in Table 4.5.

4.3.1 Grey Water Characteristics

The investigated grey water consisted of combined wastewater from laundry, shower and hand washbasins. The pollutants include detergents and components washed from humans and/or textile. Table 4.3 gives the COD of the total grey water and the suspended solids, colloidal and soluble percentage of the grey water.

Table 4.3. Grey water COD fractions found in this study and in the literature

COD	Units	Grey water Sources: Dormitory at Jordan University Campus			Sneek in Groningen, the Netherlands		Flintenbreite' settlement Luebeck-Germany	
Total	mg L^{-1}	362	(100)	[54]	827	(204)	649	(124)
Suspended	%	28	(9)	[54]	49		50	
Colloidal	%	32	(8)	[54]	24		30	
Dissolved	%	40	(7)	[54]	27		20	
references		This study			Hernandez *et al.* (2008)		Elmitwalli and Otterpohl (2007)	

( ); standard deviation.
[ ]; number of samples.

The COD of Jordanian dormitory grey water, as measured in this study, is less than half of the concentrations reported for grey water measured by Elmitwalli and Otterpohl (2007) and Hernandez *et al*. (2008) (Table 4.3). The likely reason for this is that the dormitory grey water used in this study is a combination of shower, laundry and washbasin streams, whereas the wastewater sources of the other two studies include the kitchen stream. The kitchen wastewater is the most concentrated stream in terms of COD and has the highest content of suspended solids (Abu Ghunmi *et al*., 2008).

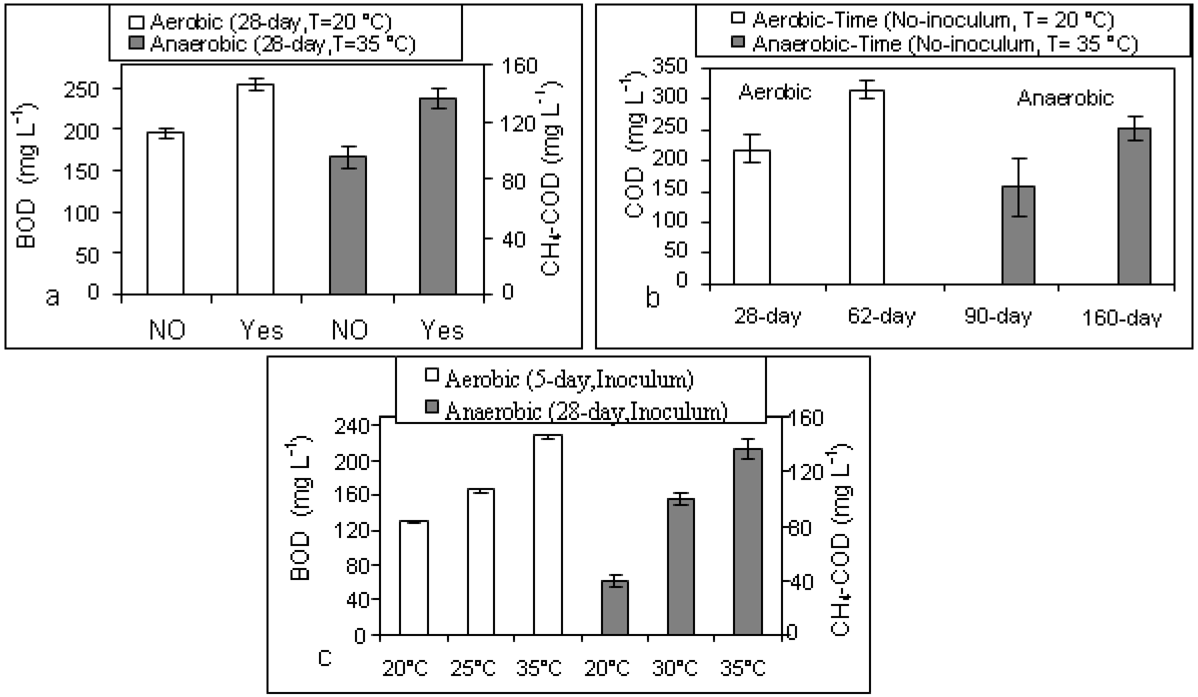
4.3.2 Biodegradability Characteristics

4.3.2.1 Influencing Factors

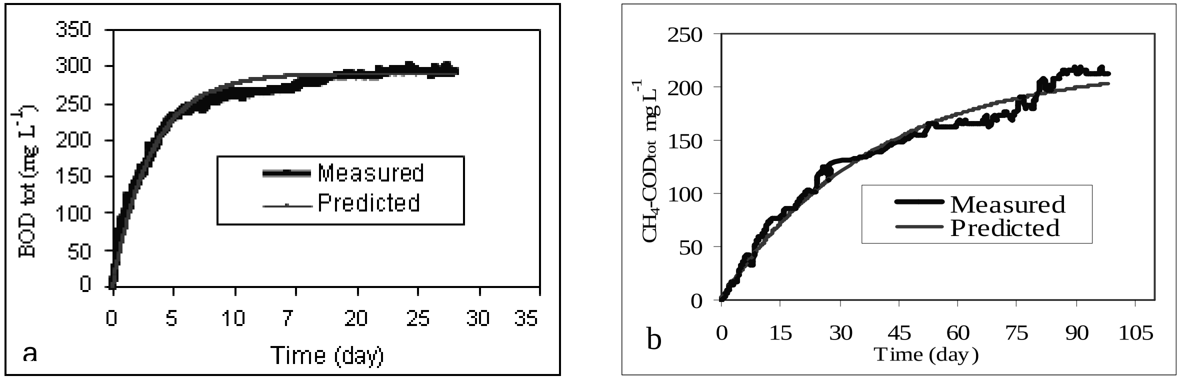
The biodegradability of grey water is affected by the inoculum addition, applied incubation time and temperature (Figure 4.1). The addition of inoculum has, at a degradation time of 28 days, a positive effect on both $BOD_{28\text{-}20℃}$ and CH_4-$COD_{28\text{-}35℃}$ production (Figure 4.1 a). This is attributed to the lack of initial natural microbial activity in the grey water, which is insufficient in a period of 28 days compared to tests where inoculum has been added. However, when the incubation time of the mineralization tests without inoculum is extended, the amount of mineralized COD increases (Figure 4.1 b). Theoretically, not adding inoculum has no influence at infinite time scales because ultimately the few microorganisms present in the grey water will multiply and have ample time to degrade all substrate. In practice, however, shorter time scales are relevant and adding adapted inoculum will accelerate the degradation process. The effect of increasing temperature on the biodegradation is, like adding inoculum, an increase in BOD_5 and CH_4-COD_{28} production caused by microbial activity that accelerates with rising temperature (Figure 4.1 c).

4.3.2.2 Biodegradability

The biodegradability measured as amount COD mineralized, BOD and/or CH_4-COD varies largely with inoculation, temperature and degradation time as shown in Figure 4.1. Finding the Ultimate BOD (BOD_U) and maximum CH_4-COD i.e. Biological Methane Potential (BMP) minimizes these variations to a single value. Figure 4.2 a and 4.2 b show the cumulative oxygen consumption (BOD) and methane production (CH_4-COD) versus time of the grey water. The maximum biodegradation is reached where the lines level horizontally. In Table 4.4 the BOD_U and BMP are given as percentage of the influent COD.



Figures 4.1. (a) Effect of adding inoculum on $BOD_{28\text{-}20°C}$ (mg L^{-1}) and $CH_4\text{-}COD_{28\text{-}35°C}$ production (mg L^{-1}) , (b) effect of incubation time on the aerobic and anaerobic COD mineralized fraction (mg L^{-1}) and (c) effect of temperature on the BOD_5 (mg L^{-1}) and $CH_4\text{-}COD_{28}$ (mg L^{-1}) production. Legends notations: experiment variables and between brackets the experiment fixed conditions. All samples are total grey water.



Figures 4.2. (a) BOD and (b) CH_4-COD production versus time of total grey water. Test was performed with inoculums at 35℃. The predicted values were calcul ated for the aerobic tests using equation 4.6 and for anaerobic tests equation 4.7.

The BOD_U/COD_{tot} is slightly lower than the mineralized-COD fraction after 60 days aerobic incubation (Table 4.4). The same is true for the BMP/COD_{tot} and mineralized-COD fraction after 160 days anaerobic incubation. The mineralization on the basis of COD can be overestimated when substrate adsorbs over time to the bottle walls, such as lipids (Sanders, 2001), and is not sampled for the determination of the end COD. Furthermore, measuring anaerobic biodegradation by means of pressure monitoring in the presence of soda lime pellets to exclude CO_2 can cause problems. This negative effect might be due to an increase of the pH (Pabon *et al*., in prep). However, the pH, in this study, did not change between the starting, 7.5±0.5, and the end, 7.2± 0.5, of the experiments viz. 98 days. Moreover, when testing the pressure development on glucose (this study, results not presented), for 3 days, at 300 and 1200 mg COD L^{-1} with and without soda lime pellets the pH remained within (6.5-7.5) favourable operation limits or did not change at all.

Table 4.4. Mineralized COD fraction, BOD_U and BMP as a percentage of the influent COD for total grey water and the various fractions.

Tests	Parameters	COD Fractions: Total (tot) %	Suspended (ss) %	Colloidal (col) %	Dissolved (dis) %	Sludge
Aerobic biodegradation	$*COD_{28d-20℃}$	62 (6) [30]				No
	$*COD_{60d-20℃}$	86 (4) [6]	78 (1) [6]	90 (3) [6]	87 (2) [6]	
	$**BOD_{U-28d-20℃} / COD_{tot}$	75 (4) [2]	76 (5) [2]	85 (6) [2]	86 (4) [2]	Yes
Anaerobic biodegradation	$*COD_{90d-35℃}$	44 (13) [24]				No
	$*COD_{160d-35℃}$	70 (6) [4]	60 (6) [4]	77 (6) [4]	71 (1) [4]	
	$**BMP_{98d-35℃} / COD_{tot}$	57 (5) [2]	51 (3) [2]	67 (7) [2]	56 (3) [2]	Yes

( ); standard deviations:

[ ]; number of the batches:

*; Only based on determined COD removal:

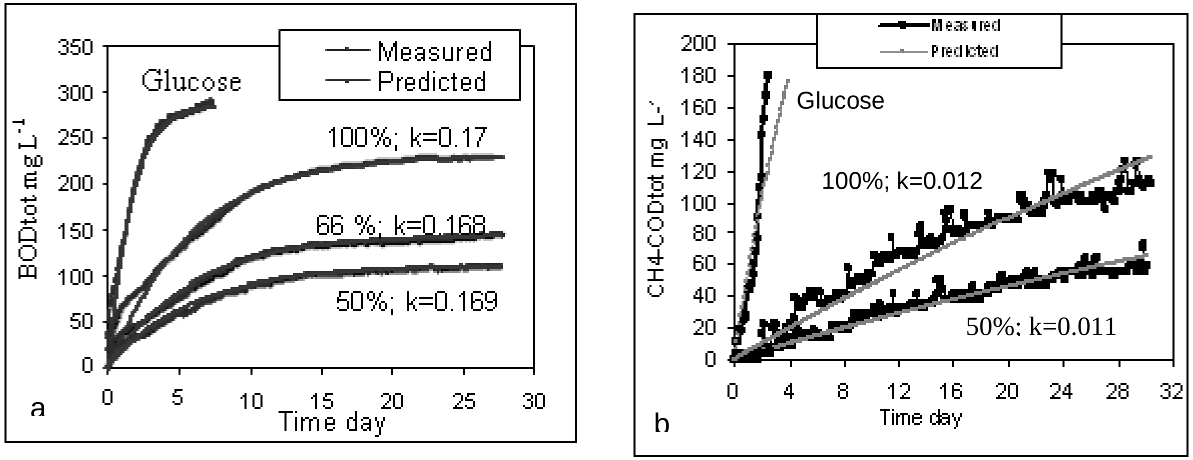
**; based on determined CH_4 production or oxygen consumption

In Table 4.4 mineralization of total grey water and the fractions is shown. Under aerobic and anaerobic conditions the conversion percentages of the colloidal (col) and dissolved (dis) fractions are significantly higher (t-values >2) than the conversion percentage of the suspended (ss) fraction.

The BOD_5/COD ratios of 43± 4, found in this study for a temperature of 20℃ are within the range found in literature of 25 -64% (Jefferson *et al*., 1999; Friedler *et al*., 2005; 2006; Winward *et al*., 2008). The anaerobic biodegradability values found in this study for the different COD fractions (Table 4.4) are similar to the values reported by Elmitwalli and Otterpohl (2007); which are 74% for total grey water, 70% for suspended solids, 84% for colloidal matter and 70% for the dissolved fraction.

4.3.2.3 Biodegradation rate

The amount of biodegradable matter digested in a certain period of time and the kinetic rate constants (k) vary with the inoculum addition and temperature (Figure 4.1 a and 4.1 b). To minimize the variation in the measured values of k, the mineralization process should not be limited by inoculum availability, but only by the nature of the organic matter. Figure 4.3 a and 4.3 b show grey water aerobic and anaerobic biodegradation. The oxygen consumed (BOD) or methane produced (BMP-COD) versus time is apparently not limited by the biomass availability: at different dilutions of grey water the amount of oxygen consumed or methane produced (mg) versus time was inversely proportional to the dilution. The degradation rate of grey water was compared to a reference sample containing glucose as carbon source under the same conditions. The grey water biodegradation rate (Table 4.5) was 4x slower aerobically (k_{tot-25}) and 12x slower anaerobically (k_{tot-30}) than with the glucose. Thus the grey water biodegradation rate is apparently limited by the chemical nature of its constituents. Grey water aerobic and anaerobic biodegradation rates can be described by first order kinetics (Figure 4.2 a and 4.2 b; 4.3 a and 4.3 b). The different COD fractions in grey water are degraded at different rates (Figure 4.2 a and 4.2 b; Table 4.5). The dissolved fraction is biodegraded at the lowest rate, both for aerobic and anaerobic conditions (Table 4.5).



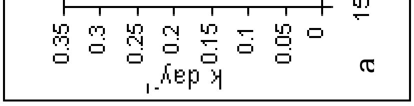
Figures 4.3. (a) BOD for 100, 66 and 50%and (b) CH_4-COD production for 100, and 50% grey water sample in water versus time, with glucose as a reference. The degradation tests were performed with inoculums and soda lime pellets at 25℃ for aerobic and 30℃ for anaerobic conditions. The predicted values were calculated for the aerobic tests using equation 4.6 and for anaerobic tests equation 4.7.

Table 4.5. The effect of temperature on anaerobic and aerobic degradation rate constants of grey water.

Tests	Parameters	Units	COD-Fractions (fra) Total	Suspended	Colloidal	Dissolved	Sludge
Aerobic biodegradation	$k_{20℃}$	day^{-1}	0.119 (0.005) [2]				Yes
	$k_{25℃}$	day^{-1}	0.174 (0.006) [2]				
	$k_{35℃}$	day^{-1}	0.318 (0.004) [2]	0.334 (0.004) [2]	0.343 (0.005) [2]	0.264 (0.004) [2]	
Anaerobic biodegradation	$k_{20℃}$	day^{-1}	0.005 (0.0008) [2]				Yes
	$k_{30℃}$	day^{-1}	0.011 (0.001) [2]				
	$k_{35℃}$	day^{-1}	0.023 (0.001) [2]	0.069 (0.002) [2]	0.036 (0.001) [2]	0.011 (0.001) [2]	

( ); standard deviations
[ ]; number of the batches.

The overall aerobic biodegradation rate (k_{tot}) is higher than the anaerobic rate, for instance, at 20℃ the aerobic rate is 24 times higher than the anaerobic rate (Table 4.4). Both the aerobic and anaerobic rates are lower than the aerobic biodegradation rate of domestic wastewater ($k_{20℃}$ 0.23 day^{-1}) (Metcalf and Eddy, 2003). To control the effect of use soda pellets for CO_2 removal, on the measured conversion rate, anaerobic batch experiments with and without pellets were conducted. For testing this anaerobic granular sludge was used with sufficient methangenic activity and a substrate concentration of 300 mg glucose L^{-1}, no effect of soda pellets was observed on the conversion rate. The increase in temperature from 20 to 35℃ results in an exponential increase in the rate constants values (k) (Fig. 4.4 a and 4.4b), which can be described via an Arrhenius relation, with an Arrhenius coefficient θ_{20-35} of 1.069 for aerobic and 1.099 for anaerobic conditions.



b

Figures 4.4. a and b Aerobic and anaerobic first order biodegradation rate constants (k) versus temperature. The tests were performed with added inoculums and soda lime pellets. The calculated k is derived from equations 4.6 and 4.7 and the predicted k is based on the calculated values of $\theta_{20\text{-}35}$, from equation 4.8.

4.3.4 Features of grey water biological processes

Designing biological processes for treatment of grey water, the parameters of concern are sludge retention time (SRT) and the temperature. In the winter time when the temperature in the reactor drops from 25℃ to 20℃ the biodegradation rates show a 1.8x and 1.4x reduction for aerobic and anaerobic degradation, respectively (Table 4.5). The sludge retention should be sufficient to provide enough stabilisation time, and the reactor should be designed for 20℃.

The anaerobic conversion of COD_{ss} and aerobic conversion COD_{tot} as a function of time at 20 ℃ were calculated, using equation 4.9, and depicted in Figure 4.5. It shows that long an SRT is required to treat grey water, both anaerobically and aerobically. For example, for 60% COD_{ss} digestibility 90 days SRT is required anaerobically and for COD_{tot} digestibility 12.6 days SRT is required aerobically. As a consequence, a reasonable treatment in terms of time and volume requires that the applied unit should have the capacity to trap pollutants in a short hydraulic retention time (HRT), while also maintaining a sufficient sludge retention time (SRT) for treatment. For example, Elmitwalli and Otterpohl (2007) treated grey water in an UASB at an HRT of 6, 10 and 16 hours, and at minimum SRT of 27, 64, and 93 days respectively, the COD_{ss} removal efficiencies were 68±17, 79±8 and 83±5% respectively.

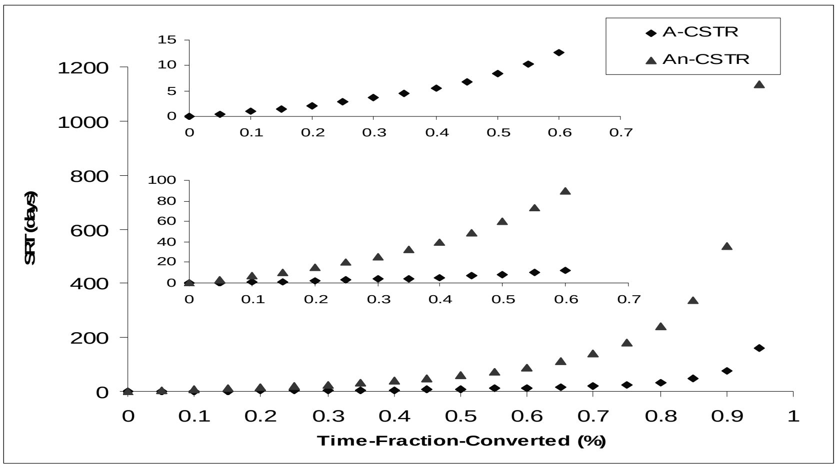


Figure 4.5. Grey water aerobic and anaerobic treatment in a Continuous Stirred Tank Reactor (CSTR): SRT versus COD converted fraction at 20℃. A refers to Aerobic and An refers to Anaerobic. The two small inset drawings zoom in the scale from 0.0 to 0.6 to depict the trends.

4.4 Conclusion

Grey water, including laundry, hand washbasin and shower wastewater, is characterised by a suspended, colloidal and dissolved COD fraction of respectively 28%, 32% and 37%.

Incubation time, inoculum addition, and temperature affect the aerobic and anaerobic biodegradability results.

The maximum biodegradability of the suspended COD fraction in grey water is lowest in comparison with the colloidal and dissolved fraction.

The biodegradation of grey water and the 3 distinguished COD fractions can be described with first order kinetics.

The dissolved COD fraction of domestic wastewater is degraded at the lowest rate.

Aerobic and anaerobic treatment of grey water needs at 20℃ long SRT, for instance 12.6 and 90 days, for 60% COD_{ss} and COD_{tot} conversion respectively.

Acknowledgment

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Chapter Five

Grey water treatment in a sequencing anaerobic – aerobic system for irrigation

Abstract:

This study aims at treatment of grey water for irrigation, focusing on a treatment technology that is robust, simple to operate and with a minimum energy consumption. The tested system consists of an anaerobic unit followed by an aerobic unit, integrating treatment and storage of the grey water. The treatment concept is optimized for the time period of the operational cycle, the discharge time, the flow directions and the aeration technique. The optimized effluents and sludge are characterised and the effluents are checked if they comply with irrigation standards. The result is an optimized system consisting of an anaerobic unit operated in upflow mode, with a 1 day operational cycle, a constant effluent flow rate and varying liquid volume. The subsequent aerobic step is equipped with mechanical aeration and the

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Abu Ghunmi, L., Zeeman, G., Fayyad, M. and van Lier, J. B. Grey water treatment in a sequencing anaerobic –aerobic system for irrigation

system is insulated for sustaining efficiency under winter conditions. The COD removal achieved by the anaerobic and aerobic units in summer and winter are 45%, 39 % and 53%, 64%, respectively. The sludge in the anaerobic and aerobic reactor has a concentration of 168 and 11 mg VS L^{-1}, respectively. The stability of the sludge in the anaerobic and aerobic reactors is 80% and 93 % respectively, based on COD. The aerobic effluent quality, except for the pathogens, agrees with the proposed irrigation water quality guidelines for reclaimed water in Jordan.

Keywords: grey water, aerobic, anaerobic, temperature, irrigation.

Nomenclature

AnF	Anaerobic Filter	PVC	Polyvinyl Chloride
BOD	Biochemical Oxygen Demand	SBF	Submerged Biofilter
COD	Chemical Oxygen Demand	SBR	Sequencing Batch Reactor
col	Colloidal	SMA	Sludge Methanogenic Activity
CSTR	Continuous Stir Tank Reactor	SRT	Sludge Retention Time
dis	dissolved	ss	Suspended Solids
E. coli	*Escherichia coli*	Tot	Total
HRT	Hydraulic Retention Time	TS	Total Solids
k	Conversion rate constant	UASB	Upflow Anaerobic Sludge Blanket
max	Maximum	VS	Volatile solids
min	Minimum	Ø	Diameter

5.1 Introduction

Grey water consists of diluted domestic wastewater streams, namely, from the shower, laundry facilities and/or the washbasins, whereas in some studies grey water includes the kitchen wastewater (Nolde, 1999; Jefferson *et al*., 1999; 2000; 2004; Erikson *et al*., 2002; Elmitwalli and Otterpohl, 2007). Biological treatment is a prerequisite for storing and/or reusing grey water (Nolde, 1999; Jefferson *et al*., 2004; Abu Ghunmi *et al*., 2008). The feature of the recommended biological treatment is a system that consists of an anaerobic pre-treatment followed by aerobic post treatment (Chapter 1; Abu Ghunmi *et al*., 2008), applying systems providing a long sludge retention time (SRT) and short hydraulic retention times (HRT), (Chapters 1 and 3). A long SRT is required to maximise the pollutants conversion and system stability because of slow biodegradability of the grey water pollutants (Chapter 3). A Short HRT is required to minimize treatment unit volume.

A distinguishing important feature of biological treatment is that it stabilises in a controlled manner the organic matter that otherwise causes problems; such as malodour, mosquito breeding, water and soil pollution, and clogging in the distribution systems (Christova-Boal *et al*., 1995; Diaper *et al*., 2001; Erikson *et al*., 2002). A treatment system that consists of an anaerobic step followed by an aerobic step has the advantages of reducing the oxygen demand and converting organic matter into methane.

In literature tested processes for treating grey water aiming at the maximum stabilisation of the wastewater, in principle apply either long HRTs such as constructed wetlands with HRTs of 2 to 14 days (Fittschen and Niemczynowicz, 1997; Winward *et al*., 2008) or membrane/ultra-filtration bio-reactors, which apply long SRT, with an HRT of less than 1.5 day (Jefferson *et al*., 1999; 2001; Winward *et al*., 2008). The first mentioned system has the disadvantage that large areas are needed e.g. 1.4 to 3.25 m^2 Cap^{-1} (Nolde and Dott, 1992; Hegemann, 1993; Li *et al*., 2003). Moreover, evaporation and rainfall, especially in the dry regions, are crucial issues (Fittschen and Niemczynowicz, 1997). The second systems, the membrane/ultra-filtration bio-reactors, have high energy demands and maintenance requirements (Jefferson *et al*., 1999; 2000), and are therefore not suited for on-site situations in developing countries. Use of treated grey water for irrigation in countries with low precipitation is an attractive option, as grey water is low in salts (Abu Ghunmi *et al*., 2008) and not polluted with toilet waste(water). Applying on-site treatment and use of grey water, provided that black waters is treated on-site as well, minimizes transport costs; i.e. distinctly reducing investments for installing and maintenance of sewerages system (van Riper and Geselbracht, 1998; Lettinga *et al*., 2001).

The grey water production rate varies considerably over the day (Butler *et al*., 1995; Imura *et al*., 1995; Abu Ghunmi *et al*., 2008). To prevent the introduction of a storage tank, the treatment concept should be designed to cope with varying loading rates. Three different concepts, integrating storage and treatment of the grey water, are tested within this study, viz. (a) a system operated in fed-batch/batch mode at varying inflow rates over the day, (b) a system operated with a constant liquid reactor volume, 1-day HRT, at varying inflow rates over the day and (c) a system operated at varying inflow rates and varying liquid reactor volume, at constant outflow rate. The objective of the study is to achieve on-site treatment of the wastewater to produce an effluent that complies with irrigation requirements. Such processes should be robust to deal with intermittent flow and changing weather conditions, simple to operate, and with a minimum consumption of energy. The approach employed in this study is (1) to construct a pilot plant that applies the anaerobic-aerobic biological treatment concept with an internal (design) structure that accelerates the physical removal of the pollutants. (2) To monitor the performance of the pilot plant in terms of the operational conditions: the time period of the operational cycle, discharge time, flow direction and aeration techniques. (3) To characterise effluent and sludge quality and to test the effluent suitability for irrigation at maximum removal capacity of the pilot plant.

5.2 Methodology

5.2.1 Grey water source

The source of the grey water is a dormitory for 150 students, at the Jordan university campus, discharging the waste streams of shower, laundry and washbasins into a single outlet pipe to the sewer. The outlet pipe was retrofitted to discharge, out of 7-10 m^3 day^{-1}, 1 m^3 day^{-1} of the wastewater directly into the pilot plant. The remaining wastewater was discharged to the sewer. The wastewater entered the pilot plant by gravitational force (Figure 5.1) following the discharge pattern of the dormitory. The inflow pattern was measured, by Abu Ghunmi *et al*. (2008), using a water recorder (Steven 68) and digitised by ArcGIS software (ESRI product-ArcGIS 8.1). During the university holidays, 21 days in winter and 30 days in summer, the inflow to the pilot plant was zero.

5.2.2 Pilot plant features

The basis of the construction was the design of an internal structure for the pilot plant to increase the solids removal in order to enhance its retention. The pilot plant consisted of two Polyvinyl Chloride (PVC) tanks, the first as the anaerobic unit and the second as the aerobic compartment. The internal structure of each of the PVC units was composed of 6 vertical PVC rods; 2.54 cm Ø with a length of 1.38 m or 1.12 m, attached to the bottom of the tank. The rods held up 14-15 corrugated polyethylene discs; 30 cm Ø, 3 mm thickness, 5 grooves with 3 mm depth. PVC rings; 5 cm Ø, 3 mm thickness and 5 cm length were inserted between the consecutive discs. The influent was loaded into the treatment system gravitationally in a down flow or up flow mode. For the down flow mode the grey water entered at the top of the water surface, and for the up flow the wastewater entered the unit through an internal PVC tube at the bottom of the tank. The anaerobic and aerobic working volumes were 1.18 m^3 and 0.98 m^3, respectively. Each tank was equipped with four taps; the heights starting from the bottom 0.13, 0.2, 0.32 and 0.83 m. Furthermore, the anaerobic tank was equipped with a water level controller (5 cm Ø) to prevent overflow. The pilot plant layout design is depicted in Figure 5.1.

Two approaches were tested for aerating the aerobic tank. The first approach was a natural aeration aimed at direct contact of liquid and sludge with the air. The technique applied was feed batch (Table 5.1) combined with a surface distribution system, installed at the top of the tank. The distribution system consisted of 3 m long perforated tubes with holes Ø 3 mm every 3 cm. The second approach was mechanical aeration by pumping air 24 hours. The air was pumped into the unit through two PVC tubes (diameter 1 cm) inserted through the bottom of the tank and connected to an air compressor (model -Z-0.12/8; MFG no. 2522). Both the aerobic and anaerobic unit were insulated in winter using fibreglass sheets.

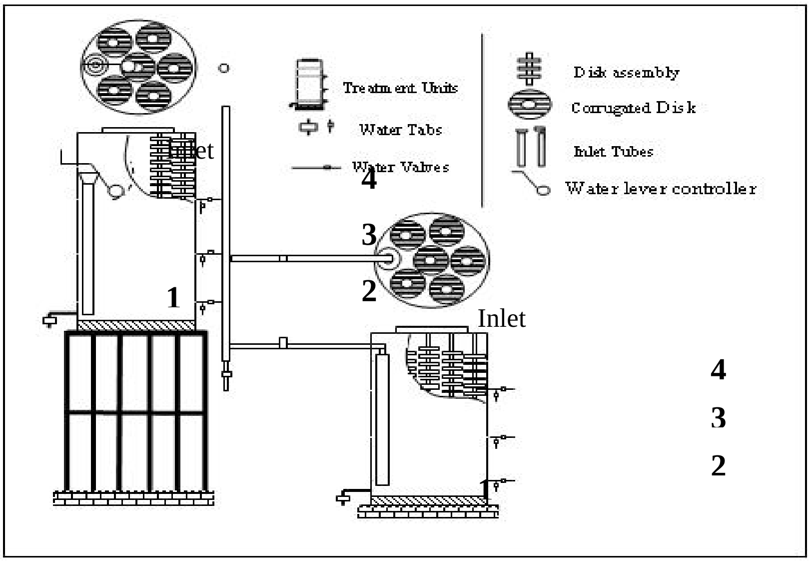

Figure 5.1. Pilot plant setup; the outside and inside structures. For detailed information on the use of the anaerobic and aerobic discharge taps see Table 5.1.

Table 5.1. The pilot plant operational conditions; operational phases, flow direction, liquid reactor volume, aeration techniques and testing period

Treatment units modes of operations	Operational Conditions							Testing Period days	Discharging points
	Operational phases				Flow-direction	Liquid Reactor Volume	Aeration Techniques		
	Fed batch (hrs)	Digestion Batch (hrs)	Discharging period (hrs)	Total Operational cycle (days)					
Anaerobic									
[c]	Natural[a] (0); Fig 5.2c	NO	24 continuous	1	Up	Variable		280	tap 3
[a]	Natural (13); Fig 5.2a	8	3 continuous	1	Up	Variable		20	tap 2
		32		2				20	
		56		3				21	
		80		4				20	
		104		5				20	
[b]	Natural (0); Fig 5.2b	11	13 Natural	1	Up	Constant		60	tap 4
[c]	Natural (0); Fig 5.2c	NO	24 continuous	1	Down	Variable		60	tap 3
[c]	Natural (0); Fig 5.2c	NO	24 continuous	1	Up	Variable		370	tap 3
Aerobic									
[d]	Constant volume (21)	NO	3 continuous	1	Down	Variable	Natural	145	tap 2
[e]	Constant volume (0)	NO	24 continuous	1	Up	Constant	Compressor [b]	220	tap 4

a; natural = intermittent and variable pattern: [ ]; operational modes: b; The compressor was regulated to pump air to keep the oxygen level in the liquid phase over 2 mg L^{-1}.

5.2.3 Pilot plant performance

5.2.3.1 Operating the system

The anaerobic unit was inoculated with 100 litres of anaerobic flocculant sludge from the Abu Nussier UASB plant that treats domestic wastewater. The aerobic unit was inoculated with 100 litres of aerobic flocculant sludge from the activated sludge unit of Abu Nussier.

The anaerobic unit was operated for 871 days, during the weekday, at three different operational modes, viz. (a) operated in fed-batch/batch mode at varying inflow rates over the day; the flow during the batch mode, after filling the tank, was discharged to the sewer. (b) operated at varying inflow rates over the day and constant liquid reactor volume and (c) operated at varying inflow rates and varying liquid reactor volume, at constant outflow rate. The operating time schedule and the operational conditions are given in Table 5.1. The aerobic unit was operated for 395 days, starting on day 476 of the anaerobic unit operation. The aerobic unit operational modes are (d) natural and (e) mechanical aeration, the operating time schedule and the operational conditions are given in Table 5.1. Each unit showed stable reactor performance in 90 days, based on the achievement of a stable effluent in terms of the COD concentration. Moreover, no sludge was discharged from either of the units during the whole operational period.

5.2.3.2 Sampling and analysis

Sampling techniques: An Auto-sampler collected daily composite samples from the inlet and the outlet tubes from the anaerobic tank. The auto-sampler (ISCO, 6712), consisting of 24 1 L bottles, was programmed to withdraw every 15 minutes 250 ml of grey water. For analysis of a composite day sample, the 24 bottles were emptied in one container and this mixed sample was subsequently analyzed. A collection tank was used to accumulate the daily aerobic effluent, from where the sample was taken for the effluent analysis. Sampling frequency was three times a week, the volume of sample from each source was 3 litres.

The anaerobic and the aerobic sludge were tested at the end of the operating period. Grab sludge samples were collected during the morning filling periods. These sludge samples were collected from the sludge discharge tap and from taps one, two and three (Figure 5.1). The volume of the sample that was collected from each tap was 1 litre, the samples were separately analyzed.

Analyses: the influent, the effluent and the sludge were fractioned into total, paper (pf) and membrane (mf) filtrates; the total sample filtered through no. 40 Whatman filter paper, then the fraction of the latter filtrate was subsequently filtered through 0.45 µm membrane filter paper. Part, less than 10%, of the influent and effluent samples were only analyzed for total and/or dissolved fractions. The total influent and effluent samples were analysed for BOD and pathogens (Total Coliform, Faecal Coliform and *E*. *coli*). The influent and effluent samples and their fractions were analysed for Chemical Oxygen Demand (COD), Nitrogen (N), Phosphorous (P), solids and Volatile Solids (VS). The analysis frequency was 3 times a week, for all the measurements except for pathogens, N and P, which were measured twice per month. In winter time the samples were just analyzed for COD fractions 3 times a week. All analyses were performed following standard methods (APHA, 1995).

To establish the sludge stabilization, or rather the fraction of non stabilized material, in the reactors the anaerobic sludge and the aerobic sludge digestibility were determined. The anaerobic sludge and the aerobic sludge digestibility were determined in serum bottles, in duplicate, and for 160 days at 30 and 20℃ respecti vely. The serum bottles content was 150 ml of anaerobic or aerobic sludge, 10 ml of 100 mM phosphate buffer and 1 ml of micronutrients and 1 ml of macronutrients stock solution (Chapter 3). The anaerobic sludge serum bottles were flushed with nitrogen for 5 min to create anaerobic conditions. Biogas production was monitored over time by a pressure measuring device (SPER-manometer: 840083). The aerobic sludge serum bottles was aerated for 30 min to create aerobic conditions, every day for 10 days, after that every 3 days for 10 days, and then every 10 days till the end of the experiment. For the anaerobic and aerobic sludge the initial and end COD, and the initial Total Solids (TS)/VS were determined, furthermore the TS/VS ratio of the anaerobic sludge was determined at the end.

The anaerobic sludge maximum specific methanogenic activity is measured in serum bottles by monitoring the methane production rate with Oxitops in duplicate, using glucose as carbon source, for 8 days at 30℃. The serum bottles were filled with 1.25 g VS L^{-1} anaerobic sludge, 100 ml of 1000 mg COD L^{-1} glucose, 10 ml of 200 mM phosphate buffer and 1 ml of each of the micronutrient and macronutrients (Chapter 3). The serum bottles were flushed with nitrogen for 5 min to create anaerobic conditions, soda pellets added, and then the bottles were closed tightly. The slope of the steepest part of the pressure curve was taken to calculate the maximum methanogenic rate.

5.2.4 Calculations

5.2.4.1 Characteristics

The constituents of the raw grey water, the pilot plant effluents, and sludge were classified into total, suspended, colloidal and dissolved fractions. The total is the total sample, the colloidal is the deference between total and the paper filtrate (pf), and the dissolved is the membrane (mf) filtrates.

5.2.4.2 Tank Volume

Equation 5.1 is applied to calculate the hourly discharging volume, and equation 5.2 to calculate the tank liquid volume on an hourly basis.

$$Q_{eff} = Q_{in} \text{ and the HRT is 1 day; or } Q_{eff} = \left(\frac{\text{daily inflow volume}}{\text{discharge hours}} \right) \qquad (5.1)$$

$$V_L = \sum_{t=0}^{t} Q_{in} \, t\,(hr) - \sum_{t=0}^{t} Q_{eff} \, t\,(hr) \qquad (5.2)$$

Where: V_L; Liquid Volume (L); Q_{in} and Q_{eff} are respectively influent and effluent (discharging) flow rates (L day^{-1}).

5.2.4.3 Sludge Retention Time (SRT)

Equations 5.3 and 5.4 are applied to calculate the minimum and maximum SRT, respectively.

$$SRT_{max} = \left(\frac{V\,X}{Q_s\,X_s} \right) \tag{5.3}$$

$$SRT_{min} = \left(\frac{V\,X}{Q_s\,X_s + Q_{eff}\,X_{eff}} \right) \tag{5.4}$$

Where: SRT; Sludge Retention Time (day), V; Sludge Volume (L); X and X_{eff} ; are respectively sludge concentration in the reactor (g VS L^{-1}) and suspended sludge concentration in the effluent (g VS L^{-1}), X_s; concentration of the discharged sludge (g VS L^{-1}). Q_s and Q_{eff} are respectively sludge daily discharge volume and effluent (discharging) flow rate (L day^{-1}).

5.2.4.3 COD conversion

Equations 5.5 and 5.6 are applied to calculate the COD conversion fractions in batch reactor and Continuous Stirred Tank Reactor (CSTR) respectively, assuming first order kinetics.

$$\frac{C_o - C}{C_o} = \left(1 - \mathrm{Exp}(-kt)\right) \tag{5.5}$$

$$\frac{C_o - C}{C_o} = \frac{k\theta}{(1 + k\theta)} \tag{5.6}$$

Where: C_o and C (mg L^{-1}); substrate concentration at time zero and t or Ɵ, respectively, K (day^{-1}); conversion rate constant.

5.3 Results and discussion

5.3.1 Operational modes

The minimum reactor volume required for treating grey water in 1 day operational cycle is the daily wastewater volume (Figures 5.2 a, b and c). The volume is used for treating as well as for storing grey water. The changes in the discharging modes do not affect the volume required in the reactor at an operational cycle of one day (Figure 5.2). The grey water inflow characteristics show no variation in the weekdays (Abu Ghunmi *et al*., 2008). Furthermore, the daily inflow pattern is intermittent in nature and shows a large peak that occurs in a short time (Figure 5.2). The results presented in Figure 5.2 were achieved by applying the three different discharging patterns. The various discharge patterns applied were found to affect the reactor performance (Figures 5.3, 5.4 and 5.5).

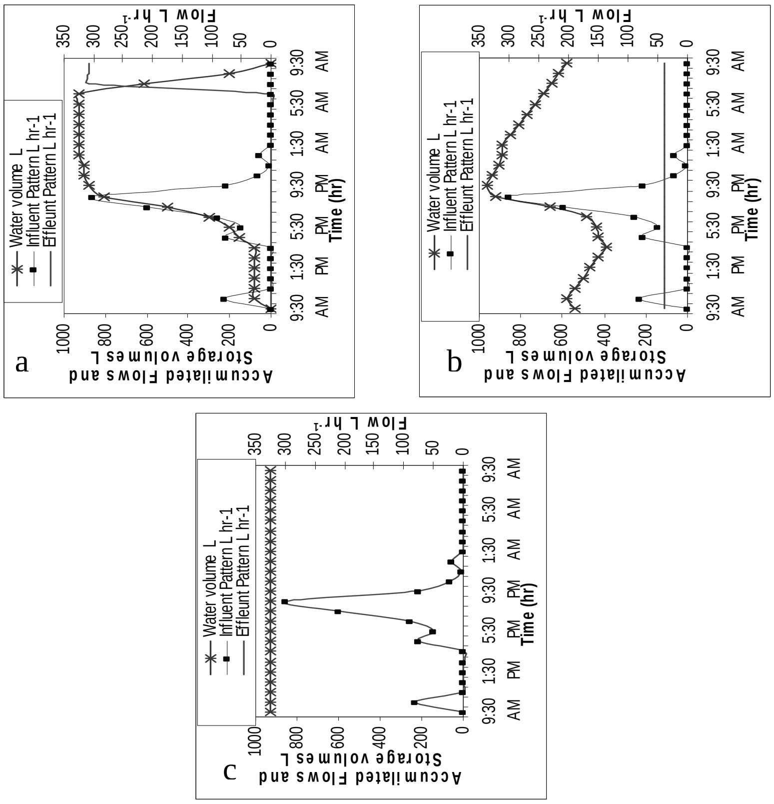

Figure 5.2. average daily inflow rate (adopted and modified from Abu Ghunmi *et al*., 2008), outflow rate and liquid reactor volume as a function of time, (a) a system operated at fed-batch/batch mode at varying inflow rate over the day, (b) a system operated at varying inflow rate over the day, 1 day HRT, and constant the liquid reactor volume, (c) a system operated at varying inflow rate, varying liquid reactor volume, and constant outflow rate.

c

5.3.2 Anaerobic treatment

5.3.2.1 Fed-batch/Batch mode

The performance of the anaerobic reactor, Figure 5.2a, fed during 13 hours with a natural daily intermittent inflow pattern, followed by 8 to 104 hours batch digestion and a discharge period of 3 hours is shown in Figure 5.3. Increasing the batch digestion time from 8 to 104 hours increases the removal efficiency of the different COD fractions. Using the conversion rate constant $k_{25\,℃} = 0.009$ day^{-1} (Chapter 3) would result after 8 to 104 hours of digestion time in a 0-4% COD removal. The in comparison much higher 20-58% of COD removal found in the reactor (Figure 5.3) indicates that the main mechanism of removal is physical, viz. by settling and adsorption.

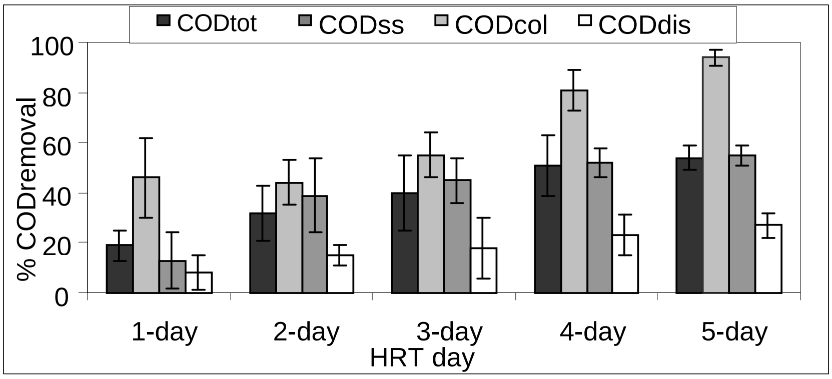

Figure 5.3. The COD fractions removal efficiencies at operational cycles of 1 to 5 days; (mean values over an operational period of 60 days). Fed in batch mode for 13 hours with a natural daily intermittent inflow pattern, followed by 8 to 104 hours batch anaerobic digestion and a discharge period of 3 hours.

5.3.2.2 Constant liquid reactor volume mode

Applying a 24 hours operational cycle consisting of 13 hours loading and discharging simultaneously at the natural daily intermittent inflow pattern (Figure 5.2b), and then 11 hours batch digestion achieves a COD_{tot} removal efficiency of 23%. Especially suspended COD removal (Figure 5.4) is increased in comparison to the fed-batch/batch mode even with a 72 hours operational cycle (Figure 5.3). Increasing the discharging time, results in a lower average discharge flow rate. Thus the discharging time, in addition to the up-flow discharge regime, improves the performance of the anaerobic unit more than only increasing the digestion time. The improved removal efficiency is mainly caused by improved COD_{ss} removal.

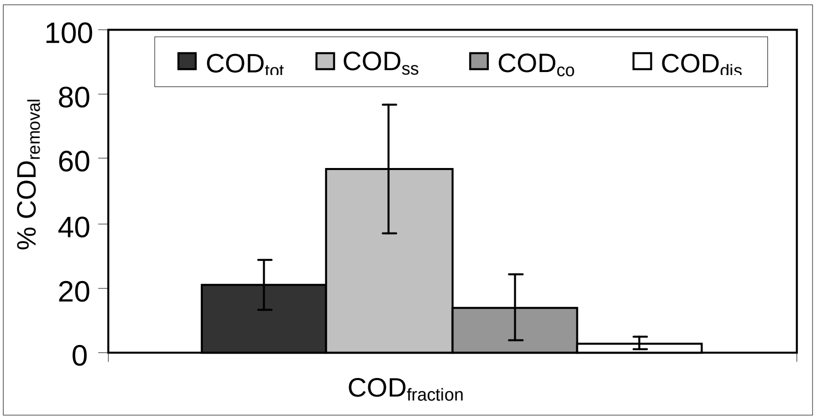

Figure 5.4. The COD fractions removal efficiency at an operational cycle of 1 day (mean values of an operational period of 60 days). A 24 hours operational cycle with a varying inflow rate over the day and constant liquid anaerobic reactor volume.

5.3.2.3 Variable liquid reactor volume

The best COD removal results for the anaerobic tank were found when applying a 24 hrs operational cycle with a constant effluent flow rate and a varying liquid volume in the reactor as is depicted in Figure 5.2c. The best results were obtained when operating the system in up flow mode as compared to down flow (Figure 5.5). The up-flow regime reduces the washout of settled sludge as can be seen from the results of the COD_{ss}.

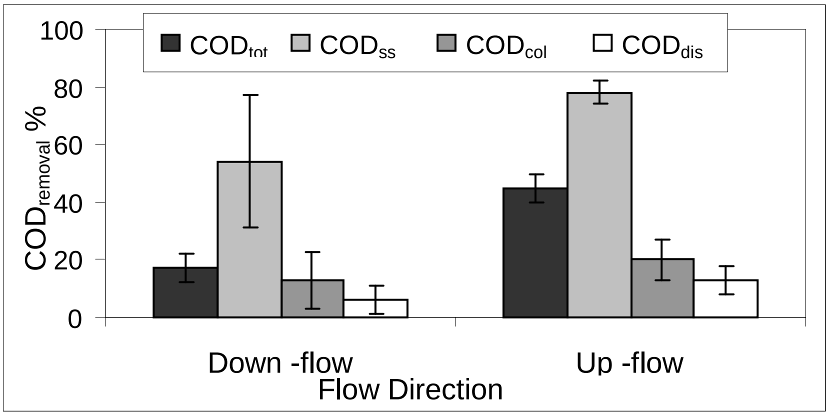

Figure 5.5. The COD fractions removal efficiencies at an operational cycle of 1 day (mean values over an operational period of 360 days); A 24 hrs operational cycle with a constant effluent flow rate and a varying liquid volume in the anaerobic reactor.

5.3.2.3.1 Removal efficiency

The reactor performance, at optimum operational conditions, was monitored in terms of COD, Solids, VS, P and N fractions, in addition to BOD_5, BOD_{21} and pathogens that are measured as Total Coliform, Faecal Coliform and/or *E*. *coli*. The operational cycle entailed feeding at natural daily intermittent inflow pattern over 13 hours simultaneously with discharging over 24 hours (Figure 5.2c).

The removal of COD_{ss} is as high as 71%, while SS is removed for 82% (Table 5.2). However, the removal efficiency for the total wastewater in terms of COD_{tot}, TS, TN and TP was rather low. This can be attributed to the fact that 69% of the COD, 90% of the N, 80% of the TP and 84% of the TS of the total influent are found in the colloidal and dissolved fraction. The effluent of the anaerobic unit is better biodegradable than the influent when expressed as the BOD_5 to BOD_{28} ratio. The concentrations of dissolved nitrogen and phosphorous do not significantly increase after anaerobic treatment (Table 5.2). The anaerobic unit reduces the Total Coli, *E*. *coli* content by 1 to 2 log units

(Table 5.2), probably due to adsorption (Stevik *et al*., 2004) or settling processes. The effluent concentrations for SS, N and P comply with the Jordanian standards for irrigation, indicated in Table 5.2. The COD, BOD_5, and *E.Coli* values of the anaerobic effluent do not comply with both (1) the Jordanian standards for irrigation inside cities and vegetables eaten cooked. And (2) the proposed irrigation water quality guidelines for Jordan (Vallentin, 2006); therefore post-treatment is required.

The removal efficiency of the COD in the anaerobic unit is similar to that of the Upflow Anaerobic Sludge Blanket (UASB) operated by both Elmitwalli *et al*. (2006) and Hernandez *et al*. (2008) (Table 5.3), but lower than that achieved by Imura *et al*. (1995) and Elmitwalli and Otterpohl (2007) who applied respectively an Anaerobic Filter (AnF) and UASB (Table 5.3). However, the AnF shows an unstable performance. The anaerobic unit presented in this study is as efficient as the UASB of Elmitwalli and Otterpohl (2007) in the removal of COD_{ss} (Table 5.3). Grey water influent CODss is 29% of the total COD in this study while 50% in the study of Elmitwalli and Otterpohl (2007), therefore latter removal of COD_{tot} is better.

The investigated anaerobic unit showed with an intermittent inflow pattern a stable performance, viz. providing a stable effluent. The performance and the stability of a UASB with an intermittent inflow pattern have not been studied so far. The UASB of Elmitwalli and Otterpohl (2007), with a high removal efficiency (Table 5.3), was operated under controlled inflow and controlled high temperatures, in contrast to the anaerobic unit of this study, which was tested for grey water at ambient temperatures. Considering the flow pattern, combined storage and treatment and water temperature, the anaerobic unit of this study is efficient and competitive with the other units tested for treating grey water.

Table 5.2.‡‡‡Anaerobic unit influent concentrations and removal efficiencies in summer time, i.e. temperature 20-36℃, at optimal tested operation conditions as depicted in Figure 5.2c.

Parameters		Units	tot	ss	col	dis	NH_4-N	PO_4^{3}-P	Standards (1)	Standards (2)
COD	Influent	mg L^{-1}	366 (165)	93 (66)	133 (58)	139 (64)			120	100
	Removal	%	45 (5)	71 (4)	44 (7)	25 (5)				
BOD_5	Influent	mg L^{-1}	150 (31)						60	30
	Removal	%	37 (9)							
BOD_5/BOD_{28}	Influent	%	55 (7)							
	Effluent	%	76 (8)							
Solids	Influent	g L^{-1}	1.081 (0.062)	0.169 (0.060)	0.094 (0.023)	0.797 (0.083)				0.15*
	Removal	%	24 (7)	81 (9)	42 (24)	7 (6)				
Volatile Solids	Influent	g L^{-1}	0.237 (0.133)	0.077 (0.017)	0.044 (0.019)	0.110 (0.031)				
	Removal	%	33 (5)	57 (1)	15 (0.2)	4 (4)	8 (2)			
Nitrogen	Influent	mg L^{-1}	12 (0.6)	1.2 (0.1)	0.6 (0)	9 (0.5)	7 (3)			45**
	Effluent	mg L^{-1}	10 (5)	1.5 (1)	0.3 (0.2)	8 (4)	8 (2)			
Phosphorous	Influent	mg L^{-1}	11 (8)	1 (1)	3 (1)	7 (1)		6 (2)		
	Effluent	mg L^{-1}	10 (4)	1 (2)	2 (2)	6 (3)		6 (1)		
Total Coli-form	Influent	MPN/ 100ml	7.1 E+7 (1.27E+8)							
	Effluent	MPN/ 100ml	1.9 E+5 (1.95E+5)							
E. Coli	Influent	MPN/ 100ml	1.4 E+6 (1.9 E+6)						1000	100
	Effluent	MPN/ 100ml	1.0 E+5 (7.0 E+4)							

‡ Influent mean values differ from those reported in Chapter 2. The values collected in Chapter 2 are only of the winter period, the students and cleaners were not aware of the research going on. In this chapter the period included

winter and summer, the students and cleaners were aware of the research going on. The latter might be of importance.
‡‡; The difference between the total analyzed samples and the summation of the samples' fractions (suspended, colloidal and dissolved) was due to a higher number of the total and the dissolved compared with suspended and colloidal samples
( ); standard deviation: influent = raw grey water
(1); proposed irrigation water quality guidelines for Jordan (Vallentin, 2006)
(2); Jordanian standards 893/2002 for use of reclaimed wastewater for irrigation vegetables eaten cooked, parks, playgrounds and sides of roads within city limits
*; suspended solid and **; total.

Table 5.3. Anaerobic units treating grey water in literature; operational conditions and performances.

Parameters		Units	AnF	UASB	UASB	UASB
HRT		hours	16.5-19	12	16	12
Temperature		℃	14.3-19.5	23	30	35
COD						
Influent	Total	mg L^{-1}	40-130		618±130	830±211
	suspended	mg L^{-1}			325 ±132	427±181
Effluent	Total	mg L^{-1}	17-74			
	suspended	mg L^{-1}				
Removal	Total	%		41	64±5	47±15
	suspended	%			83.5±5.4	
References			Imura *et al*. (1995)	Elmitwalli *et al*. (2006)	Elmitwalli and Otterpohl (2007)	Hernandez *et al*. (2007;2008)

5.3.2.3.2 Sludge characteristics

The anaerobic unit in this study produced highly concentrated sludge (Table 5.4) compared with concentrations reported in literature, by Elmitwalli and Otterpohl (2007) and Hernandez *et al*. (2008), for UASB systems treating grey water. The sludge depth is however limited, viz. 13 cm, which is 10% of the tank depth (Figure 5.6). Elmitwalli and Otterpohl (2007) reported a sludge bed height of 122 cm, which is 75% of the tank depth; the reported sludge profile concentration varies from 0 to 25 g VS L^{-1} with an average of 12.6 g VS L^{-1}, as shown in Table 5.3. The difference in sludge bed height could be a consequence of the techniques applied to keep the sludge in contact with the grey water or the dimensions of the units i.e. depth to width ratio. For instance, in the present study discs are used to distribute the sludge and keep it in contact with liquid over the depth of the tank whereas in an UASB the influent up-flow velocity suspends the sludge. Moreover, the depth to width ratio in the unit of this study is 1.33 while the UASB used by Elmitwalli and Otterpohl (2007) was 22.

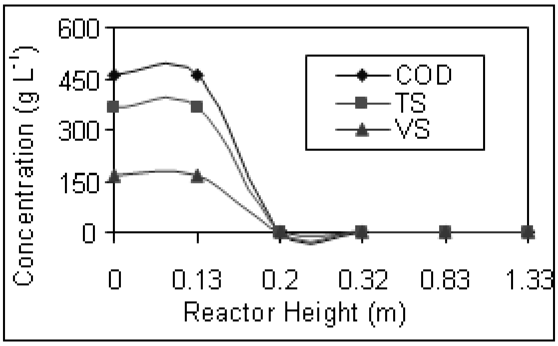

Figure 5.6. Profile of COD, TS and VS concentration of the anaerobic sludge.

Table 5.4. Anaerobic sludge characteristics found in this study and reported in the literature.

	Units	This study	Elmitwall and Otterpohl (2007)	Hernandez *et al.* (2008)
HRT	Hours	≤ 24	16	12
Temperature	℃	17-28	30	35
VS	g L^{-1}	168 (5)	12.6	12.7±3.9
VS/TS	%	46 (4)	55 (5)	
COD	g L^{-1}	459 (2)	28	
COD/VS		2.7 (0.1)	2.1 (0.1)	
Sludge digestibility	g $COD_{digested}$ /g COD %	20 (2)	12 (3)	
SMA_{max}	g COD g VS^{-1} d^{-1}	0.12 (0.02)	0.18 (0.03)	
SRT_{min}	day	30	93	
SRT_{max}	day	Infinite	481	

The ratio COD to VS of the sludge is double than that of grey water (2.74 compared to1.3 mg COD mg VSS^{-1}) and close to the 2.1 reported by Elmitwalli and Otterpohl (2007). The reason for this could be the accumulation of some of the detergents, such as the surfactants LAS ($C_{11.6}$ linear alkyl chain $NaC_{18}H_{29}SO_3$) and SLS ($NaC_{12}H_{25}SO_4$) which have a calculated COD to VS ratio of 3.6 and 3.5 respectively.

The minimum SRT (SRT_{min}) in this study is lower than that reported by Elmitwalli and Otterpohl (2007), therefore the sludge digestibility as compared to Elmitwalli and Otterpohl (2007) is also lower (Table 5.4). Other reasons for the lower stability might be the difference in treatment temperature which was lower in this study (Table 5.4). The anaerobic sludge of the reactor in this study was stable as it had a digestibility of 8.5% g $VS_{digested}$ /g VS. Halalsheh, (2002) reported sludge treating domestic wastewater with a digestibility of 6-11% g $VS_{digested}$ /g VS was stable. The SMA_{max} found in this study (0.12 g COD g VS^{-1} d^{-1}) was lower than that found by Elmitwalli and Otterpohl (2007), but was within the range of 0.10 to 0.15 g COD g VS^{-1} d^{-1} reported by (Halalsheh, 2002) for the sludge of the UASB treating domestic wastewater of Kherbit Al-Samra in Jordan.

5.3.3 Aerobic post treatment

5.3.3.1 Aeration techniques

The performance of the system with forced aeration was better than that with natural aeration: 54% and 30% total COD removal, respectively (Figure 5.7). Both performances, however, are better than the performance estimated based on the conversion rate constant $k_{25\ ℃}$ = 0.174 day^{-1} (Chapter 3) after 1 day operational cycle. This would result, when calculated, in 15% total COD removal. Forced aeration does supply enough oxygen (≥ 2 mg L^{-1}) (Metcalf and Eddy, 2003) while natural aeration does not (≤ 0.5 mg L^{-1}). Forced aeration also enhances the removal of the pollutants by coagulation and flocculation processes. However, also at forced aeration, dissolved and colloidal pollutants are not sufficiently removed. Detailed results on the performance of the forced aeration system are listed in Table 5.5.

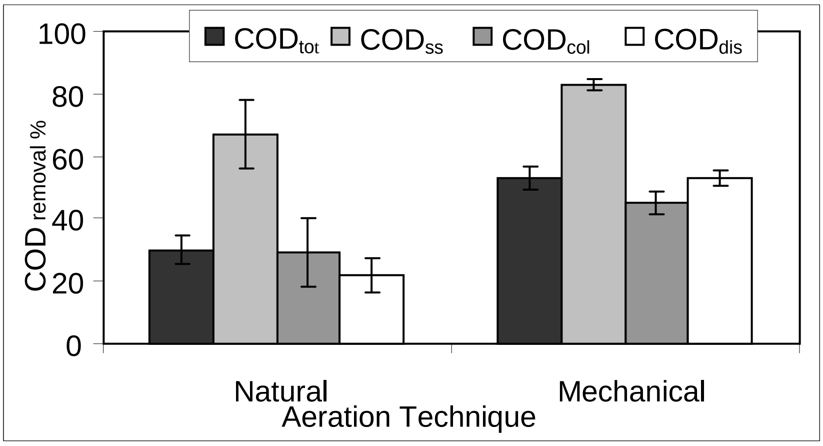

Figure 5.7. The COD fractions removal efficiencies at an operational cycle 24 hours applying natural and mechanical aeration. (mean values over an operational period of 145 days for natural aeration and 220 days for mechanical aeration).

5.3.3.2 Removal efficiency

Aerobic treatment improves the anaerobic effluent quality with regard to the organic matter, i.e. the BOD concentration. (Table 5.5). The aerobic effluent BOD_5 to BOD_{28} ratio is lower than in the raw grey water and anaerobic effluent (Table 5.5). Phosphorus and nitrogen removal is insignificant in the dissolved fractions. Furthermore the aerobic unit shows no reduction in the TC, and the *E. coli* in the effluent. Based on the above findings, the aerobic unit has limited capacity in treating the anaerobic effluent. Nevertheless, the effluent, except for the pathogens, complies with the proposed irrigation water quality guidelines for Jordan in Table 5.2 (Vallentin, 2006). Furthermore, the effluent qualities should be improved with respect to BOD_5 and pathogens content for use as irrigation water inside cities and for vegetables, eaten cooked.

In literature, Sequencing Batch Reactors (SBR) and Submerged Biolfilters (SBF) were tested for post treating grey water that was pre-treated anaerobically (Hernandez *et al*., 2007; 2008; Imura *et al*., 1995). Compared to the SBR and the SBF the removal efficiency of COD by the aerobic unit in this study was low: SBR removal efficiency 80±9 %, SBF 85%, aerobic unit in this study 53±9 %. The effluent quality with regard to COD, of both the aerobic unit of this study and the SBR of Hernandez *et al*. (2007; 2008) are similar. Hernandez *et al*. (2008) tried to improve the quality by treating raw grey water aerobically at an HRT of 12 hours, instead of anaerobic-aerobic treatment at an HRT of respectively 6 and 7 hours. This, however, had no effect, the effluent qualities were similar. Further improvement of the effluent might be possible by application of filtration techniques (Imura *et al*., 1995) or by addition of coagulants (Cui and Ren, 2005).

Table 5.5. ‡‡Aerobic unit influent concentrations and removal efficiencies in summer time, i.e. temperature 20-36°C, at optimal tested operation conditions, i.e. mechanical aeration.

Parameters		Units	tot	ss	col	dis	NH_3-N	PO_4^{-3}-P
COD	Influent	mg L^{-1}	201 (62)	27 (11)	75 (28)	104 (38)		
	Removal	%	53 (9)	83 (4)	54 (7)	59 (5)		
BOD_5	Influent	mg L^{-1}	101 (18)					
	Removal	%	40 (6)					
BOD_5/BOD_{28}	Influent	%	76 (8)					
	Effluent	%	49 (4)					
TS	Influent	g L^{-1}	0.822 (0.089)	0.032 (0.015)	0.055 (0.027)	0.739 (0.083)		
	Removal	%	5 (2)	40 (4)	37 (4)	1 (6)		
Volatile Solids	Influent	g L^{-1}	0.159 (0.06)	0.033 (0.041)	0.038 (0.025)	0.106 (0.0421)		
	Removal	%	18 (4)	0.0 (3)	4 (0.2)	1 (4)	8 (2)	
Nitrogen	Influent	mg L^{-1}	10 (5)	1.5 (1)	0.3 (0.2)	9 (4)	8 (2)	
	Effluent	mg L^{-1}	8 (2)	0.2 (0.1)	0.4 (0.2)	7 (2)	6 (1)	
Phosphorous	Influent	mg L^{-1}	10 (4)	1 (2)	2 (2)	9 (3)		6 (1)
	Effluent	mg L^{-1}	7 (4)	1 (0)	2 (2)	6 (1)		5 (2)
Total Coli-form	Influent	MPN/100ml	1.9E+5 (1.95E+5)					
	Effluent	MPN/100ml	1.6E+5 (1.4E+5)					
Fecal Coli-form	Influent	MPN/100ml	2.0E+4 (7.5E+3)					
	Effluent	MPN/100ml	6.4E+4 (4.8E+4)					
E. Coli	Influent	MPN/100ml	1.0E+5 (7.0E+4)					
	Effluent	MPN/100ml	5.9E+4 (7.7E+4)					

‡‡;The difference between the total analyzed samples and the summation of the samples' fractions (suspended, colloidal and dissolved) was due to a higher number of the total and the dissolved compared with suspended and colloidal
( ); standard deviation; influent is effluent of anaerobic unit.

5.3.3.3 Sludge characteristics

Consistent with the anaerobic sludge results, the aerobic unit produced a concentrated sludge of 11 g VS L^{-1}, much higher compared to the 3.4 and 5.6 g VS L^{-1} found by Hernandez *et al*. (2008). The sludge accumulated in 10% of the tank depth (Figure 5.8). The aerobic sludge is stable as it has an aerobic digestibility of 7% in 160 days. The VS/TS ratio of the aerobic sludge is 73%, which is much higher than the 46% of the anaerobic sludge, and the 20% in the aerobic influent which indicates yield of cell material is the dominant factor (Table 5.6). The ratio of COD to VS for the aerobic sludge is half that of the anaerobic sludge and similar to the 1.29 found for grey water (Table 5.6). McAvoy *et al*. (1993) reported that aerobically stabilized sludge had a LAS content of <500 mg L^{-1}, which is lower than that of an anaerobically stabilized sludge: viz. 5000 to 10000 mg L^{-1}.

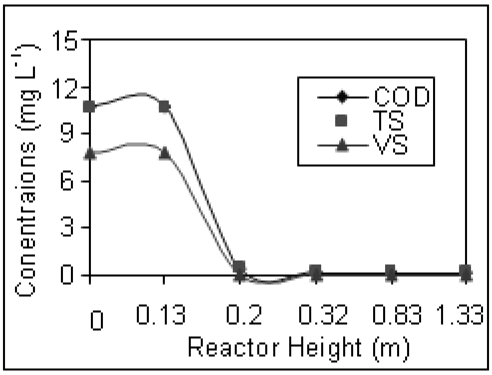

Figure 5.8. Profile of COD, TS and VS concentration of the aerobic sludge

Table 5.6. Aerobic sludge characteristics

	Units	Aerobic Sludge
HRT	hr	≤ 24
TS	g L^{-1}	11 (1)
VS	g L^{-1}	8 (1)
SS/TS	%	95 (3)
VS/TS	%	73 (2)
COD	g L^{-1}	11 (0.8)
COD/VS		1.38 (0.2)
Sludge digestibility	COD %	7 (1)
SRT_{min}	day	30
SRT_{max}	day	Infinite

5.3.4 Performance of the total system at summer and winter conditions

This study tested the system during summer and winter time. The results show that insulating the anaerobic and the aerobic units keeps the performance of the combined system stable during the whole year (Table 5.7) despite large variation in the ambient temperatures of summer (20-36ºC) and winter (7-15ºC). The insulation keeps the internal temperatures of the anaerobic and the aerobic units in winter (17-25ºC) close to the temperatures in summer (22-28ºC). Although the difference in the temperature reduces the performance of the anaerobic unit by 9% (Table 5.7), the decrease is compensated by the increase in the aerobic unit performance and hence maintains the stability of the system performance throughout the year. Furthermore, the zero inflow period, data not reported, had no influence on the performance of the two units.

Table 5.7. Performance of the anaerobic and aerobic units, the removal of the different COD fractions, in winter and summer

		Units	COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}
Summer	Influent	mg L^{-1}	306 (165)	83 (66)	103 (58)	119 (64)
Anaerobic + Aerobic	Effluent	%	70 (4)	85 (5)	54 (8)	58 (3)
Winter	Influent	mg L^{-1}	313 (193)	102 (132)	93 (66)	118 (75)
Anaerobic	Effluent	%	39 (7)	65 (6)	16 (9)	17 (6)
Aerobic	Effluent	%	64(6)	84 (5)	57 (7)	49 (5)
Anaerobic + Aerobic	Effluent	%	67 (7)	87 (6)	47 (5)	58 (6)
Summer+ Winter	Influent	mg L^{-1}	316 (188)	103 (88)	94 (67)	119 (72)
Anaerobic + Aerobic	Effluent	%	69 (5)	85 (7)	50 (8)	55 (5)

5.4 Conclusion

An anaerobic treatment unit operated in up-flow mode, at a feeding pattern following the dormitory grey water discharge pattern and at a continuous constant discharge pattern with a varying liquid reactor volume, in a 1 day operational cycle, can combine storage and treatment of grey water, while accommodating the variation in flow pattern.

The COD removal from anaerobic treatment of grey water, combining storage and treatment, reaches up to 42 and 89%, for respectively the total and suspended COD fractions.

The COD removal from aerobic post-treatment of anaerobically treated grey water at an HRT of 1 day amounts to 54, 53 and 59%, for respectively the total, colloidal and dissolved COD fractions.

The sludge concentration in the anaerobic reactor is high, viz.168 mg VS L^{-1}; sludge bed height is low.

The anaerobic sludge attains a stability of 80%, while the maximum specific methanogenic activity is 0.12 g COD g VS^{-1} d^{-1}.

The sludge concentration in the aerobic reactor is high, viz. 11 g VS L^{-1}; the sludge bed height is low.

The aerobic sludge attains a stability of 93%; the sludge bed height is low.

The aerobic effluent quality complies, except for pathogens, with the proposed irrigation guidelines for reclaimed wastewater in Jordan.

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Chapter Six

Summary and Discussion

6.1 Introduction

Water shortage, water pollution, and appropriate management of (domestic) wastewater are water concerning issues of prime importance in (semi)arid countries like Jordan. The Decentralized Sanitation and Reuse (DeSaR) concept addresses these issues by supplying the practical tools to provide sanitation, thus preventing domestic wastewater being a source for water pollution, to recycle nutrients and furthermore to mitigate the water shortage by recycling the treated water for multiple usage. The tools of the DeSaR concept are decentralization of sanitation and source separation of domestic wastewater streams according to their nature and degree of pollution (Lettinga, 1996; MacKenzie, 1996; Zeeman and Lettinga, 1999; van Lier and Lettinga, 1999; Lettinga *et al*., 2001).

Grey water characteristics, treatment technologies and potentials for reuse form the scope of this thesis, mainly focusing on the treatment aspects. The applied approach is: (a) Reviewing the literature for an efficient, simple, affordable system to treat grey wastewater (Chapter 1). (b) Characterizing grey water in Jordan from different sources, and assessing the potential and reuse requirements for on-site application (Chapter 2). (c) Investigating the biodegradability characteristics of grey water to indentify design and operation guidelines (Chapter 3). (d) Using the results of Chapters 1, 2 and 3 to design and operate a pilot plant treating on-site dormitory grey water and using the treated effluent for irrigation (Chapter 4). Chapter (5) finally summarizes and compiles the four previous chapters, discussing the potentials for grey water recycling and applying the DeSaR concept in Jordan.

6.2 Summary

6.2.1 Review of treatment system for grey water

The variation in the quantity and the quality characteristics of grey water in time (Butler *et al*., 1995) and between the different sources (Nolde, 1999; Eriksson *et al*.,2002; Jefferson *et al*., 2004), as well as the variation in the reuse options (Jefferson *et al*., 2001; Eriksson *et* *al*.,2002) resulted in a plethora of treatment technologies (Jefferson *et al*., 1999). In addition, reviewing the examined technologies in Chapter 1, shows that the technology performances are not optimized. Moreover their design criteria and/or the operational conditions are not or only partially reported. However, the review of the technology performance together with the characteristics of the treated grey water and the reuse standards, reveal that grey water could be effectively pre-treated in an anaerobic unit followed by an aerobic process with a disinfection unit as an optional step. The anaerobic step removes 40-63% of the grey water COD (Elmitwalli and Otterpohl, 2007; Hernandez *et al*., 2007; 2008), and thus reduces the energy consumption in the subsequent step (Lettinga *et al*., 1980). The **a**erobic step post-treats the anaerobic effluent for better quality in terms of the BOD (Imura *et al*., 1995), and converts NH_4^+ to NO_3^- (Shin *et al*., 1998). The disinfection step depends on the quality required for reuse and the effluent quality of the aerobic step. Chapter 1 comments on revising and classifying the effluent reuse standards, considering the reuse options and requirements.

6.2.2 Characteristics of grey water for reuse requirement and treatment alternatives

Grey water characteristics vary with the source and also within the same source (Eriksson *et al*., 2002). Furthermore, Eriksson *et al*. (2002) request information about grey water characteristics in order to evaluate the potential for reuse. The latter is a first step in any plan for applying (treated) grey water for reuse. In Chapter 2 (Abu Ghunmi *et al*., 2008) the characteristics of grey water in Jordan are studied extensively. Because Jordan suffers from water scarcity (WAJ, 1999), the potentials and reuse requirements are evaluated.

The assessment of the grey water in Chapter 2 is based on the characteristics of grey water in Jordan and the reuse options. The characteristics are determined in terms of (1) the quantity i.e. the volume and the hydrographs and (2) the quality i.e. characteristics and pollutographs. Hydrographs, illustrating the sub-daily flow pattern as a function of time, are designed for rural and urban residential places in addition to dormitory places. Quality of different grey water streams are characterized with respect to the physical properties, the chemical and biological components, based on sampling and analysis from different locations. In addition pollutographs, illustrating the sub-daily concentration and organic load patterns as a function of time, are designed for rural and urban residential places in addition to dormitory places.

The assessment revealed that the grey water volumes and the concentrations of TS, BOD_5, COD and pathogens vary between the different sources. This is in agreement with Nolde (1999) for the variation in volume and concentrations and Eriksson *et al*. (2002) for the quality characteristics. In all the studied locations in Jordan the total amount of grey water (shower, laundry, washbasin and kitchen) produced on-site exceeds the total water volume required for on-site reuse options, namely toilet flushing and laundry in addition to garden irrigation in Amman city. Houses in Amman with a garden, use 5% of its drinking water consumption for irrigation (Jamrah *et al*., 2006). Surendran and Wheatley, (1998) reported that shower and laundry wastewater provide sufficient water for toilet flushing. However, a daily storage of treated grey water is inevitable to meet the daily reuse requirements in terms of volume and timing. The latter is consistent with findings of Jefferson *et al* (1999), based on the data of Surendran and Wheatley (1998), who considered reuse of grey water for toilet flushing in the UK. Grey water qualitative parameters, like solids, BOD_5, COD and pathogens indicate that grey water needs treatment before storage and reuse can be applied. On the other hand, the inorganic content of grey water and the sodium adsorption ratio (SAR) values, except for laundry wastewaters, comply with the reuse guidelines for irrigation in Jordan.

In agreement with the results of the review in Chapter 1, the results of the characterization of grey water presented in Chapter 2 lead to the recommendation of avoiding physical treatment as a pretreatment step and avoidance of storage of non-treated grey water. The application of a biological treatment system (Nolde, 1999; Jefferson *et al*., 2004), consisting of an anaerobic and aerobic step, in combination with pre-storage is the main recommendation for the treatment concept. The recommended biological system copes with the fluctuations in the hydrographs and pollutographs. The developed hydrographs and pollutographs indicate that the hydraulic load is the determining design parameter for a biological treatment unit for dormitories and hotels. For family houses, the organic load is most decisive for the design of a biological treatment unit.

6.2.3 Grey water biodegradability

The basic problem of raw grey water is the content of organic compounds that may create odour problems (Diaper *et al*., 2001). Open grey water storage may also lead to mosquito breeding and the suspended solids may result in clogging of irrigation distribution systems (Christova-Boal *et al.,* 1995). The basic design parameters for an appropriate grey water treatment system able to cope with the above constraints are the volumetric quantity, the maximum amount of biodegradable organic matter, and the rate of biodegradation at different temperatures. Scarce literature on this subject is from Elmitwalli and Otterpohl (2007) and Zeeman *et al.* (2008), and concerns the biodegradability of grey water constituents.

Chapter 3 aims at investigating the aerobic and anaerobic biodegradability of grey water. The study is applied on grey water from a Jordanian University Campus hostel. The research focused on studying the factors affecting the biodegradation, i.e. the maximum biodegradability and the conversion rates of the grey water constituents. The latter parameters were determined for one or more of the COD fractions of the wastewater, i.e. the total (COD_{tot}), suspended (COD_{ss}), colloidal (COD_{col}) and/or dissolved (COD_{diss}) COD, at one or more of the following temperatures 20, 25, 30 and 35 ℃. The determined parameters are used for calculating the SRT to be applied in aerobic and anaerobic biological treatment processes.

The determined maximum biodegradability, conversion rates, and Arrhenius temperature coefficients for anaerobic and aerobic biodegradation of grey water, are summarized in Table 6.1. Results show that the determined biodegradability differs between the different COD fractions, viz. COD_{ss}, COD_{col} and COD_{diss}. Aerobic and anaerobic biodegradability is lowest for the COD_{ss} fraction, whereas the lowest conversion rate is shown for the COD_{diss} fraction. Aerobic and anaerobic treatments require a long SRT.

Table 6.1. Summarized results of aerobic and anaerobic batch tests on raw grey water.

	Aerobic	Anaerobic
Maximum biodegradability (%)	86	70
K_{20} Conversion rate (day^{-1})	0.119	0.005
Temperature coefficients	1.069	1.099

6.2.4 Grey water treatment for irrigation

Based on the results reported in Chapters 1-3 a treatment system is designed, facilitating storage and treatment in a single unit, with the lowest possible volume. The produced effluent should meet the irrigation reuse guidelines as applied in Jordan.

The treatment system includes anaerobic and aerobic conversion processes. The treatment units were constructed using local materials. Three different concepts, integrating pre-storage and treatment of the grey water are tested within this study, viz. (a) a system operated in fed-batch/batch mode accepting fluctuating inflow rates over the day, (b) a system operated at a constant liquid reactor volume, accepting fluctuating inflow rates over the day and (c) a system operated at non-fixed liquid reactor volumes, accepting fluctuating inflow rates and a constant outflow rate. The effluent quality of both, the anaerobic and aerobic unit, was tested and its compatibility with irrigation standards was evaluated.

The results in Chapter 4 reveal that working with an influent flow following the daily intermittent flow pattern, the optimal conditions for the anaerobic treatment unit were an operational cycle of 24 hrs in an upflow regime, while discharging effluent for 24 hours at a constant flow rate. The tested aerobic post-treatment reactor set-up, which was initially operated in down flow mode, was not appropriate for providing sufficient oxygen in the reactor bulk solution. The aerobic post-treatment was, therefore, amended, providing mechanical aeration and an upflow mode of operation. To economise on mechanical aeration, other reactor concepts, using natural aeration techniques are to be considered for future application. Tandukar *et al* (2007) successfully applied a combination of a UASB and DHS (Downflow Hanging Sponge) reactor for the treatment of domestic sewage. They concluded that mechanical aeration is not required for achieving similar removal efficiency in comparison to an activated sludge system. It was concluded that a combined UASB-DHS system can be a cost-effective and viable option for the treatment of municipal sewage, especially for low-income countries (Tandukar *et al*., 2007). Though it was shown within this thesis that the treatment requirements for grey water differ from domestic sewage, the application of a DHS system for post treatment of grey water will likely improve the effluent quality. Tandukar *et al*. (2007) reported that the combined UASB-DHS system out-competes an activated sludge system with respect to the removal of pathogens. The removal of Total Coliform by a UASB-DHS is about 4 log (Tandukar *et al*., 2007), whereas 2.4±0.4 log removal is found in activated sludge systems (Wen *et al*., 2009).

The anaerobic unit, treating dormitory grey water, achieves 89% removal of the COD_{ss} fraction. The removal of COD_{tot} is only limited, viz. 42 %. The removal efficiency in the aerobic unit, treating the anaerobic effluent, is also limited, viz. 53% of the COD_{tot}, 59% of the COD_{col} and 54% of the COD_{dis}. However, the system copes with the intermittent influent flow even at a high peak value, and with changing temperature. Insulation of the reactors keeps the reactor temperature between 17-25 ℃. The aerobic effluent quality, except for pathog ens, complies with the proposed Jordanian guidelines for lawn irrigation.

Anaerobic and aerobic sludge digestibility in 160 days is 20% and 7%, respectively. Apparently, there is still room for improvement since stable anaerobic sludge with a digestibility of 7% to 12% was produced in the reactors of Elmitwalli and Otterpohl (2007). The system did not show any operational problems such as sludge wash out.

6.3 Discussion

6.3.1 General discussion

In literature, the recognized issues for grey water recycling are the variation in 1) the characteristics (Eriksson *et al*., 2002), 2) the treatment technology (Nolde, 1999; Winward *et al*., 2008), 3) the reuse options (Christova-Boal *et al.,* 1995; Otterpohl *et al*., 1999; Eriksson *et al*., 2002) and/or 4) standards issues (Jefferson *et al*., 1999; Gross *et al*., 2007).

This thesis illustrates that grey waters vary in concentration, depending on the local conditions. As such, grey water can be characterised as a low, medium or even strong domestic wastewater stream. This, in contrast to earlier reports stating that grey water is a diluted wastewater stream (Christova-Boal *et al.,* 1995). Grey water can be a high strength wastewater, in terms of COD, BOD, TS and/ or pathogens, like in Jordanian rural areas (Chapter 2), or a medium to strong stream like in Sneek in the Netherlands (Hernandez *et al*., 2008) or in the Flintenbreite settlement in Luebeck in Germany (Elmitwalli and Otterpohl, 2007). But it can also be characterised as medium strength to weak as in the dormitories at Jordan University in Jordan (Chapter 2). The reason for the observed high strength, in e.g. Jordan, is the lower water consumption, as occurring in water scarcity areas (Otterpohl *et al*., 1999; Chapter 2) or due to inclusion of kitchen wastewater in the grey water stream (Chapter 2). The dormitory grey water as used for the main part of this research does not include kitchen wastewater and can be defined as low concentrated wastewater.

In order to properly design a treatment system for grey water, the anaerobic and aerobic biodegradability and conversion rates are determined. The biodegradable organic matter has low conversion rates compared with e.g. domestic wastewater (Chapter 3). In addition ≥ 50% of the grey water COD content is of dissolved and colloidal nature (Chapter 3; Elmitwalli and Otterpohl, 2007; Hernandez *et al*., 2007; 2008). The challenge of this thesis was to develop a treatment technology that copes with these conditions to produce a stable effluent that meet the Jordan irrigation standards. Therefore, this thesis developed guidelines for the treatment concept of grey water, including its technical specifications. By integrating pre-storage and biological treatment the treatment concept becomes relatively simple, viz. an anaerobic vessel is followed by an aerobic vessel, and no pumping has to be applied. Insulation in the winter time is required.

Unlike the high anaerobic and aerobic biodegradability figures observed in batch experiments, i.e. 70% and 86% respectively, only 42% of the COD is removed during the anaerobic pre-treatment step and 70% of the COD in the combined anaerobic-aerobic continuous system. Therefore, the treatment efficiency of the continuous treatment system can, in principle, be improved. Enhancing both the anaerobic and aerobic effluents might be possible by inclusion of filtration as applied by Imura *et al.* (1995) or addition of coagulants as tested by Cui and Ren (2005). This thesis research does not include methane collection facilities and the possible application of the produced methane. The estimated amount of methane that potentially can be produced from a dormitory grey water, occupied by 150 people, is about 2.2 L CH_4 cap^{-1} day^{-1}. The collected biogas can be used for e.g. bio-gas lighting (ACCU).

This thesis recommends developing multi-category standards considering the aesthetic, health and environmental aspects, since the various reuse options require different qualities of water. Developing multi-category reuse standards could mean that the reuse quality e.g. for toilet flushing can be the same as for irrigation, except for the pathogen concentrations, but should be much more strict for reuse as bathing water or car washing. Furthermore, similar to WHO guidelines (2006) for safe reuse of grey water for irrigation, the developed reuse standards are recommended to include guidelines for safe practice for each of the reuse options e.g. for toilet flushing.

6.3.2 DeSaR in Jordan

The prevailing water shortages in Jordan makes the separation, treatment and reuse of grey water an attractive option, and is on a limited scale already applied in practice. The results of this study show possibilities to improve grey water accessibility for reuse by introducing cost-effective treatment techniques. On the other hand, it must be realised that a wider application of grey water separation, treatment and reuse, might affect the transport and treatment of the remaining domestic wastewater stream, especially in situations where central sewer systems and treatment systems are applied, i.e. in 47% of the sanitation facilities (WAJ, 2000). Separation of grey water, results in highly concentrated remaining wastewater streams, i.e. the so-called “black waters”. Obviously, transport of black water by the generally applied gravity sewers could result in blockage of transport lines. Since amendments to the already constructed sewerage infrastructure is not very likely to occur, grey water diversion projects are particularly of interest in new construction areas or renovation sites. In those areas, on-site collection and treatment of black water for the recovery of energy and nutrients becomes an interesting option as well. Cost-effective anaerobic treatment technologies may play a crucial role in such approach.

Black water includes the main part of the nitrogen, phosphorus and potassium (Gulyas *et al*., 2004; Kujawa-Roeleveld and Zeeman, 2006). The recovery of these nutrients for agricultural use is a topic of increasing interest (e.g. Otterpohl *et al*., 1999). Anaerobic treatment of black water is already applied at demonstration scale in the Netherlands (Zeeman *et al*., 2007). So far, in Jordan, the anaerobic treatment at ambient conditions is successfully demonstrated for the pre-treatment of municipal sewage (Halahsheh, 2002). The anaerobic treatment of black water is certainly an option in Jordan as well. The type of toilets and, therewith, dilution of the black water and moreover the degree of grey water separation determines the final concentration of the remaining wastewater stream. The concentration will also determine the further possibilities of recovery and reuse of the nutrients. In new building situations, collection (toilets), transport and treatment should be optimized for achieving a concentrated black water stream suitable for recovery of energy and nutrients. In Germany (Otterpohl *et al*., 1997; Wendland and Oldenburg, 2003) and the Netherlands (Zeeman *et al*., 2007) vacuum collection and transport is introduced for achieving a concentrated black water stream. The applicability of such systems in Jordan should be evaluated. Large hotels might be a good location for first demonstrations.

The situation in the rural areas is rather different than in the urban situation. As 95% of the sanitation facilities are cesspools. When introducing grey water separation in these situations the hydraulic load of the cesspool will decrease. Detrimental effects of grey water separation are therefore not to be expected. Further optimisation of the currently used systems is certainly possible. The application of cesspools results in the loss of the major part of the nutrients included in black water, and moreover can result in contamination of ground water. Depending on the water use for toilet flushing the cesspool could be used as an Accumulation (Ac) system (Kujawa-Roeleveld and Zeeman, 2006) or reconstructed as an UASB-septic tank for stabilisation of the black water organic matter (Zeeman *et al*., 2007). The liquid effluent from the UASB-septic tank and sludge produced in Ac-system and UASB septic tank could be reused for fertilisation, though precautions should be taken considering the pathogenic content. One step further is the possible introduction of urine separation toilets. Urine contains the major fraction of nutrients and is produced in a volume of 0.6-2.5 litres per person per day (STOWA, 2001; Vinnerås, 2002). After 6 months storage, urine is hygienically safe for reuse in agriculture (Schönning, 2001). When urine separation is applied, an existing cesspool could be kept in use for the remaining brown water treatment. As brown water contains only a small part of the nutrients (Kujawa-Roeleveld and Zeeman, 2006); loss of nutrients is minimized.

The proposed concept for rural situation is illustrated in Figure 6.1. This concept provides optimisation of recovery and reuse of (grey) water and nutrients while using existing infrastructure and minimising emissions to the ground water. Social acceptance of such new concepts is of major importance, where social and technical aspects should be developed hand in hand (Hegger, 2007).

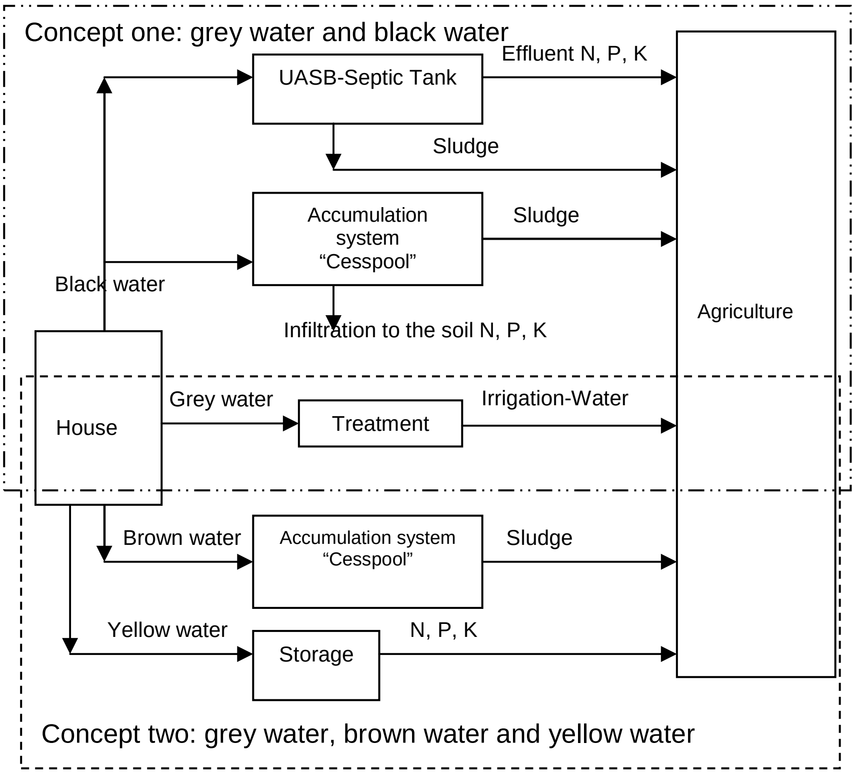

Figure 6.I sanitation: the concepts consist of source separation, treatment and reuse. Black water is the mixture of brown and yellow waters.

6.4 Concluding remarks

Grey water can be characterised as low, medium or strong domestic wastewater.

The investigated dormitory grey water consists of shower, laundry and washbasin streams. The dormitory grey water constituents, measured as COD, are ≥ 50% dissolved and colloidal, 70 % anaerobically and 86% aerobically slow biodegradable matter.

The grey water treatment concept is plain, integrating pre-storage and biological treatment, viz. an anaerobic reactor is followed by aerobic one, and no pumping is needed. Furthermore, to keep the performance stable the anaerobic and aerobic units need insulation to overcome the drop in treatment performance resulting from temperature drops in winter periods.

Grey water reuse needs multi-category reuse standards with guidelines for proper and safe reuse application. The core of the standards is based on parameters related to aesthetic, health and environmental problems.

Grey water treatment in the rural areas needs further research. And in conjunction with this, black water recycling, i.e. collection, treatment and reuse in Jordan, needs to be investigated. The applicability of a DeSaR concept is determined by the successful recycling of grey and black water. The main challenge for recycling grey and black water is the development of affordable technologies that produce products of interest for reuse, without having negative effects for aesthetic, health and environmental issues.

6.5 Recommendations and future research

Recommendations:

To improve the biodegradability of grey water pollutants, i.e. to promote that materials that end up in the grey water are better degradable, such as biodegradable detergents.

To develop reliable multi-category standards for grey water reuse, combined with guidelines for safe practice.

Future research:

Investigating the physical properties of grey water pollutants, such as the particles’ size distribution and surface charges, in order to improve the performance of both biological and physico-chemical processes needed for the cost-effective treatment of grey water.

studying the potential of integrating grey and black waters recycling processes, in Jordan as well as the proper management of it.

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Chapter Seven

Samenvatting en Discussie

7.1 Inleiding

Waterschaarste, watervervuiling en management van (huishoudelijk) afvalwater zijn zeer belangrijke water gerelateerde onderwerpen in semi-aride landen zoals Jordanië. Het decentrale sanitatie en hergebruik concept (Decentrale Sanitatie en Hergebruik: DeSaH) richt zich op deze watergerelateerde onderwerpen en biedt praktische aanknopingspunten voor de toepassing van een sanitatie- en behandelingconcept dat voorkomt dat huishoudelijk afvalwater een bron van verontreiniging wordt, dat resulteert in recycling van nutriënten en het watertekort vermindert door gebruik van het gezuiverde water voor diverse doeleinden. Het DeSaH concept wordt gekenmerkt door decentralisatie en scheiding van huishoudelijke afvalwaterstromen aan de bron, afhankelijk van de mate en type van vervuiling (Lettinga, 1996; MacKenzie, 1996; Zeeman and Lettinga, 1999; van Lier and Lettinga, 1999; Lettinga *et al*., 2001).

In dit proefschrift zijn karakteristieken, de toe te passen zuiveringstechnologieën en hergebruikmogelijkheden van grijswater nader bestudeerd, waarbij de nadruk ligt op het zuiveringsaspect. De gevolgde aanpak omvat:

(a) Literatuurstudie naar een efficiënt, simpel en betaalbaar grijswater zuiveringsysteem (Hoofdstuk 1).

(b) Karakterisering van Jordaans grijswater afkomstig van diverse locaties en onderzoek naar de mogelijkheden van on-site behandeling. Daarnaast zijn de hergebruikvoorwaarden voor toepassing bediscussieerd (Hoofdstuk 2).

(c) Onderzoek naar de afbreekbaarheid van de organische componenten in grijswater bij diverse redoxcondities, ten behoeve van ontwerp en type bedrijfsvoering van een zuivering (Hoofdstuk 3).

(d) Het ontwerpen en bedrijven van een pilot zuivering voor het behandelen van grijswater van een studentenhuis, tot irrigatie kwaliteit (Hoofdstuk 4). Bij het ontwerp van de pilot is gebruik gemaakt van de resultaten van Hoofdstuk 1, 2 en 3.

(e) Algemene discussie over het potentieel voor grijswater hergebruik en de toepassing van het DeSaH concept in Jordanië (Hoofdstuk 5).

7.2 Samenvatting

7.2.1 Literatuur onderzoek duurzame behandeling van grijswater

De term “grijswater” is een kwalitatieve benaming die aangeeft op welke wijze het afvalwater is geproduceerd en ingezameld. De exacte samenstelling is veel minder bekend en kent grote verschillen tussen de diverse locaties. In de literatuur zijn vele zuiveringstechnologieën voor grijswater beschreven (o.a. Jefferson *et al*., 1999), waarbij technologiekeuze en mate van zuivering afhankelijk is van i) de variatie in samenstelling en hoeveelheid in de tijd (Butler *et al*., 1995); ii) van de verschillende bronnen van herkomst (Nolde, 1999; Eriksson *et al*.,2002; Jefferson *et al*., 2004); en iii) de verschillende hergebruikopties (Jefferson *et al*., 2001; Eriksson *et al*.,2002). In Hoofdstuk 1 is een vergelijking gemaakt van de diverse zuiveringstechnologieën, waaruit blijkt dat de bestaande technieken nog niet zijn geoptimaliseerd. Bovendien zijn de ontwerpcriteria en/of the operationele condities niet of nauwelijks beschreven. De literatuurstudie naar de behaalde zuiveringsrendementen in combinatie met de grijswaterkarakteristieken en de hergebruikvoorwaarden laat zien dat grijswater effectief kan worden behandeld in een anaerobe voorzuiveringstap gevolgd door een aeroob proces en eventueel een desinfectie stap. De anaerobe stap verwijdert 40-63% van de grijswater organische stof, uitgedrukt als chemisch zuurstof verbruik (CZV) (Elmitwalli and Otterpohl, 2007; Hernandez *et al*., 2007; 2008), waarmee de energiebehoefte van de vervolgstap significant wordt verminderd (Lettinga *et al*., 1980). De aerobe stap behandelt het anaerobe effluent voor een lagere effluentconcentratie aan biochemisch zuurstof verbruik (BZV) (Imura *et al*., 1995) en zet NH_4^+ om in NO_3^- (Shin *et al*., 1998). De desinfectie stap hangt af van de benodigde kwaliteit voor hergebruik en van de aerobe effluentkwaliteit. Hoofdstuk 1 bediscussieert herziening en herclassificatie van de effluent hergebruikvoorwaarden met inachtneming van de hergebruik opties en benodigdheden.

7.2.2 Karakteristieken van grijswater voor behandelingsalternatieven en hergebruik

Grijswater karakteristieken zijn sterk afhankelijk van de plaats van productie (b.v. wasmachine of vaatwasser), waarbij de kwaliteit en kwantiteit van één type grijswater sterk verschilt in tijd en tussen de diverse locaties (Eriksson *et al*., 2002). Laatstgenoemde auteurs willen tevens meer informatie over grijswater karakteristieken om het potentieel voor hergebruik te kunnen evalueren; een noodzakelijke eerste stap in elke grijswater ‘hergebruiksplan’. Jordanië heeft momenteel te maken met ernstige waterschaarste (WAJ, 1999). In Hoofdstuk 2 (Abu Ghunmi *et al*., 2008) zijn de karakteristieken van grijswater in Jordanië uitgebreid bestudeerd om zo het potentieel en hergebruikvoorwaarden van grijswater in Jordanië te kunnen vaststellen. De karakteristieken zijn bepaald in termen van:

(1) kwantiteit, zijnde volumina’s en hydrografieken, d.w.z. kwantitatieve variaties in de tijd;
(2) de kwaliteit, zijnde concentraties van verontreinigingen en verontreiniginggrafieken, d.w.z. de kwalitatieve variaties in de tijd.

De hydrografieken zijn geschikt voor landelijke en stedelijke locaties, inclusief een campus. De kwaliteit van de diverse grijswaterstromen wordt gekenmerkt door de fysische eigenschappen en de chemische en biologische componenten, gebaseerd op analyses van monsters van verschillende locaties. De verontreiniginggrafieken geven de dagelijkse variatie in concentraties en organische beladingpatronen tegen de tijd weer en zijn ontworpen voor landelijke en stedelijke locaties, inclusief een campus.

De grijswater karakteristieken illustreren de enorme verschillen tussen de verschillende bronnen wat betreft volumina’s, de drogestof, BZV_5, CZV en pathogenen concentraties. Onze resultaten komen overeen met Nolde (1999) voor de variatie in volume en concentraties en met Eriksson *et al*. (2002) voor de kwaliteitskarakteristieken. In alle onderzochte gebieden in Jordanië was de hoeveelheid on-site geproduceerd grijswater (douche, wasgoed, wasbassin en keuken) meer dan het volume dat in principe benodigd is voor on-site hergebruik opties, zoals toiletspoeling, kleren wassen, en irrigatie van de privé tuin, zelfs in de stad Amman. Huizen met een tuin besteden ongeveer 5% van hun drinkwater voor irrigatie (Jamrah *et al*., 2006). Surendran en Wheatley, (1998) rapporteerden dat de grijswater stroom uit douche en was(goed)water voldoende is voor het spoelen van het toilet. Echter, een dagelijkse opslag van behandeld grijswater is absoluut noodzakelijk om aan de dagelijkse hergebruik hoeveelheid en timing te kunnen voldoen. Dat laatste komt overeen met de bevindingen van Jefferson *et al* (1999), gebaseerd op de data van Surendran en Wheatley (1998), betreffende hergebruik van grijswater voor toilet spoelen in het Verenigd Koninkrijk.

De kwalitatieve grijswater parameters zoals vaste stof, BZV_5, CZV en pathogenen geven duidelijk aan dat behandeling van grijswater noodzakelijk is vóór opslag en hergebruik. Echter, de concentratie aan anorganische stoffen in het grijswater en de natrium-adsorptie-ratio (SAR) waarden, met uitzondering die van het was(goed)water, voldoen aan de hergebruikrichtlijnen voor irrigatie in Jordanië. In overeenstemming met de bevindingen uit de literatuurstudie in Hoofdstuk 1, wordt op basis van de grijswaterkarakterisering in Hoofdstuk 2, sterk afgeraden om onbehandeld of louter fysisch behandeld grijswater op te slaan. De belangrijkste aanbeveling uit dit hoofdstuk is toepassing van een biologisch zuiveringsysteem (b.v. Nolde, 1999; Jefferson *et al*., 2004), bestaande uit een anaerobe en aerobe stap, waarbij de vooropslag en behandeling zijn gecombineerd. Een dergelijk biologisch systeem is in staat om de fluctuaties in volume en verontreinigingen op te vangen. Uit de hydrografieken en verontreiniginggrafieken blijkt dat voor het ontwerpen van de biologische zuivering op campussen en hotels de hydraulische belasting als belangrijkste parameter gebruikt kan worden. Voor huishoudens moet de organische belasting worden aangehouden.

7.2.3 Grijswater biologische afbreekbaarheid

De organische componenten in grijswater kunnen tot ernstig stankoverlast leiden (Diaper *et al*., 2001), maar ook to verstoppingen van het distributiesystemen en zelfs tot muggenbroedplaatsen (Christova-Boal *et al.,* 1995). Het ontwerp van een grijswater zuiveringsysteem voor huishoudens om dergelijke problemen te voorkomen is gebaseerd op de maximale biologische omzettingscapaciteit en de biologische afbreekbaarheid van de organische stof bij verschillende temperaturen. Tot dusver zijn slechts enkele onderzoeken verricht naar de afbreekbaarheid van grijswater componenten (Elmitwalli en Otterpohl, 2007; en Zeeman *et al.*, 2008).

In Hoofdstuk 3 zijn de aerobe en anaerobe biologische afbreekbaarheid van de grijswater componenten, evenals de bijbehorende kinetische parameters, nader onderzocht. Het onderzoek is uitgevoerd met grijswater afkomstig uit een Jordaanse universiteitscampus (University of Jordan, Amman). Tevens is de maximale biologische afbreekbaarheid en omzettingsnelheid bepaald van een of meerdere CZV fracties van het afvalwater, te weten totaal, gesuspendeerd, colloïdaal en opgelost CZV, bij 20, 25, 30 en/of 35 ℃. De resultaten zijn gebruikt voor de berekening van de slibleeftijd in een potentieel aeroob en anaeroob zuiveringproces. De gemeten biologische afbreekbaarheid, omzettingsnelheden en Arrhenius temperatuurcoëfficiënten voor anaerobe en aerobe afbraak van grijswater zijn samengevat in Tabel 7.1. De biologische afbreekbaarheid verschilt bij de verschillende CZV fracties. De aerobe en anaerobe afbreekbaarheid is het laagst voor de gesuspendeerde CZV fractie, de omzettingsnelheid is het laagst voor de opgeloste CZV fractie. Aerobe en anaerobe zuivering vereist een lange slibleeftijd.

Tabel 7.1. Samenvatting resultaten van aerobe en anaerobe batch testen met totaal grijswater.

	Aeroob	Anaeroob
Maximumbiologische afbreekbaarheid (%)	86	70
K_{20} Omzettingsnelheid (dag^{-1})	0.119	0.005
Temperatuur coëfficiënten	1.069	1.099

7.2.4 Grijswater behandeling voor irrigatie doeleinden

Op basis van de resultaten van Hoofdstuk 1-3 is een zuiveringsysteem ontworpen met opslag mogelijkheid voor een maximaal volume t.b.v. hergebruik. Het geproduceerde effluent zou moeten voldoen aan de Jordaanse irrigatierichtlijnen.

Het zuiveringsysteem bevat anaerobe en aerobe conversie processen. De zuiveringsunits zijn vervaardigd van lokaal verkrijgbare materialen. Drie verschillende concepten voor geïntegreerde vooropslag en zuivering van het grijswater zijn onderzocht in deze studie, te weten:

(a) een ladingsgewijs gevoed systeem bij een variërend influent debiet gedurende de dag,
(b) een systeem gevoed met variërende influent debieten gedurende de dag en een constant vloeistof volume in de reactor
(c) een systeem gevoed met variërende influent debieten gedurende de dag, een variërend vloeistof volume in de reactor en een constant effluent debiet.

De effluentkwaliteit van zowel de anaerobe als de aerobe unit is bepaald en de waarden zijn geëvalueerd voor geschiktheid voor irrigatie conform de richtlijnen.

De resultaten in Hoofdstuk 4 laten zien dat met een influent belading die het natuurlijke onregelmatige patroon volgt de optimale condities voor de anaerobe unit een operationele cyclus van 24 uur is, gevoed onder een opstroom regime en met een continue effluent lozing met een constant debiet. De geteste aerobe nazuivering werd bedreven met een neerwaarts stromingregime en een passieve beluchting. De waargenomen beluchtingscapaciteit bleek echter onvoldoende. Daarom werd de aerobe nazuivering bedreven met behulp van mechanische aeratie en met een opstroom regime. Om mechanische aeratie te vermijden kunnen andere reactorconcepten met natuurlijke aeratie worden overwogen voor een toekomstige toepassing. Tandukar *et al* (2007) beschrijven een succesvolle toepassing van een combinatie van een UASB en DSH (neerwaartse stroming hangende spons) reactor voor de behandeling van huishoudelijk rioolwater. Men concludeerde dat aeratie niet nodig was en men een gelijkwaardig verwijderingpercentage behaalde als met een actiefslib installatie. De conclusie was dat een gecombineerd UASB-DHS systeem kosteneffectief en geschikt kan zijn voor de zuivering van huishoudelijk afvalwater, vooral in lage-lonen landen (Tandukar *et al*., 2007). Ofschoon in dit proefschrift is aangetoond dat de zuivering van grijswater andere vereisten heeft dan huishoudelijk rioolwater, lijkt het waarschijnlijk dat de toepassing van een DHS systeem voor de nazuivering van grijswater de effluentkwaliteit zeker zal verbeteren. Tandukar *et al*. (2007) rapporteerde dat het gecombineerde UASB-DHS system gelijkwaardig aan een actiefslib installatie is wat betreft de verwijdering van pathogenen, bijvoorbeeld de totaal coliform verwijdering van het UASB-DHS systeem is 4 log eenheden en 2,4±0,4 van een actiefslib installatie (Wen *et al*., 2009). De anaerobe zuiveringstap behaalt 89% gesuspendeerd CZV verwijdering met grijswater. De verwijdering van totaal CZV is beperkt tot 42%. De verwijderingefficiëntie in de aerobe unit gevoed met het anaerobe effluent is ook beperkt tot 53% van de totaal CZV, 59% van de colloïdaal CZV en 54% van de opgeloste CZV. Het systeem is echter in staat om het onregelmatige influentdebiet te verwerken, zelfs bij piekbelasting, en met temperatuur fluctuaties. Isolatie van de reactoren is zeker in de winter noodzakelijk en houdt de reactorinhoud tussen 17-25 ℃. De aerobe effluent kwaliteit, voldoet aan de voorgeschreven Jordaanse richtlijnen voor gazon irrigatie, met uitzondering van de concentratie pathogene indicator organismen. De anaerobe en aerobe afbreekbaarheid (stabiliteit) van het geproduceerde slib na 160 dagen is respectievelijk 20 en 7%. Elmitwalli and Otterpohl (2007) rapporteerden stabiel anaeroob slib met een afbreekbaarheid van 7-12%. Het zuiveringsysteem vertoonde geen operationele problemen zoals slibuitspoeling.

7.3 Discussie

7.3.1 Algemene discussie

De in de vakliteratuur meest besproken problemen in relatie tot hergebruik van grijswater zijn i) de variatie in de karakteristieken

(Eriksson *et al*., 2002); ii) de toe te passen zuiveringstechnologie (Nolde, 1999; Winward *et al*., 2008); iii) de hergebruikopties (Christova-Boal *et al.,* 1995; Otterpohl *et al*., 1999; Eriksson *et al*., 2002); en/of iv) de richtlijnen voor hergebruik, ofwel wetgeving (Jefferson *et al*., 1999; Gross *et al*., 2007).

De resultaten uit dit proefschrift laten duidelijk zien dat grijswater concentraties variëren afhankelijk van de locale condities. Grijswater kan worden gedefinieerd als laag, medium en zelfs als hoog-geconcentreerd huishoudelijk afvalwater. Dit in tegenstelling tot eerdere rapportages, die grijswater als een verdunde afvalstroom karakteriseren (Christova-Boal *et al.,* 1995). Grijswater kan hoog geconcentreerd zijn wat betreft CZV, BZV, droge stof en/of pathogenen, zoals te vinden in rurale gebieden in Jordanië (Hoofdstuk 2), of een medium tot hoog geconcentreerde stroom zoals in Sneek (Hernandez *et al*., 2008) of in Flintenbreite in Luebeck in Duitsland (Elmitwalli and Otterpohl, 2007), of als medium tot laag geconcentreerd zoals te vinden is op de campus van Jordan Universiteit. (Hoofdstuk 2). De reden voor de hoge concentraties van het grijswater in Jordanië is de lagere waterconsumptie in gebieden van waterschaarste (Otterpohl *et al*., 1999, Hoofdstuk 2), of het meenemen van het keukenafvalwater bij het grijswater (Hoofdstuk 2). Het campusafvalwater dat voornamelijk in deze studie is gebruikt bevat geen keukenafvalwater en kan als laag geconcentreerd worden beschouwd.

Om een ontwerp te maken voor een zuiveringsysteem zijn de anaerobe en aerobe afbreekbaarheid en omzettingsnelheden bepaald. In vergelijking met het huishoudelijk afvalwater is het organische materiaal in grijswater slechts zeer langzaam afbreekbaar (Hoofdstuk 3). Meer dan 50% van het grijswater CZV is opgelost of colloïdaal (Hoofdstuk 3; Elmitwalli en Otterpohl, 2007; Hernandez *et al*., 2007; 2008). De uitdaging was het ontwikkelen van een zuiveringsysteem dat onder voorgenoemde omstandigheden een stabiel effluent kan produceren dat voldoet aan de Jordaanse irrigatie standaarden. Dit proefschrift geeft richtlijnen voor het zuiveringsconcept voor grijswater en technische specificaties. Door integratie van vooropslag en biologische zuivering wordt het totaalconcept relatief eenvoudig; bijvoorbeeld een anaerobe tank gevolgd door een aerobe tank zonder dat een pomp nodig is. In de winter is isolatie noodzakelijk. Tijdens de continue experimenten werd slechts 42% van het CZV verwijderd in de anaerobe voorzuivering en 70% in de gecombineerde anaerobe en aerobe zuivering. Dit in tegenstelling tot de hoge anaerobe en aerobe biologische afbreekbaarheid in de ladingsgewijze testen, waarbij respectievelijk 70 en 86% CZV verwijdering werd behaald. In principe zou het het verwijderingrendement van het continue zuiveringsysteem dus verbeterd moeten kunnen worden. Verbetering van het anaerobe en aerobe effluent is mogelijk met filtratie zoals toegepast door Imura *et al*. (1995) of door toevoeging van een coagulant zoals getest door Cui en Ren (2005). In dit proefschrift is geen onderzoek verricht naar methaanopvang en toepassing. De geschatte hoeveelheid methaan uit de vergisting van campus grijswater van 150 personen is 2,18L CH_4 p.p.d. Het opgevangen biogas kan worden gebruikt voor gaslampen (ACCU)

In plaats van rigide richtlijnen wordt in dit proefschrift voorgesteld om multi-categorieën voor hergebruikstandaarden te ontwikkeld, aangezien de verschillende hergebruikopties verschillende waterkwaliteiten benodigen, rekening houdend met de esthetische, gezondheid, en milieu-aspecten. Multi-categorie hergebruikstandaarden bieden mogelijkheden dat de hergebruikkwaliteit van water, bijvoorbeeld voor toilet spoeling, het zelfde kan zijn als voor irrigatie, met uitzondering van de concentratie aan pathogene organismen. Water voor hergebruik als badwater en autowassen zou aan strengere eisen moeten voldoen. Zoals de WHO richtlijnen (2006) voor veilig hergebruik van grijswater voor irrigatie, zou de ontwikkeling van hergebruikstandaarden tevens richtlijnen moeten hebben voor veilige toepassingen voor elke hergebruikoptie.

7.3.2 DeSaR in Jordanie

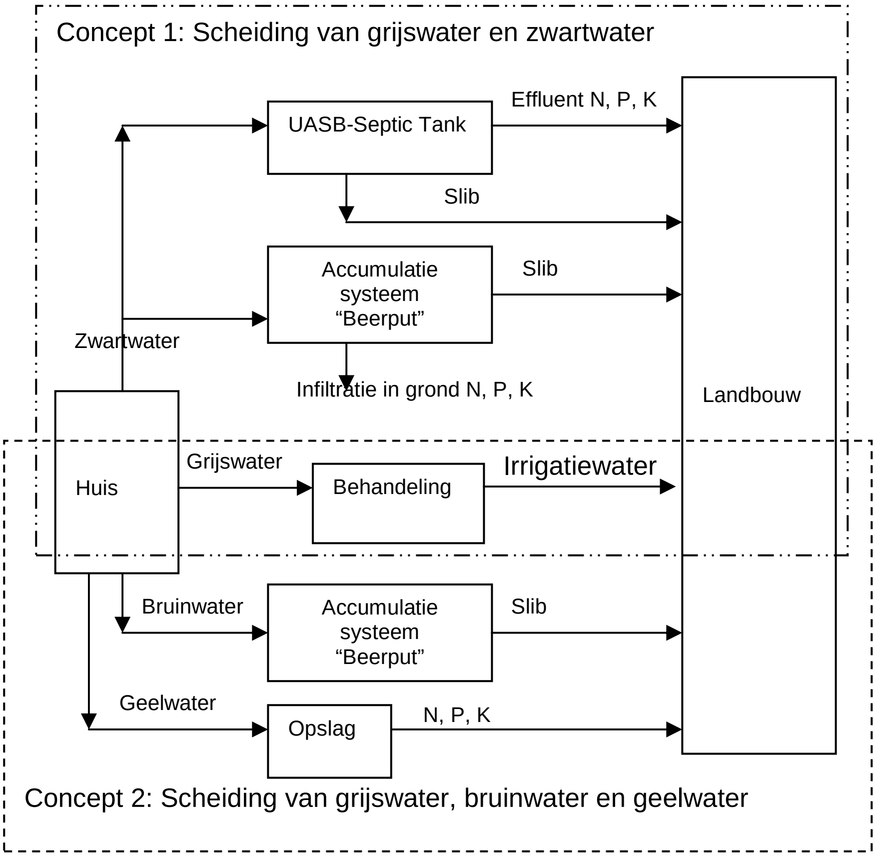
Als gevolg van de wijd verspreide waterschaarste in Jordanië is de scheiding, zuivering en hergebruik van grijswater een zeer attractieve mogelijkheid. Op bescheiden schaal wordt grijswaterhergebruik reeds toegepast. De resultaten uit onderhavige studie geven diverse mogelijkheden voor een kosten-effectieve behandeling van grijswater, waardoor grotere hoeveelheden behandeld grijswater beschikbaar zouden kunnen komen voor hergebruik. Echter, een bredere toepassing van grijswater scheiding, behandeling en hergebruik kan invloed hebben op het transport en zuivering van de rest van het huishoudelijk afvalwater, zeker daar waar een centraal riolering- en zuiveringsysteem wordt toegepast (= 47% van de sanitatie systemen, WAJ, 2000). Volledige afscheiding van het grijswater leidt tot een hoog geconcentreerde rest afvalwaterstroom, het zogenaamde ‘zwartwater’. Transport van dit zwartwater in riolen met behulp van zwaartekracht, zoals dit meestal het geval is, kan tot verstoppingen leiden. Aangezien wijzigingen in bestaande civiele infrastructuren niet snel te verwachten zijn, zijn projecten gebaseerd op scheiding van grijswater met name interessant voor nieuwe constructies bij stadsuitbreidingen of bij renovaties. In dergelijke gebieden biedt ook de afzonderlijke inzameling van zwart water voor het terugwinnen van energie en meststoffen zeer interessante mogelijkheden. Bij een dergelijk aanpak kunnen kosten-effectieve anaerobe zuiveringstechnologieën een cruciale rol spelen.

Zwartwater bevat het grootste aandeel van stikstof, fosfaat en kalium (Gulyas *et al*., 2004; Kujawa-Roeleveld en Zeeman, 2006). De terugwinning van deze nutriënten voor gebruik in de landbouw is belangrijk (Otterpohl *et al*., 1999). Anaerobe zuivering van zwartwater wordt al toegepast op pilot schaal in Nederland (Zeeman *et al*., 2007). In Jordanië wordt tot nu toe anaerobe zuivering slechts succesvol toegepast als voorbehandeling van huishoudelijk afvalwater (Halahsheh, 2002). Echter, ook voor de behandeling van zwartwater in Jordanië is anaerobe zuivering een zeer interessante optie. Het type toilet en daarmee de mate van verdunning van zwartwater, en de mate van grijswaterscheiding, bepalen de eindconcentratie van verontreinigende stoffen in het restafvalwater. Deze concentratie bepaalt in hoeverre het verder concentreren en terugwinnen van meststoffen tot een reeele optie behoort. Met name in nieuwbouwprojecten kan men op relatief eenvoudige wijze het inzamelen met speciale toiletten, evenals transport en behandeling van het zwarte water kunnen optimaliseren. In feite is deze optimalisatie een voorwaarde om geconcentreerd zwartwater te verkrijgen dat geschikt is voor winning van energie en nutriënten. In Duitsland (Otterpohl *et al*., 1997; Wendland en Oldenburg, 2003) en Nederland (Zeeman *et al*., 2007) zijn diverse proefprojecten gestart waarbij met behulp van vacuüm'spoeling' en transport geconcentreerd zwartwater wordt verkregen. Het wordt aanbevolen om de toepasbaarheid van zulke systemen in Jordanië nader te testen en te evalueren. Grote hotels zouden een goede testlocatie kunnen zijn waarbij het gescheiden ingezamelde grijswater voor irrigatiedoeleinden in b.v de tuinen kan worden gebruikt.

De situatie op landelijke locaties verschilt nogal van de stedelijke omgeving. In de landelijke omgeving zijn 95% van de sanitaire voorzieningen aangesloten op beerputten. Bij afscheiding van de grijswaterstroom zou de belading van de beerput verminderen. Afzonderlijk inzameling en behandeling van zwartwater heeft hier dus een positief effect. Immers, de toepassing van beerputten voor de totale afvalwaterstroom resulteert in het verlies van een groot deel van de in zwartwater aanwezige meststoffen en kan leiden tot besmetting van het grondwater. Afhankelijk van het waterverbruik voor het spoelen van het toilet, kan de beerput worden gebruikt als een accumulatie systeem (Kujawa *et al*., 2006) of hij kan worden omgebouwd tot een UASB septic tank ter stabilisatie van het organisch materiaal in het zwartwater

(Zeeman *et al*., 2007). Het effluent van de UASB septic tank en het slib van het accumulatie systeem zouden kunnen worden hergebruikt voor bemesting, waarbij uiteraard voorzichtigheid moet worden geboden gezien het mogelijke besmettingsgevaar met pathogene organismen uit deze stromen.

De introductie van gescheiden urineopvang is eveneens een interessante optie. Urine bevat de hoogste concentratie aan meststoffen van alle huishoudelijke stromen en het geproduceerd volume is slechts 0.6-2.5 L p.p.d. (STOWA, 2001; Vinnerås, 2002). Na 6 maanden opslag is urine hygiënisch veilig te hergebruiken in de landbouw (Schönning, 2001). Indien urine apart wordt opgevangen kan de bestaande beerput worden gebruikt voor de behandeling van het resterende ‘bruinwater’. Bruinwater bevat slechts een klein gedeelte van de meststoffen, waarmee het verlies aan stikstof en fosfaten wordt geminimaliseerd. Het voorgestelde concept voor landelijke locaties is weergegeven in Figuur 7.1. Dit concept voorziet in de optimalisatie van gescheiden afvoer en hergebruik van grijswater en meststoffen met gebruikmaking van bestaande infrastructuur en minimalisatie van emissies naar het grondwater. Sociale acceptatie van een dergelijk nieuw concept is cruciaal, waarbij sociale en technische aspecten gelijktijdig ontwikkeld moeten worden (Hegger, 2007).



Figuur 7.1. Twee potentiële concepten voor toepassing van DeSaR in landelijke omgeving met beerput sanitatie: de concepten bestaan uit bron scheiding, behandeling en hergebruik. Zwartwater is een mengsel van bruin- en geelwater.

7.4 Conclusies

Grijswater kan worden gezien als een laag, medium of hoog geconcentreerd huishoudelijk afvalwater.

Het onderzochte campus grijswater bestaat uit douche, was(goed) en wasbassin waterstromen. De organische stoffen in het grijswater, gemeten als CZV, zijn voor meer dan 50% opgelost en colloïdaal en is voor 70% anaeroob en 86% aeroob matig afbreekbaar.

Het voorgestelde grijswater behandelingsconcept zeer eenvoudig: integratie van vooropslag en biologische behandeling, te weten, een anaerobe reactor gevolgd door een aerobe reactor, zonder toepassing van een pomp. Om de werking van de biologische units stabiel te houden gedurende de winter periode is isolatie noodzakelijk.

Grijswater hergebruik vereist duurzame multi-categorie standaarden met richtlijnen voor correcte en veilige hergebruiktoepassing. De kern van de standaarden is gericht op esthetische, gezondheid en milieu problemen.

Grijswaterbehandeling in de landelijke omgeving moet verder worden onderzocht. In samenhang hiermee moet ook nader onderzoek worden uitgevoerd naar de mogelijkheden om zwartwater te hergebruiken en op welke wijze zwartwater kan worden ingezameld en gezuiverd. De toepasbaarheid van het DeSaR concept wordt bepaald door het succesvol hergebruiken van grijs- en zwartwater componenten. De uitdaging daarvoor is de ontwikkeling van betaalbare technologieën die producten opleveren voor hergebruik zonder negatieve effecten voor esthetiek, gezondheid en het milieu.

7.5 Aanbevelingen en toekomstig onderzoek

Aanbevelingen:

Verbetering van de biologische afbreekbaarheid van grijswater verontreinigingen: het stimuleren en propageren van toepassing van makkelijker afbreekbare materialen die in het grijswater eindigen, bijvoorbeeld biologisch afbreekbare detergenten.

Ontwikkelen van betrouwbare multi-categorie standaarden voor grijswater hergebruik gecombineerd met richtlijnen voor veilig gebruik.

Toekomstig onderzoek

Onderzoek naar de fysische eigenschappen van grijswater verontreinigingen zoals deeltjesgrootteverdeling en oppervlaktelading, om de grijswater biologische zuiveringsprocessen te verbeteren.

Onderzoek naar de potentiële integratie van grijswater- en zwartwaterhergebruik in Jordanië in bestaande en nieuwe concepten, evenals onderzoek naar de meeste optimale management structuur van dergelijke concepten.

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Conferences contributions

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Papers in progress

Abu Ghunmi, L., Zeeman, G., Fayyad, M. and van Lier, J. B. Grey water biodegradability. *Submitted*

Abu Ghunmi, L., Zeeman, G., Fayyad, M. and van Lier, J. B. Grey water treatment in a sequencing anaerobic –aerobic system for irrigation. *Submitted*

About the author

Lina Nawwash Abdel-Hafiz Abu Ghunmi was born on March 22^{ed}, 1976 in Amman-Jordan. In 1999 she got her bachelor degree in Chemical Engineering from Jordan University, and in 2002 she got Masters degree in Civil Engineering/ Environmental Engineering and Water Resource, from the same University. In 2002 she worked as a research assistant, at the Water and Environment Research Study Center at Jordan University, on CORTECH project which was funded by the EU. In 2004 she was the candidate of Jordan University to get the Nuffic, Dutch government, scholarship to do PhD in Environmental Technology. In February, 2005 she started her PhD studies at the Sub-department of Environmental Technology in Wageningen University, The Netherlands.

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