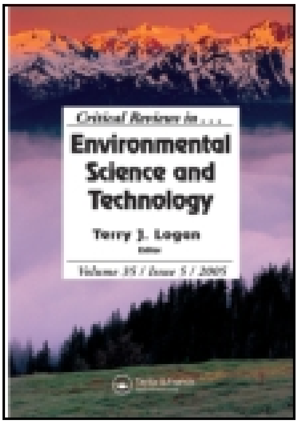

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Grey Water Treatment Systems: A Review

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Grey Water Treatment Systems: A Review

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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.

KEY WORDS: biological, chemical, grey water, physical, reuse, standards, treatment technologies

1 INTRODUCTION

Water shortage and water pollution have become global issues; related issues are scarcity of water resources, mismanagement, population growth, and climate change (Arnell, 1999; Bouwer, 2000; Falkenmark, 1990). Industrial and domestic wastewaters' constituents contribute to water resource and soil pollution (Metcalf and Eddy, 2003). Wastewater treatment and recycling

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of useful products (i.e., water, nutrients, and organic matter) mitigates 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 (Otterpohl et al., 1999; Otterpohl et al., 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; Eriksson et al., 2002; Jefferson et al., 1999; Otterpohl et al., 1999).

Grey water contribution to domestic wastewater is 60–75% of the water volume (Gulyas and Raj Gajurel, 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. First is reuse with or without simple treatment (Al-Jayyousi, 2002; Christova-Boal et al., 1995). Second is recycling for indoor use, such as flushing toilets, washing clothes, and bathing (e.g., Bingley, 1996; Christova-Boal et al., 1995; Cui and Ren, 2005; Jefferson et al., 1999, 2001; Li et al., 2003; Nolde, 1999; Shrestha et al., 2001a, 2001b), 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 groundwater (e.g., Al-Jayyousi, 2002, 2003; Bingley, 1996; Christova-Boal et al., 1995; Eriksson et al., 2002; Fittschen and Niemczynowicz, 1997; Jefferson et al., 2001; Nolde, 1999; Okun, 2000; Otterpohl, 1999; Shrestha et al., 2001a, 2001b). Third, standards are mainly related to health and social aspects in order to improve the control of the recycling process (e.g., Cui and Ren, 2005; Jefferson et al., 1999, 2000, 2001; Li et al., 2003; Nolde, 1999). Fourth is the obtaining of affordable treatment technologies to cope with the quantity and quality variation of grey water sources (Eriksson et al., 2002; Imura et al., 1995), and the recycling requirements (e.g., Cui and Ren, 2005; Jefferson et al., 1999, 2000, 2001; Li et al., 2003; Nolde, 1999). A wide range of treatment technologies have been applied and examined for grey water and considering one or more of the grey water issues, producing effluents with different qualities. In this review, therefore, we examine 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, simple in operation with a 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 regrowth is minimal. The issues considered in the selection of grey water treatment systems are grey water characteristics, used standards, technology performance, and costs.

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 slow or nonbiodegradable under anaerobic conditions (Garcia et al., 1999; Matthew and Malcolm, 2000). Storing grey water for 48 hr at 19–26°C deteriorates its quality (Dixon et al., 1999), and biological degradation produces malodorous compounds, causing an aesthetic problem (Christova-Boal et al., 1995; Dixon et al., 1999), pathogens breeding (Christova-Boal et al., 1995; Dixon et al., 1999; Rose et al., 1991) and mosquito breeding (Christova-Boal et al., 1995), which are a health threat. Use of raw grey water clogs the recycling system due to buildup of suspended material and/or the biological growth in the systems (Christova-Boal et al., 1995). Raw grey water quality characteristics do not comply with the standards (Table 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 (Jefferson et al., 1999, 2004; Nolde, 1999).

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, groundwater discharge, or fire extinguishing (e.g., Eriksson et al., 2002; Jefferson et al., 1999). The adopted standards (Table 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; nondetectable Fecal Coliform (FC) and Total Coliform (TC) are lower than bathing water standards in the United Kingdom. Furthermore, China created differentiated standards (e.g., for toilet flushing, car cleaning, 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 1) are neither uniform nor globally standardized. Development of multicategory standards is required for an optimal use of grey water. The standards should include

TABLE 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/ 100 mL	FC CFU/ 100 mL	EC CFU/ 100 mL	References
USA-EPA											U.S. EPA (2004)
Unrestricted use[a]	2	≤10	—	—			6–9	—	ND		
Restricted use[b]		≤30		≤30			6–9		≤200		
WHO											WHO (2001)
Restricted irrigation	—	—	—	—			—	≤1E5	—	—	
Unrestricted irrigation[c]								≤1E3			
Drinking quality[d]	≤5				50		6.5–8.5				
UK—Bathing water								5E2[g]–1E4[m]	1E2 [g]–2E3[m]		Parker and Frost (2000)
				China[e]							China (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											Ogoshi et al. (2001)
Toilet flushing							5.8–8.6	≤1000			
Landscape irrigation							5.8–8.6	ND			
Jordan											Jordan (2002)
Recharge aquifer	2	15	50	50	30	15	6–9	—	<2.2	<2.3	
Unrestricted irrigation[f]	10	30	100	50	45		6–9	—	100	101	

Note. ND = not detectable; g = guidelines; m = mandatory.
[a]urban uses, crops eaten raw, recreational impoundments.
[b]restricted access area irrigation, processed food crops, nonfood crops, aesthetic impoundments, construction uses, industrial cooling, and environmental reuse.
[c]crops eaten raw.
[d]drinking water quality, 1993.
[e]Nitrogen are for ammonia measurements.
[f]irrigation of vegetables (to be cooked before consumption), parks, playgrounds, and roadsides or roads within city limits.

different aspects such as health, aesthetic, and environment. For instance, the WHO (2006) guidelines for use of grey water have two categories, 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).

3 GREY WATER TREATMENT SYSTEMS

A grey water treatment system consists of different treatment steps that may be considered, depending on the required quality of the effluent (Figure 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.

3.1 Filtration and Physiochemical Processes

Several types of macro- and membrane-filtration units for grey water treatment have been tested. The tested macrofiltration units include a strainer series with pore size ≥0.17 mm, nylon sock-type filters, geotextile (filter sock) filters, fibrous (cloth) filters, coarse filters (CF), and sand filters (SF; Al-Jayyousi, 2003; Christova-Boal et al., 1995; Friedler et al., 2006; Jefferson et al., 1999). The tested membrane-filtration units, sheet or tubular, were (a) microfilter 0.1 μm Membrane Fibrous Filters (M[F]F; Ahn et al., 1998) and $\leq$ 0.2 μm M(F)F (Shin 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), and 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. Last, 75, 80, and 90% $CaCl_2$ rejection

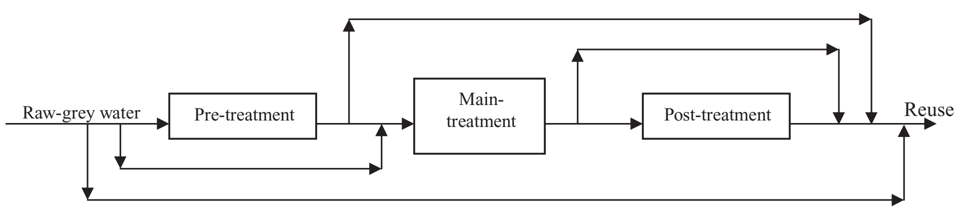

FIGURE 1. Gray water recycling and treatment possible steps and tracks.

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). 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 with UF; the underlying reason is the presence of organic matter with low molecular weight in grey water that cannot be rejected by UF. Table 2 also 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 ultrafiltration 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, have been tested for nutrients removal of 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 that 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; Friedler et al., 2006; Jefferson et al., 1999). 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

TABLE 2. Filters or Physicochemical Units Treating Grey Water; Influent and Effluent Qualities and Operation Conditions

Treatments	BOD 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}									
						Macrofilters							
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		143			44.5							
Effluent	12		35.7			32.3							
ST+ Back-washing 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				316								
Effluent	392				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 days, 15 min 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)				
						Microfilters							
M(F)F	Volume 20 m^3, membrane hollow fiber polyproplyne with 0.2 μm nominal pore size. Air automatic backwashing and chemical for cleaning after long operation												Shin et al., 1998
GW-Source	Japanese Office building; Cooking, bathing, washing												
Influent					19–113								
Effluent					around 1								

(Continued on next page)

TABLE 2. Filters or Physicochemical Units Treating Grey Water; Influent and Effluent Qualities and Operation Conditions *(Continued)*

Treatments	BOD mg L^{-1}		COD mg L^{-1}		SS	Turbidity	N	P	FC				
	BOD_{tot}	BOD_{dis}	COD_{tot}	COD_{dis}	mg L^{-1}	NTU	mg L^{-1}	mg L^{-1}	CFU/100 mL	Chemical	Energy	Cost	References
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		143			44.5							
Effluent	5		22.2			0.34			0[a]				
GW-Source													
Influent	25–185		86–410			12–100			2E0-3.1E5[b]				
Effluent	1–19		21–112			<1			ND-2.4E3[b]				
grey													
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 200 kDa	Polyvinyslidenofluoride												
Influent	20–25	6–10											
Effluent	11	8											
MWCO 6 kDa	Modified polyethersulphone												
Influent	20–25	6–10											
Effluent	5	5											
MWCO 4 kDa	Tight polyethersulphone												
Influent	20–25	6–10											
Effluent	6	5											
75% $CaCl_2$ rejection	Polyamine film												
Influent	20–25	6–10											
Effluent	3	3											
80% $CaCl_2$ rejection	Polyamine film												
Influent	20–25	6–10											
Effluent	2	2											
90% $CaCl_2$ rejection	Cellulose acetate												
Influent	20–25	6–10											
Effluent	5	5											
Grey water source	sport centre from public shower of the												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 rejects 400 kDa							
Influent		80 (21.5)			1.4 (0.4)		
% removal		45			92		
MWCO rejects 200 kDa							
Influent		74 (28.6)			1 (0.5)		
% removal		49			94		
MWCO rejects 30 kDa							
Influent		50.6 (6.6)			0.8 (0.2)		
% removal		69			97		
grey **N(F)F**	Tubular nanofilter 30 cm length, 1.25 inner diameter, and 0.014 m^2 filtration area, applied pressure 6–10 bar, cross flow filtration unit, 150 L hr^{-1} flow rate						Ramon et al., 2004
MWCO rejects 200 Da							
Influent		226			29.5 (0.6)		
% removal		93			98		
Physicochemical 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)[c]				9E5[b]	
Effluent		25 (TOC)[c]				ND[b]	
Multiple physico-chemical 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] *E. coli*.
[b] TC = total coliform; ND = not detectable.
[c] TOC = total organic carbon.

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 pretreatment of raw grey water in storage and 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 pretreatment tank should be optimized to prevent deterioration of its effluent quality (Imura et al., 1995; Shrestha et al., 2001a, 2001b). Adding coagulants such as $AL_2(SO_4)_3$, $FeCl_3$, polyaluminium chloride, and PFS in combination with mixing enhances the performance of the pretreatment tank (Cui and Ren, 2005). Application of physicochemical processes, as shown by Pidou et al. (2007), is promising for grey water treatment, certainly when considering the short hydraulic retention time (HRT; <1 hr) that can be applied. However, more primary sludge is produced, resulting in an increase in operational costs. Different types of posttreatment units have been used to enhance the filters' effluent turbidity, suspended solids, organic matter, or pathogens qualities. The tested units (Table 2) are Ultra Membrane Filtration (Cui and Ren, 2005; Hills et al., 2001), Activated Carbon Absorber and Ultraviolet radiation (Cui and Ren, 2005), and disinfection by halogens (Al-Jayyousi, 2002; Christova-Boal et al., 1995). 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 1). Therefore, membrane filtration (i.e., micro-, ultra-, and nanofilters) could be an option for posttreating grey water to achieve the most conservative standards.

3.2 Modified Filters

Filters' performances have been improved by modifying the operational conditions, such as flow direction, 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.

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 subclassified according to the tested flow directions. Unplanted filters are Intermittent Vertical-Flow Soil Filter (IVSF; Nolde and Dott, 1992), Subsurface 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 Subsurface Filter (SSrF; Dallas et al., 2005), Recycled Vertical Flow Constructed Wetlands (RVFCW; Gross et al., 2007b), and Green Roof Water Recycling System (GRWRS; Winward et al., 2008). The first four planted filters are also called root-zone facilities, or Vertical, Horizontal, and Subsurface Constructed Wetlands (V-CW, H-CW, SSr-CW; Fittschen and Niemczynowicz, 1997; Li et al., 2003; Otterpohl et al., 1999) or (V-, H- and SSr-) Reedbed (Dallas et al., 2005; Winward et al., 2008), respectively.

Performance tests (Table 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 macrofilters (Table 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 or applying vertical flow (Table 3). Moreover, as shown in Table 3, H-CW (Fittschen and Niemczynowicz, 1997), V-CWs (Shrestha et al., 2001a, 2001b; Winward et al., 2008), SSrF-CW (Dallas et al., 2005), and GRWRS (Winward et al., 2008) produce effluent qualities in terms of BOD that comply with all standards (Table 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, 14, and 5.1–8.5 days, respectively, compared with other tested systems (Tables 2, 4–7). The tested systems (Table 3) show different capacities in treating pathogens according to differences in their internal structure and the applied HRT. However, none of the effluents comply with all standards with regard to pathogens (Table 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 (Dallas et al., 2005; Shrestha et al., 2001a, 2001b). Local conditions must be considered in the design, such as temperature, rainfall, and wastewater composition (Dallas et al., 2005; Fittschen and Niemczynowicz, 1997; Shrestha et al., 2001a, 2001b). Consequently, Shrestha et al. (2001a, 2001b) concluded that development of appropriate design guidelines for constructed wetlands is imperative.

Pretreatment of grey water in settling tanks for constructed wetlands has been tested (Fittschen and Niemczynowicz, 1997; Li et al., 2003; Shrestha et al., 2001a, 2001b). Shrestha et al. (2001a, 2001b) 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. Posttreating CW effluent in a sand filter enhances the quality (Table 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 3) and complies easily with European

TABLE 3. Soil Filters and Planted Soil Filters (CWs) Combined with Others Units Treating Grey Water; Influent and Effluent Qualities and Operation

Treatments	BOD (mg L^{-1})	COD (mg L^{-1})	SS (mg L^{-1})	Turbidity NTU	TN (mg L^{-1})	TP (mg L^{-1})	TC CFU/100 mL	Surface Areas m^2 cap^{-1}	Volume m^3	Hydraulic Load m^3 days^{-1}	HRT days^{-1}	Chemical	Energy	Cost	References
							Soil filters								
IVSF									1.4		38–75				Nolde and Dott, 1992
GW-Source	Grey water without kitchen wastewater														
Effluent	<3														
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	Dry season	8.5 E7 ± 5.7E7[a]							5					
Effluents															
SSrF (PET)	16 ± 4						1.1E6 ± 2.0E6[a]		0.075		5.6				
SSrF (Rock)	9.0 ±1						2.9 E5 ± 6.9E5[a]		0.075		4.6				
Influent	155 ± 14	Dry season					3.8E6 ± 1.0E6[a]			10					
Effluents															
SSrF (PET)	18 ± 2						1.1E6 ± 2.3E6[a]		0.075		3.3				
SSrF (Rock)	19 ± 4						1.3E5 ± 2.9E5[a]		0.075		2.7				
Influent	290 ± 36	Wet season					1.0E8 ± 1.0E8[a]			5					
Effluents															
SSrF (PET)	14 ± 5						2.7E5 ± 1.7E5[a]		0.075		2.9				
SSrF (Rock)	14 ± 11						2.2E4 ± 3.3 E4[a]		0.075		2.9				
Influent	285 ± 106	Wet season								10					
Effluents															
SSrF (PET)	31 ± 2								0.075		2.1				
SSrF (Rock)	28 ± 0								0.075		1.7				
SSo	Slanted Soil system; soft particle with 1 cm of Kanuma soil that comprises alumina and hydrated silica, the setup; three stacks of 100 × 50 × 17.5 cm slanted plastic foam trays; each trey has three 6 cm ridges to prevent clogging. 12.5 cm the 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[b]	23.3[b,c]	9			1.78	0.323			0.1					
Effluent	7	3.6	2			0.38	0.046								

grey **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 0.2 × 0.35 × 0.5 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 (31)	46 (3.0)		3.5 (0.1)	3.5 (0.1)^{days}	1.9 (0.2)	5E4 (1.3E0)		0.2 × 0.35 × 0.5 m		2–3		
Effluent	47 (6)	3.0 (0.0)		1.8 (0.3)	1.0 (0.06)^{days}	0.5 (0.1)	1.3E0 (1.1E 0)						
							Planted soil filter						
HPSF													Hegemann, 1993
Influent								3.25					
Effluent	10–40												
HR-CW Multistage 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	165	361			18.1	3.9	5.4E5–3.3E6[e]						
HR-CW	Water flows horizontally, a reed bed planted with *Phragmites communis*								600 m^2 × 0.6 m		14		
Influent	165	361			18.1	3.9	5.4E5–3.3E6[e]						
Effluent	<5	46.4			7.4	1.4	1.E2–3.3E4[e]						
VFSF									300 m^2 × 0.8 m		2–4[f]		
Influent	<5	46.4			7.4	1.4	1.E2–3.3E4[e]						
Effluent	<4	43.3			1.3	0.79	0–2E1[e]						
AP	Collected Storm Water								130 m^2 × 1.0 m				
Influent	<4	43.3			1.3	0.79							
Effluent	<4	56.3			<0.43	0.23							
grey **HR-CW**	Horizontal Flow Reed Bed (**HFRB**); water flows continuously to Sand/soil/compost mix media (≤1 mm diameter), planted with *Phragmites australis*												Winward et al., 2008
GW-Source	Bathroom sinks, baths and showers of 18-student flat on the Grandfield University Campus												

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TABLE 3. Soil Filters and Planted Soil Filters (CWs) Combined with Others Units Treating Grey Water; Influent and Effluent Qualities and Operation *(Continued)*

Treatments	BOD (mg L^{-1})	COD (mg L^{-1})	SS (mg L^{-1})	Turbidity NTU	TN (mg L^{-1})	TP (mg L^{-1})	TC CFU/100 mL	Surface Areas m^2 cap^{-1}	Volume m^3	Hydraulic Load m^3 $days^{-1}$	HRT $days^{-1}$	Chemical	Energy	Cost	References
Influent	20 (11)	87 (38)	29 (32)	19.6 (14)			2.51E + 05 ± 6.31E + 00		6 m^2 × 0.7 m	0.48	2.1				
Effluent	2 (1)	29 (9)	9 (8)	16.9 (16)			3.981E + 02								
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 × 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*.														Dallas et al., 2005
GW-Source	Monteverde Institute's grey water.													Operation and maintenance costs are generally low	
Influent	216 ± 55	Dry season					8.5 E7 ± 5.7E7[a]			5					
Effluents															
SSrF-CW (PET)	4 ± 4						2.2 E3 ± 2.4E3[a]		0.075		8.5				
SSrF-CW (Rock)	7 ± 4						2.4 E3 ± 4.7 E3[a]		0.075		5.1				
Influent	155 ± 14	Dry season					3.8E6 ± 1.0E6[a]			10					
Effluents															
SSrF-CW (PET)	13 ± 2						2.1E3 ± 1.7E3[a]		0.075		4.2				
SSrF-CW (Rock)	18 ± 1						2.6E5 ±5.1E5[a]		0.075		2.5				
Influent	290 ± 36	Wet season					1.0E8 ± 1.0E8[a]			5					
Effluents															
SSrF-CW (PET)	10 ± 6						1.5E3 ± 1.7E3[a]		0.075		4.8				
SSrF-CW (Rock)	18 ± 4								0.075		2.4				
Influent	285 ± 106	Wet season								10					
Effluents															
SSrF-CW (PET)	26 ± 6								0.075		2.8				
SSrF-CW (Rock)	26 ± 2								0.075		1.7				

grey												
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, 2001b
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[d]							
Effluent												
V-CW	Intermittent vertical water flow, planted with reed; *Phragmites karka*, filled with coarse sand and grey water is hydromechanically flushed 3 to 4 times per day							0.86				
Effluent												
Storage Tank									0.7			
Effluent	0–12	6.8–72			0.02–1.98[d]							
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[g]			9.7–16.6	5.2–9.6	7.5E3-2.6E5[h]					
V-CW	Intermittent vertical water flow, filled with gravels are between 4–8 mm							2				
Influent		80–94[g]			9.7–16.6	5.2–9.6	7.5E3–2.6E5[hr]					

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TABLE 3. Soil Filters and Planted Soil Filters (CWs) Combined with Others Units Treating Grey Water; Influent and Effluent Qualities and Operation *(Continued)*

Treatments	BOD (mg L^{-1})	COD (mg L^{-1})	SS (mg L^{-1})	Turbidity NTU	TN (mg L^{-1})	TP (mg L^{-1})	TC CFU/100 mL	Surface Areas m^2 cap^{-1}	Volume m^3	Hydraulic Load m^3 $days^{-1}$	HRT $days^{-1}$	Chemical	Energy	Cost	References
Effluent															
Storage Tank															
Influent															
Effluent		<5–28[g]			1.18–5	5.6–6.8	3.3 E2–2.6 E4 [hr]								
Disinfection	The optimum TiO_2 dosage (among 1, 3, 5 and 10 g L^{-1}) and UV irradiation 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 hr UV radiation.														
Influent							58[h]								
Effluent grey							1[h]								

Treatments	BOD (mg L^{-1})	COD (mg L^{-1})	SS (mg L^{-1})	Turbidity NTU	NO_3-N (mg L^{-1})	TN (mg L^{-1})	TP (mg L^{-1})	TC CFU/100 mL	Volume m^3	Hydraulic Load m^3 $days^{-1}$	HRT $days^{-1}$	Chemical	Energy	Cost	References
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 0.4 $kWhm^{-3}$ 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 × 0.7 m	0.48	20[f]				
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 × 0.7 m	0.48	20[f]				
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 he bed. The effluent discharge to 40 L secondary ST and then is used for irrigation.													US 600$ investment cost and 100 labor and maintenance cost.	Gross et al., 2007b
GW-Source	Estimated 450 L $days^{-1}$ 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)[a]			8–24[f]	
Effluent	1 (0)	157 (62)	3 (1)		8.6 (4.3)	10.8 (3.4)	6.6 (1.1)	2E5 (1E5)[a]				
GRWRS	Green roof water recycling system (**GRWRS**): water flows to continuously five rows of shallow troughs (depth), Optiroc expanded clay media (10 mm diameter) topped with gravel chippings (20 mm diameter), planted with a variety of aquatic plants. Baffles and weirs create plug flow and additional aeration for 1 hr per days.											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 m² × 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 m² × 0.1 m	0.48	2.1	
Effluent	80 (38)	159 (64)	20 (8)	28.8 (10.7)				3.16E + 01				

Note. Values in parentheses are standard deviations.
[a]fecal coliform.
[b]The units of the influent and effluent are g m^{-2} $days^{-1}$.
[c]COD based on Mn measurement.
[days]ammonia.
[e]thermostable coliform bacteria.
[f]hours.
[g]TOC = total organic carbon.
[hr]*E. coli*

TABLE 4. Modified Filters: Biofilters Combined With Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions

Treatments	BOD mg L^{-1}		COD	SS	Turbidity	TN	TP	TC CFU/	Volume	HRT	Hydraulic Load	Organic Load Kg BOD m^{-3}	Chemical	Energy	Cost	References
	BOD$_{tot}$	BOD$_{dis}$	mg L^{-1}	mg L^{-1}	NTU	mg L^{-1}		100 mL	m^{-3}	hr	L m^{-2} hr^{-1}	days^{-1}				
Macrobiofilters																
Multistage 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 Lmin^{-1} blower was aerating and mixing (biological processes) the content; water + microorganisms								1.24–1.43							
Effluent	5–10			3–13		1.3–10	0.9–1.6									
ST									0.37–0.46							
Disinfection	solid-chlorine								0.019							
Effluent	5–10			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 2001 article), 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 multistory building									2.7–0.46		1.09 ± 0.73				Jefferson et al., 1999 and 2001
Influent	41 ± 30			52 ± 58				2E0-2E7								
Effluent	4 ± 4			5.9–5.5	3.2 ± 8.9			4E0-1E5								
GW-Source	Artificial grey water; mimicking grey water discharge from multistory 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 multistory 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 grey			52.3 ± 28	18 ± 20	12.6 ± 27			1E4 ± 1E7								

Treatments	BOD mg L^{-1} BOD$_{tot}$	BOD$_{dis}$	COD mg L^{-1}	SS mg L^{-1}	Turbidity NTU	TN mg L^{-1}	TP mg L^{-1}	TC CFU/ 100 mL	HRT hr	Hydraulic Load L m^{-2} hr^{-1}	Organic Load Kg BOD m^{-3} days^{-1}	Chemical	Energy	Cost	References
								Microbiofilters							
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														Jefferson et al., 1999, 2001
GW-Source	Artificial Grey water; mimicking grey water discharge from multistory building									31.5–3.4		0.14 ± 0.07		Substantial capital cost	
Influent	41 ± 30		120 ± 74.7					2E0 ± 2E7							
Effluent	1 ± 2		9.6 ± 7.4		0.32 ± 0.28			2E0-2E1							
GW-Source	Artificial Grey water; mimicking grey water discharge from multistory building											acid cleaning	4.0E3 kWh kg product^{-1}	sub critical flux lowers membrane replacement cost, but significantly increases investment cost	Jefferson et al., 2000
Influent	9–100							2E2-5E7							
Effluent	<10				<2			≤1E1							
GW-Source	Primary sewage + (Artificial Grey water mimics grey water of multistory building)								29.2–5.5		0.16 ± 0.08				Jefferson et al., 2001
Influent			144 ± 85.7	62.6 ± 40				2E2 ± 5E8							
Effluent			10.6 ± 5.5	3.6 ± 3.7	0.4 ± 0.28			1E0-1E3							
GW-Source	Primary sewage													compares with BAF	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	13 ± 27			1E0 –<2E2							
M(F)BR	Type-Submerged; working volume 0.0073 m^3, membrane surface area 0.04 m^{2}m^{-3}, fiber 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 multistory 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 fiber 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} days^{-1}. F/M 256 (118–390) mg COD^{-1}g vss^{-1}. Grey water filtered through 1 × 1cm then by 1 × 1 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)				0.5 (0.3)	5.7 (1.9)	1.3 (0.5)								

(Continued on next page)

TABLE 4. Modified Filters: Biofilters Combined With Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions *(Continued)*

Treatments	BOD mg L^{-1}		COD mg L^{-1}		SS	Turbidity	TN	TP	TC CFU/	Volume	HRT	Hydraulic Load L	Organic Load Kg BOD m^{-3}				
	BOD_{tot}	BOD_{dis}	COD_{tot}	COD_{dis}	mg L^{-1}	NTU	mg L^{-1}		100 mL	m^3	hr	m^{-2} hr^{-1}	$days^{-1}$	Chemical	Energy	Cost	References
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 $days^{-1}$ and solids retention time 68 days.																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 × 0.034	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 × 0.034	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 2bar, cross-flow velocity 4 m s^{-1}. 1 days 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 mem- branes			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 330 L 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 Dalton, diameter 0.0155 m total surface area 0.34 m^2, permeate flux varies from 0.0588 m^3 m^{-2} hr^{-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) days																
Influent	69 (33)	36 (20)	211 (141)	108 (47)	92 (115)	65 (68)			3.4E5 (1.8E5)[b]								
Effluent	1 (2)	0.5 (0)	40 (16)	37 (14)	12 (8)	0.2 (0.1)			27 (56)[b]								

Note. Values in parentheses are standard deviations.

[a]standards values, reported by The Institute of Public Health, Ministry of Health and Welfare (1999).

[b]fecal coliform.

[c]reported as BOD; ND = not detectable.

TABLE 5. Modified Filters: Chemfilters Combined with Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions

Treatments	BOD mg L^{-1} BOD_{tot}	BOD mg L^{-1} BOD_{dis}	COD mg L^{-1}	SS mg L^{-1}	Turbidity NTU	TN mg L^{-1}	TP mg L^{-1}	TC CFU/ 100 mL	Volume m^3	HRT hr	Hydraulic Load L m^{-2} hr^{-1}	Chemical	Energy	Cost	References
M(T)CR	Two units: Photo-catalytic Oxidation Reactor (PCOR) followed by M(T)F unit														Rivero et al., 2006
GW-Source	Shower wastewater														
PCOR	8L stainless steel tank reactor with 25-W 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		252–324		15.6–18.7										
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		56–72		0.35–3.57			Meets WHO standards							
M(T)CR	9 L PCOR with four submerged 25 W, UV-C lamps and side stream air-lift M(T)F											TiO_2			Winward et al., 2008
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 $days^{-1}$.														
Effluent	3 (2)		43 (14)	ND (ND)	0.1 (0.0)			ND							
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							

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TABLE 5. Modified Filters: Chemfilters Combined with Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions *(Continued)*

Treatments	BOD mg L^{-1}	COD mg L^{-1} COD_{tot}	COD mg L^{-1} COD_{dis}	Turbidity NTU	SS mgL^{-1}	TN mg L^{-1}	NH_4-N mg L^{-1}	NO_3-N mg L^{-1}	TP mg L^{-1}	TC CFU/100 mL	FC CFU/100 mL	HRT hr	Hydraulic Load m^3 $days^{-1}$	Chemical	Energy	cost	References
								Attached biological treatment system									
Multistage 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	Multistory Building; Showers, bath and hand-washing basin.																
ST																	
Effluent	50–125	100–430				5–10			0.2–0.6	1E2-1E6	1E1-1E6						
RBCs																	
Effluent	<5																
Disinfection	Ultra Violet																
Effluent										2E-2-2E0	2E-2-1E-1						
Multistage 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[a]
GW-Source	Bath, shower and washbasin streams discharge from seven student apartments at Technion campus; accommodate 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 (30)	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	7 (10)	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 11time a days, 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	7 (10)	46 (19)	47(27)	1.9 (2.3)	16 (14.5)						9.7E3 ± 3.0E4						
Effluent	2 (2)	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						

Note. Values in parentheses represent standard deviations. ND = not detectable.

TABLE 6. Aerobic Biological Processes: Suspended and Attached Combined with Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions

Treatments	BOD mg L^{-1}	COD mg L^{-1} COD_{tot}	COD mg L^{-1} COD_{dis}	Turbidity NTU	SS mg L^{-1}	TN mg L^{-1}	NH_4-N mg L^{-1}	NO_3-N mg L^{-1}	TP mg L^{-1}	TC CFU/100 mL	FC CFU/100 mL	HRT hr	Hydraulic Load m^3 $days^{-1}$	Chemical	Energy	Cost	References
							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	113–633								1E4-1E6	1E4-1E3						
Effluent	<5									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 fuel ash media was used in both columns.																Hills et al., 2001
GW-Source	Artificial Grey water; mimicking grey water discharge from hand basins																
Influent	$60^{b}30^{c}$																
Effluent	$20–25^{b}$ $6–10^{c}$																
MultiStage 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 hr then anoxic phase 5 hr (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									

(Continued on next page)

TABLE 6. Aerobic Biological Processes: Suspended and Attached Combined with Others Units Treating Grey Water; Influent and Effluent Qualities, and Operation Conditions *(Continued)*

Treatments	BOD mg L^{-1}	COD mg L^{-1} COD_{tot}	COD_{dis}	Turbidity NTU	SS mg L^{-1}	TN (mg L^{-1})	NH4-N (mg L^{-1})	NO3-N (mg L^{-1})	P mg L^{-1}	FC CFU/ 100 mL	HRT hr	Organic Load Kg COD m^3 $days^{-1}$	Chemical	Energy	Cost	References
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 were 0.05 g VSS g^{-1} COD 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)							

Note. Values in parentheses are standard deviations-

[a]similar to data of Friedler et al. (2005, 2006); data adapted from Friedler et al. (2005).

[b]BOD (total).

[c]BOD (dissolved).

TABLE 7. Anaerobic and Aerobic Biological Processes: UASB and UASB Followed by SBR Treating Grey Water; Influent and Effluent Qualities and Operation Conditions

	COD mg L^{-1}				N mg L^{-1}			P mg L^{-1}									
Treatments	COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}	TN	NH4-N	Particulate-N	Total	Ortho	particulate	Temperature °C	SRT days	HRT hr	Chemical	Energy	Cost	References
UASB																	Elmitwalli and Otterpohl, 2007
GW-Source																	
Influent											23		12				
% 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}																Elmitwalli and Otterpohl, 2007
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)	84 (5)	52 (19)	51 (9)	30 (5)	–70 (44)	53 (11)	15 (4)	-6 (11)	53 (11)							
Influent	647 (137)	353 (131)	177 (81)	117 (40)	27 (5)	3.9 (1.0)	23 (4)	9.7 (0.7)	8.7 (1.2)	1.0 (0.5)	30	64–377	10				
% removal	52 (5)	79 (8)	29 (20)	30 (8)	22 (5)	15 (36)	31 (13)	17 (5)	15 (9)	43 (33)							
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 (12)	68 (17)	37 (18)	35 (21)		47.2 (53.6)		21 (7)	19 (1)	30 (4.0)							
UASB	3.6 and 5.0 L																Hernandez et al., 2007
GW-Source	Eco-village in Groningen, and DESAR project Sneek																

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TABLE 7. Anaerobic and Aerobic Biological Processes: UASB and UASB Followed by SBR Treating Grey Water; Influent and Effluent Qualities and Operation Conditions *(Continued)*

	COD mg L^{-1}				N mg L^{-1}			P mg L^{-1}									
Treatments	COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}	TN	NH4-N	Particulate-N	Total	Ortho	particulate	Temperature °C	SRT days	HRT hr	Chemical	Energy	Cost	References
Influent	200–2700	50–2100	135–402	135–722							20–30		12–24				
% removal	40	56	33	25													
	(40)																
UASB	3.6 L																Hernandez et al., 2008
Influent	827	385	246	196	29.9	0.8		8.5			35	393	12				
	(204)	(167)	(92)	(52)	(11.9)[a]	(0.6)		(9.2)									
% removal	47				3	-616		8									
	(2)				(57)	(642)		(36)									
grey																	
UASB+SBR	UASB followed by SBR																Hernandez et al., 2008
UASB	5.0 L																
Influent	830	427	212	234	53.6	1.2		7.7			35	97	7				
	(211)	(181)	(81)	(70)	(50.7)[a]	(1.3)		(5.6)									
% removal	36				−8	−856		5									
	(2)				(56)	(806)		(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				32	5	5.7					378	6				
	(190)				(14)[a]	(4)	(2.2)										
% removal	80				26	91	11										
	(9)				(27)	(6)	(31)										

Note. Values in parentheses are standard deviations.
[a]Total Kjeldahl. nitrogen (TKj).

bathing water standards (Table 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 Chemfilters section.

3.2.2 Biofilters

The tested biofilters can be classified as macro- and membrane biofilters. Macrobiofilters can be further classified into two subcategories: attached and suspended. Membrane subcategories are submerged and side-stream. Attached macrobiofilters 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, 2001). Anaerobic Biofilters (AnBF) and Bed-Submerged Biofilters (BSB) combine macrofiltration 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 Winward 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 Andersen et al. (2002) and an M(T)BR was tested by Friedler et al. (2006). All filters, except the anaerobic filter, are supplied with an external oxygen source.

The performance differences of micro- and macromembrane biofilters are presented in Table 4. The microsystems 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 the HRT (Table 4), which is similar to the conclusion for the physical filters and CW. Biofilters' nitrogen and phosphorous removal performance were not tested, except by Merz et al. (2007), who reported 63% and 19% removal, respectively (Table 4). Jefferson et al. (1999, 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°C and 0.4 g VSS L^{-1} compared with 20°C 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 4–20 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 (Friedler et al., 2006; Jefferson et al., 1999; 2000; Merz et al., 2007).

Biofilters show problems with cleaning, membrane fouling, and operational cost. Table 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 macrosolids such as hair and precipitated soaps. Jefferson et al. (2000, 2001) showed that submerged MBR pilot plants treating artificial, low suspended solids and grey water suffer from fouling and need frequent cleaning. In contrast, the side-stream MBR pilot plants tested by Andersen 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 subcritical 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 desludging and maintenance costs, are considered similar for the different systems. But the MBRs require 4 times the energy of the conventional systems. Fletcher and Judd (2007) justified their conclusions on a study of prefabricated units installed on site, treating medium-strength municipal wastewaters of 6–20 persons.

Pretreatment of biofilter's influent is advisable for MBR (Melin et al., 2006) and optional for other filters. Influent pretreatment reduces blockage, fouling problems, and cleaning frequency. It also produces better effluent qualities in some cases (i.e., BAF). Applied pretreatment techniques are a primary settling tank prior to biofilter (Imura et al., 1995), and screens (1 × 1 cm followed by 1 × 1 mm) prior to an MBR (Merz et al., 2007). Posttreatment of anaerobic filter effluent improves its qualities (Table 4), in terms of BOD, TN, TP, SS and pathogens (Imura et al., 1995).

3.2.3 Chemfilters

A Membrane Tubular Chemical Reactor (M[T]CR) combines a Photocatalytic 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; Winward 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 TC, which can comply with the most conservative standards (Table 5). However, Winward 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 cost, membrane fouling, recovery of TiO_2 (Li et al., 2003), and high effluents' COD fluctuation (Winward et al., 2008). Pretreatment of grey water with high SS is required before feeding the MCR. Influent pretreatment reduces the blockage, the fouling problems, and the cleaning frequency. Posttreatment 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.

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) recommended application of attached biomass and avoiding activated sludge systems. Both systems have been examined: Nolde (1999) and Friedler et al. (2006) examined attached systems, and Shin 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 pretreatment of grey waters.

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) were examined by Nolde (1999) and Friedler (Friedler et al., 2005; Friedler et al., 2006). Table 6 shows a two-stage aerobic FBR and multistage 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, 2006). However, both an FBR and an RBC are not successful in pathogen removal. Friedler (Friedler et al., 2005; Friedler et al., 2006) reported that an RBC removes BOD more efficiently than COD, which is attributed to the presence of nonbiodegradable or slowly biodegradable organic matter in the grey water.

Nolde (1999) and Friedler (Friedler et al., 2005; Friedler et al., 2006) 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 (Friedler et al., 2005; Friedler et al., 2006). 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 6) leads to a reduction in the weekly maintenance time to 0.2 hr and an energy requirement for treatment, disinfection, and service water of less than 1.5 KWh m^{-3} (Nolde, 1999). Costs for the treatment of produced primary sludge are, however, not taken into consideration.

3.3.2 Aerobic Suspended-Growth Processes

A Sequencing Batch Reactor (SBR) operated by Shine et al. (1998) produced an unstable effluent quality in terms of SS, but was stable regarding BOD. The BOD values were close to the effluent BOD of FBR, RBCs, and MBR (Table 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 cost in terms of energy consumption. Shin et al. (1998) optimized SBR operational modes to achieve the highest nitrogen removal through testing step-feeding, cyclic aeration, and conventional modes. The results (Table 6) illustrate that an SBR operated at an HRT of 12 hr produces an effluent in terms of nitrogen comparable to that of CW operated at an HRT of 14 days (Table 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, 0–5 mg NH_3-N L^{-1} (Akunna and Shepherd, 2001). Hernandez et al. (2007, 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 hr and 1 day. This contradicts Otterpohl et al. (1999), who recommended that an activated sludge process 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 (Abu Ghunmi et al., 2008; Butler et al., 1995). 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 was 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 posttreatment, an MF coped with the variation in the an SBR effluent and produced a stable effluent in terms of SS (Shin et al., 1998) (Tables 2 and 6).

3.3.3 Anaerobic Biological Processes

A UASB treating grey water produced a stable effluent quality and sludge (Table 7) compared with a primary settling tank (Table 4; Elmitwalli and Otterpohl, 2007). However, Hernandez et al. (2008) reported that a 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 stabilized. Table 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 (Abu-Ghunmi et al., 2008; Eriksson et al., 2002), operating the UASB at 30°C as recommended by Elmitwalli and Otterpohl (2007) cannot be achieved throughout the whole operation period even with a proper insulation of UASB and connection pipes and installing it in the building cellar. Nevertheless, anaerobic pretreatment of grey water is recommended, particularly when grey water concentrations are high. The reasons are (a) that 74% of grey water pollutants are anaerobically biodegradable (Elmitwalli and Otterpohl, 2007; Zeeman et al., 2008); (b) the probable deficiency in the macronutrients, nitrogen, and phosphorus, to sustain microorganism's growth in aerobic treatment (Abu Ghunmi et al., 2008); (c) that anaerobic treatment could produce less and stable sludge that is easily dewatered; (d) that no energy is required for aeration; and (e) that methane is produced that can be used as energy source (Lettinga et al., 1980). Thus, pretreating 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 posttreatment of domestic wastewater in a 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. On the other hand, the removal efficiency of the anaerobic processes could be improved by incorporating filtration (e.g., AnB; Imura et al., 1995) or physicochemical processes (e.g., activated carbon; Cui and Ren, 2005).

3.4 Toward Optimal Treatment Systems for Grey Water

3.4.1 Selection Factors

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

3.4.1.1 Characteristics and reuse requirements. Tables 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/100 mL, or FC or EC < 200 CFU/100 mL. 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 a 5–14 days HRT). The second process is the microfiltration or biofiltration with a relatively short HRT of 9.7 hr. The third process is physicochemical treatment, such as oxidation and coagulation. Grey water characteristics, 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 2–7 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 3) and an SBR (Table 6) can reduce the N or the P to less than 1 mg L^{-1}, which is far below the irrigation standards in Jordan.

3.4.1.2 The Technology Performance. Tables 2–7 show that the detailed design criteria and the operational conditions of most of the tested systems are not reported. The performance of the technologies is 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°C) 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 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 8 presents grey water treatment units and processes, the possibility of improving their performances considering grey water characteristics, and proposes HRT and SRT that can result in good effluent quality. Table 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-, chem.-, and biofiltration units produce effluents that meet the highest achievable quality concerning pathogens. Tables 2–7 give no information about sludge. Table 8 gives estimates of sludge type and handling, based on the applied technology, 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.

3.4.1.3 Energy demand, chemicals, and costs. Reported data for energy demand, chemical requirements, and costs are in general qualitative (Tables 2–7). Based on basic requirements to operate and maintain systems and on reported data, estimations of technology demand and costs are made (Table 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 ultramembrane filtration and biofiltration processes, followed by aerobic and physicochemical processes, and then coarse and sand filter, CW, and anaerobic processes. The chemicals are required as part of some treatments, such

TABLE 8. Grey Water Treatment Units and Processes: Compliancy with the Standards, Technology Demands, and Costs

	Effluent compliances with the reuse standards						Technology demands: Extra units				Cost		
Treatment type	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	*	*	**	**	*		***		*	*
HR-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	***	***	*	**	**	**	*		***	***
Grey													
Grey													

(Continued on next page)

TABLE 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 Operational Investment		
							Extra-Units						
	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	*	*	*	*	*	**	***		*	*

Note. Effluent compliances with the reuse standards in Table 2 are the most conservative standards reported in the literature. The BOD, COD, SS, and pathogen 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 = primary; S = secondary. 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.

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 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.

3.4.2 Optimizing the Treatment Systems for Grey Water

Table 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, physicochemical and chemfilter units should be avoided as main units for treating grey water. To minimize energy consumption and operation and maintenance costs, the use of membrane biofilters is highly questionable in the biological process options. Table 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 such as 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 and detailed costs information is required and for the CW option, it is necessary to consider the geographical location and climatic conditions as well. Table 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, minimize possibility of pathogen regrowth in the treated effluent, a disinfection step is recommended. The disinfection techniques, according to the conducted studies, are $<0.2\ \mu$m MF, ultrafilters, UV + TiO_2, or chlorine/bromine disinfection. Nevertheless, grey water standards should be revised in order to have multicategory standards for the different use options.

4 CONCLUSION

The tested grey water treatment processes are not optimized. Some of the effluents do not comply 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-stabilized sludge. To save on energy requirements, an anaerobic–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 anaerobic, aerobic, and disinfection units is recommended.

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NOMENCLATURE

AnBF	Anaerobic Biofilter
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
DHS	Down-flow Hanging Sponge
dis	dissolved
EB	Equalization Basin
E. coli	*Escherichia coli*
EPA	Environmental Protection Agency
FBR	Fluidized Bed Reactor
FC	Fecal Coliform
GRWRS	Green Roof Water Recycling System
GW	Grey Water
H	Horizontal
HPSF	Horizontal-Flow Planted Soil Filer
HRT	Hydraulic Retention Time
IVPSF	Intermittent Vertical-Flow Planted Soil Filter
IVSF	Intermittent Vertical-Flow Soil Filter
K	Potassium
LAS	Linear Alkyl Benzene Sulfonates
MBR	Membrane Biological Reactor
MF	Membrane Filter
M(F)BR	Membrane (Fibrous) Biological Reactor

M(F)CR	Membrane Chemical Reactor
MRT	Maximum Retention Time
MWCO	Molecular-Weight Cutoff
N	Nitrogen
NTU	Nepthelometric Turbidity Unit
P	Phosphorous
PCOR	Photocatalytic Oxidation Reactor
PFS	Polyferric Sulfate
PFST	Prefiltration 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
SSr	Subsurface
SSrF	Subsurface Filters
ST	Settling Tank
TC	Total Coliform
TF	Trickling Filter
TiO_2	Titanium Dioxide
Tkj	Total Kjeldahl nitrogen
TOC	Total Organic Carbon
TON	Threshold Odor Number
tot	Total
UF	Ultrafiltration
UV	Ultraviolet
V	Vertical
WHO	Word Health Organization

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