Performance and Economic Evaluation of a Semibatch Vertical-Flow Wetland for Onsite Residential Bathroom Graywater Treatment

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The performance and economic feasibility of residential bathroom graywater treatment using a semibatch vertical-flow wetland was evaluated over an eight-month field study in a single-family home in Los Angeles, Calif. Graywater treatment time of only 3 h produced an effluent with turbidity (~0.3 ntu) and 5-day biochemical oxygen demand (~3.1 mg/L) that met graywater reuse guidelines for nonpotable applications. After chlorine disinfection, total coliform count was below the mandated level of 2.2 most probable number per 100 mL for aboveground water reuse. Process modeling demonstrated that the treatment process followed first-order kinetics with optimal operation at a hydraulic retention time of 0.3 h. Treatment capacity of 2 m^3/day was feasible with the present system, which had a modest footprint area of 0.68 m^2. Economic analysis demonstrated a breakeven period of between two to six years depending on the daily treatment volume. Higher water costs favor graywater treatment, especially when power costs are modest or low.

Keywords: *field study, graywater economics, onsite treatment, residential graywater recycling, vertical-flow wetland*

Graywater, generally defined as domestic wastewater that does not originate from toilets or urinals (Friedler et al. 2004, Jefferson et al. 2004, Eriksson et al. 2002), is a potential resource to alleviate water shortage, increase water security, and foster long-term water sustainability in various regions around the world. It is a stable alternative water supply for nonpotable water use (e.g., irrigation, toilet flushing) in urban and rural environments (Al-Jayyousi 2002, Nolde 2000). A recent review of residential graywater reuse regulations in the United States indicates that reuse of untreated graywater is restricted to subsurface irrigation or ground disposal (Yu et al. 2013). Residential graywater reuse regulations vary throughout the world (Li et al. 2009) and even regionally within a given country (Yu et al. 2013) in terms of the permitted graywater storage, reuse applications, and required treatment levels. Clearly, to expand graywater reuse to the residential sector and allow it to be reused for aboveground water applications (Yu et al. 2013), treatment is necessary for safeguarding public health and meeting regulatory requirements. Moreover, low-cost and simple treatment approaches will be critical for widespread adoption of distributed graywater recycling.

To meet aboveground graywater reuse requirements (Table 1), biological treatment, filtration, and disinfection are necessary for removal of organics, suspended solids, and pathogens, respectively (Li et al. 2009). Complex biological treatment processes such as membrane bioreactors, along with a filtration process train (NSW Health Department 2010), have been developed for nonpotable aboveground graywater reuse onsite at residential homes. However, high capital and operations and maintenance (O&M) costs (EMRC 2011, Farrelly & Brown 2011) have hindered homeowners' adoption of such commercial treatment systems (Blundell 2010, Nolde 2005). In contrast, constructed wetlands have long been considered a suitable low-cost alternative technology for graywater treatment (Zhang et al. 2014).

Previous studies have evaluated subsurface horizontal flow (Dallas & Ho 2005, Dallas et al. 2004, Niemczynowicz & Fittschen 1997), vertical flow (VF) (Katukiza et al. 2014, Gross et al. 2007a, Shrestha et al. 2001), and hybrid constructed wetlands (Paulo et al. 2013, Schonborn et al. 1997) for graywater treatment (Table 2). Results from these studies indicate that, to produce treated graywater effluent with <10 mg/L of biochemical oxygen demand (BOD) (Table 1) to comply with the water reuse water quality standard for aboveground nonpotable beneficial water applications as presented in Table 1, continuous flow, single-pass wetlands would require long hydraulic retention time (four to eight days). Merely reducing the treatment system area and hydraulic retention time in single-pass flow systems produced effluent of poor quality (Katukiza et al. 2014). The previously proposed continuous flow, single-pass wetland systems were also of low daily treatment volume-to-area ratio (~0.04–0.08 m^3/m^2/day; Table 2). Such a performance level, for example, would require a wetland area of 4.3–8.5 m^2 to treat ~340 L/day of bathroom and clothes-washing graywater generated in a single-family home in California (DeOreo 2011). Such space requirements and associated construction costs (Kadlec & Wallace 2009) would make conventional wetlands impractical for graywater treatment in residential homes in most urban areas. Furthermore, long hydraulic retention time of four to eight days would require the provision of significant onsite storage capacity for collected raw graywater, which could create an odor nuisance (Dixon et al. 2000). In various regions, graywater storage is limited in terms of both storage time and capacity (Yu et al. 2013). Clearly, short hydraulic retention times would be desirable to reduce raw graywater storage needs; thus, the preference is for an alternative compact and low-cost wetland design that is suitable for residential use.

Operation of wetlands in semibatch (SB) mode with graywater recirculation is an approach that reduces the required hydraulic retention time and thus is an attractive alternative to the single-pass conventional constructed wetlands. Previous work has reported on an SB recirculating VF wetland for treatment of graywater that achieved five-day BOD (BOD_5) and fecal coliform removal at essentially 100% and 99%, respectively. This performance level was achieved using a system with a relatively small footprint (~1 m^2) and treatment time of 8 to 12 h (Gross et al. 2007a), and the system's performance was shown to be comparable to the performance of much larger horizontal flow wetland treatment (Table 2). This VF wetland design used stratified heterogenous layers of silica-based soil and a small, randomly packed bed of spherical plastic media and rocks (Gross et al. 2007b). Its operational mode involved recirculating water dripped from the wetland into a holding tank below the wetland. The silica-based soil layer served as a filtration zone, and the plastic media provided surface area for biofilm growth. The "raindrop" effect created by water flow from the wetland to the receiving reservoir below promoted aerobic conditions. It was reported that the treatment capacity was 0.45 m^3/m^2/day for untreated graywater containing 158 ± 30 mg/L total suspended solids (TSS) and 839 ± 47 mg/L chemical oxygen demand (COD). This system produced, for a treatment time of 12 h, effluent that had a low TSS range of 0 to 6 mg/L and a low BOD_5 range of 0 to 1.5 mg/L (Gross et al. 2007a). Despite such a low BOD_5, the effluent COD range of 60 to 220 mg/L was relatively high for the BOD_5 concentrations achieved compared with the BOD and COD data reported for other wetlands that achieved similar BOD removals (Table 2). In

TABLE 1 Water quality criteria for aboveground nonpotable graywater reuse in selected countries

Standards	Type of Reuse	Treatment Level Equivalent	Water Quality Criteria
United Kingdom (Environment Agency 2011)	Sprinkler, car washing, toilet flushing, garden watering, pressure washing, washing machine use		<10 ntu turbidity pH 5–9.5 <2 mg/L residential chlorine <0.5 mg/L residential chlorine for nonspray garden watering 0.0 mg/L residual bromine for all spray application and nonspray garden watering
California (CBSC 2010)	Aboveground nonpotable reuse	Disinfected tertiary	Turbidity: 2 ntu (average), 5 ntu (maximum) Total coliform: 2.2 MPN/100 mL (average), 23 MPN/100 mL (maximum in 30 days)
Wisconsin (WDOC 2015)	Surface irrigation except food crops, clothes and vehicle washing, air conditioning, soil compaction, dust control, washing aggregate, making concrete	Disinfected tertiary	pH: 6–9 BOD_5: ≤10 mg/L TSS: ≤5 mg/L Free chlorine residual: 1.0–10 mg/L
	Toilet and urinal flushing	Disinfected primary with filtration	pH: 6–9 BOD_5: ≤200 mg/L TSS: ≤5 mg/L Free chlorine residual: 0.1–4.0 mg/L
NSF/ANSI 350R (NSF 2011)	Restricted indoor and unrestricted outdoor water reuse	Residential capacity ≤1,500 gpd	pH: 6–9 $CBOD_5$: ≤10 mg/L of (average), 25 mg/L (maximum) TSS: ≤10 mg/L (average), 30 mg/L (maximum) Turbidity: 5 ntu (average), 10 ntu (maximum) *Escherichia coli*: 14 MPN/100 mL (mean), 240 MPN/100 mL (maximum) Storage vessel disinfection: ≥0.5 to ≤25 mg/L
New South Wales, Australia (NSW Health Department 2011)	Toilet flushing, cold water supply to washing machines, garden irrigation with local approval	Disinfected secondary	BOD_5: <20 mg/L TSS: <20 mg/L Fecal coliforms: <10 cfu/100 mL
Western Australia, Australia (Government of Western Australia, Department of Health 2010)	Toilet flushing, cold water supply to washing machines, irrigation	Disinfected tertiary	BOD_5: <10 mg/L TSS: <10 mg/L *E. coli*: <1 MPN/100 mL Coliphages: <1 pfu/100 mL Clostridia: <1 cfu/100 mL
Victoria, Australia (EPA Victoria 2013)	Toilet flushing, cold water supply to washing machines, surface irrigation, subsurface irrigation	Disinfected tertiary	BOD_5: <10 mg/L TSS: <10 mg/L Fecal coliforms: <10 cfu/100 mL

BOD_5—five-day biochemical oxygen demand, $CBOD_5$—carbonaceous BOD_5, MPN—most probable number, TSS—total suspended solids

TABLE 2 Various wetland designs and their reported performance for treatment of graywater

Country (citation)	Graywater Sources	Wetland Type	Media	Recirculation	Mode of Operation	Treatment Capacity *m^3/day*	Wetland area *m^2*	Hydraulic Loading Rate *m^3/m^2/day*	HRT *days*	Turbidity *ntu (% removal)*	Organics *mg/L (% removal)*	TSS *mg/L (% removal)*	Total Coliform % *removed per 100 mL*
Sweden (Niemczynowicz & Fittschen 1997)	Sinks, kitchen, shower, washing machines	HF	Soil, NS	No	CF	40	600	0.07	4	—	BOD_5: <5 (95.8%) $CBOD_5$: 46 (87%)	—	—
Sweden (Schonborn et al. 1997)	Sinks, kitchen, shower, washing machines, urinal	VF-HF	Fine grained sand, iron armament pieces, sandy and stony loam	No	CF	40	38	0.04	—	—	BOD_5: 5 (96%) $CBOD_5$: 27 (91%)	—	—
Nepal (Shreshtha et al. 2001)	Bathroom, shower, washing machines, kitchen sinks	VF	Coarse sand	No	CF	0.5	6	0.08	—	—	BOD_5: 5.2 (97%)	2.6 (97%)	—
Costa Rica (Dallas & Ho 2005)	Mixed, NS	HF	PET	No	Laboratory, batch	0.01	0.38	0.03	4.2	—	BOD_5: 13 (92%)	—	Fecal coliform 99.99% (2.1×10^3 cfu)
	Mixed, NS	HF	Crushed rocks	No	Laboratory, batch	0.01	0.38	0.03	2.5	—	BOD_5: 18 (88%)	—	Fecal coliform 99% (2.6×10^5 cfu)
Costa Rica (Dallas et al. 2004)	Mixed, NS	HF	Crushed rocks	No	CF	2.5	30	0.08	7.90	2 (98%)	BOD_5: 3 (99%)	—	Fecal coliform: 99.999% (16 cfu)
Israel (Sklarz et al. 2009, Gross et al. 2007a)	Mixed, NS	VF	Peat, soil and randomly packed plastic media or tuff	Yes	Semibatch	0.45	0.9	0.5	0.3–0.5	6 (87%)	BOD_5: 0.7 (100%) $CBOD_5$: 157 (81%)	3 (98%)	Fecal coliform 99% (2×10^5 cfu)
Uganda (Katukiza et al. 2014)	Bathroom, shower, laundry, kitchen sinks	VF	Crushed lava rock	No	Batch, single pass	0.22	0.2	1.1	0.06/10 L-batch	–	BOD_5: 390 (72%) $CBOD_5$: 198 (90%) TOC: 277 (69%)	80 (93%)	99.9% (3.38×10^4 cfu)
Brazil (Paulo et al. 2013)	Laundry, bathroom	HF-VF	Coarse and fine gravel	No	CF	0.7	7.8	0.09	0.8	13 (95%)	BOD_5: 22 (95%)	9.6 (92%)	—

BOD_5—five-day biochemical oxygen demand, $CBOD_5$—carbonaceous BOD_5, CF—continuous flow, HF—horizontal flow, HRT—hydraulic retention time, NS—not specified, PET—polyethylene terephthalate, TOC—total organic carbon, VF—vertical flow

Values in parentheses are reported average effluent concentration values. Dashes indicate the data were not reported in the reference literature.

this system, particles that leached from the silica-based soil resulted in an average effluent turbidity of 6 ntu with a range of 2 to 12 ntu (Sklarz et al. 2009). Such a turbidity range exceeds the most stringent standards for aboveground nonpotable water uses (Table 1) and may also decrease disinfection effectiveness (Metcalf & Eddy et al. 2004). Preventing fine soil leaching and increasing hydraulic loading rate are necessary to address the current limitations of recirculating VF wetland (VFW) designs. A possible solution would be to replace the high-density silica-based soil with a lower-density nonsilty soil substitute. For example, recent studies have proposed that organic materials as low-density soil substitutes, such as palm tree mulch (Herrera-Melián et al. 2014) and tree bark (Dalahmeh et al. 2014), may be be suitable for use in VFWs. It was reported, however, that palm tree mulch was not as effective for organics and TSS removal compared with sand filters, achieving only an average of 53% BOD, 38% COD, and 70% TSS removals compared with 85% BOD, 62% COD, and 95% TSS removals achieved using silica-based soil (Herrera-Melián et al. 2014).

The design of a VFW (whether operated in single-pass or multipass mode) requires a specific system performance model to tailor the system to the target treatment level for the expected range of raw graywater quality. A first-order plug-flow reactor developed for conventional wetlands (Kadlec 2000), but modified for recirculation, was used to calculate needed wetland area. This approach provided a correlation between the required recirculation flow rate and needed wetland area, but without evaluating the effectiveness of organic loading on the treatment time. Another first-order kinetics model for a VFW was proposed (Sklarz et al. 2010), based on the assumption of a completely mixed batch reactor, for determining the needed wetland volume to achieve TSS removal for a given treatment time. It was concluded that the treatment efficiency (with a wetland area of 0.9 m^2 and a treatment capacity of 450 L/day) was independent of a recirculation flow rate greater than a recirculation flow rate of 3 m^3/h. It was suggested that performance improvement could be attained by increasing the wetland bed volumes and decreasing the treatment volume. Although these studies have provided useful insights regarding the operation of VFWs, there remains a critical need for systematic evaluation and optimization of the residential VFW approach to graywater treatment with respect to key operational and design parameters and their interplay, including treatment time, hydraulic retention time, and wetland area and depth.

The present work focused on demonstrating the technical and economic feasibility of a VFW system for residential graywater treatment that overcomes the stated shortcomings of previous approaches in the following ways: (1) using coconut coir dust as a soil substitute of high hydraulic conductivity for the wetland, (2) replacing the conventional large gravels with crossflow plastic

TABLE 3 Average influent and effluent water quality over the eight-month study period

	Influent	Effluent
pH (6)	7.35 ± 0.07	6.90 ± 0.09
TDS (6)—*mg/L*	261.00 ± 8.98	257.76 ± 14.11
DO (5)—*mg/L*	1.33 ± 0.45	7.20 ± 1.54
Temperature (6)—*°C*	18.48 ± 1.58	16.14 ± 2.55
Turbidity (20)—*ntu*	20.59 ± 6.71	0.30 ± 0.27
TSS (5)—*mg/L*	40.02 ± 20.60	0.5 ± 0.12
TOC (21)—*mg/L*	33.62 ± 4.63	7.34 ± 1.14
bDOC (21)—*mg/L*	12.89 ± 3.89	0.41 ± 0.37
TOC minus DOC (21)—*mg/L*	14.93 ± 2.92	0.59 ± 0.37
COD (3)—*mg/L*	148.03 ± 20.25	16.37 ± 2.92
BOD_5 (3)—*mg/L*	49 ± 6.56	3.1 ± 1.18
Total coliforms, before chlorine disinfection (4)—*MPN/100 mL*	1.35×10^8 (9.65×10^7–1.84×10^8)	6.49×10^5 (4.25×10^5 to 9.42×10^5)
Total coliforms, after chlorine disinfection (1)—*MPN/100 mL*	—	<1 (BDL)
Fecal coliform (2)—*MPN/100 mL*	<1 (BDL)	<1 (BDL)
Septic odor[a] (40)	Noticeable	None

BDL—below detected limits, bDOC—biodegradable dissolved organic carbon, BOD_5—five-day biochemical oxygen demand, COD—chemical oxygen demand, DO—dissolved oxygen, DOC—dissolved organic carbon, MPN—most probable number, TDS—total dissolved solids, TOC—total organic carbon, TSS—total suspended solids

[a] Septic odor was noted after raw graywater had been stored in the storage tank for ~24 h.
Numbers in parentheses after constituent are the numbers of observations. Dashes indicate that the data were not reported in the referenced literature.

media consisting of large flow channels and high surface area that allow for both biofilm growth film and aeration, and (3) ensuring adequate distribution of graywater onto the wetland. The VFW was evaluated during an eight-month period in a single-family home for treatment of graywater from bathroom sinks, showers, and baths. A first-order kinetics model and collected field data were then used to evaluate the relationships between operational parameters and treatment performance, thereby providing the basis for scale-up. Furthermore, the economic feasibility of onsite residential graywater treatment was evaluated on the basis of capital and O&M costs derived from the field study. The overall goals were to demonstrate that a suitably designed SB-VFW can (1) produce treated effluent that meets stringent water quality requirements for aboveground graywater reuse, (2) have high treatment capacity per unit area of >1 m^3/m^2/day, and (3) be economically feasible in different parts of the world even in the absense of financial subsidies.

MATERIALS AND METHODS

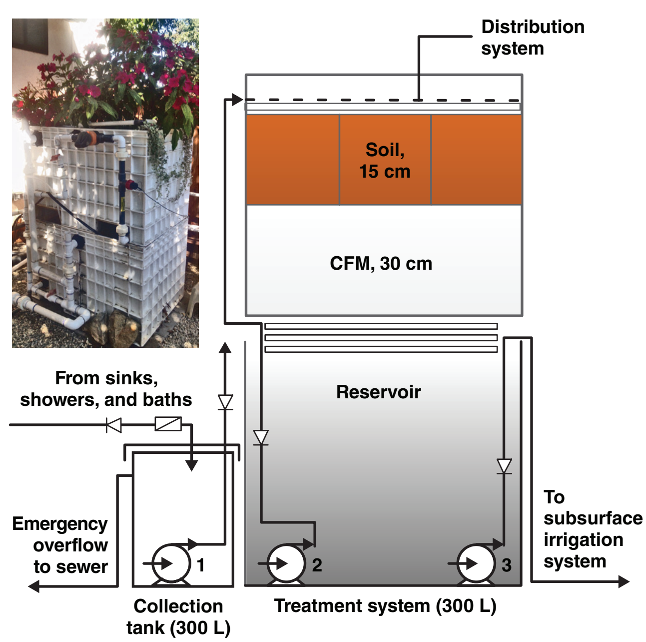
Graywater source, instrumentation, and analytical methods. Onsite treatment of bathroom graywater generated from bathroom sinks, showers, and baths using an SB-VFW was investigated during an eight-month period in a single-family home. In the present arrangement (Figure 1), graywater was diverted to a belowground collection tank by gravity. Subsequently, graywater was pumped from the collection tank into the treatment system using a sump pump.[1] Ten 300-mL grab samples were collected directly from the midwater column of the collection tank (influent) and the reservoir (effluent) of the pilot system using two autosamplers[2] during each batch of the experiment. These 10 samples, including one influent sample collected just before pumping the graywater into the treatment system and nine grab effluent samples from the treatment unit, were collected for evaluation over a 24-h period at 0, 0.15, 0.5, 0.75, 1, 2, 3, 8, and 24 h. Collected samples were kept in a cooler (<4°C) before they were transported back to the laboratory for water quality analysis.

Total organic carbon (TOC) and dissolved organic carbon (DOC) were determined via method 5310C (*Standard Methods* 2012) with a TOC analyzer.[3] Samples for TOC and DOC analysis were processed within 24 h of arriving in the laboratory. Samples for DOC analysis were prepared by filtering the water samples through 0.8-µm filters (mixed cellulose ester with modified acrylic housing).[4] TOC and DOC were monitored during the entire study period, but, during the startup period (i.e., the first three months), TOC was measured less frequently. Graywater was treated every other day because of the lower than expected bathroom graywater production. There were 21 treatment batch runs evaluated after the startup period with eight samples per treatment batch, making a total of 168 samples tested from the field study. In addition to these samples, collected influent and effluent samples were analyzed for COD (Hach 2009) and BOD (method 5210B [*Standard Methods* 2012]) to establish the ratios between TOC, DOC, COD, and BOD. BOD tests were conducted by a California state–certified laboratory.[5] The TOC-to-BOD and -COD ratios remained relatively constant throughout the study; this was expected because the wastewater conditions and house occupancy did not change. The house tenants were part of the study and agreed not to change conditions without notifying the authors. Average BOD_5 and COD values were calculated on the basis of the correlation established between the BOD_5-TOC and COD-TOC ratios.

Turbidity (USEPA 1993), measured with a portable turbidity meter,[6] was the primary monitoring parameter to assess reduction in suspended particles and colloidal matter. Turbidity measurements were taken immediately after the samples arrived in the laboratory. Other monitored inorganic water quality parameters included pH, conductivity,[7] temperature, total inorganic carbon[3] (method 5310C [*Standard Methods* 2012]), TSS, volatile suspended solids[8] (method 2540D), and dissolved oxygen.[9] Finally, total and fecal coliforms were measured[10] (method 9223B [*Standard Methods* 2012]) in the influent and effluent samples as well as after chlorine disinfection. Samples for total and fecal coliforms were processed immediately after the samples arrived at the laboratory. The effectiveness of chlorination was evaluated by adding hypochlorous acid (HOCl) to 200-mL effluent samples to attain an HOCl concentration of 4 mg/L in the sample that, after a period of 10 min, was quenched with sodium metabisulfite. This HOCl concentration was sufficient to meet the disinfection goal.

Graywater treatment system. Residential graywater treatment in a single-family home was accomplished with a modular SB-VFW (Figure 1). The upper section of the system (1.1 m [L] × 0.7 m [W] × 0.7 m [H]) consisted of a wetland (top) and

FIGURE 1 Schematic and photograph of the graywater treatment system



CFM—cross-flow media

The upper wetland unit consists of a soil layer and CFM. Graywater intake (pump 1), recirculation (pump 2), and discharge (pump 3) are controlled by three submerged pumps.

a reservoir (bottom). The upper unit was composed of a layer of coconut coir dust (the "soil" layer) sitting on top of a layer of cross-flow media (CFM). The soil layer was 15 cm thick and was compartmentalized using fabric bags[11] (33 cm [W] × 33 cm [L] × 20 cm [D]). A variety of ordinary garden plants along with wetland plant species (*Carex spissa* and *Phyla nodiflora*) were planted in the wetland. Household garden plant species planted in the wetland were *Aeonium purpureum* and *Crassula ovate, Equisetum hyemale, Nasturtium, Narcissus, Impatiens, Anigozanthos,* and *Colocasia.* The viability of the plants was visually assessed on the basis of their overall appearance (e.g., leaf greenness, ability to produce flowers for flowering plants). The CFM,[12] which provided additional surface for biofilm growth and structural support for the upper soil bed, had a specific surface area of 226 m^2/m^3. The CFM layer was 30 cm deep. The open channel configuration of the CFM enabled significant aeration while avoiding channel clogging (Parker & Merrill 1984). The raindrop effect created by water droplets (from the wetland) impinging on the graywater-holding reservoir below the CFM also provided additional aeration (Banks et al. 1984).

The SB-VFW was operated in an SB mode to accommodate the intermittent generation of graywater. A collection tank with a 300-L storage capacity was used to collect and store untreated graywater between treatment cycles. Raw graywater flowed into the collection tank (from the residential home) by gravity. A 200-µm nylon fabric screen filter was installed at the inlet of the collection tank for removal of hair and other stringy matter. Once filled, graywater was pumped using a sump pump[1] from the collection tank to the wetland through a distribution system. The treatment cycle began by pumping the contents of the collection tank into the distribution system above the wetland. The effluent of the treatment system was then collected in the reservoir and recirculated to the wetland until the treatment cycle was completed (when the collection tank refilled, approximately 24 to 48 h). A second sump pump[1] in the reservoir emptied the treated graywater reservoir to the irrigation field. Automated operation of the graywater treatment with the three pumps (Figure 1) was facilitated using an irrigation controller.

The application of graywater onto the wetland surface, during both delivery of the initial charge and recirculation, was accomplished using a flow distribution system of enclosed conduits (installed just above the soil) with bottom discharge holes. Uniformity of flow distribution was verified by determining the local permeation flow rate (through the wetland soil and CFM) at different locations beneath the CFM. The recirculation flow rate was measured by an inline paddlewheel flowmeter[13] connected to a powered transmitter/totalizer.[14]

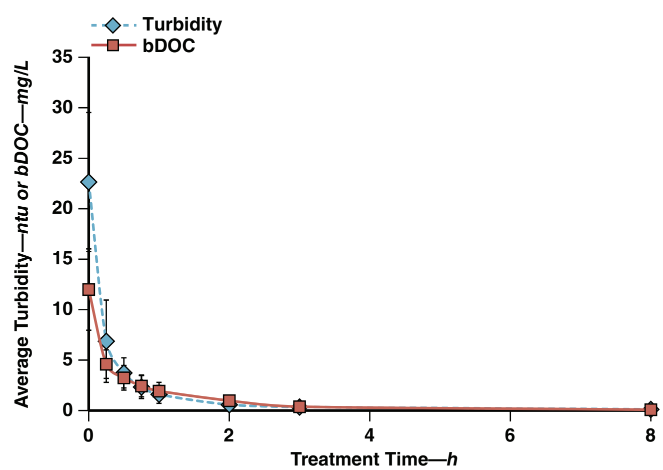
RESULTS AND DISCUSSIONS

Influent and effluent water quality. Untreated bathroom graywater contained organics, TSS, and turbidity at levels that did not meet water quality requirements for aboveground nonpotable graywater reuse presented in Table 1. The SB-VFW was effective in reducing the concentration of physical and chemical contaminants to levels that complied with aboveground graywater reuse standards (Table 3). Significant removal of turbidity, which averaged 0.3 ntu for the treated effluent (relative to ~21 ntu for the raw graywater), was achieved, which is well below the turbidity requirement of <2 ntu for aboveground graywater reuse in California. The effluent also had a low BOD_5 of 3.1 mg/L and TSS of 0.5 mg/L (representing a reduction of ~94 and ~99% relative to the influent graywater), which also meets organic and physical contaminant requirements for aboveground graywater reuse (Table 1). The treatment performance did not change during the study period. Untreated graywater pH was 7.35 (relative to ~6.4 for tap water) and decreased to 6.9 after treatment. Raw graywater total dissolved solids were 261 mg/L (~18% higher than tap water) and did not change significantly upon treatment. The treatment system was able to produce effluent water quality as described in only about 3 h, with water quality showing marginal improvement thereafter, as noted in Figure 2.

The wetland section of the treatment system performed as a packed-bed media filter that captured suspended solids and reduced turbidity. Water recirculation increased the residence time in the soil layer and thus enhanced turbidity removal. Greater than 60% turbidity removal was achieved within the first 15 min of treatment and >90% removal within the first hour (Figure 2). Moreover, effluent turbidity treated for about 2 h was well below 2 ntu. System performance over the eight-month field study demonstrated performance robustness with respect to turbidity removal without the problem of leaching of fine silts or clay associated with the use of silica-type soils (Sklarz et al. 2009).

The SB-VFW was effective in removing TOC (with an average organic loading rate of 0.10 kg of TOC/m^3/day) and especially the degradable TOC fraction. Over the course of the study, TOC

FIGURE 2 Average turbidity and bDOC removal with respect to treatment time shows that the system was effective in removing both turbidity and bDOC



bDOC—biodegradable dissolved organic carbon

Effluent turbidity and bDOC after 2 h of recirculation were consistently low despite fluctuations in influent concentrations. Effluent turbidity and bDOC concentrations after 3 and 8 h of treatment remained throughout the study period. Error bars represent 1 standard deviation of a range of results of multiple runs (*n* = 16). All points have error bars, but some are not visible because they are smaller than the symbol.

was typically reduced by ~80% (from 33.6 ± 4.6 to 7.3 ± 1.1 mg/L) within just 3 h of treatment. More frequent grab-sampling during the first treatment hour revealed that ~62% of the biodegradable dissolved organic carbon (bDOC) was removed within the first 15 min of treatment and that 97% bDOC removal was achieved within 3 h (Figure 2). The apparent rapid removal of organics and turbidity by SB-VFW within the first 30 min of treatment is consistent with observations reported by other studies (Sklarz et al. 2010, Gross et al. 2007a). The level of bDOC removal was consistent with the low-effluent BOD_5, which was reduced by 93 ± 3% after 3 h of treatment (Figure 2). Plant uptake of organic material was not expected (Brix & Schierup 1990).

VFWs typically provide higher dissolved oxygen (DO) levels in the soil matrix that promotes efficient organic removal and nitrification (Sun et al. 2005, Brix 1993). High DO levels were attributed to the use of the CFM that enhanced aeration (Figure 1). Operation of the treatment system without the CFM attained a lower effluent DO level of ~3.8 ± 0.7 mg/L. This result indicated that the CFM provided improved aeration that resulted in high-effluent DO (~7.2 ± 1.5 mg/L; Table 3) without the need for mechanical mixing or direct aeration. These effluent DO results are comparable to those reported by Ammari et al. (2014) of 5.2 to 8.7 mg/L. The CFM did not show a visible biofilm, but a thin, slimy biofilm was present and observable to the touch. The microbial community in the wetland soil is expected to be diverse (Zhang et al. 2010, Truu et al. 2009, Vacca et al. 2005). The influent ammonia and nitrate averaged 1.8 mg N/L and 0.2 mg N/L, respectively. Effluent ammonia averaged 0.2 mg N/L, whereas nitrate increased to 5.6 mg N/L. The wetland effluent was always high in DO, so denitrification was not expected as confirmed by the results. The detergents used in the household were phosphate free and the raw graywater phosphorus as phosphate was only 2.3 mg/L.

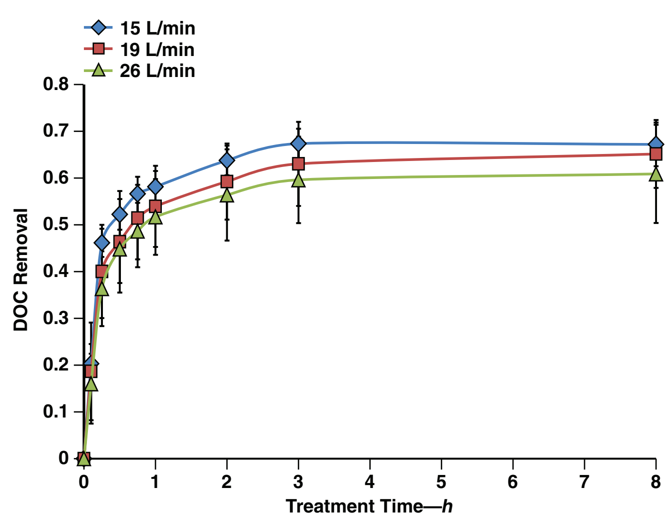
The high hydraulic conductivity of the soil layer (0.09 ± 0.04 cm/s) made it possible to operate the system at high recirculation flow rates, with the highest recirculation rate of 36 L/min (or a hydraulic dosing rate of 53 L/m^2/min) permitted by the recirculation pump, and provided a short complete volume turnover every ~7 min. However, introduction of vegetation into the wetland reduced the soil-water hydraulic conductivity, which limited the maximum achievable recirculation flow rate (15 to 26 L/min or a hydraulic dosing rate of 22 to 38 L/m^2/min). This recirculation flow is still significantly higher than reported in previous work, with a hydraulic dosing rate of 6.8 L/m^2/min, using silica-type soil (Gross et al. 2007a).

The effect of recirculation flow rate on dissolved organics removal is illustrated in Figure 3 for the flow rate range of 15 to 26 L/min, with the upper and lower limits being those of the pump's feasible operation. There was no measurable difference in DOC removal over this range of recirculation flow rates; hence, for the present system, the lower recirculation flow rate of 15 L/min was adopted as suitable for graywater treatment. However, DOC removal did decrease by about 10%, with increasing recirculation rates from the previously stated low to high limits. This performance result may have been caused by scouring or sloughing of organics from the soil (Peyton 1996). One should expect that a higher recirculation flow rate would increase the water shear velocity through the wetland unit, a technique commonly used in controlling biofilm thickness in trickling filters (Metcalf & Eddy et al. 2004, Peyton 1996). This behavior suggests that lower recirculation flow rates are preferable with the added benefit that ultralow power pumps (e.g., solar fountain pumps) can be used.

The origin of turbidity was investigated by comparing the particulate fraction of the organic carbon obtained from subtracting measured DOC from TOC with the overall measured turbidity. The linear correlation between turbidity and particulate organic carbon (Figure 4) suggests that turbidity was primarily of organic origin, an assertion that was also supported by the high fraction of volatile suspended solids (~99%) of the TSS. This observation was consistent with the determination that the personal care products used by the residents contained 128 ingredients (as reported by the manufacturers), of which 95% were organic compounds with fragrances, glycerin, and cocamidopropyl betaine being the most common ingredients. Given this information, efficient turbidity removal also resulted in TOC removal. The recirculation vertical flow regime of the SB-VFW enhanced removal of particulates and turbidity through filtration (Brix 1993).

The SB-VFW achieved an ~2-log reduction of total coliforms (i.e., from average influent coliform count 1.35×10^8 most probable number (MPN)/100 mL to the average of 6.49×10^5 MPN/100 mL in the treated effluent). Fecal coliforms were not detected in the influent or effluent samples. Moreover, upon chlorine disinfection to achieve residual chlorine concentration of 4 mg/L, total

FIGURE 3 Dependence of DOC on treatment time for different recirculation flow rates



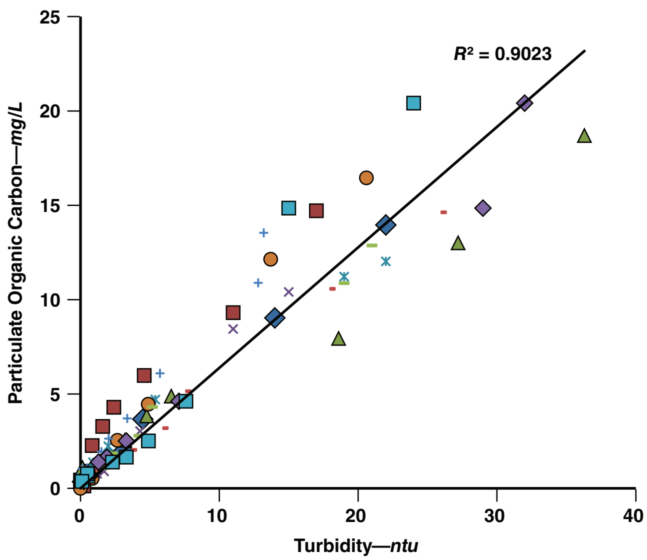
DOC—dissolved organic carbon

The results indicate that 3 h of treatment was sufficient with marginal gain for a longer treatment period. Lower recirculation flow rates resulted in higher DOC removal. Error bars represent 1 standard deviation of a range of results of multiple runs.

coliforms were reduced to below the detection limit of 1 MPN/100 mL. This indicates that simple chlorination of the treated effluent should be sufficient for meeting the stringent California total coliform count limit of 2.2 MPN/100 mL for aboveground nonpotable water reuse.

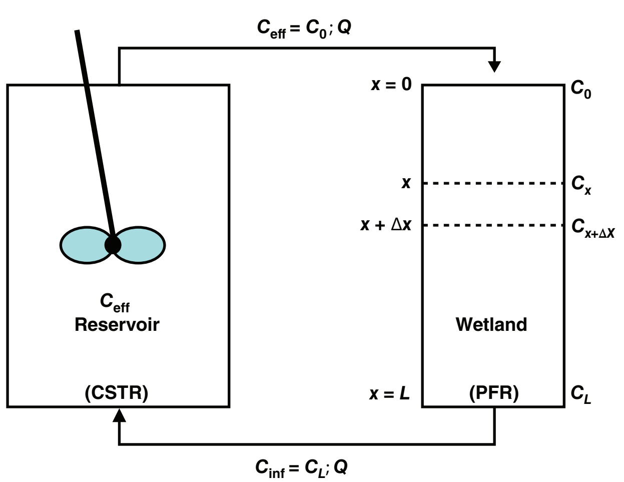
Finally, it is interesting to note that over the course of the eight-month study period, the ordinary household plants *Aeonium purpureum*, *Crassula ovate*, *Equisetum hyemale*, *Nasturtium*, *Narcissus*, *Impatiens*, *Anigozanthos*, and *Colocasia* thrived and appeared to be most adaptable in the wetland environment. Worms, which are known to reduce sludge accumulation (Moshiri 1993), were also found in the wetland during the study period.

FIGURE 4 Linear relationship of particulate organic carbon and turbidity



Symbols represent different sampling events.

FIGURE 5 Schematic of the batch graywater treatment process



C_{eff}—concentration in the water outflow after water has passed through the wetland, C_{inf}—inflow concentration in the graywater input to the wetland system, CSTR—continuously stirred tank reactor, *L*—wetland depth, PFR—plug flow reactor, *Q*—recirculation flow rate

Evaluation of operational and design parameters. The system had a volumetric treatment capacity of 0.3 m^3 per treatment cycle (~3-h treatment time per batch and short system charge and emptying times of ~15 to 20 min), thereby enabling total daily graywater treatment of ~2.1 m^3 in seven treatment cycles. This translates to a hydraulic loading rate of 3.1 $m^3/m^2/day$ for the system area of 0.68 m^2. In practical terms, the present system design is sufficient for treating graywater from one up to about six single-family homes (DeOreo & Hayden 2008). Scale-up to multi-family dwellings should be feasible given the capacity and modularity of the SB-VFW design.

To further evaluate the potential scale-up of the SB-VFW approach to graywater treatment, a simple mathematical model was developed for the idealized process, as depicted in Figure 5. In the model formulation, the SB-VFW was approximated as a plug-flow reactor without dispersion, in which the reservoir below the wetland was approximated as a completely mixed reactor. For this system, bDOC level was modeled subject to the following assumptions: (1) bDOC removal can be described by a first-order kinetics model, (2) biomass density was in steady-state over the treatment period, (3) water evaporation was negligible over the course of a single-batch treatment cycle, (4) water distribution over the wetland was approximately uniform (verified via a series of hydraulic studies, and (5) biofilm sloughing was negligible at the low recirculation flow rate used. Accordingly, the bDOC C_x (mg/L) change, along the length (L) of the wetland, can be expressed by the following differential mass balance:

$$\frac{dC_x}{dx} = -\mathrm{k}\,\frac{A\epsilon}{Q}\,C_x \qquad (1)$$

where Q is the graywater recirculation flow rate (m^3/h), A is the wetland surface area (m^2), ϵ is the wetland porosity, k is the removal rate constant (h^{-1}), and x is the distance from the wetland surface downward. This approach is similar to that used in trickling filter models (Roesler & Smith 1969, Schulze 1960, Velz 1948). The solution of Eq 1, subject to the upper boundary conditions of concentration (C) = C_0 at $x = 0$, and a lower boundary condition of $C = C_L$ at $x = L$, is as follows:

$$C_L = C_0 \exp\left(-\mathrm{k}\,\frac{A\epsilon L}{Q}\right) \qquad (2)$$

The effluent from (C_L) and inflow to (C_0) in the wetland are equal to the influent bDOC (C_{inf}) from and effluent bDOC (C_{eff}) to the reservoir below the wetland (i.e., $C_L = C_{inf}$ and $C_{eff} = C_0$), respectively. Assuming that bDOC removal in the well-mixed reservoir with volume V_R was negligible compared with that in the wetland, the rate of change of bDOC in the reservoir is given in Eq 3:

$$\frac{dC_{eff}}{dt} = \frac{Q}{V_R}\,(C_{inf} - C_{eff}) \qquad (3)$$

which can be solved setting $C_{eff} = C_0$, noting that $C_{inf} = C_L$ and then combining Eqs 2 and 3 to yield Eq 4:

$$\frac{dC_{\text{eff}}}{dt} = -\frac{Q}{V_R}\left[1 - \exp\left(-k\,\frac{A\epsilon L}{Q}\right)\right] C_{\text{eff}} \tag{4}$$

Upon integration of Eq 4, with the initial value $C_{eff} = C_{i,gr}$ (i.e., bDOC concentration in the initial raw graywater batch) at $t = 0$, where t is time (h), the time-dependent treated graywater bDOC in the reservoir (C_{eff}) is then given by Eq 5:

$$C_{\text{eff}} = C_{\text{i,gr}} \cdot \exp\left[-\frac{t}{\theta}\left[1 - \exp\left(-k\,\frac{A\epsilon L}{Q}\right)\right]\right] \tag{5}$$

in which θ is the hydraulic retention time in the wetland layer (i.e., the ratio V_R-Q). The average bDOC removal rate constant (k) in Eq 5 can be extracted from the time-evolution data of the graywater effluent bDOC. In this approach, k is considered an overall bDOC removal rate constant, which can be a function of the graywater properties as well as the wetland's biomass content. Using operational data over periods of two to three weeks each for three wetland hydraulic retention times (0.19, 0.27, and 0.33 h, equivalent to hydraulic dosing rates of 3.18, 2.29, and 1.32 $m^3/m^2/h$, respectively), the average k value was determined to be $5.8 \pm 0.9\ h^{-1}$. The model fit to the data (with an average $R^2 = \sim 0.99$ for the comparison of predicted versus measured values and average absolute relative error of 2%) for the different cases is shown in Figure 6 for different values of the θ in the wetland, area (A), and soil depth (L). The removal rate constant of $5.8\ h^{-1}$ was adopted as a reasonable value for assessing the expected treatment performance and economic feasibility of the present SB-VFW system.

Performance sensitivity evaluation of the graywater treatment system with respect to bDOC was conducted with the model per Eq 5 using the average bDOC removal rate constant of $5.8\ h^{-1}$, as a function of (1) hydraulic retention time (θ) in the wetland; (2) wetland bed thickness (L); (3) wetland area (A), and (4) removal rate constant (k). In all cases, graywater treatment effectiveness was evaluated based on a 3-h treatment time or seven cycles per day. As shown in Figure 6, part A, reducing the hydraulic retention time in the wetland by nearly a factor of 5 (from 0.3 to 0.06 h) had little impact on the treatment time necessary to achieve 98% bDOC removal. Increasing hydraulic retention time beyond 1 h significantly increased the required treatment time (e.g., treatment time of 6 to 8 h for a corresponding θ range of 1 to 1.5 h). This behavior can be rationalized by noting that, for the 3-h treatment time, the number of turnovers (i.e., of the reservoir water volume) decreases with increasing hydraulic retention time, resulting in reduced level of treatment. For the present system design, it appears that a hydraulic retention time of ~0.3 h is already near optimal for treatment of graywater at 0.3 m^3/batch. A deeper bed would shorten the treatment time necessary to achieve 98% bDOC removal (relative to the influent bDOC) (Figure 6, part B) as an alternative to using a wetland of larger superficial area (Figure 6, part C). However, the bDOC reduction (down to 1 mg/L) achieved with the present system (0.15 m soil depth, area of 0.68 m^2) was already well below the typical effluent concentrations from secondary wastewater treatment plants practicing biological nutrient removal (Babcock et al. 2001).

This analysis demonstrates that the current system design is adequate for treatment of bathroom graywater for aboveground nonpotable reuse purposes per the highest levels of treated graywater quality required in the United States (California and Wisconsin), United Kingdom, and Australia. Moreover, as shown by the analysis, the flexibility in design (e.g., system area, soil depth) and operability (e.g., hydraulic retention time) of the SB-VFW could be advantageous when space in residential homes is limited. The application of such a system in different climates will need to consider the effects of temperature (Stefanakis & Tsihrintzis 2012).

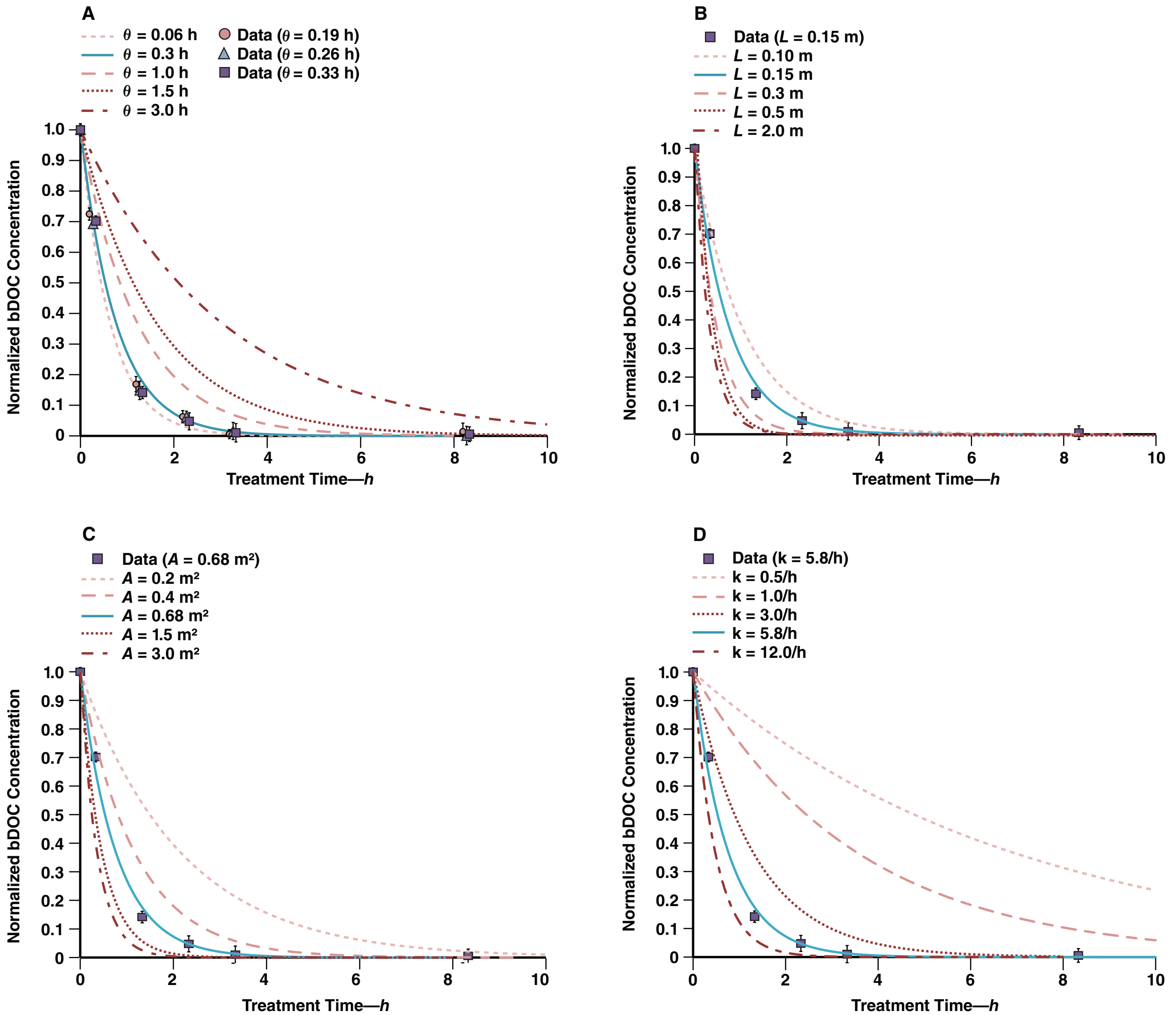
Economic assessment of residential deployment of the treatment system. Economic analyses of breakeven periods and return on investment (ROI) were carried out with respect to daily graywater treatment volume for deployment of an SB-VFW system in a single-family home. The capital and O&M costs used in the analyses were based on system construction costs and O&M cost data generated from the field study. It is estimated that the capital cost of a prefabricated system (including system delivery and installation) could range from $1,000 to $2,500 depending on the level of system automation. Specifically, capital cost for a manual system is estimated to be in the range of $1,000 to $1,500, whereas the capital cost for an automated system would be in the range of $2,000 to $2,500. The upper bounds of capital costs for manual and automated systems were used in a conservative calculation of the ROIs for deployment in different countries. These costs were adjusted according to the consumer price indexes, which were 100 in the year 2005 (World Bank 2016). The cost savings and operational costs (i.e., electricity cost) were calculated considering local utility rates. The labor costs associated with annual maintenance were adjusted according to the gross national incomes relative to the United States. Table 4 lists the utility rates, consumer price indexes, and gross national incomes of selected countries used in the ROI analysis.

The breakeven period, Y, was defined as the time required for the accumulative total net cost-savings from water conserved and avoided sewer charges to be equal to system capital costs:

$$Y = \frac{P}{\alpha Q \cdot V_d - \alpha E \cdot R \cdot V_d - M'} \tag{6}$$

in which P is the system capital costs ($), Y is the breakeven period (years), W is the water rate ($/$m^3$), V_d is the daily volume to be treated and reused (m^3/day), E is the daily power consumption (kW·h/m^3), R is the electricity rate ($/kW·h), α is the conversion factor (365 days/year), and M is the annual maintenance cost ($/year). The breakeven period depends on system capital cost (including installation), O&M costs, daily volume of graywater generated, system treatment capacity, and amount of conserved potable water. Given the relatively low cost of the treatment systems, financing cost was assumed to be negligible. Also, because the cost estimate focused on the treatment system itself, costs of house plumbing retrofit were not considered as part of the system capital or O&M costs. Retrofitting costs can be highly variable depending on the type of house construction and are probably lowest for new construction. Following this approach, ROI, determined as the ratio between net cost savings from avoided water-consumption cost (including the cost of water and sewer charges,

FIGURE 6 Graywater treatment performance for the vertical flow wetland with respect to (A) θ, (B) *L*, (C) *A*, and (D) bDOC k



θ—hydraulic retention time, *A*—wetland area, bDOC—biodegradable dissolved organic carbon, k—removal rate constant, *L*—wetland bed thickness

A hydraulic retention time of θ = 0.33 h was used in generating B–D. The bDOC removal rate constant k = 5.8 h^{-1} was used in A–C. Error bars represent 1 standard deviation of the sample data.

TABLE 4 Average combined water and sanitation tariffs and electricity rates used to calculate cost savings from water conservation and the operational costs for the graywater treatment system

Variable	Australia	Denmark	Germany	Greece	Mexico	United States	Los Angeles, Calif.
Average water cost—*$US/m^3^* (OECD 2010)[a]	3.40	8.61	5.56	1.58	0.37	2.10	2.96 (LADWP 2011)
Electricity rate—*$US/kW•h* (NUS Consulting Group 2014)	0.40	0.40	0.35	0.17	0.19	0.13	0.21
Consumer price indexes (2005 = 100) (World Bank 2016)	108.51	114.87	87.14	83.79	46.96	77.39	—
Gross national income per capita—*$*	59,360	59,850	44,260	23,260	9,640	52,340	—

[a]Includes water cost and sewer cost if present

if present) and system capital cost, as an indicator for economic viability of the treatment system, was determined with Eq 7:

$$ROI = \frac{(\alpha W \cdot V_d - \alpha E \cdot R \cdot V_d - M)}{P} \times 100\% \quad (7)$$

After this, the breakeven periods and ROIs for deployment of an SB-VFW in a single-family home in Los Angeles, Calif., and in selected countries were calculated, respectively. The breakeven period shows that there is a significant economy of scale. The higher-priced automated system has a longer breakeven period but, as expected, the difference diminishes as the daily treatment volume increases. At daily treatment volumes >0.6 m^3/day, the breakeven period for the manual system is between 2.5 and 3.5 years, whereas a breakeven period of four to six years would be required for the automated system. A breakeven period shorter than two years is expected for daily treatment volume >0.9 m^3/day (three batches/day) for a manual system and >1.2 m^3/day (four batches/day) for an automated system. Given that bathroom graywater production is ~64 L/per capita/day in California (DeOreo 2011), property owners of large households or multifamily homes could break even within a relatively short period even in the absence of financial subsidies.

In the absence of financial subsidies, water cost will govern the financial benefits of onsite graywater reuse for end users. Higher water costs, such as in Denmark and Germany, would provide high ROIs even for a system that would treat only a small daily volume of graywater. In countries or areas with a high cost of electricity relative to that of water, such as in Mexico, the ROI will be lower. In extreme cases of high electricity cost, treating graywater may not be economically beneficial, as is indicated by negative ROIs. Clearly, higher water costs relative to electricity costs, will make onsite graywater recycling more economical for single-family home owners. In countries or regions with low water costs relative to electricity cost, graywater treatment will be less favorable and may be economically feasible only for larger scale deployment. In some special cases, such as in areas with limited access to centralized sewers and wastewater treatment systems, graywater treatment may be justified on the benefits of reducing sewage flows. Graywater treatment may also be advantageous in developed areas with collection and treatment systems that are at capacity or are overloaded. The addition of graywater treatment may extend the life of existing systems and avoid capital investment for upgrades while providing an alternative avenue for increasing water reuse.

CONCLUSIONS

Graywater treatment performance and economics of a low-cost and compact SB-VFW for residential deployment were investigated in an eight-month pilot field study. Treatment of bathroom graywater (from showers, baths, and sinks) produced effluent of high quality within a treatment time of 3 h. The treatment process followed first-order kinetics, and treated effluent turbidity (~0.3 ntu) and BOD_5 (~3.1 mg/L) met graywater reuse guidelines for nonpotable applications. After disinfection, total coliform counts were below the required 2.2 MPN/100 mL for aboveground water reuse. Economic analysis showed that, for the present type of treatment systems (costing $1,000 to $2,500 for a treatment capacity of 2.1 m^3/day), the breakeven period was in the range of two to six years, depending on the daily treatment volume. Higher water costs (water cost plus sewer charges, if present) favor graywater treatment, especially when electricity costs are modest or low. However, if electricity costs are high relative to water cost, graywater treatment is less favorable or may even result in a negative ROI. With appropriate financial conditions or subsidies, onsite graywater recycling can have positive environmental and societal effects in areas of limited or no sewer or wastewater treatment access. A decentralized graywater treatment approach may also provide financial benefits from reduced need for capital expansion or retrofitting existing centralized systems while increasing water reuse opportunities. The results of this study suggest that there is merit in exploring distributed deployment of residential graywater systems and further evaluating the efficiency of the approach for treatment that also includes laundry water.

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ENDNOTES

[1]5-ASP sump pump, Little Giant, Fort Wayne, Ind.
[2]MAX900 autosampler, Hach Sigma, Loveland, Colo.
[3]Aurora 1030D TOC Analyzer, OI Analytical, College Station, Tex.
[4]Millipore Millex Sterile Syringe Filters, EMD Millipore, Billerica, Mass.
[5]Weck Laboratories, City of Industry, Calif.
[6]2020 Portable Turbidity Meter, LaMotte, Chestertown, Md.
[7]Oakton CON 11 Economy Meter, Vernon Hills, Ill.
[8]Whatman Grade GA/F filter paper, Pittsburgh, Pa.
[9]Dissolved Oxygen Tracer PocketTester, LaMotte, Chestertown, Md.
[10]IDEXX Colilert and Colilert-18, Westbrook, Maine
[11]SmartPots, Oklahoma City, Okla.
[12]AccuPac Cross Flow Media, Brentwood Industries, Reading, Pa.
[13]P51530-P0 flowmeter, GF Signet, El Monte, Calif.
[14]3-8550-2 transmitter/totalizer, GF Signet, El Monte, Calif.

PEER REVIEW

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