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Physicochemical treatment of office and public buildings greywater

Eran Friedler and Yuval Alfiya

ABSTRACT

The current study analyses the performance of deep sand filtration of greywater from an office building and the performance of a combined physicochemical process comprising of coagulation, sedimentation and filtration. Raw greywater quality exhibited very high variability with average turbidity of 35 NTU, and TSS, COD_t, and BOD of 45, 240, 75 mg/l respectively. The stand-alone filter removed 50 and 70% of the turbidity and TSS, but failed to remove COD and BOD. Quality of the produced effluent was too low to allow any reuse. Clogging rate of the filter was high and under hydraulic loading of 3–4 $m^3/(m^2\,h)$ the filtration cycle had to be terminated after 5–8 h. Clogging occurred mainly on the upper layer, indicating the dominance of "cake" filtration mechanism. Addition of coagulation and sedimentation prior to sedimentation dramatically improved effluent quality, reaching overall removal efficiencies of 92, 94, 65 and 57% of turbidity, TSS COD_t and BOD respectively. The filtration cycle could be prolonged to 20 h. The effluent produced was of much better quality, yet, it has to be further treated (either biological treatment or membrane filtration). Most of the removal occurred in the coagulation-sedimentation step, while the filter acted as a polishing unit.

Key words | greywater, offices, on-site reuse, physicochemical treatment

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INTRODUCTION

Non-domestic water consumption in urban areas consists 10–25% of the urban water demand (Table 1). The largest proportion this non-domestic demand is consumed in office and public buildings (OPBs). Toilet and urinals flushing comprises 60–80% of the indoor water use in OPBs, while the rest is generally generated as greywater (Table 2). Indoor water consumption of OPBs typically consists 31–37% of the total water demand of the building, cooling towers are responsible for 31–48%, landscaping 1–18%, and other uses 1–3% (Chanan *et al.* 2003; Quinn *et al.* 2006). A moderate size building of 10,000 m^2, typically consumes 20 m^3/d. Chanan *et al.* (2003) state that up to 50% of the water consumption in offices can be saved by on-site treatment and reuse of greywater (GW) for toilet flushing and/or landscape irrigation.

Although the overall GW reuse potential in residential buildings is higher than in OPBs, GW reuse in OPBs has several advantages over domestic reuse, the main of which are described herewith.

- Administrative simplicity–OPBs are usually owned by a single entity, unlike residential houses that are often owned by many dwellers. Thus, construction, operation, management and monitoring of GW reuse schemes in OPBs are expected to be much simpler from an administrative point of view. Further, as there would be one central system for the whole building, operation and maintenance could be performed by professional workers, employed (or out-sourced) by a maintenance company who usually runs these types of buildings.
- Implementation simplicity–Since, as stated above, usually one entity owns/runs the OPB, funding the construction and operation of the systems should be easier. Moreover, as these buildings are owned and

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Table 1 | Urban water consumers—proportional demand (in %)

Country	IL[*]	AU[†]	AU[‡,§]	UK[‖]
Households	61	40		
Urban	25	15	10–20	
(commercial)		(10)		(15)
(public services)		(5)		
Industrial	4	45		
(Manufacturing)		(15)		
(electricity)		(30)		
Losses	10			

[*]Israel Water authority (2008); [†]Lenzena & Foran (2001); [‡]Chanan *et. al.* (2003); [§]Quinn *et al.* (2006); [‖]Surendran *et al.* (2004).

operated by one entity, institutional barriers are expected to be lower. Based on experience from Israel, governmental authorities/ministries tend to be more willing to approve on-site GW reuse schemes when buildings are operated by a single company.

- Positive image–By implementing GW treatment and reuse, the organisation (private or public) may gain a better/positive public image, as a result of showing environmental awareness.
- Specific costs–OPBs tend to be larger than residential houses, accommodating many more people. The specific costs (cost/$m^3_{treated}$) of GW treatment and reuse systems were shown to decline significantly with size (Friedler 2008). Thus, the specific costs of GW systems in OPBs are expected to be lower than in residential houses. Nevertheless, the specific GW production (m^3/(person d)) in OPBs is expected to be lower than in residential homes, and this may mask the effect of size.

Domestic GW is generated from five to six appliances: Kitchen sink, dishwasher, washing machine, washbasin,

Table 2 | Proportional water demand in OPBs

	Proportional demand (%)		
Appliance	**DoE (1992)**	**Hills *et al.* (2002)**	**Shuler (2007)**
Blackwater			
WC flushing	43	48	78
Urinals	20	7	3
Greywater			
Hand Washing	27	13	19
Cleaning	1	32	
Canteen	9		

and shower and/or bath. In OPBs three major GW generating appliances are identified: washbasins, sinks in kitchenettes, and showers (do not exist in all establishments of this type). Domestic washbasins are used for washing hands after excretion, for tooth brushing, shaving, etc., while washbasins in OPBs are predominantly used for washing hands after excretion. As a result, greywater generated by washing basins in OPBs is expected to less polluted than greywater generated by domestic washbasins. The same is true for OPBs kitchenettes' sinks, which are used mostly for washing cups and dishes of light meals, unlike domestic kitchens that are often used for washing dished "heavy meals". To conclude, GW generated in OPBs should generally less polluted than domestic GW, as demonstrated in Table 3.

In densely populated urban areas, due to the high cost of space and its limited availability, it is important that on-site GW treatment systems are compact. Physical treatment units are an attractive option for this setting since they have small footprint and were proved to be reliable. Stand alone

Table 3 | Quality characteristics of greywater from domestic houses and office/public buildings

Parameter		OPBs GW			Domestic GW
Source	**Unit**	**1**	**2**	**3**	**4**
COD	mg/l	22.9	514	79[*]	822
BOD	mg/l		257		477
TKN	mg/l			29	
TN	mg/l		15.5		
NH_4-N	mg/l		1.1	9	1.6
NO_3-N	mg/l		0.9		
NO_2-N	mg/l		0.2		
TP	mg/l		7.3	1.7	61
TSS	mg/l			185	298
Turbidity	NTU	12.6			
pH	–	7.27	6.9	7	
MBAS	mg/l		64.6		37
Cationic surfactants	mg/l		3.8		
Nonionic surfactants	mg/l		55		

1. Kim *et al.* (2007); 2. Shuler (2007)—GW from washbasins and dishwashers—the latter is characterised by high concentrations of surfactants; 3. Shin *et al.* (1998)—Consists of kitchenettes GW (64%), washbasins and laundry GW (10%), washbasins in restrooms (26%); 4. Friedler (2004)—Combined stream of light domestic GW originating from bath (20%), shower (20%), washbasin (16%), kitchen sink (26%), dishwasher (5%) and washing machine (13%).

[*]Dissolved COD.

filtration has an economical advantage over advanced biological treatment like Membrane Bioreactor (MBR) and Rotating Biological Contactor (RBC). However, previous works showed that stand alone filtration does not produce effluent of acceptable quality and fail to meet quality requirements of reused water (March *et al.* 2004; Friedler *et al.* 2006). Therefore, sand filtration should be combined with other treatment. This was the goal of the current study, the objective of which were to analyse the performance of sand filtration treatment of light GW from an office building and to compare its performance with a combined physicochemical process consisting of coagulation, sedimentation and filtration.

METHODS

The experimental system

The building of the Faculty of Civil and Environmental Engineering in the Technion (Israel Institute of Technology) served as a case study for this research. The building is seven story high, consisting of a combination of classes and offices. The first four floors comprise of classes, library, cafeteria and secretarial offices. Staff offices are located in the top three floors. In a typical working day the building accommodates about 400–600 students and 100 staff. During lunchtime about 100–200 more people enter the building in order to dine in the cafeteria.

The building is equipped with a dual drainage system, collecting two separate steams: (1) Blackwater from toilets and wastewater from the cafeteria; and (2) A combined stream of GW originating mainly from washbasins and condensate water from the central air-conditioning system. Greywater was collected from a wet well in the basement of the building. Experiments were conducted during spring (from March to May 2009), therefore the contribution of condensate water was minimal. Thus, pollutants' concentrations in the greywater were higher than during summer when the air-condition condensate water had a dilution effect (data not shown).

The treatment system consisted of two options: either stand-alone sand filtration, or combined treatment of flocculation-sedimentation-filtration (Figure 1). The filtration column was made of 0.090 m diameter (0.084 m internal diameter) and 2.6 m high circular PVC tube. The bottom 0.10 m of the column was filled with quartz gravel (3.5–5.0 mm; D_{10} 3.1 mm ± 5%; U.C. < 1.5) that acted as a drainage layer. On top of the gravel 0.70 m were filled with quartz sand number 0 (0.60–0.84 mm) having D_{10} of 0.585 mm ± 5% (diameter of the 10th percentile sand size, known also as effective size) and U.C. < 1.5 (uniformity coefficient = D_{60}/D_{10}). Backwash was performed manually.

Stand-alone filtration experiments

Filtration rate in the stand-alone filtration experiments was in the range of 2.5–10 $m^3/(m^2\,h)$. The filter was operated until clogging of the sand media was observed. Clogging occurred mainly on the surface of the upper part of the media causing the flow through the media to become unsaturated. Therefore, a siphon was added to ensure saturated flow conditions through the media. During each filtration cycle, samples of raw and filtered GW were taken every hour for laboratory analyses.

Combined coagulation-sedimentation-filtration system

Ferric chloride ($FeCl_3$) was used as coagulant. $FeCl_3$ dose was determined by a Jar Test, following the following

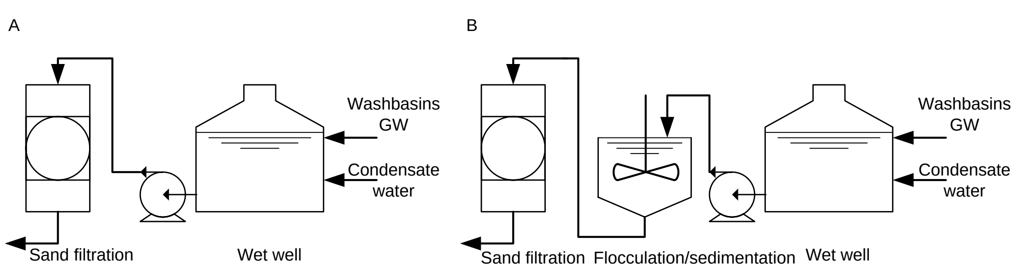

Figure 1 | Schematic of the two treatment options. A—Stand alone sand filtration; B—Flocculation-sedimentation-filtration.

Table 4 | Raw GW quality characteristics

Parameter	Turbidity	TSS	COD_t	COD_d	COD_d/COD_t	BOD_5	COD_t/BOD_5
Units	(NTU)	(mg/l)	(mg-O_2/l)	(mg-O_2/l)	–	(mg-O_2/l)	–
Average	35	46	244	195	0.75	74	4.2
Median	31	46	221	211	0.75	61	4.8
STD	19	29	121	119	0.25	27	2.4
CV*	55%	63%	50%	61%	33%	36%	57%
n	28	6	6	6	6	5	5

*CV—Coefficient of variation.

procedure: $FeCl_3$ was dosed in increasing concentrations to six 500 ml glasses filled with raw GW, then rapid mixing (100 RPM) was performed for 5 min, followed by 30 min of slow mixing (25 RPM), then after 30 minutes of settling (idle) residual turbidity was measured and the optimal coagulant dose was derived.

Due to high temporal variability of raw greywater quality, the combined coagulation-sedimentation-filtration experiment was performed in a semi-batch mode. Raw GW was pumped from the wet well to a 800 L tank. Then a sample was taken for determination of optimal coagulant dose (Jar Test). The coagulant was dosed to the tank to reach its optimal concentration. The tank was mixed at 60 RPM for 5 min followed by 30 min of slow mixing (30 RPM), and 40 min settling (no mixing). At the end of the settling period sludge was carefully drown out from the bottom of the tank. The settled GW effluent was fed to the filtration unit, which was operated in the same way as in the stand-alone filtration experiments.

Analytical methods

Raw GW, coagulated-settled GW effluent, and filtration effluent (with and without pre-treatment) were analysed for turbidity, TSS, BOD_5 (total), and total and dissolved COD. All analyses were performed according to the Standard Methods (APHA AWWA WEF 2005).

RESULTS AND DISCUSSION

Raw GW quality

As expected, the quality of the raw GW exhibited high variability (Table 4). Turbidity ranged from 11 to 76 NTU, with none of the observations being below 10 NTU, 36% lying in the range of 11–20 NTU (the highest proportion), 46% in the range of 20–50 NTU and 21% between 50 and 76 NTU. This high variability is reflected by the high CV (coefficient of variation, 55%). Other parameters exhibited high variability too, while the variability of the BOD was somewhat lower (CV 36%). The reason for the variation in the GW quality could have been a result of the water use pattern in the building. COD_t/BOD ratio exhibited high variability rising significantly with the COD_t of the raw GW (Figure 2(A)). This indicates that when the raw GW was more polluted by organic pollutants the proportion of slowly biodegradable organics was higher. Most of the COD was in the dissolved form (about 75%) and this ratio was quite

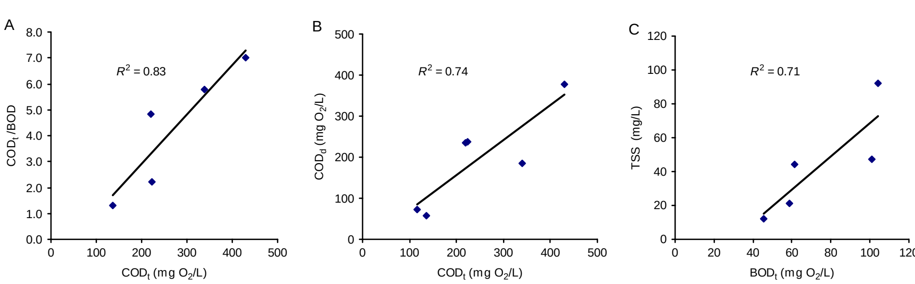

Figure 2 | Raw GW quality: A—(COD_t/BOD) vs. COD_t; B—COD_d vs. COD_t; C—TSS vs. BOD.

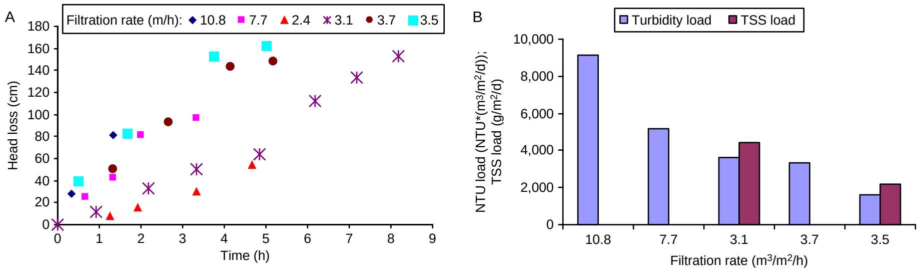

Figure 3 | Stand-alone filtration: A—Head-loss development; B—Turbidity and TSS loads under varying filtration rates.

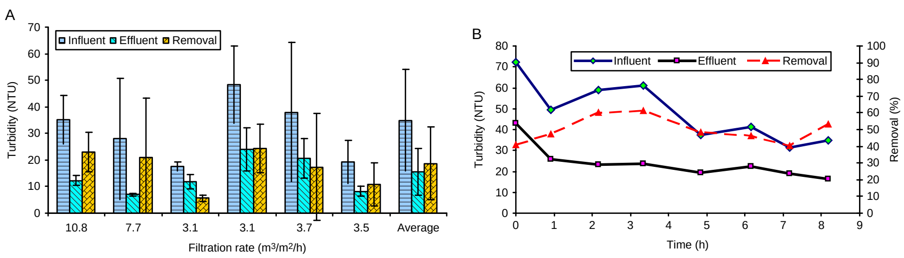

Figure 4 | Stand-alone filtration: Turbidity of raw and filtered GW; A—Different filtration rates; B—Along one filtration cycle (filtration rate 3.1 m/h).

insensitive to the COD_t in the raw GW (Figure 2(B)). TSS did not exhibit correlation with COD_t, COD_d or suspended COD, but quite a good one with BOD ($R^2 = 0.71$, Figure 2(C)). This may indicate that biodegradable organics in the raw GW may have been in a suspended form.

Stand alone filtration

During the filtration cycles clogging of the sand developed quite rapidly and the head-loss along the sand media increased until the sand was completely clogged (Figure 3(A)). Close observation of the media revealed that clogging occurred mainly on the upper layers. This phenomenon is typical for "cake" filtration where solids are intercepted on the top of the sand media and a "cake" builds up. The cake prevents from smaller particles to penetrate the media. This type of filtration is less efficient than deep bed filtration, because not all the media is being actually used. Loading the filter with 9,000 (NTU m^3)/(m^2 d), equivalent to filtration rate of 10.8 $m^3/(m^2\,h)$ (Figure 3(B)), resulted in clogging of the media after 2 h of operation. Therefore, turbidity load was lowered to 3,100–3,700 (NTU m^3)/(m^2 d), equivalent to filtration rate of 3.1–3.7 $m^3/(m^2\,h)$ and TSS load of ~4,500 g/(m^2 d). Under this load, complete clogging of the media occurred after 5–8 h.

The average turbidity of the influent, effluent and the removed turbidity under different filtration rate are presented in Figure 4(A). The removed turbidity (average removal 51% (±15%)) was strongly depended on the turbidity of the influent, while the residual turbidity of the effluent was more stable (average 15 NTU, STD 7.0 NTU). Turbidity of raw GW was highly variable as demonstrated in Figure 4(B), where in the beginning of the filtration cycle it was 72 NTU dropping to 31 NTU after 7 h. Effluent residual turbidity was much more stable ranging from 26 NTU (1 h into the filtration cycle) to 16 after ~8 h. Average TSS removal was 24.1 (±16) mg/l, with removal efficiency of 69 (±8.8)% and residual TSS (effluent) of 9.9 (±5.1) mg/l. Only 5% of the BOD was removed and no removal of COD_t and COD_d was observed. This falls in line with the fact that the majority of the COD in the raw

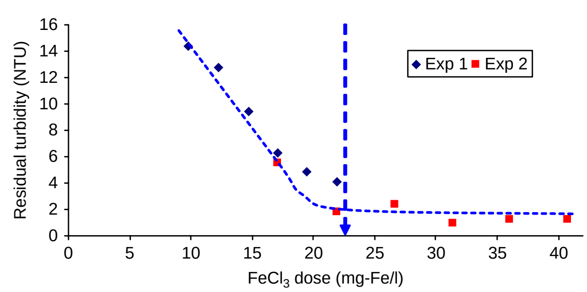

Figure 5 | Optimal $FeCl_3$ dose as derived from jar tests.

Table 5 | Coagulation-sedimentation-filtration: quality of raw GW and treated effluent ($n = 6–7$)

Parameter	Units	Raw GW	Coagulation + sedimentation effluent Concentration	Coagulation + sedimentation effluent Removal	Filtration effluent Concentration	Filtration effluent Removal*	Overall removal
Turbidity	NTU	46 (±23)†	5.7 (±4.5)	88%	3.9 (±3.4)	32%	92%
TSS	mg/l	70 (±32)	7.4 (±6.2)	89%	4.4 (±3.8)	41%	94%
COD_t	mg-O_2/l	180 (±61)	80 (±76)	56%	63 (±77)	21%	65%
COD_d	mg-O_2/l	148 (±127)	34 (±32)	77%	31 (±25)	8.8%	79%
BOD_5	mg-O_2/l	103 (±2.1)	50 (±8.6)	51%	44 (±19)	12%	57%

*Specific removal of the filtration unit.
†Value—Average value; Number in brackets—one standard deviation.

GW was dissolved (75%) and indicates that the rest was in fine particles that passed through the filter media.

Combined process coagulation, sedimentation and filtration

The optimal dose of Ferric chloride was found to be 22 mg-Fe/l (Figure 5). The coagulation and sedimentation stage were very efficient, producing effluent of very good quality (Table 5), with very high removal efficiencies of turbidity and TSS (88 and 89% respectively) and moderate removal efficiency of COD_t, COD_d and BOD (56, 77 and 51% respectively). The latter findings are important, since the stand-alone filter failed to remove these three pollutants.

Since the coagulation and sedimentation produced good quality effluent, the filtration rate could be increased to 6.5 $m^3/(m^2\,h)$. Nevertheless, quite obviously, all loads were lower, with turbidity load of 615 (NTU m^3)/m^2 d), 732 g/(m^2 d) TSS, 4,890 g-O_2/(m^2 d) COD_t, 8,450 g-O_2/(m^2 d) COD_d and 5,960 g-O_2/(m^2 d) BOD. Clogging of the media developed at a much slower rate as compared with the clogging rate of the stand-alone filtration. Further, no development of a "cake" on the upper layers of the media was observed, indicating that the whole media depth was active in the process. The filtration cycle could be prolonged to 20 h (in the stand-alone filtration, the duration of the filtration cycle was 5–8 hours). As a result of the high efficiency of the coagulation-sedimentation stage, the influent to the filtration unit was of good quality. Therefore, the removal efficiency of the filtration stage was relatively low. Nevertheless the filter acted as a polishing unit, removing 32, 41, 21, 8.8 and 12% of the turbidity, TSS, COD_t, COD_d and BOD entering the filter respectively. The quality of the treated GW effluent was very high in regard to turbidity and TSS, but not good enough regarding COD and BOD. The concentrations of these two in the treated effluent not only were higher than permitted values in reuse regulation, but could also lead to bacterial regrowth and to negative aesthetic and environmental effects. Nevertheless, addition of coagulation and sedimentation as pretreatment to filtration improved the overall efficiency dramatically and the combined process or can serve as pretreatment to biological treatment or direct membrane filtration (Friedler *et al.* 2008).

CONCLUSIONS

Although GW reuse potential in residential homes is higher than in OPBs, GW reuse in OPBs has several advantages over domestic reuse, namely: administrative and implementation simplicity, creation of positive "green" image, and probably lower specific costs. Moreover, due to the nature of water use within OPBs, the generated GW is expected to be less polluted than domestic GW. These advantages make OPBs good candidates for initiation of on-site reuse schemes.

This study analysed the performance of deep sand filtration treatment of light GW from an office building and compared its performance with combined physicochemical process of coagulation, sedimentation and filtration. The study was performed in the building of the Civil and Environmental Engineering Faculty in the Technion, which is equipped with dual collection system.

Raw GW quality exhibited very high variability (CV ~ 40–60%), with average turbidity of about 35 NTU, and TSS, COD_t, COD_d and BOD of about 45, 240, 200, 75 mg/l respectively. About 75% of the COD was in the dissolved form. The COD_t/BOD ratio exhibited positive correlation with COD_t, indicating that as the organic load in the raw GW rose, it became less biodegradable. Positive correlation was also found between TSS and BOD in the raw GW.

The stand-alone sand filter removed about 50 and 70% of the turbidity and TSS, but failed to remove COD or BOD. This is due to the fact that most of the COD in the raw GW was dissolved. Quality of the produced effluent was, as expected, too low to allow any reuse option. Clogging rate of the filter media was relatively high and under hydraulic load of 3–4 $m^3/(m^2\,h)$ and TSS load of ~4,500 $g/(m^2\,d)$ the filtration cycle lasted not more than 5–8 h. Clogging occurred mainly on the upper layers, indicating that the filtration mechanism was “cake filtration”.

Adding coagulation and sedimentation stages prior to filtration dramatically improved effluent quality and enabled removal of COD and BOD as well, with overall removal efficiencies of 92, 94, 65, 79 and 57% of turbidity, TSS COD_t, COD_d and BOD respectively. The effluent produced was of very high quality regarding turbidity and TSS, and moderate quality regarding COD and BOD. Thus, the treated effluent has to be further treated before it can be safely reused. Nevertheless, addition of coagulation-sedimentation as pretreatment dramatically improved the overall process efficiency and the combined process or can serve as pretreatment to biological or membrane treatment options. In the combined coagulation-sedimentation-filtration process most of the removal occurred in the coagulation-sedimentation step, while the filter acted as a polishing unit. The filtration mechanism was “deep sand filtration”. Development of clogging and head-loss in the filter media was much slower than in the stand-alone filtration, leading to a much longer filtration cycle (20 h vs. 5–8 h), to lower frequency of backwashes and thus to better process efficiency.

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