Environment Protection Engineering

Vol. 41 2015 No. 3

DOI: 10.5277/epe150309

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UPGRADING THE EFFLUENT QUALITY OF AN AERATED LAGOON WITH HORIZONTAL ROUGHING FILTRATION

Removal of suspended solids and microorganisms from an aerated lagoon effluent with a horizontal roughing filter (HRF) was investigated. The aerated lagoon receives Qom municipal wastewater. The HRF was operated at three filtration rates of 0.5, 1.0 and 1.5 $m^3/(m^2\cdot h)$ during four month operation period. The measured values of turbidity, TSS, COD, pH, temperature and flow rate of HRF at the former filtration rate were 79±12 NTU, 100±11 mg/dm^3, 190±12 mg/dm^3, 7±0.1 °C, 17±8 °C and 0.82 dm^3/min, respectively. The differences between inlet and outlet values of pH and temperature were not significant ($P > 0.05$). Measures turbidity, TSS and COD in HRF final effluent were 15±13.7 NTU, 37±295 mg/dm^3, 64±39.7 mg/dm^3, respectively, which corresponds to 81.1%, 63% and 66.3% removal efficiencies, respectively. A decrease of removal efficiency was observed upon increasing filtration rates. The Spearman correlation coefficients between the head-loss and removal efficiencies ranged from 0.578 to 0.968 pointing to a direct relationship. Results of modeling approach revealed appropriate compliance between the values of the observed and predicted TSS for higher filtration rates.

1. INTRODUCTION

Using aerated lagoons for municipal wastewater treatment are common practices, especially in developing countries [1]. Due to lack of return activate sludge (RAS) line, an aerated lagoon process produces relatively high levels of suspended solids in the system affecting the quality of the final effluent negatively [2]. Rich et al. [3] reported the average BOD_5 of 65 mg/dm^3 during 18 months operation in South Carolina (USA)

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wastewater treatment plant in which aerated lagoon was used as a biological treatment system. A number of technologies have been used to upgrade lagoon effluents for suspended solid and BOD removal [1], including intermittent sand filters, as well as wetland and aquatic treatment systems. Intermittent sand filtering is one of the prevalent processes in aerated lagoon effluent treatment. But this filtration technology has disadvantages such as land and regular maintenance requirements, odor problems, filter media availability , media clogging and sensitivity to low temperatures [2].

HRF consists of a long trough open to the atmosphere equipped with series of flow-through compartments (cells) filled by descending sizes of gravel. The gravel often includes crushed river rock media from an average 20 mm diameter in the first compartment, to an average 4 mm diameter in the latter one. The common filtration rates range from 0.3 to 1.5 $m^3/(m^2 \cdot h)$ of the filter flow exposed area [4].

HRFs are mainly used to pretreatment of raw water for clay removal. This application tends to lengthen the slow sand filter (SSF) filtration cycles by decreasing the inlet contaminant loading [5]. The idea of connecting three successive cells in HRF is to provide the maximum capacity for deposition of suspended solids [6]. Various kinds of microorganisms may be active during the roughing filtration run which can eliminate organic matters [4]. HRF also has a noticeable ability to remove microorganism from water. Collins et al. [7] reported up to 90% removal of coliforms from surface water with roughing filters. Few authors investigated use of HRF as wastewater post-treatment.

This study was aimed at using a HRF as a post-treatment unit of a wastewater treatment plant which used an aerated lagoon as its biological system. Removal of suspended solids, total coliforms, fecal coliforms, turbidity and COD were considered.

2. EXPERIMENTAL

2.1. STUDY AREA

Figure 1 illustrates the location of Qom province in the central part of Iran. The Qom wastewater treatment plant (WWTP) is located in the northeastern part of the city, beside the Qomroud River. The Qom WWTP is an aerated lagoon system consisting of two parallel sets of cells. Each set of cells is four basins (lagoons). The dimension of each basin is 100 m (length) × 80 m (width) × 4 m (depth). Basins have been connected to each other in series. Two parallel sets of basins were in use during the study. Except the final basin of each lagoon series, three other upstream lagoons are equipped with mechanical floating aerators. The final effluent is used for agricultural irrigation and the overflow effluent is discharged to the Qomroud River.

A pumping station located beside the final effluent channel discharged irrigation water or supplied it to the river. The station consisted of two parallel pumps intermit

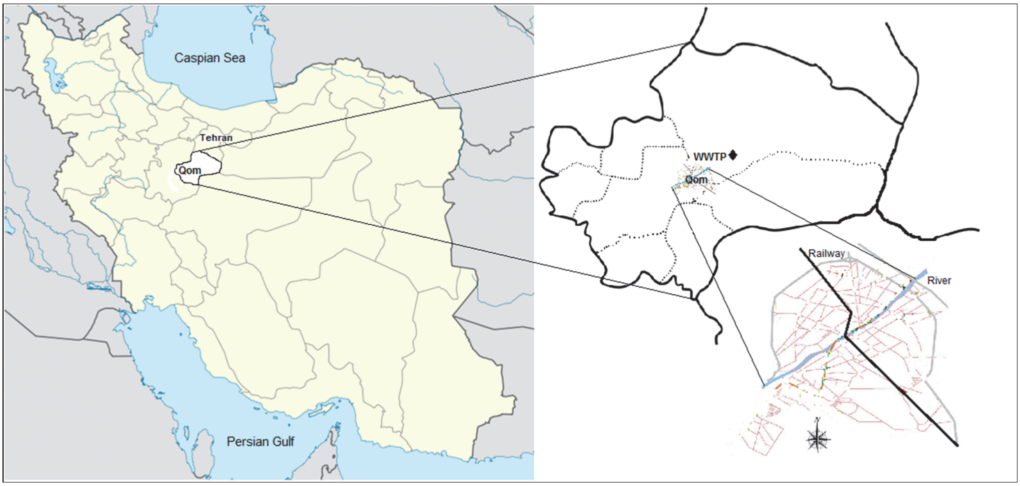

Fig. 1. Location of the municipal wastewater treatment plant (WWTP) in Qom province, Iran

tently discharging the effluent, therefore discharging was performed continuously during the study period. The main deficiencies of the Qom WWTP were high concentration of suspended solids, high turbidity and elevated content of organic materials in the final effluent. Table 1 shows the main parameters of the Qom WWTP effluent for 74 grab samples during the study period, from February through May 2009.

Table 1

Characteristics of the influent feeding the pilot-scale (effluent of Qom WWTP)

Parameter	Hydraulic loading rate [$m^3/(m^2 \cdot h)$]		
	0.5	1	1.5
Temperature, °C	17±8	20±3	23±2
pH	7±0.1	8±0.2	8±0.3
Turbidity, NTU	79±12	79±11	69±7
Suspended solids, mg/dm^3	100±11	86±5	93±10
Total coliform, 10^8 cfu/100 cm^3	357±231	420±229	410±210
Fecal coliform, 10^8 cfu/100 cm^3	229±156	200±95	206±101
COD, mg/dm^3	190±12	197±10	190±13

2.2. DESCRIPTION OF THE PILOT PLANT

The pilot system was installed adjacent to the effluent channel of the Qom WWTP. The HRF system based on the aerated lagoon effluent. Thus, the composition of the inlet flow was unpredictable and could only be monitored via daily measurements.

The pilot scale system (Fig. 2) was a HRF consisting of three subsequent compartments built in steel sheets with the dimensions of 4 m (length) × 0.5 m (width) × 0.45 m (depth), (the lengths of inlet and outlet zones are considered). Each compartment was filled with crushed river gravel as filtration medium. Three compartments were separated from each other and also from the inlet and outlet zones with perforated walls. Perforation density of walls was average 2 holes per cm^2. The opening diameter of each hole was 3 mm.

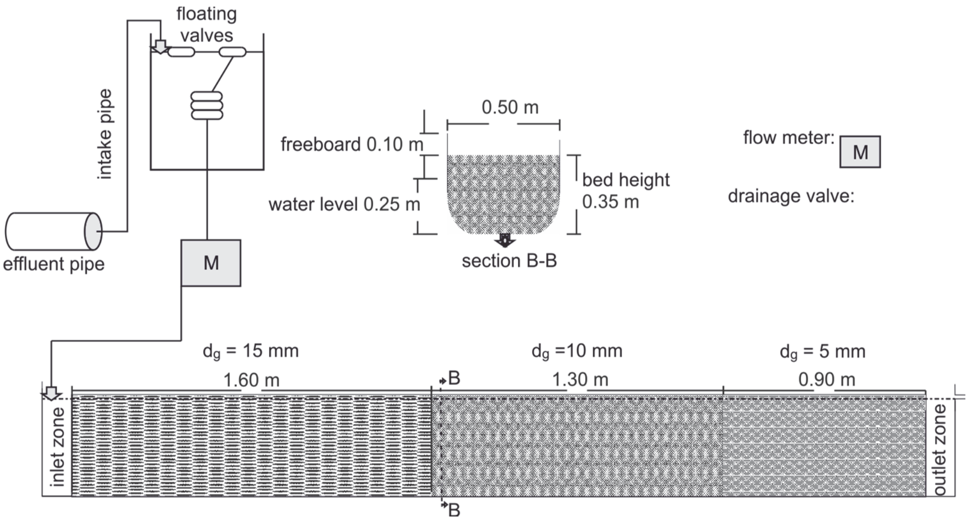

Fig. 2. Schematic layout and details of the HRF pilot system

The surface area is defined as the vertical wet surface of the gravel bed exposed to the horizontal flow of water during the filter run. Thus, the filtration rate (V_f) was calculated from following equation

$$V_f = \frac{Q_{in}}{S} \tag{1}$$

where Q_{in} is the inlet flow rate by applying a continuous flowmeter on the HRF inlet pipe and S is the surface area.

To avoid short-circuiting and also preventing algal growth on the horizontal surface of filter beds, water was adjusted to flow 10 cm beneath the bed surface based on Collins et al. recommendations [6]. Thus, the height of wet media at the beginning of filtration was 35 cm. However, during the filtration run, due to clogging the upper dry media were submerged gradually during the head loss raising. Water appearing on the horizontal surface of the beds was the sign of over-clogging. Further operating of the system after over-clogging would create a short-circuit flow over the filter bed from the inlet to the outlet zone. Thus, the filtration had to stop for hydraulic washing (flushing). As illustrated in Fig. 2, hydraulic washing was performed via applying hydraulic loading exceeding 10–30 times the ordinary operation rate and then, opening six flushing valves installed in the filter floor for disposal of sediments [4].

The washing system was designed to allow flushing rates higher than 30 $m^3/(m^2\cdot h)$ [6]. For improving the efficacy of hydraulic flushing, the compartment walls were designed based on a curved-pattern to obtain a sufficient slop during the residual discharge which based on the Torabian et al. [8] experiment.

Table 2

Operation design criteria for the HRF pilot system

Parameter	Compartment			Total
	1	2	3	
Wet surface area, m^2	0.1			–
Height, m	0.45			–
Length, m	1.6	1.3	0.9	3.8
Volume, m^3	0.16	0.13	0.09	0.38
Bed weight, kg	424	345	239	1008
Gravel effective size, mm	6.8	5	3.2	–
No. of drainage valves	3	2	1	–
Slope, %	1			
Bed height, m	35			–
Submerged height, m	25			–

A 0.22 m^3 elevation tank equipped with an inlet floating valve, outlet floating gate, mixer and drainage valve was installed adjacent to the HRF inlet zone. The elevation tank was fed continuously from the WWTP effluent pipe via a narrow intake pipe and the respective flow was determined with a flow meter at the HRF inlet zone during the study period. The HRF medium with the specific surface area of approximately 90 m^3/m^2 is locally available at a low cost. Table 2 illustrates the operational design criteria of HRF which were obtained based on the Wegelin et al. [4] suggestions.

2.3. MATERIALS, SAMPLING AND ANALYTICAL METHODS

River gravels used as filter media were prepared from available the Qomroud riverbanks. They were washed, dried and separated to respective sizes (Table 2) using standard ASTM laboratory sieves.

From the inlet and outlet zone of the HRF, 74 duplicated samples were taken during the measurement period. The inlet zone means the beginning of the first compartment and the outlet one – the end of the third compartment (Fig. 2). Sampling from the outlets of first and second compartments was not useful during this experiment.

According to the previous works [4, 10], the appropriate results have been obtained at the filtration rates from 0.3 to 1.5 $m^3/(m^2\cdot h)$. So, three distinctive filtration rates within this range (0.5, 1.0 and 1.5 $m^3/(m^2\cdot h)$ were evaluated to determine the optimum situation and also to develop a model for predicting the TSS values.

The first filtration run lasted 45 days with an average two day sampling interval. The time interval between subsequent paired samples in second and third filtration runs attributed to 1.0 and 1.5 $m^3/(m^2\cdot h)$, respectively, was 1 day. Sampling of the first filtration run was continued to obtain the steady state. Relative long time to reach a stable outlet in the first run could have been due to low temperature of water (Table 1) in winter which hindered some temperature-related mechanisms.

All materials used in laboratory analysis were purchased from the Merck Company (Merck®, Germany). Grab samples were analyzed for turbidity using a portable turbidimeter, Aqualytic® model, suspended solids SS by the method 2540 D [9], COD (5220 D) with a spectrophotometer, model DR-4000, Hatch® Company, USA, total coliforms (9221 B) and fecal coliforms (9221 E) [9].

The head loss was measured by daily recording of the distance between the water level in the inlet zone and the gravel bed surface of the first compartment.

2.4. MODELING THE CONCENTRATION OF SUSPENDED SOLIDS IN THE EFFLUENT OF HRF

Various approaches have been developed for modeling the horizontal flow modes of depth filtration. The 1/3–2/3 theory by Wegelin [10] is a simple model to describe the performance of the HRF based on gravel media. While a suspended particle in water passes through a bed filled up with gravel, there are equal chances for the particle to escape either through left or right or get entrapped on the surface of the gravel. Thus, the success probability of settling on the gravel surface and escaping from it is 1/3 and 2/3. This concept forms the fundament of Weglin's theory.

Consequently, as the filtration run time proceeds, the effect of multiple layers (known as compartments) of the HRF tends to separate more of the particles. Weglin's 1/3–2/3 theory has been applied to develop the models for describing the removal efficiency of the roughing filter. Based on the existing filter theories and also on Fick's law, the filter efficiency is determined by the filter coefficient according to following differential equation

$$\frac{dc}{dx} = -\lambda c \tag{2}$$

where c is the solid concentration (mg/dm^3), x is the filter depth (m) and λ is the filter coefficient. Thus the rate of particle removal with a roughing filter is a function of the initial concentration of particles in water.

The filter length defines the number of conceptual parallel plates (randomly curved surfaces of gravels in reality) being multistage reactors. So, the removal performance of the HRF is determined based on the data obtained from these tiny filter units. The final TSS concentration after passing through the length of Δx gives the following equations

$$C_e = \sum_i C_0 \exp\left(-\lambda_i \Delta x\right) \tag{3}$$

where λ_i is the filter coefficient of each filter cell and Δx is the depth of experimental filter cells. C_e and C_0 are the TSS concentrations in the HRF inlet and outlet, respectively. It should be noted that $\exp(-\lambda_i\Delta x)$ is the efficiency E_i of the filter cell.

Based on the Wegelin theory, it can be assumed that the integration of filter cell efficiencies E_i can be expressed as the compartment efficiency E. Thus, for a HRF system consists of n compartments, the predicted final TSS effluent can be estimated as follows

$$C_e = C_0 E_1 E_2 ... E_n \tag{4}$$

where E_n is the filter efficiency attributed to compartment n. The values of E_n can be found either by using tables or graphical nomograms [10].

One way analysis of variance (ANOVA) with the Tucky Test served as average comparative statistical analysis. The Spearman correlation coefficient analysis was used to determine the correlation between the head loss and removal efficiencies. SPSS version 18.5 was applied for data analysis and Microsoft Excel 2007 was used for calculations.

3. RESULTS AND DISCUSSION

3.1. SUSPENDED SOLIDS, TURBIDITY AND COD REMOVAL

Table 3 shows the suspended solids, turbidity and COD values from the aerated lagoon effluent which entered the HRF during the study period. There was no significant difference and also, there was no obvious increasing or decreasing trend between the HRF inflow values during the three filter runs ($P = 0.38$). The results listed in Table 3 show the statistical summary of values measured from the HRF outlet samples. There is a significant difference between turbidity, TSS and COD average loads in three filtration rates ($P < 0.05$). Collins et al. [5] reported the turbidity removal between 70–80% by using a pilot scale of HFR which was fed by storm water with turbidity in the range 30–80 NTU.

Al-Saed et al. [11] during a pilot experiment lasting four months (from March through August 2006) on effluent of a facultative pond with a horizontal rock filter, found that 77.3% TSS removal occurred. According to Barman et al. [12], measured turbidity values of the HRF inlet, which was the effluent of a stabilization pond, ranged from 60 to 120 NTU.

Table 3

Process performance for the HRF at various hydraulic loading rates

V_f [m³/(m²·h)]		Turbidity [NTU]	TSS [mg/dm³]	Total coliform [10⁸ cfu/100 cm³]	Fecal coliform [10⁸ cfu/100 cm³]	COD [mg/dm³]
0.5	average	14.9	36.9	40	28.2	64.0
	SD	13.7	29.5	33.3	18.0	39.7
	min	3.0	11.0	8.2	7.2	21
	max	57.0	98.0	120.0	63.0	163
1	average	27.1	34.4	56.1	28.8	112.6
	SD	14.5	11.6	33.4	17.7	28.7
	min	10.0	17.0	27.0	7.8	80
	max	54.0	55.0	130.0	66.0	167
1.5	average	32.6	42.9	68	37	113.3
	SD	12.9	16.7	35.1	12.3	34.2
	min	18.0	23.0	30.0	19.0	64.0
	max	58.0	78.0	135.0	68.0	172.0

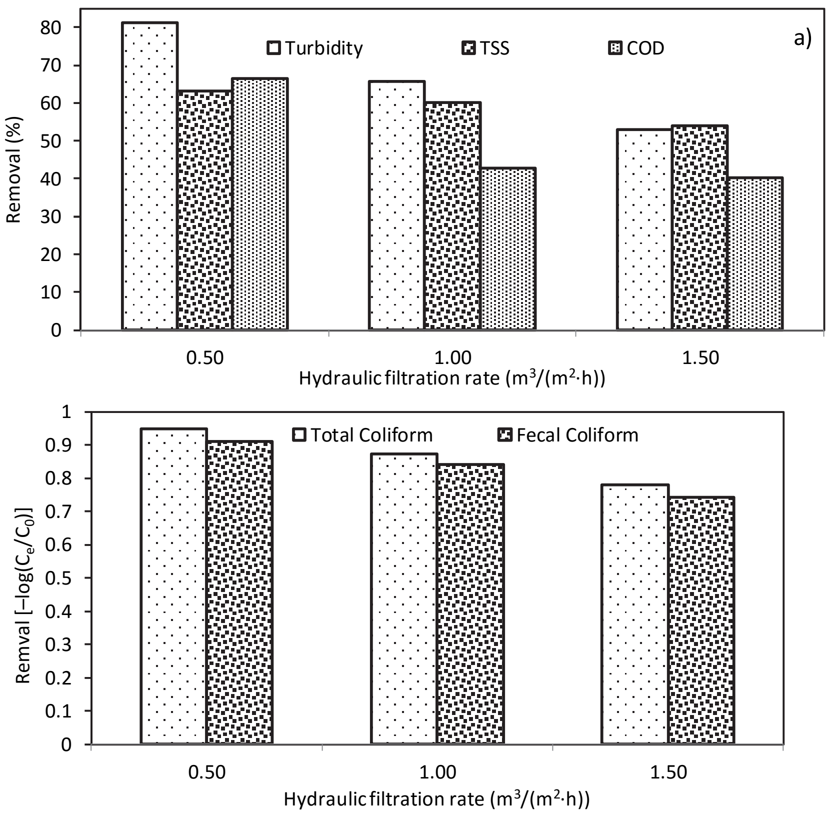

Fig. 3. Removal efficiencies of a) turbidity, TSS, COD,
b) coliforms at three hydraulic filtration rates (m³/(m²·h))

They reported 90% turbidity removal at 0.3 $m^3/(m^2 \cdot h)$ filtration rate [12], while Dastanaie et al. [13] reported 63.4% and 89.7% removal of turbidity and TSS, respectively, with an HRF installed beside a riverbank from February through September 2006 [13]. El-Taweel et al. [14] found 83% removal of turbidity with HRF. The roughing filters removed clay particles more effectively when the filter was ripened with algal cells [15].

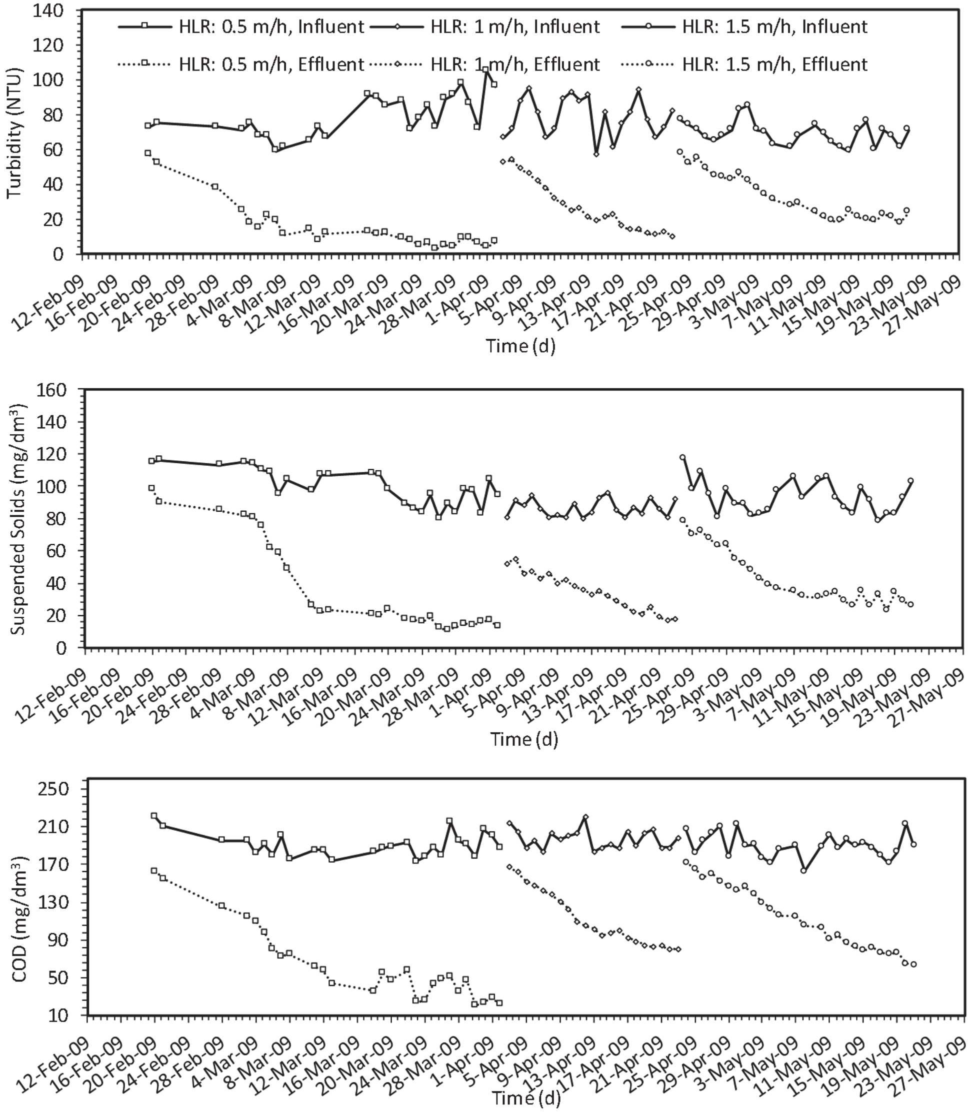

Fig. 4. Values of a) turbidity, b) TSS and c) COD from influent and effluent flows of HRF at three subsequent *Vf*

Figure 3a shows that for the filtration rates below 1.5 m/h the average percentage removal of turbidity and TSS decrease upon V_f increasing. During the experiments at $V_f = 0.5$ m^3/(m^2·h), the average removals of turbidity, TSS and COD were 81.1%, 63% and 66.3%, respectively. At $V_f = 1.5$ m^3/(m^2·h), the mean turbidity and TSS removals were 52.8%, 54% and 40.3%, respectively.

Nkwonta et al. [16] reported 63% and 89% removal of turbidity and TSS achieved with the HRF at the filtration rate of 0.75 m/h. Rooklidg et al. [17] found 80% and 95% removal of turbidity and TSS with a pilot scale HRF based on crushed dolomite gravel as media. Figure 4 shows the turbidity, TSS and COD values measured in double samples taken from the inlet and outlet zones of HRF at various filtration rates. The maximum reductions in turbidity (96%), TSS (85%) and COD (88%) were achieved after almost 50 day continuous run at the filtration rate of 0.5 m/h. Also, significant removals of turbidity, TSS and COD at the filtration rate of 1.5 m/h achieved after 21 days were 66.1%, 74.7% and 66.3%, respectively. This finding is in agreement with those of Lee et al. [18] who reported 63.2% turbidity reduction at the filtration rate of 1.5 m/h with a full scale roughing filter. Also, gradually decreasing trend of outlet values is observed at three filtration rates (Fig. 4) which supports findings of Wegelin [4]. The experiments of Reed et al. [19] performed with polystyrene beads as media of HRF showed ca. 45% removal of turbidity which was much lower than those observed in this study. It may be due to the low surface area of polystyrene beads compared with gravel media.

It can be observed from Fig. 3a that increasing the V_f from 0.5 to 1 m^3/(m^2·h) tend to 24% percent decrease in COD removal efficiency. During the first experimental run (V_f 0.5 m^3/(m^2·h)), the mean COD removals was 66.3%. In the third experimental run (V_f 1.5 m^3/(m^2·h)), the mean COD removals dropped to 40.3%. As noted in Table 3, the average effluent COD concentrations were 64, 112.6 and 113.3 mg/dm^3 for V_f 0.5, 1.0 and 1.5 m^{3}m^{-2} h^{-1}, respectively. Al-Saed et al. [11] during a pilot experiment lasting four months (from March through August) on effluent of a facultative pond with a horizontal rock filter, found that 80.5% removal of COD was occurred [11].

3.2. REMOVAL OF TOTAL AND FECAL COLIFORMS

The data on influent and effluent coliform loads are presented in Tables 1 and 3, respectively. As can be seen from Fig. 3b, HRF removed on average of 0.94 log of the influent total coliforms during the first filter run (0.5 m^{3}m^{-2} h^{-1}). During the subsequent filter runs, the coliform removal efficiency significantly decreased ($P < 0.05$) and reached 0.74login the third filter run (1.5 m^3/(m^2·h)). Also, Figure 3b shows the 0.16log decrease in fecal coliform removal by increasing the V_f from 0.5 to 1.5 m^3/(m^2·h). Ochieng et al. [20] found 82% of total coliform removal from surface raw water with a horizontal roughing filter.

As illustrated in Figure 5, almost 5 weeks after starting the first filter run, the mean effluent coliform concentrations (both total and fecal) reached steady state. The stable HRF effluent condition in the second and third runs were much shorter than in the first run (almost two weeks). It may be due to the lack of biological layer on the gravel surfaces at the first filter run which has been attributed to the improvement of removal efficiency in the previous studies [6, 21].

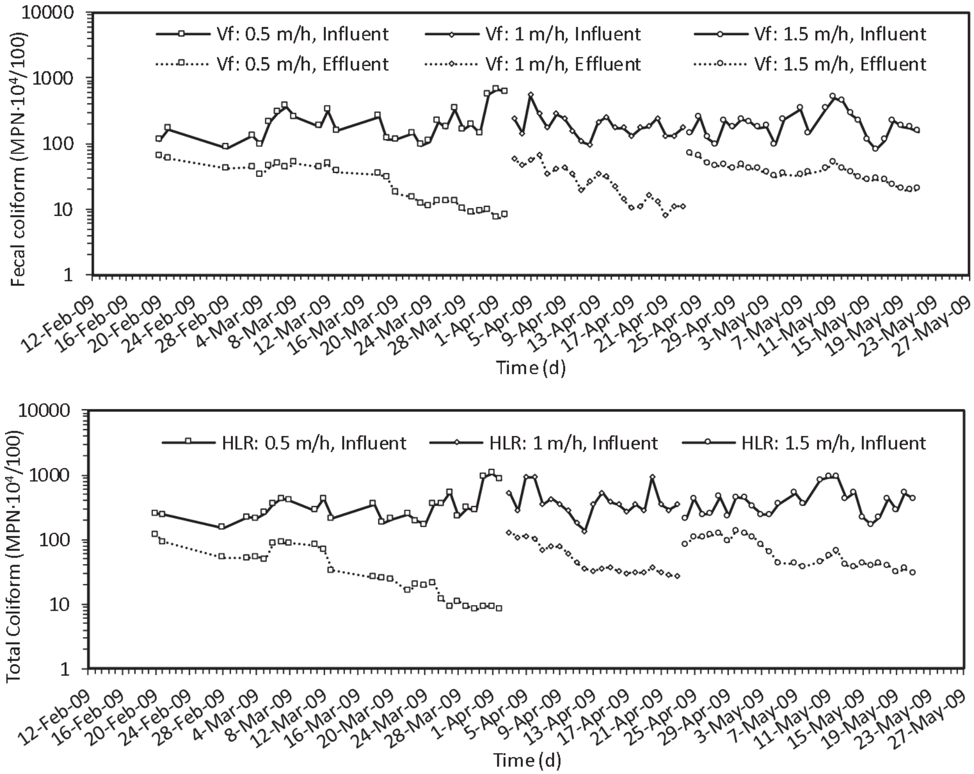

Fig. 5. Concentrations of a) fecal coliforms and b) total coliforms from influent and effluent of HRF for three subsequent V_r

In the study of Ochieng et al. [20], the steady concentration of the effluent coliform occurred after 20 days of the filtration run. Al-Saed et al. [11] found 2 log decrease in fecal coliforms after 4 month filtration run with a horizontal rock filter which was used for stabilization pond post treatment. Barman et al. [12] found 90–98% removal of total coliform by HRF which was served as a pretreatment unit for slow sand filter. Galvis et al. [5, 22] governed pilot studies of various roughing filter configurations (horizontal flow, up flow and down flow) which reduced fecal coliform bacteria by 86.3%. Dastanai et al. [13] reported 94% removal of total coliforms with a HRF installed beside a riverbank.

3.3. PREDICTION THE EFFLUENT TSS OF HRF BY APPLYING THE E_n VALUE CONCEPT

By using the table and graphical nomogram developed by Wegelin [10], the values of E_n attributed to each HRF compartments were obtained from Eq. (4) and presented in Table 4, where Lf is the compartment filter length, dg – gravel size. The total values of E attributed to each filtration rate were calculated by multiplying three values of E_n of the respective filtration rates.

Table 4

Values E_{total} for various filtration rates attributed to each filter compartment

V_f [m³/(m²·h)]	d_g [mm]	L_f [m]	E_n [%]	E_{total} [dec]
0.5	5	1	15.2	0.0146
	10	1.3	28.73	
	15	1.6	33.46	
1	5	1	39.9	0.131
	10	1.3	54.62	
	15	1.6	59.98	
1.5	5	1	59	0.333
	10	1.3	72.48	
	15	1.6	77.9	

The predicted TSS final concentrations of HRF at three filtration rates are illustrated in Fig. 6. The values of predicted suspended solids content at the filtration rate of 0.5 m/h are much lower than the observed (measured) values.

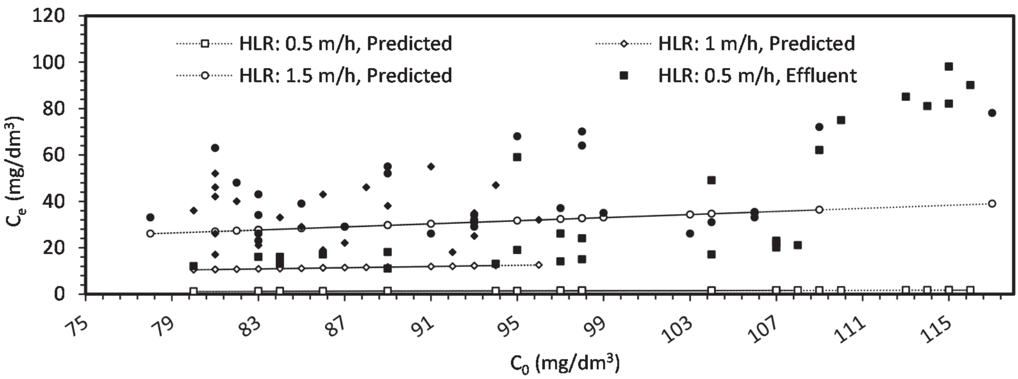

Fig. 6. Observed versus predicted TSS concentrations in HRF effluent for various filtration rates

These differences may be due to the lack of further solid deposition in the first filter run which tend to decrease the filtration performance [5, 6]. At the two sequence filtration rates the values of predicted TSS concentration in the filter effluent were higher and so a noticeable compliance with the observed results was achieved. Similar results have also been observed by Ochieng et al. [20, 23]. Nkwonta et al. [24] reported the total value of *E* of 0.026 for the filtration rate of 0.75 m/h and for three compartments of length equal to 0.45 m.

3.4. HEAD LOSS DEVELOPMENT AND REMOVAL EFFICIENCY AT THE FIRST FILTRATION RATE

From Figure 7, it can be observed that at the beginning of the first filtration run (V_f = 0.5 m/h), TSS removal was less than 20% but after five weeks, the removal efficiency of 80% was achieved. This improvement can also be observed for the other parameters including turbidity, COD, total coliforms and fecal coliforms. The gradually increasing HRF head loss has a noticeable compliance with the removal efficiencies of measured parameters. It can be seen from Fig. 7 that 0.15 cm/d increase of the head loss was developed during the period of first filtration rate (V_f = 0.5 m/h). However, measured daily head loss is less than those of Pacini et al. [25] and also Ochieng et al. [20] experiments, in which, they found the 0.8 cm/d and 0.66 cm/d values. These differences may be due to the use of synthetic HRF concentrate influent in those studies while current study relied on diluted aerated lagoon effluent.

Results of analysis of the Spearman correlation coefficient point to strong relationship ($P < 0.01$) between the removal efficiency of measured parameters and HRF head loss (Table 5). The correlation coefficient between HRF head loss and removal efficiencies for all parameters ranged from 0.578 to 0.968 which revealed a noticeable direct relationship. Biofilm formation on gravel surfaces probably was the major reason for efficiency improvement [6]. Based on Cleary et al. [21], in the case of roughing filtration, growth of biofilm on the media decreases its interstitial pore space, which may increase the collection efficiency.

Table 5

Results of analysis of the Spearman correlation coefficients (CC) for removal efficiency of measured parameters and HRF head loss

Vf	TSS		Turbidity		COD		TC		FC	
[m^3/(m^2·h)]	CC	*P*	CC	*P*	CC	*P*	CC	*P*	CC	*P*
0.5	0.932[a]		0.917		0.924		0.965		0.927	0.000
1.5	0.968	0.000	0.837	0.000	0.891	0.000	0.732	0.000	0.853	
2.0	0.823		0.873		0.942		0.989		0.578	0.002

[a]Correlation is significant at the 0.01 level (2-tailed).

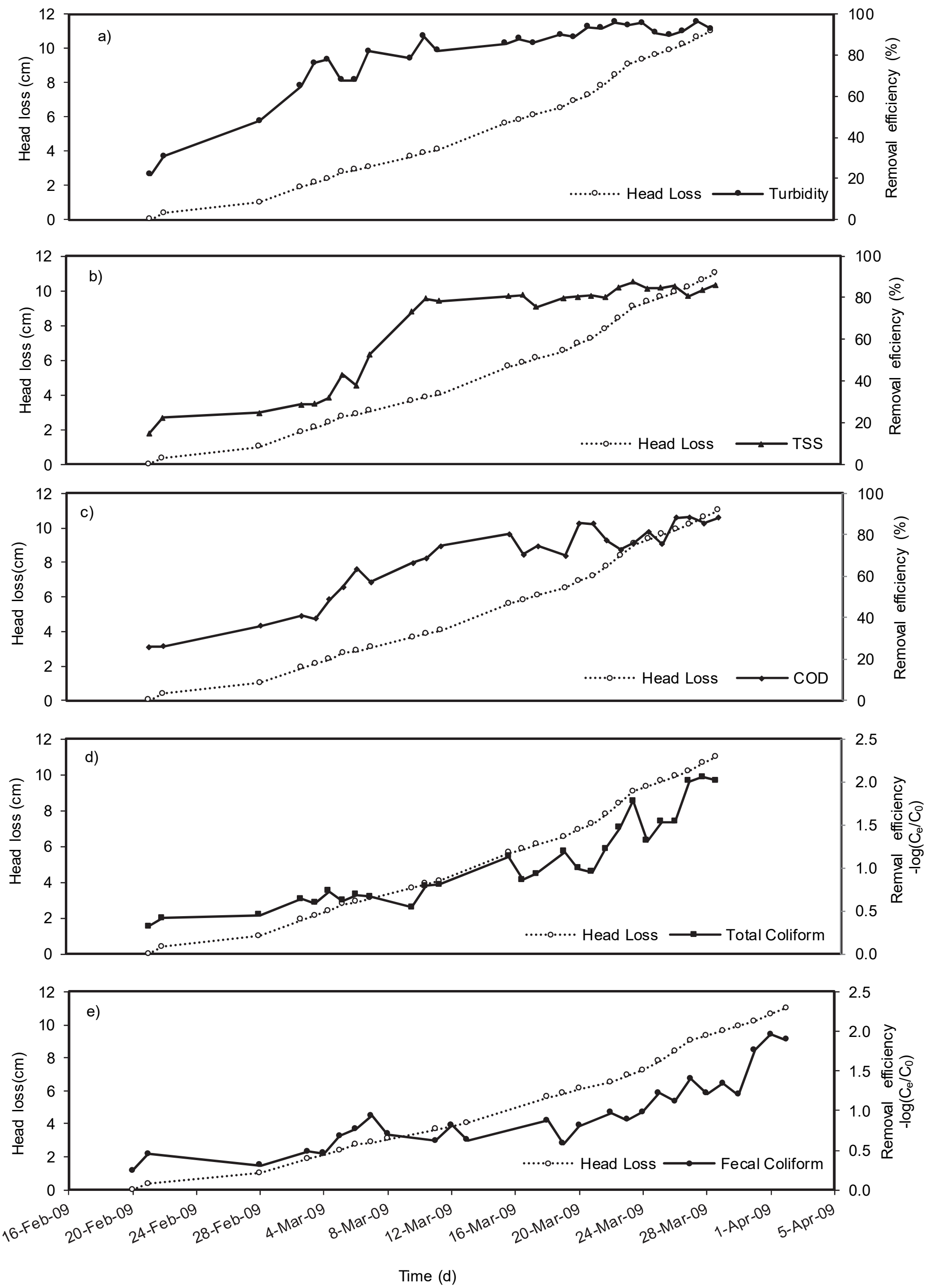

Fig. 7. Removal efficiencies of a) turbidity, b)TSS, c) COD, d) total coliforms, e) fecal coliforms compared with HR_f head loss at $V_f = 0.5$ $m^3/(m^2 \cdot h)$

4. CONCLUSIONS

HRF is a low cost technology for upgrading the physical and biological quality of turbid surface waters. Due to the relatively plain structure, easy designing and construction, availability of construction material and simple operation, HRF is considered an appropriate candidate for removal of water turbidity especially in developing countries. This research is an effort to study the filtration of aerated lagoon effluent by using the HRF. The following main conclusions were derived from the study:

- At the first filtration rate ($V_f = 0.5$ m/h), the removal of turbidity, TSS, COD, total coliforms and fecal coliforms were 81.1%, 63%, 66.33%, 0.94 log and 0.91 log, respectively.
- For the effluent of HRF, there were significant differences between turbidity, TSS and COD average loads in three filtration rates ($P < 0.05$) but these differences were not significant for the influent of HRF ($P = 0.23$).
- Prediction of the effluent TSS of HRF by applying the value of E_n concept represents that increasing the filtration rate from 0.5 to 1.5 m/h tends to decrease the distances between the values of observed and predicted TSS in the filter effluent based on the Wegelin model.
- There is a significant ($P < 0.01$) relationship between the removal efficiency of measured parameters and HRF head loss alteration. Also the correlation coefficients between HRF head loss and removal efficiencies for all parameters reveal a strong direct relationship.
- By achieving TSS final effluent of 37 mg/dm^3, turbidity of 14 NTU and COD of 64 mg/dm^3, the HRF system seems to be a viable option as an upgrading technique for aerated lagoon effluents.
- Although by using of HRF pilot system almost 1 log of coliforms removal were achieved but the HRF final effluent still contained around 10^8 cfu/100 cm^3. Hence, using disinfection alternatives along with HRF systems are recommended for future experiments.

ACKNOWLEDGMENTS

This work was supported by Tehran University of Medical Sciences, School of Public Health (TUMS SPH). The authors express their gratitude to the laboratory staff of TUMS-SPH and also to the experts of water and wastewater organization of Qom province (ABFA-Qom) for their advices and assistance with this study.

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