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An experimental investigation on the hydraulic behavior of declining rate filtration

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ABSTRACT

Hydraulic behavior of declining rate filtration (DRF) was investigated by means of pilot-scale experiments. A bank of four declining rate filters was constructed and operated. The common water level in the filters, individual filter velocities, and head losses in the filters were carefully monitored and recorded as functions of time. Many of the previous studies on DRF employed either a single average filtration velocity or a single coagulant dosage. In this study, however, filter runs were repeated treating the same water at several different filtration rates and coagulant dosages. Using such an experimental matrix of several different rates and coagulant dosages allowed an evaluation of the behavior of the declining rate filtration system under different operating conditions and the effects of the mentioned variables on various hydraulic characteristics of the DRF system. The validity of certain simplifying assumptions used in design calculations was also tested in these experiments. It is believed that the new data presented herein will lead to a better understanding of DRF.

Keywords: Constant rate filtration; Declining rate filtration; Filtration; Water treatment

1. Introduction

Declining rate filtration (DRF), which is also known as variable declining rate filtration (VDRF), is a well-established modern filter control method and is described in detail in a number of Refs. [1–6]. In a DRF system, the filters in the bank are interconnected by means of a submerged inlet channel or pipe. There are no influent or effluent control devices on the filters. Flow to the filters enters below the minimum water level and there is free communication of flow between all the operating filters. As a result, all the operating filters have approximately the same water level. At any instant in time, therefore, the dirtiest filter operates at the lowest filtration rate and the cleanest filter operates at the highest filtration rate in the bank.

The water level variations in VDRF are shown in Fig. 1. (Level-1, the level that would result if all filters are operated clean at the design average rate, is not shown since it is not relevant in what follows.) Levels 2–4 are as defined by Cleasby and Di Bernardo [7] and Level-5 was defined by Akgiray and Saatçı [8]. Level-2 is the minimum water level. As the filters in the bank get clogged, the water level rises slowly to Level-3. At that point, the dirtiest filter is taken off-line to be backwashed. When that one filter is being backwashed, the water level in the operating filters

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rises at a faster rate to Level-4. Next, when the inlet to the newly backwashed filter is opened, the water level quickly drops to Level-5. This sharp drop in water level after the backwash is due to the redistribution of the accumulated water from the working filters to the clean filter. After equalizing at Level-5 quickly, the water level would continue to fall more slowly to Level-2. The water level starts rising again at that point, and keeps changing periodically between the minimum (Level-2) and the peak (Level-4) water levels. It should be noted that Fig. 1 illustrates the water level variations after all the filters have been in service long enough to have been backwashed at least once, and not during a start-up period.

There have been a number of attempts to model and predict the hydraulic behavior of a DRF system. Of particular interest is the prediction of water levels that occur during the operation of a DRF plant. Cleasby [3], Cleasby and Di Bernardo [7], Arboleda et al. [9], Di Bernardo [10], and Gupta and Hayes [11] presented approximate calculation methods based on various simplifying assumptions and/or experimental observations. Akgiray and Saatçi [8] and Chaudhry [12] presented mathematical models that predict not only water level variations, but also filtration rates and effluent suspended solid concentrations as functions of time. The use of these latter mathematical models, however, necessitates the collection of pilot or full-scale filtration data to determine the appropriate attachment and perhaps detachment coefficients for the particular suspension being filtered. Furthermore, rate dependence of these coefficients may have to be determined as well. These requirements and the complexity of solutions limit the use of such models in routine plant design.

The calculation method described by Cleasby [3] is probably the simplest, the most practical and the most widely used method for DRF plant design. This calculation method employs the following assumptions: (1) Level-5 is the same as Level-3. (2) The maximum rate (the rate of the cleanest filter) is always selected as $V_{max} = 1.5\,V_{av}$, and the minimum rate (the rate of the dirtiest filter) is then assumed to be $V_{min} = 0.7\,V_{av}$. Here, V_{av} denotes the average (i.e. design) filtration rate. (3) The slope of the head loss curve for a bank of several DR filters operating at a particular average rate is the same as the slope for a CR filter operating at the same rate. This last assumption—which was also employed by Gupta and Hayes-—was critically examined by Akgiray and Saatçi [4] using theoretical arguments and it was concluded that this assumption cannot be generally valid. The other two assumptions were discussed in detail by Akgiray and Saatçi [5] who argued that there is no reason why these assumptions should be generally valid. They did not, however, present any new experimental data to check the validity of the mentioned assumptions.

Fig. 1 is based mainly on the work of Cleasby and his co-workers [3,7,13–15], and the computer simulation results are obtained by Chaudhry [12] and Akgiray and Saatçi [8]. Fig. 2 illustrates the rate of an individual filter as a function of time. A newly backwashed filter starts at the maximum rate and the rate of this filter declines whenever another filter in the bank is backwashed. This figure (the curve with solid lines) shows constant rates between backwashes and a stepwise decrease in rate as the filter in question gets clogged. This description is based mostly on the work of Cleasby and his co-workers [3]. The dashed lines are based on computer simulation models by Chaudhry [12] and Akgiray and Saatçi [8]. These models predict non-constant (non-linear and declining) rate profiles between backwashes, especially during the initial stages after a backwashed filter is put back into service. After a few backwashes (of other filters), the rate profile takes a more horizontal, stepwise appearance according to the calculated results of these models.

While the type of behavior illustrated in Figs. 1 and 2 (solid lines) as well as the information given in the above paragraphs have received general acceptance in the DRF literature, and there are a number of questions that remain unanswered or only partially answered and a number of findings that require further verification by means of tests under different

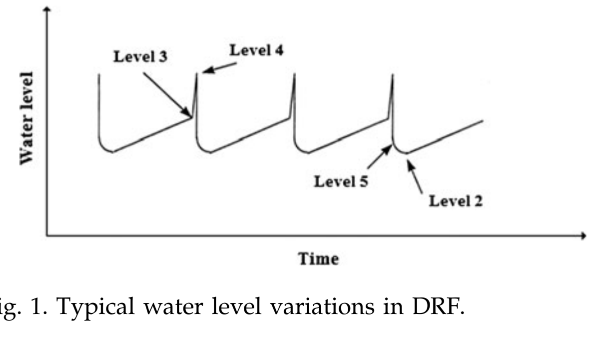

Fig. 1. Typical water level variations in DRF.

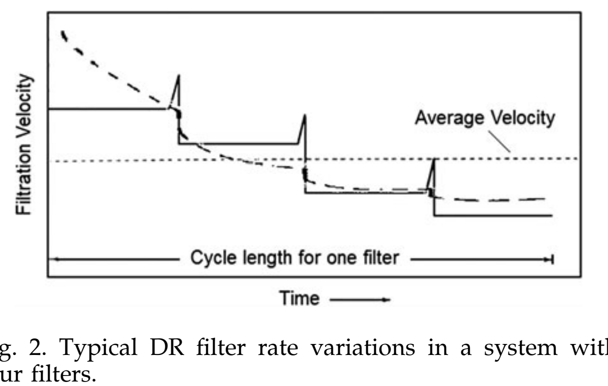

Fig. 2. Typical DR filter rate variations in a system with four filters.

conditions. The goal of this work was to study the following aspects of DRF:

(1) Are the water level profiles always as described in Fig. 1, i.e. an approximately straight line from Level-2 to Level-3, another straight and much steeper line from Level-3 to Level-4, a sharp and quick drop from Level-4 to Level-5, a further slower drop from Level-5 to Level-2?
(2) What is the position of Level-5 with respect to other levels?
(3) Are Level-3 and Level-5 equal, as assumed by Cleasby [3]?
(4) How are the various water levels influenced by average (i.e. design) filtration rate?
(5) How are the mentioned water levels affected by coagulant dosage (suspended solids load on the filters)?
(6) How does the rate of an individual filter change as a function of time? Is it always constant between backwashes? Does it always decrease in a stepwise manner? What are the effects of coagulant dosage and average filtration rate on the individual filter rate profile?
(7) What is the relationship between the maximum and minimum filtration rates? Are they symmetric about the average rate or is there another relationship?
(8) Is it generally true, as assumed by Cleasby [3], that the minimum velocity is about 0.7 times the average velocity when the maximum velocity is 1.5 times the average velocity?
(9) Are the slopes of the head loss curves for DRF and constant rate filtration (CRF) the same, as assumed by Cleasby [3] and Gupta and Hayes [11]?

The experimental set-up and the tests carried out to investigate these questions are described in the following section.

2. Experimental

2.1. Preparation and storage of the raw water

Mikron's Kaolin 5 (Mikron Company, Istanbul, Turkey) was mixed with tap water for the preparation of the raw water used in the study. The water prepared in this way had a turbidity of 10 NTU and was stored in a 3 m^3 polyethylene tank. The water in the tank was mixed at a speed of 300 rpm with a turbine mixer located in the tank to prevent sedimentation and to maintain a uniform turbidity in the tank. Raw water in the tank was replenished semi-continuously as the water was used in the filtration experiments.

2.2. Properties of the filter medium

The silica sand used in the experiments is the sand used in all the water treatment plants in Istanbul and was obtained from İSKİ (Istanbul Water and Sewerage Administration) Kağıthane Water Treatment Plant. The sand had an effective size of 0.82 mm and a uniformity coefficient of 1.45. Exactly the same amount (6 kg) of sand was placed in each column, giving a bed height of 109 cm in each filter. This height was preserved during filtration runs and after each backwashing.

2.3. Pilot filters

Four plexiglas columns, each with an internal diameter of 7 cm, were used as filter columns. Another 7 cm-ID column was employed as the "inlet main" to feed the raw water into the four DR filters. A spare fifth filter column was also constructed, but only four parallel DR filters were used in the tests described here. A picture of the pilot system is given in Fig. 3 and a schematic display of a filter can be seen in Fig. 4. The raw water flowed from the vertical distribution column into a horizontal 7 cm-ID pipe and then from this pipe to the filters with four equivalent distribution pipes each with a 2.5 cm internal diameter and 40 cm length. These distribution pipes were sloped towards the filters (with an angle of 60° with respect to the horizontal) so as to make sure that any sand that entered into these pipes during backwashes settled back into the filter columns.

2.4. Filtration experiments

The raw water was pumped from the 3 m^3 storage tank to the top of the vertical distribution column. The coagulant (alum) was injected into the line connecting the storage tank to the vertical distribution column. The water then flowed downward by gravity within the distribution column which is open to the atmosphere. This column served as the common inlet channel for the DRF system. The water level in the distribution column was always the same as the water level in the operating filters during the filtration experiments. This was made possible by keeping the head losses in the connections from the distribution column to the filters negligibly small. The total water inflow flow rate to the DRF system was adjusted to the desired value by means of a gate valve and rotameter that were placed at the inlet side of the DRF system. The raw water entered 11 cm above the sand bed by means of inclined distribution pipes. The common water level in the filters remained above the

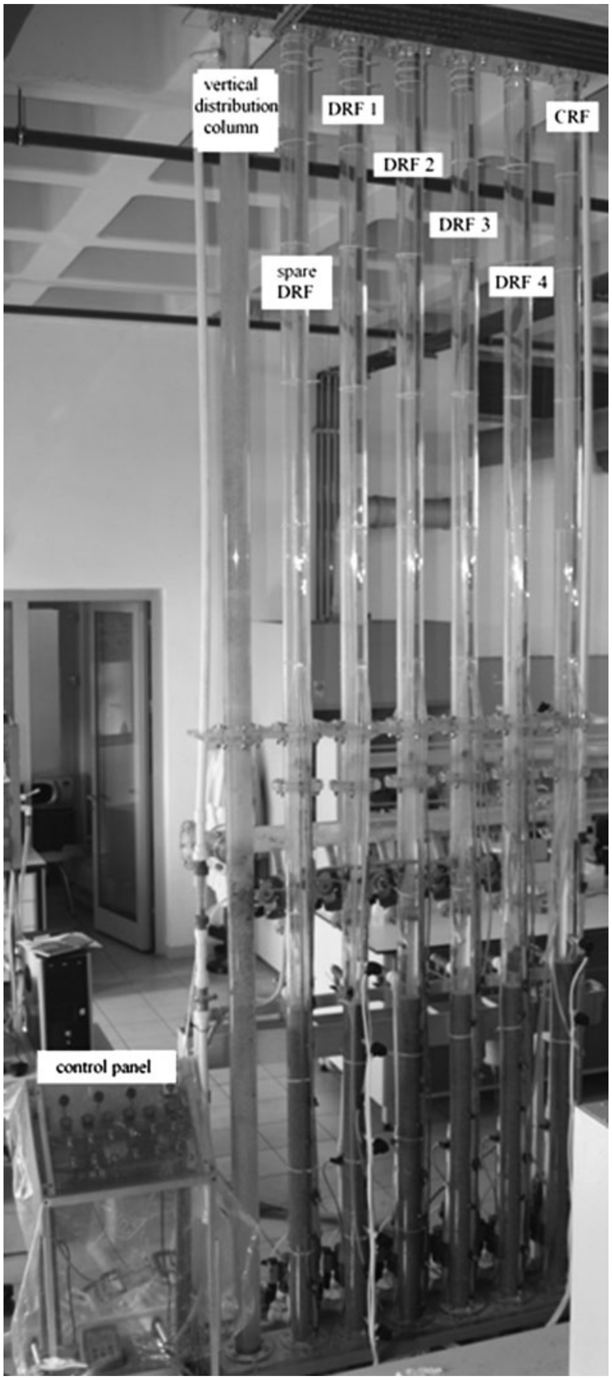

Fig. 3. The pilot DRF system used in this work.

inlet pipes and above the bottom of the horizontal distribution pipe throughout the filtration experiments so that the filters always remained interconnected. The filter columns were also open to the atmosphere. The horizontal filter outlet pipes turn vertically upwards with 90˚ bends and then are directed horizontally again with another set of 90˚ bends. With this construction, the outlet water level was the same as the level of the top of the sand bed. This design prevents negative pressures and air-binding in the filters and is consistent with typical DRF system design [1,3]. The output of each filter flowed into a common open channel. The outputs of the individual filters are combined and mixed in the mentioned open channel. This common outlet channel was sloped towards one side and the mixed output was collected at this side of the channel.

The pressure at five points in one of the DRF filters was measured at every 5 s by means of five pressure sensors. The positions of the sensors on the filter are shown in Fig. 4. VISION 2000 turbine type flow meters (Remag AG, Bern, Germany) were mounted on the output line of the each filter to measure the rate of each filter at every 5 s. These and the pressure measurements were recorded on the computer.

Tap water stored in a 2 m^3 tank was pumped to backwash each filter. The bottom of each filter contains a 10 cm high section filled with 0.5 cm diameter glass balls to prevent jetting action and to provide a homogenous flow of backwash water into the sand bed. The sand bed and the glass balls are separated by a support grid (sieve) with 0.5 mm openings. Each filter was backwashed once every 24 h in this work. As there are four filters in the DRF system, one filter was backwashed after every six hours. Dirty backwash water is collected by means of pipe connections located 71 cm above the top of the filter sand. Each filter was backwashed in exactly the same way, for a period of 15 min and with a backwash rate of 35 m/h. The bed expansion was about 23% under these conditions. Following backwashing, each filter was brought to the fixed-bed depth of 109 cm which remained constant during the filtration process.

To compare the slopes of the head loss curves for CRF and DRF, a single CRF was constructed and operated in parallel with the DRF system (see Fig. 3). The CR filter was constructed so that it was physically equivalent to the DR filters, using the same column structure and the same filter medium. The CR filter received exactly the same raw water as input and it was operated at the design (average) rate of the DR system. This means that the inflow rate to the CR filter was one-fourth of the total input rate to the DRF system. The rate, the head loss, and water level in the CR filter were monitored and recorded continuously during the filtration runs. The CR filter was backwashed once every 24 h with the same backwash velocity and duration as the DR filters.

The DRF system was first tested without the filter sand to ensure that filters and their connections are physically equivalent. This was checked by verifying

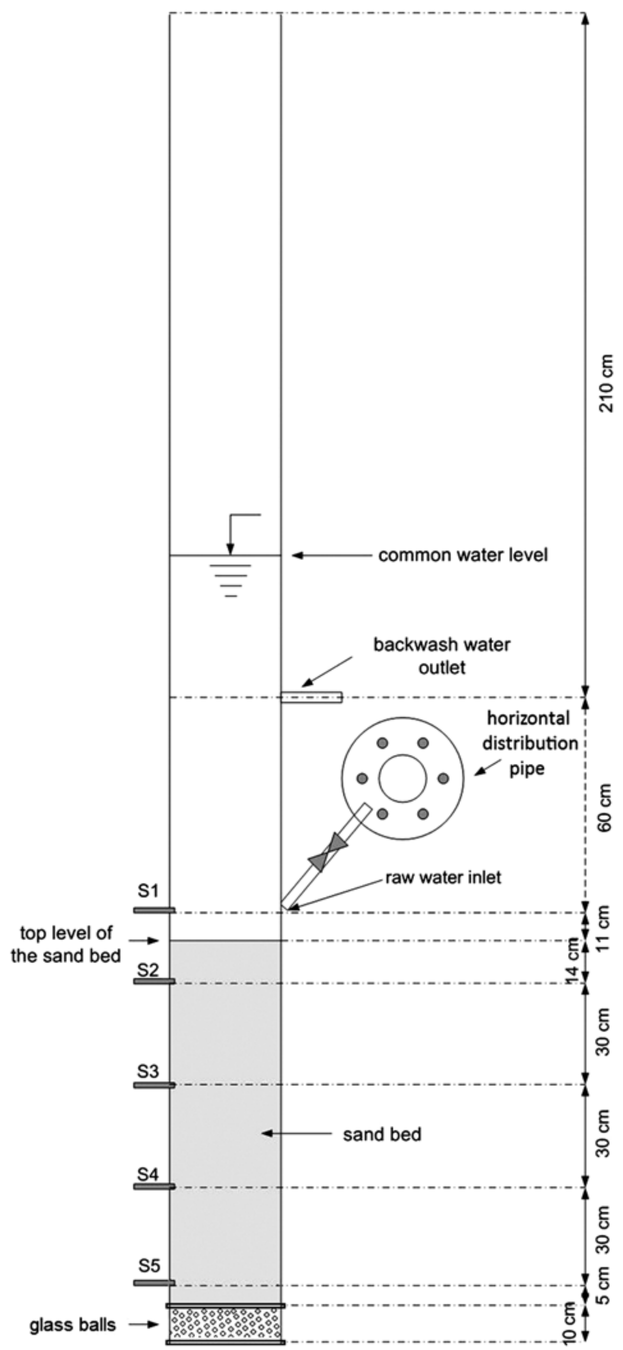

Fig. 4. Schematic side view of the pilot plant (S1–S5 are pressure sensors).

that water levels and flow rates were the same in all the filters when operated without the filter medium. Next, the filters were filled with the sand described above. The filters were tested again with clear water and without a coagulant. When operated in this manner, the filters do not clog and the common water level remains constant when the total input rate remains constant. The system was operated in this way with several different total inflow rates, and it was verified that the rates and levels were the same in all the filters and that the inlet distribution column and connections functioned as expected. Trial filtration tests were then started using the 10-NTU turbidity influent water described above. Several different rates and coagulant dosages were tried to evaluate the behavior of the system and to determine the water level variations to be expected at different coagulant dosages and with different inflow rates. Following these preparations, 18 trial runs were carried out under carefully controlled conditions. In all the mentioned runs, the constant rate filter was operated in parallel with the DRF system and received exactly the same influent water. These runs are listed in Table 1. The water temperature remained at 20.5 ± 3.5℃ in the tests. Water level variations, head losses, and individual filtration rates were recorded. Each trial lasted about a week, encompassing at least three filtration cycles (and three backwashes) for each filter with the exception of Test # 18 listed in Table 1. The data presented herein are based on about six months of intensive tests carried out in this manner.

3. Results and discussion

A large amount of data was collected in this work (cf. the experiments listed in Table 1) and it is not

Table 1
Experimental matrix

Experiment	Raw water turbidity (NTU)	Alum dosage (mg/L)	Average velocity (m/h)
1	9.9	5	7.8
2	10.0	5	10.5
3	10.0	5	12.6
4	10.0	5	14.7
5	10.6	5	17.0
6	10.0	7.5	7.9
7	10.0	7.5	10.6
8	10.0	7.5	14.6
9	10.0	7.5	17.7
10	10.0	10	7.9
11	9.9	10	10.6
12	10.0	10	12.5
13	10.0	10	14.7
14	9.4	10	17.1
15	9.9	12.5	7.8
16	9.9	12.5	10.6
17	10.0	12.5	14.7
18	9.3	12.5	17.1

possible to show all the data in graphical form. Sample results are shown in Fig. 5 through Fig. 6. The results that could not be shown graphically are reported in Table 2. Figs. 5–10 and the results in Table 2 are discussed in the following paragraphs.

The water level values shown in Figs. 5(a) and 6(a) have been calculated using the elevation of the top horizontal section of the outlet pipe as the reference level (this corresponds to an effluent weir in a full-scale DRF plan). The top of the sand also coincides with the outlet water level.

Fig. 5(a) displays the water level changes observed during the second cycle of four different experiments (Tests 6–9). Here, water levels are plotted as functions of time. Alum dosage is 7.5 mg/L and each curve in the figure corresponds to a different filtration rate. For a fixed coagulant dosage, as is the case in these sample results, higher average rates always resulted in higher water levels in the filters. This is to be expected because higher rates result in higher laminar and turbulent head losses in the media, the underdrains, and the outlet piping. Fig. 6(a), in turn, shows the water level changes observed during the second cycle of four other experiments (Tests 1, 6, 10, and 15). Each curve in this figure represents a different coagulant dosage, the rate being in the range 7.8–7.9 m/h for all the curves. Here, it is seen that the water level increases as the coagulant dosage is increased at a given average rate. This again is to be expected because higher coagulant dosages result in higher clogging head losses in the media.

Fig. 5(b) shows the individual filter rate vs. time values (for Filter # 1) observed during the second cycle of the experiments (Tests 6–9) on which Fig. 5(a) is based. Similarly, the sample filter rate data displayed in Fig. 6(b) correspond to the water level data of Fig. 6 (a) (Tests 1, 6, 10, and 15). As can be seen in the sample data of Fig. 5(b), individual filter rate profiles do not change in a noticeable way as average rate is increased. Of course, the magnitudes of individual rates increase as the average rate increases. As shown in the sample curves in this figure, rates generally declined in a stepwise manner starting at a maximum rate (observed when the filter is newly backwashed) and ending at a minimum rate (observed when the filter in question is the dirtiest filter in the bank). This is in general agreement with previously observed behavior of DR filtration. On the other hand, the mathematical models used by Chaudhry [12] and Akgiray and Saatçi [8] predict non-constant (declining) rates between backwashes (dashed line in Fig. 2). According to these models, rate of a filter declines with a

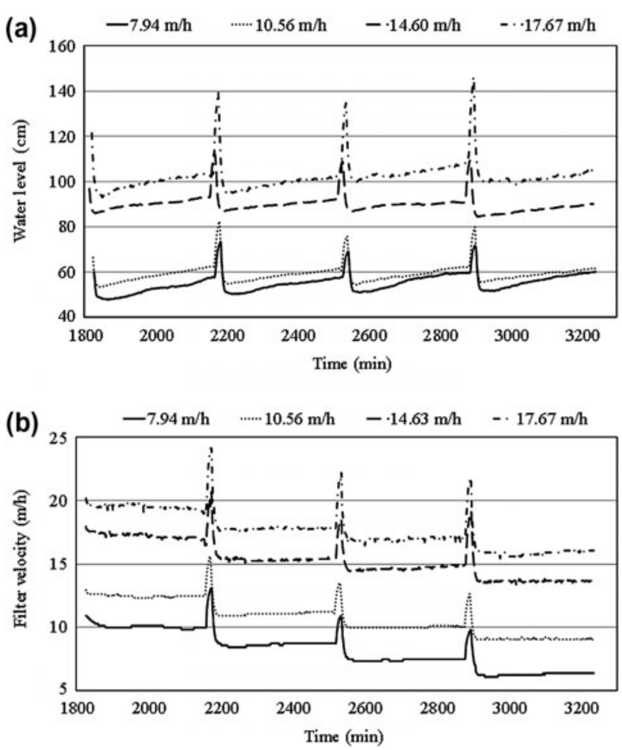

Fig. 5. Effects of flow rate (a) on water level and (b) individual filter rate changes (alum dosage: 7.5 mg/L).

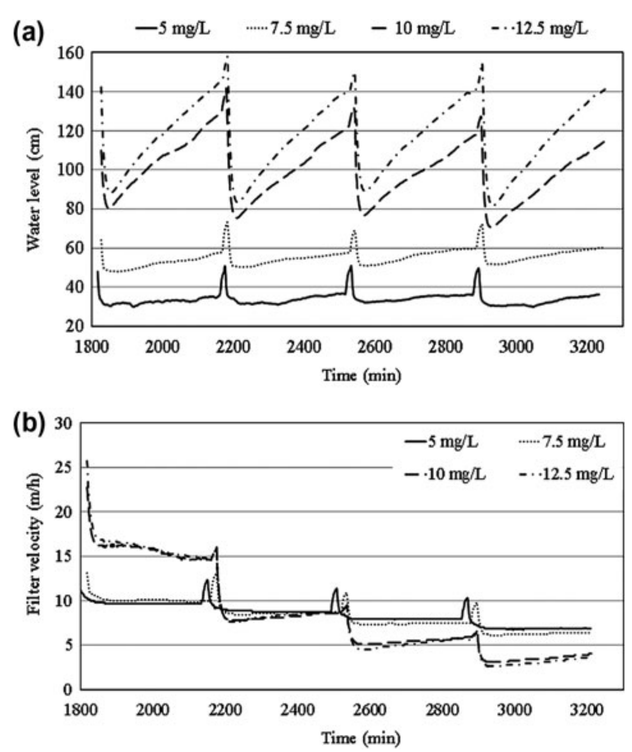

Fig. 6. Effects of coagulant dosage (a) on water level and (b) individual filter rate changes (average filtration velocity: 7.8-7.9 m/h).

Table 2
Relations between the peak, the maximum, the minimum and the average filtration rates and maximum/minimum water levels

Exp.	V_{peak}/V_{av}	V_{max}/V_{av}	V_{min}/V_{av}	V_{max}/V_{min}	h_{max}/h_{min}	h_3-h_2 (cm)	h_3 (cm)	h_4 (cm)	h_5 (cm)
1	1.53	1.36	0.83	1.64	1.14	4.5	36	51	52.8
2	1.39	1.16	0.79	1.47	1.09	5.5	65	84	79.2
3	1.45	1.16	0.83	1.40	1.09	6	75	101	92.8
4	1.36	1.13	0.95	1.19	1.07	6.5	100	130	116
5	1.33	1.07	0.90	1.19	1.07	7	103	142	125.6
6	1.64	1.37	0.79	1.75	1.16	8	58	71	68.8
7	1.48	1.21	0.84	1.44	1.13	7.3	61.5	78	74.4
8	1.42	1.21	0.92	1.31	1.08	7	92	110	100
9	1.37	1.13	0.90	1.26	1.07	7	104	141	124.8
10	2.00	1.97	0.43	4.62	1.60	46	123	134	119.2
11	2.00	1.83	0.51	3.57	1.63	59	153	171	148.8
12	1.85	1.60	0.62	2.57	1.57	56	155	169	147.2
13	1.59	1.40	0.69	2.03	1.23	28	148	180	156
14	1.70	1.52	0.62	2.44	1.40	56	195	226	192.8
15	1.80	1.53	0.40	3.83	1.68	57	141	156	136.8
16	1.76	1.54	0.46	3.32	1.41	52	180	210	180
17	1.73	1.45	0.56	2.56	1.45	70	227	247	209.6
18*	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

*In experiment 18, with 12.5 mg/L of alum at 17.1 m/h average filter rate, both the CR and the DR filters had to be stopped after 12 h because the available head was exhausted.

relatively steep slope in the first stage after backwashing, the slope of the rate curve decreases after each backwash of another filter, the rate curve becoming almost constant (taking a stepwise appearance) after a few such backwashes. There was one feature of the rate curves observed in the present study that is in partial agreement with the mentioned model predictions: While the rate remained almost constant (the curve being almost horizontal) between backwashes after the first backwash ("the first backwash" here refers to the backwash of another filter after the filter under consideration is put back into service), the rate tended to decline during the very first stage after a clean filter was put back into service and until the next filter was backwashed. This decline was more pronounced in some runs, and less noticeable in others. In general, such a non-constant (declining) first-stage rate profile tended to occur at higher dosages. It should also be added that both Chaudhry [12] and Akgiray and Saatçi [8] employed a simple filtration (kinetic) model with a constant attachment coefficient. The use of more sophisticated models, for example a model with rate-dependent attachment and detachments coefficients, may be explored to see if better agreement with the experimental observations can be attained.

The curves in Fig. 6(b) show the effect of coagulant dosage on individual filter rates at a given average filtration rate. Higher dosages lead to greater variations in filtration rate. It should be remembered that the filtration experiments in the present study were conducted using a fixed filter cycle length (i.e. backwashing each filter once every day). This means that, at a given average rate, higher coagulant dosages will lead to more completely clogged filter beds at the end of filter cycles.

Fig. 7(a) displays the observed relations between the maximum and the minimum filtration rates and the average filtration rate at a fixed alum dosage of 7.5 mg/L. This figure is based on the same experiments as Fig. 5 (Tests 6–9). Here, V_{max} and V_{min} represent the rates of the cleanest and the dirtiest filters in the bank, respectively. V_{peak} is the peak rate of the cleanest filter, i.e. the rate when another filter is being backwashed. It is seen that, the ratios V_{peak}/V_{av} and V_{max}/V_{av} both decreased as average flow rate increased. When considered together with three other similar figures (each for a different coagulant dosage and not shown here because of space limitations), it is observed that the ratio V_{min}/V_{av} increased as average velocity was increased. The ratio V_{max}/V_{min} in general decreased as average (design) filtration rate increased. This ratio was less sensitive to velocity at low coagulant dosages. V_{max}/V_{min} tended to increase when coagulant dosage was increased.

Fig. 7(b) shows the observed relations between the maximum and minimum filtration rates and the

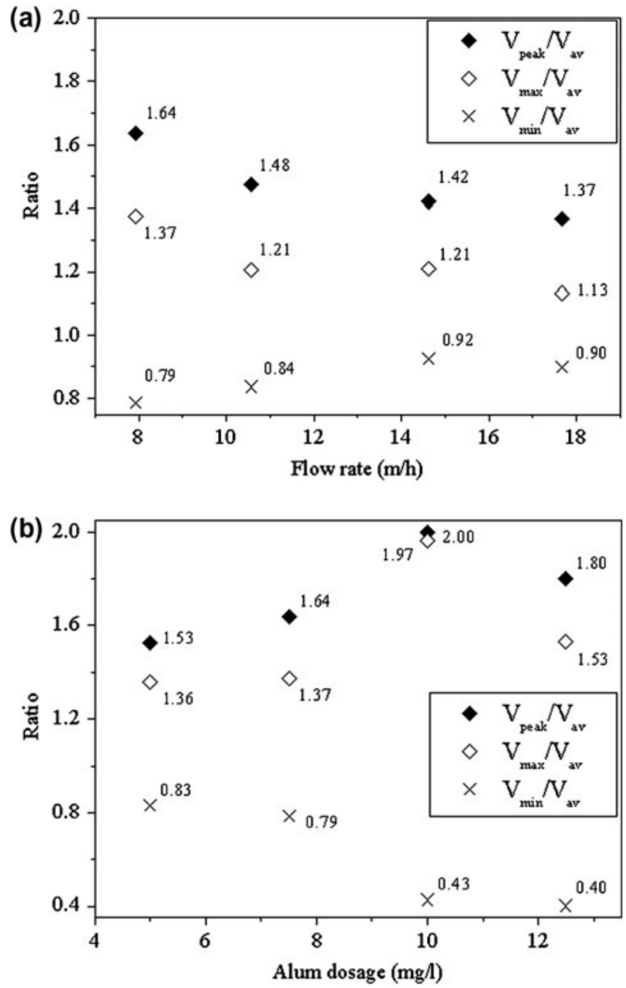

Fig. 7. Relations between the peak, the maximum, the minimum, and the average filtration rates (a) as functions of the average filtration rate (coagulant dosage: 7.5 mg/L) and (b) as functions of the coagulant dosage (average filtration velocity: 7.8–7.9 m/h).

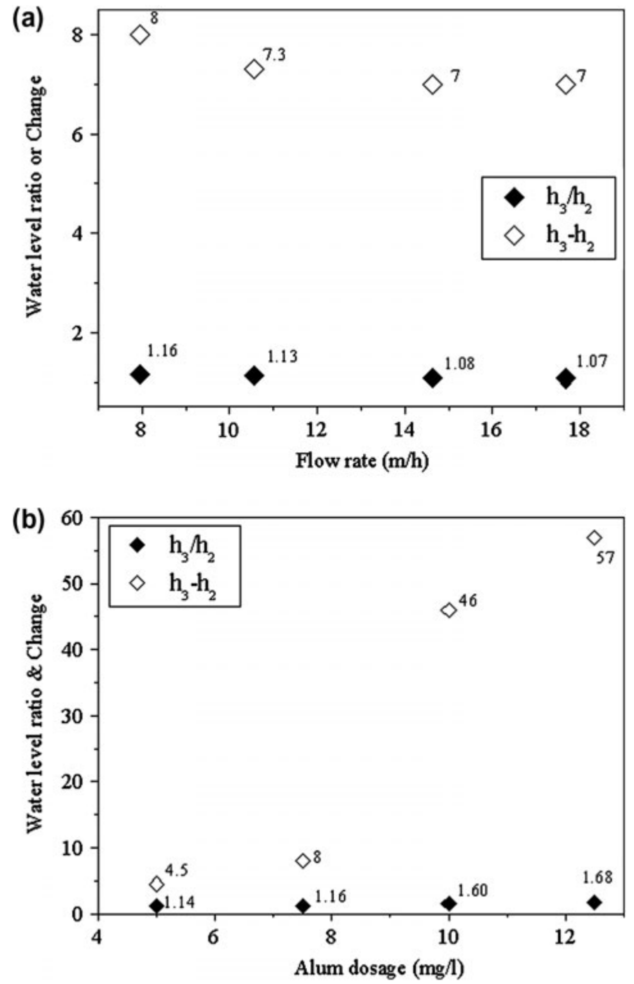

Fig. 8. Relations between the maximum and the minimum water levels at (a) alum dosage: 7.5 mg/L and (b) average filtration velocity: 7.8–7.9 m/h.

coagulant dosage at a fixed average filtration rate (7.8–7.9 m/h). This figure is based on the same experiments as Fig. 6 (Tests 1, 6, 10, and 15). As is the case in these sample results, both of the ratios V_{peak}/V_{av} and V_{max}/V_{av} generally tended to increase whereas V_{min}/V_{av} decreased when the coagulant dosage was increased. Given a fixed influent turbidity, higher coagulant dosages result in higher suspended solids loading on the filters. Therefore, higher coagulant dosages lead to greater clogging in the filter beds and, consequently, larger velocity variations are observed between the cleanest and the dirtiest filters in the bank (or, between the cleanest and the dirtiest states of a given filter).

Fig. 8(a) shows how the values of h_3/h_2 and $(h_3 - h_2)$ changed with average flow rate at a fixed coagulant dosage. Fig. 8(b), in turn, shows how these quantities varied with coagulant dosage at a fixed average rate. Examining the results of the entire set of experiments (see Table 2), the following was observed: For a fixed coagulant dosage, the ratio h_3/h_2 of the maximum water level to the minimum water level always decreased with increasing average rate. At a fixed average filtration rate, on the other hand, the ratio h_3/h_2 generally tended to increase with the coagulant dosage. The level change $(h_3 - h_2)$ remained small and did not change significantly at low coagulant dosages. It may be noted that these comments apply equally well to the slope of the water level curve which is simply the change $(h_3 - h_2)$ divided by the corresponding time period (which did not change during these experiments as the filters were always backwashed with a certain frequency). At the two highest coagulant dosages (10 and 12.5 mg/L), $(h_3 - h_2)$ became significant due to faster clogging of

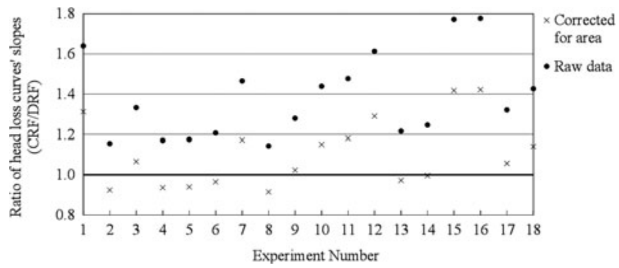

Fig. 9. Ratios of the slopes of the head loss curves for DRF and CRF.

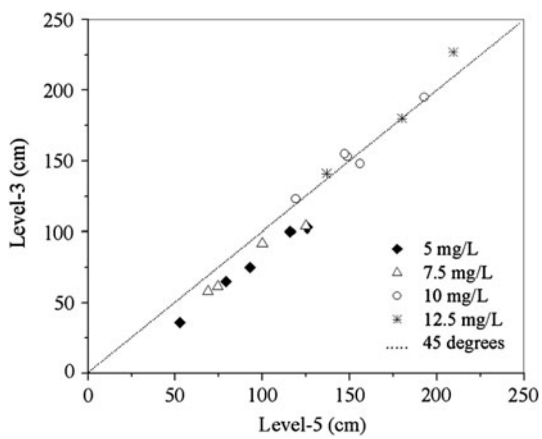

Fig. 10. Level-3 vs. Level-5 in DRF.

the filters. However, although $(h_3 - h_2)$ tended to increase with rate at 12.5 mg/L alum dosage, a clear dependence on rate was not observed at 10 mg/L. The change $(h_3 - h_2)$ always increased with increasing coagulant dosage.

For Tests 14–17 (see Table 2), the value of the ratio $V_{\max}/V_{\mathrm{av}}$ was approximately 1.5. As can be seen in this table, the ratio $V_{\min}/V_{\mathrm{av}}$ was significantly different from 0.7. This observation casts doubt on the general validity of the assumption $V_{\min} = 0.7\,V_{\mathrm{av}}$ whenever $V_{\max} = 1.5\,V_{\mathrm{av}}$ used by Cleasby [3] in his design calculations.

Fig. 9 shows the ratio of the slope of the head loss curves for DRF and CRF for the 18 independent tests carried out in this work. The last test (Test 18 in Table 1) is also included in this figure using the slopes of the water level curves during the start-up period (the first cycle). This particular test had to be stopped at 12 h because the water levels in both filters increased to the maximum allowed value. Raw data for all the tests are shown with bold black markers. It is seen that, contrary to what was reported previously [7,15] the slopes of the head loss curves for two systems were in general not equal. The deviation of this ratio from unity varied from 14.4 to 77.7%, with a mean deviation of 38.1%. This deviation did not show a noticeable variation trend with either coagulant dosage or filtration rate. The other set of markers on this figure represents "corrected" ratio values. This correction was carried out based on the following considerations. Following Akgiray and Saatçı [4], let Q denote the total volumetric inflow rate to the DRF system, and h denote the common water level in all the filter boxes with reference to the downstream effluent level. Also, let A and A_a represent the surface area of each filter column and the area available for water accumulation per filter, respectively. Furthermore, $v_i(t)$ represents the time-dependent production rate of the i-th filter in the bank. As shown by Akgiray and Saatçı [4], the slope of the water level curve (between Level-2 and Level-3) for the DRF system can be written as follows:

$$\frac{dh}{dt} = \frac{A}{A_a}\left[\frac{(Q/N)}{A} - v_m(t)\right] \tag{1}$$

where the arithmetic mean production rate $v_m(t)$ at any instant is defined as follows:

$$v_m(t) = \frac{1}{N}\sum_{i=1}^{N} v_i(t) \tag{2}$$

The derivation and a detailed discussion of Eq. (1) can be found in Akgiray and Saatçı [4]. The important point here is that the value of dh/dt decreases as A/A_a decreases. As noted by Akgiray and Saatçı [4], this is how the water level variations $h_3 - h_2$ and $h_4 - h_3$ can be minimized in practice: If submerged launders with orifices are provided in the sedimentation tanks ahead of filters, the levels in the sedimentation tanks will follow the levels in the filters [3]. In such a case, the surface area of the sedimentation tanks is included in A_a and therefore A/A_a will be very small.

The point here is that slope of the water level curve between Level-2 and Level-3 depends on the ratio A/A_a and does not have a unique value that can be expected to be equal to the slope of the clogging head loss curve for a CR filter operating at the same average rate. From the descriptions of the pilot plants used by Di Bernardo and Cleasby [13] and Hilmoe and Cleasby [15], it is apparent that A/A_a was approximately the same and close to unity for the DRF and the CRF pilot systems employed in those studies. In this study, this ratio had a value $A/A_a = 4/5$ for the DRF system whereas A/A_a was equal to 1 for the single CR filter: (The vertical column used as "inlet main" for water distribution in DRF had the same diameter as the four operating filter columns. Therefore, when the area of this column is taken into account, $A_a = A + A/4 = 5A/4$).

As a "first-order correction," the experimental values of *dh*/*dt* between Level-2 and Level-3 were therefore multiplied by $A_a/A = 5/4$ to compute the "corrected" values shown in Fig. 9. It is seen that the disagreement between the slopes of the DRF and CRF systems decreases with this correction.

Fig. 10 shows Level-3 vs. Level-5 values observed in the 17 separate tests in this study (the first 17 tests listed in Table 2). Each level shown in this figure is the arithmetic average of the values observed during each test. Since data recorded and reported herein are based on stable conditions (i.e. not on start-up conditions), these values did not change significantly from one filtration cycle to the next during a given test. As noted before, Cleasby [3] assumed—without any explanation or justification—these two levels to be equal in his design calculation procedure. For the 8 tests carried out at the higher dosages (10 mg/L and 12.5 mg/L), these two levels were indeed very close. For the two lower dosages (5 mg/L and 7.5 mg/L), Level-3 remained 15 to 20% below Level-5 (cf. Fig. 10 and the first nine rows in Table 2). For many of the runs with 5 and 7.5 mg/L alum, the water level h_3 (and, in some tests, even h_4) remained near or even below the level of backwash collection pipe (which was 71 cm above the outlet pipe). Most of the total filter wall height (281 cm above the outlet pipe level) was not utilized in these experiments because of the fact that low coagulant dosages were used and the filters did not clog enough to cause high water levels. In full-scale plants, the backwash troughs normally remain well below h_3 and h_4. Put in other words, it may be conjectured that the low-coagulant dosage runs in the current study yielded low water levels that are not typical in practice. For the tests that utilized a significant fraction of the available filter wall height, Level-3 and Level-5 did not deviate from each other significantly.

Finally, when all the tests carried out in this work are considered together, it is seen that water levels in DRF are greatly influenced by both the average filtration rate and the coagulant dosage (see Table 2). At a dosage of 5 mg/L alum, for example, Level-3 (the level at which the dirtiest filter is taken off-line to be backwashed) varies between 36 and 100 cm as the average filtration rate is increased from 7.8 to 17 m/h. This change is more drastic at 12.5 mg/L dosage: from 141 cm to the top of the filter column (which is at 281 cm). The filter run (Test 18 in Tables 1 and 2) had to be stopped in the latter case to avoid overflowing the filter column. At the fixed average rate of 7.8 m/h, on the other hand, Level-3 changed from 36 to 141 cm as the alum dosage was increased from 5 to 12.5 mg/L. At the fixed average rate of 17 m/h, Level-3 changed from 90 to 104 cm, and then to 195 cm, and finally to the top of the column as the dosage was varied from 5 to 7.5 mg/L, and then to 10 mg/L, and finally to 12.5 mg/L, respectively. An increase in average filtration rate would occur, for example, when the production of the treatment plant is increased in response to increased water demand. More typical, perhaps, would be changes in coagulant dosage (and possibly in coagulant type and/or dosages of filter aids) to optimize treatment. The results obtained here show that water levels can be influenced significantly by such changes. More generally, changes in influent water suspended solids concentration can be expected to change water levels observed in a DRF plant.

4. Concluding remarks

The goal of the experiments carried out in this work was to ascertain the hydraulic behavior of a DRF system under different operating conditions. Specifically, the effects of design filtration rate and coagulant dosage on the behavior of DRF were investigated. One of the main goals of the experiments was to evaluate the accuracy of certain simplifying assumptions used in DRF design calculations. A constant rate filter was operated in parallel treating the same water to compare the slopes of the head loss curves for CR and DR filters. Experiments were carried out at five different rates and with four different coagulant dosages. Influent water with a turbidity of 10 NTU was used in all the tests. Water level changes and individual filter rates were carefully monitored. The following were observed in these experiments:

(1) In general, in accordance with previous studies in the literature, rate of each filter decreased in a stepwise manner, remaining approximately constant between backwashes. During the first stage immediately after a clean filter is put back into service, however, the rate varies somewhat, decreasing (rather than remaining constant) until another filter is backwashed. This behavior is not explicitly mentioned in previous experimental studies.

(2) The ratio (h_3/h_2) of the maximum water level to the minimum water level always decreased as filtration velocity was increased at a given coagulant dosage. On the other hand, this ratio generally increased with coagulant dosage at a given filtration rate.

(3) The absolute value of water level change ($h_3 - h_2$) [and therefore the slope of the water level curve] remained small and relatively insensitive to rate changes at low coagulant

dosages. At the higher coagulant dosages (10 and 12.5 mg/L) used in this work, this level change became significant due to more rapid clogging of the filter beds. However, although $(h_3 - h_2)$ tended to increase with rate at 12.5 mg/L alum dosage, a clear dependence on rate was not observed at 10 mg/L. The level change $(h_3 - h_2)$ always increased as coagulant dosage was increased.

(4) The ratio (V_{max}/V_{min}) of the maximum and minimum rates of a filter (when all filters are in operation) in general decreased as average (design) filtration rate increased. It was found to be less sensitive to velocity at low coagulant dosages. This ratio (V_{max}/V_{min}) tended to increase when coagulant dosage was increased.

(5) The ratio V_{peak}/V_{min} of the peak and minimum rates of a filter (V_{peak} occurring at the maximum water level h_4 reached when one filter is taken off line to be backwashed) in general decreased as average filtration rate increased. Just like V_{max}/V_{min}, it was less sensitive to velocity at low coagulant dosages. The ratio V_{peak}/V_{min} tended to increase as coagulant dosage was increased.

(6) Trends similar to those summarized above for V_{max}/V_{min} were observed when V_{max}/V_{av} values were examined: This ratio changed between 1.0 and 2.0 in the tests reported here. It tended to increase with coagulant dosage and decreased with increasing average velocity.

(7) The ratio V_{min}/V_{av} varied between 0.4 and 0.9 and changed in accordance with the changes in V_{max}/V_{av}: It increased (approaching unity) with increasing average flow rate and decreased with increasing coagulant dosage. An important observation was that the ratio V_{min}/V_{av} was significantly different from 0.7 when V_{max}/V_{av} was about 1.5. This casts some doubts on the generality of the calculation method described by Cleasby [3] who assumed $V_{min}/V_{av\,=}\,0.7$ whenever $V_{max}/V_{av} = 1.5$.

(8) The slopes of the head loss curves for CR and DR filters were observed to be unequal with a mean deviation of 38% in these experiments. When corrected for the effect of accumulation area, the deviation of the two slopes from each other decreased to a mean value of 14%, and the slope of the CRF head loss curve in general tended to be larger than that of the DR filter.

(9) The values of Level-3 and Level-5 were found to be approximately equal for the experiments with high water levels. This finding supports one of the assumptions used by Cleasby [3] in his design method. When the coagulant dosage was so low that the peak water level remained below or slightly above the water level in a newly backwashed filter before it is connected to the other filters in the bank, however, these two levels were significantly different.

(10) For the 17 experiments for which it was possible to record Level-2, Level-4, and Level-5, the following periods were observed: Change from Level-4 to Level-5: 47 s (with a standard deviation of 22 s) and change from Level-5 to Level-2: 23.4 min (standard deviation of 4.8 min). These are in qualitative agreement with previously reported values of "a few minutes" and "30 min", respectively [7].

(11) The experiments reported in this work show that the water level variations in DRF are greatly influenced by both coagulant dosage and average filtration rate. Even for a fixed average filtration rate and a fixed coagulant type, adjustments in coagulant dosage can cause significant changes in water levels.

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