Physical clogging of soil filters under constant flow rate versus constant head

Lakshmi N. Reddi, Ming Xiao, Malay G. Hajra, and In Mo Lee

Abstract: In this study, the differences in soil filter clogging are evaluated under two operating conditions: constant head and constant flow rate. Whereas filters placed around well screens and leachate collection systems are subjected to a constant flow rate, filters in earth dams and pavement drainage systems operate under constant or slightly varying heads. In this study, the experiments revealed similar permeability reductions with respect to time in both cases; however, the permeability reduction under the condition of constant head occurred in much fewer pore volumes than under the condition of constant flow rate. Self-filtration appeared to be greater under the condition of constant head. The physical clogging model developed for the conditions of constant flow rate and constant head showed good qualitative agreement with experimental observations.

Key words: particle, clogging, filter, constant, head, flow.

Résumé : Dans cette étude, les différences de colmatage d'un filtre de sol sont évaluées entre les deux conditions d'opération de charge constante ou de débit constant. Alors que les filtres placés autour des grillages de puits et des systèmes de collecte des lexiviants sont soumis à des conditions de débit constant, les filtres dans les barrages en terre et dans les systèmes de drainage des chaussées opèrent sous charge constante ou légèrement variable. Dans cette étude, les expériences révèlent des réductions similaires de perméabilité en fonction du temps dans les deux cas; cependant, la réduction de perméabilité sous la condition de charge constante se produit dans beaucoup moins de volume de pores que dans la condition de débit constant. L'auto-filtration semblait être plus grande dans la condition de charge constante. Le modèle de colmatage physique développé pour des conditions de débit constant et de charge constante a montré une bonne concordance qualitative avec les observations expérimentales.

Mots clés : particule, colmatage, filtre, constant, charge, débit.

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Introduction

Soil filters play an important role in ensuring proper stability and performance of subsurface infrastructures in geotechnical and geoenvironmental engineering. Although filters are used primarily to protect base soils from eroding, they are also expected to serve as drainage layers in many cases. Soil filters can be successful in preventing the erosion of base soils, but they might also undergo significant reductions in permeability as a result of physical clogging caused by the accumulation of fine particles in pores. Many studies in recent decades documented reports of dam failures associated with inadequate filter design (Vaughan and Soares 1982; von Thun 1985; Peck 1990). Koerner et al. (1994) highlighted the impact of poor drainage on the performance of leachate collection systems in landfills. Koerner and Koerner (1991) concluded that particulate clogging is a major factor in flow rate reductions in drainage layers. Reddi et al. (2000) reported that permeability reductions of more than an order of magnitude are possible, even when the migrating particles are smaller than the majority of the soil pores. In that study, which was restricted to conditions of constant flow rate in soil filters, the authors developed a model for physical clogging and evaluated the effects of flow rate and the size and concentration of migrating particles in the clogging process. It should be noted that biological and chemical clogging, acting in synergy with physical clogging, could also occur and contribute to permeability reduction, particularly in the case of leachate collection systems. Studies by Brune et al. (1991), Cunningham et al. (1991), Vandevivere and Baveye (1992), Fleming et al. (1999), Hajra et al. (2000), and Fleming and Rowe (2004) showed that biological processes could significantly clog porous media. The scope of the present paper, however, is limited to physical clogging.

In practice, physical clogging of soil filters may occur under two operating conditions: constant flow rate and constant head. Flow rates are kept constant in (*i*) oil recovery processes in petroleum engineering (Gray and Rex 1966; Muecke 1979; Baghdikian et al. 1989); (*ii*) well systems for dewatering in geotechnical engineering; (*iii*) recharge wells or water supply wells (van Beek 1984) and nutrient injection

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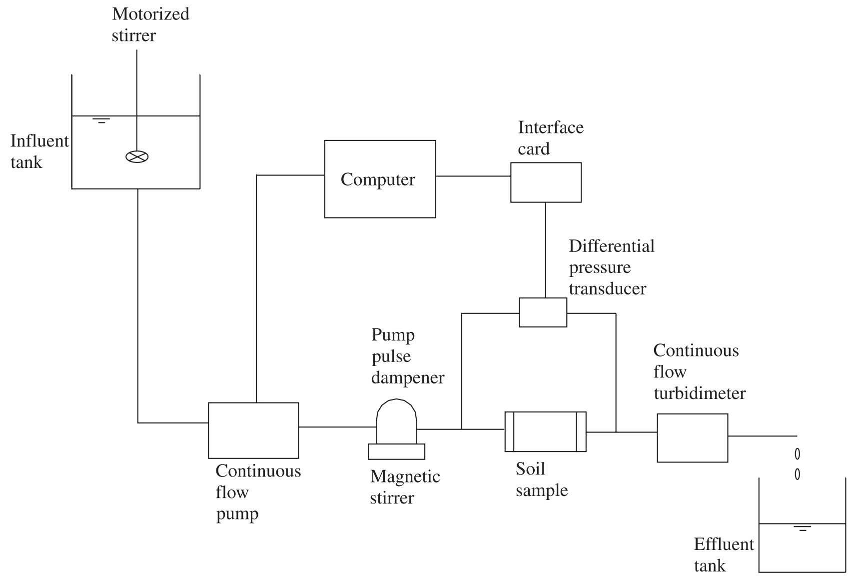
Fig. 1. Experimental setup for the constant flow rate condition.



wells (Lee et al. 1988) in site remediation efforts; and (*iv*) filters in waste water treatment plants in environmental engineering. Physical clogging under the condition of constant head occurs in the cases of (*i*) filters placed at the bottom of waste water lagoons in environmental engineering; (*ii*) sealing liners of canals and reservoirs (Rausch and Curry 1963; Miller et al. 1985) in hydraulic and agricultural engineering; (*iii*) pavement subsurface drainage systems in transportation engineering; and (*iv*) protective soil filters and drainage layers in earth-retaining structures, tunnels, and earth dams in geotechnical engineering.

The primary objective of this paper is to evaluate the differences in the mechanisms of physical clogging of soil filters under the condition of constant flow rate and under the condition of constant head. The experimental and modeling investigations reported in a previous paper (Reddi et al. 2000) were modified to achieve this objective.

Experimental materials and methods

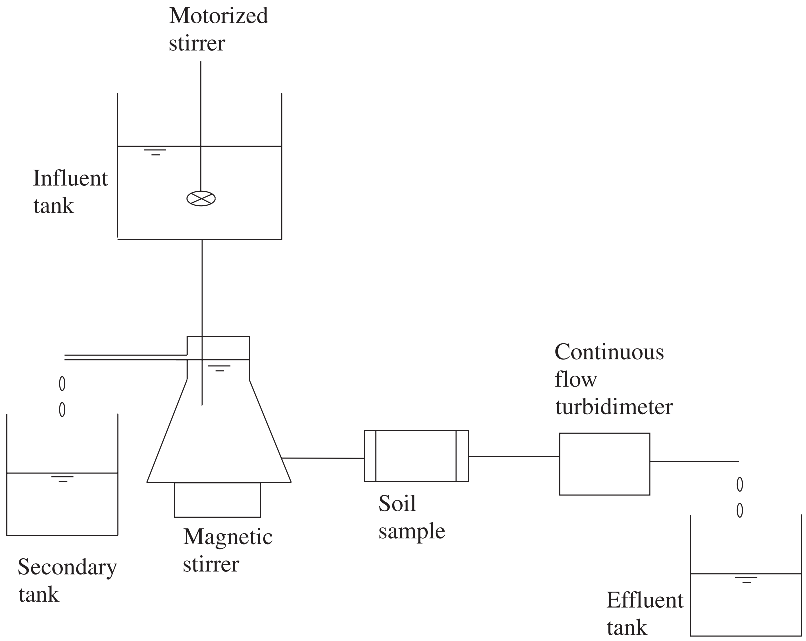
A sandy soil representative of concrete sands (uniformity coefficient C_u = 4.7), which are used in practice as filters and drainage layers for cohesive base soils, was used as the filter sand in this study.

The experiments were first conducted with only tap water, without any added particles, to evaluate the extent of self-filtration under constant head. In the particle clogging experiments, two kinds of particulate suspensions were used to permeate the soil samples: one suspension was prepared with kaolinite particles; and the other with polymer microspheres provided by Duke Scientific Corp (Palo Alto, Calif.). The diameters of the kaolinite particles were 1.9–12.0 μm, and those of the polymer microspheres were 1.0–35.0 μm. The diameter of migrating clay flocs in most filter systems is of the order of 10 μm (Vaughan and Soares 1982). Thus, the diameter ranges for the two suspensions chosen in this study cover the diameters of migrating particles in practice. The size distributions of these fine particles were discussed in detail by Reddi et al. (2000).

Figure 1 shows the experimental setup for the condition of constant flow rate. The sandy soil sample (3 in., or 7.63 cm, in diameter; 2.5 in., or 6.35 cm, in length) was loosely packed in a Plexiglas column. The sample's bulk density was 1.7 g/cm^3. A metal screen was used at both ends of the sample; the screen opening, 0.4 mm, was chosen so that the screen would prevent the sandy soils from flushing and meanwhile would not impede the flow. Two flow rates (50 and 100 mL/min) were used in the constant flow rate experiments. These flow rates cover a range of filter and drainage layer applications. The influent was prepared with either kaolinite or polymer microspheres mixed in tap water in a tank at a desired particle concentration. A motorized stirrer was used in the influent tank to keep the suspension stable and uniform. A peristaltic pump was used to pump the influent suspension at a constant rate into the soil sample. A pulse dampener was used between the pump and the sample to reduce pulses created by the pump. The dampener was located on a magnetic stirrer to prevent particle settling. As the flow rate was kept constant throughout the entire duration of the experiment, the pressure head across the soil sample increased with time as a result of the gradual accumulation of particles within the filter soil. This increase in pressure was recorded with a differential pressure transducer. The peristaltic pump and the transducer were connected to a data-logging system in a computer. The effluent coming out of the soil sample was passed through a continuous flow turbidimeter to record the turbidity values. The permeability was calculated, using Darcy's law, with the recorded pressure differences across the soil sample and the flow rate of the pump.

Figure 2 shows the experimental setup for the condition of constant head. The influent was prepared with either kaolinite or polymer microspheres mixed in tap water in a tank at a desired particle concentration. A motorized stirrer kept the suspension stable and uniform in the influent tank. The influent passed through a primary container with an opening near the top. The excess influent coming out of the primary container was diverted to a secondary container and

Fig. 2. Experimental setup for the constant head condition.



added back to the suspension in the influent tank. The primary container was placed on a magnetic stirrer to prevent particle settlement. Prior to the use of the particulate suspension, each experiment was conducted with tap water to obtain the initial permeability of the soil filter. The pressure head drop was chosen to provide an initial flow rate equal to that in the constant flow rate experiments (5 cm corresponds to 100 mL/min, and 2.5 cm corresponds to 50 mL/min). The pressure head drop was kept constant throughout the duration of the experiment. The flow rate, which decreased with time as a result of particle accumulation inside the sample, was measured and recorded with a graduated cylinder at the outlet of the soil sample at regular time intervals. The effluent coming out of the sample was passed through a continuous flow turbidimeter, which recorded the turbidity values. Darcy's law was used to calculate the soil sample permeability from the flow rate data and the constant head.

Model development

Physical clogging in soils was modeled by using the particle capture probability approach to determine reductions in porosity and hydraulic conductivity (Reddi et al. 2000). For the sake of completeness, relevant aspects of that model are described below, before the differences in modeling constant flow rate and constant head conditions are outlined. The soil filter is idealized as an ensemble of parallel capillary tubes (cylinders) of various diameters and the same length. To simulate time-dependent reductions in the pore tube diameters, the particle deposition in each pore tube is expressed as

$$\frac{\mathrm{d}N(r_i, a_j)}{\mathrm{d}t} = q(r_i)p(r_i, a_j)C(a_j) \tag{1}$$

where $N(r_i, a_j)$ is the number of particles of radius a_j deposited in the pore tube of radius r_i; $q(r_i)$ is the flow rate through the tube r_i; $p(r_i, a_j)$ is the probability that a particle of radius a_j will be deposited in a pore of radius r_i; and $C(a_j)$ is the concentration of particles of radius a_j, in terms of the numbers of particles per unit volume in the pore stream. The flow rate through each tube, $q(r_i)$, may be expressed by Poiseuille's law:

$$q(r_i) = \frac{\pi\gamma J}{8\mu} r_i^4 \tag{2}$$

where J is the hydraulic gradient across the tube; γ is the unit weight of water; and μ is the viscosity of water. The probability of a particle of radius a_j being deposited in a pore of radius r_i is $p(r_i, a_j)$, expressed as

$$p(r_i, a_j) = 4\left[\left(\frac{\theta a_j}{r_i}\right)^2 - \left(\frac{\theta a_j}{r_i}\right)^3\right] + \left(\frac{\theta a_j}{r_i}\right)^4 \tag{3}$$

where θ is a lumped parameter that takes into account the effect of several interparticle forces on deposition, such as gravitational, inertial, hydrodynamic, electric double layer, and van der Waals forces (Stein 1940; Rege and Fogler 1988), and may be written as

$$\theta = \theta_0 \exp\left[\frac{-v(r_i)}{v_{\mathrm{cr}}}\right] \tag{4}$$

where θ_0 is a constant dependent on ionic conditions and has the value of 3.0 for salt-free water (Rege and Fogler 1988; Reddi et al. 2000); $v(r_i)$ is the velocity of flow in the pore tubes of size r_i; and v_{cr} is a critical velocity beyond which no particle deposition is likely. Reddi et al. (2000) suggested that a v_{cr} value of 0.1 cm/s could be used for soil filters. For a detailed explanation of θ, the reader is referred to Reddi et al. (2000) and Hajra et al. (2002).

The differences between filters subjected to conditions of constant head and those subjected to conditions of constant flow rate lie in the manner in which the particle deposition rate is used to reduce pore size. The effect of particles deposited on the pore walls is simply to increase the resistance to flow and reduce the pore size. Based on drag forces experienced by particles in cylindrical tubes, the pressure drop, $\Delta P_{\text{particle}}(a_j)$, caused by a single particle of radius a_j in a tube of radius r_i may be expressed as (Happel and Brenner 1973) follows:

$$[5] \qquad \Delta P_{\text{particle}}(r_i, a_j) = \frac{12\mu a_j U(r_i)}{r_i^2}\left\{1-\left[1-\left(\frac{a_j}{r_i}\right)\right]^2\right\}^2 K(r_i, a_j)$$

where $U(r_i)$ is the central line velocity in a tube of size r_i, which can be taken as two times the average pore velocity, assuming the velocity profile is parabolic; and

$$[6] \qquad K(r_i, a_j) = \frac{1-0.667\left(\frac{a_j}{r_i}\right)^2-0.202\left(\frac{a_j}{r_i}\right)^5}{1-2.1\left(\frac{a_j}{r_i}\right)+2.09\left(\frac{a_j}{r_i}\right)^3-1.71\left(\frac{a_j}{r_i}\right)^5+0.73\left(\frac{a_j}{r_i}\right)^6}$$

The hydraulic gradient due to resistance to the flow caused by a single deposited particle in a pore of radius r_i is

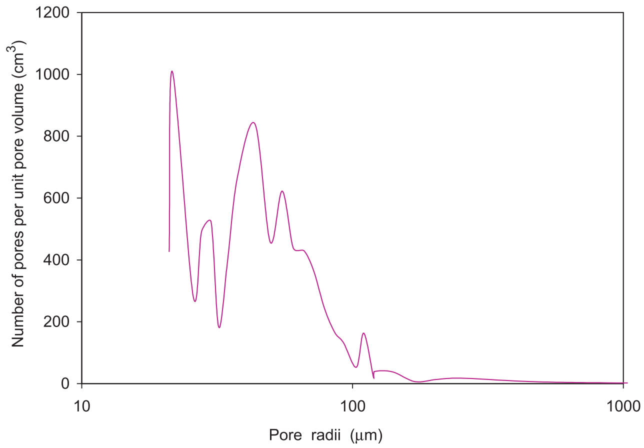
$$[7] \qquad J_{\text{particle}}(r_i, a_j) = \frac{\Delta P_{\text{particle}}(r_i, a_j)}{\gamma\alpha^*}$$

where α^* is the characteristic pore length over which the idealization of straight cylindrical pore tubes is valid. The characteristic pore length was proposed by Arya and Paris (1981) and Arya and Dierolf (1989), who estimated that α^* varied from about 0.3 to 1.5 cm for sandy soils. They suggested that an average value of 0.9 cm produces good agreement between predicted and measured soil moisture characteristics for two sandy soils. For this reason, 0.9 cm was adopted for α^* in this study. The total hydraulic gradient incurred by all deposited particles in a pore of radius r_i is

$$[8] \qquad \sum J_{\text{particle}} = \frac{\sum \Delta P_{\text{particle}}(r_i, a_j)}{\gamma\alpha^*} = \frac{12\mu U(r_i)}{\gamma\alpha^* r_i^2} \times \sum_{j=1}^{M} N(r_i, a_j)\, a_j \left\{1-\left[1-\left(\frac{a_j}{r_i}\right)\right]^2\right\}^2 K(r_i, a_j)$$

Under the condition of constant flow rate, the pressure difference across a clogged pore tube is equal to the sum of pressure differences caused by the deposited particles and the pressure difference across the clean tube with no deposition. Under the conditions of constant head, on the other hand, the flow rate in a clogged pore tube corresponds to the pressure difference between a constant head applied across the tube and the pressure head due to particles deposited in the tube. If Poiseuille's law is applied for these two conditions, the new pore radii (r_{i1}) after deposition may be related to the previous radii (r_{i0}) before deposition by means of the following expressions:

Fig. 3. Pore size distribution of filter sand.



$$[9] \qquad r_{i1} = r_{i0}\left(1+\frac{\sum J_{\text{particle}}}{J}\right)^{-1/4} \qquad \text{constant flow rate}$$

$$[10] \qquad r_{i1} = r_{i0}\left(1-\frac{\sum J_{\text{particle}}}{J}\right)^{1/4} \qquad \text{constant head}$$

The initial pore size distribution was obtained from a water retention test on filter sand, using the Haines method (Rowell 1994). On the water retention curve, each suction head was associated with a particular pore size (r_{i0}) based on capillarity, and the quantity of water sucked out at this suction head could be converted into the number of pore tubes of radius r_{i0}. The pore size distribution is plotted in Fig. 3 in terms of pore radii versus number of pores per unit pore volume. The pore radii updated according to eq. [9] for the condition of constant flow rate and to eq. [10] for the condition of constant head may then be used in the Kozeny hydraulic radius model, expressed below, to determine permeability at each time step:

$$[11] \qquad k = C_s n\left(\frac{\gamma}{\mu}\right)\left[\frac{1}{4\sum_{i=1}^{M}\frac{f(d_i)}{d_i}}\right]^2$$

where k is permeability; C_s is the shape factor (1/32 for cylindrical pores); n is porosity (which decreases as a result of particle deposition); γ is the unit weight of the pore fluid; μ is the dynamic viscosity of the pore fluid; M is the number of different pore sizes; d_i is the ith pore diameter; and $f(d_i)$ is the volumetric frequency of the pore group of size d_i. The parameters used in the model are summarized in Table 1.

Results and discussion

Figure 4 shows the results from experiments on self-filtration conducted without adding particles to influent water. The permeability of the sample, normalized by the initial permeability, is plotted against the number of pore volumes exiting the sample. The initial permeability was measured to

Fig. 4. Self-filtration in filter sand under constant flow and constant head conditions.

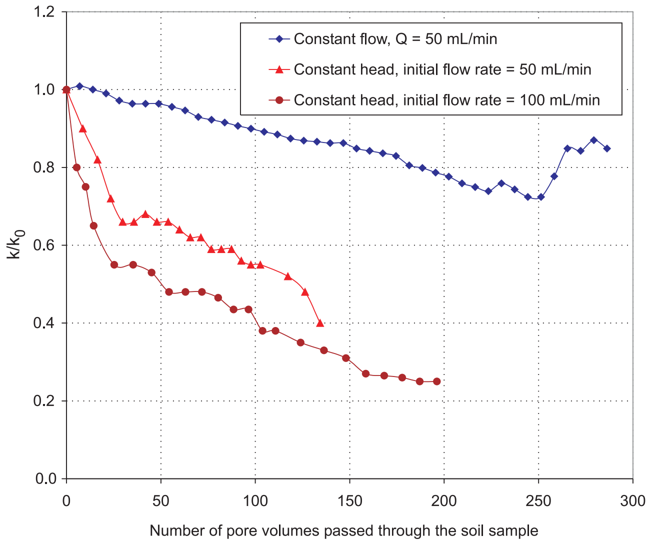


Fig. 5. Permeability reduction versus time under constant flow rate and constant head conditions.

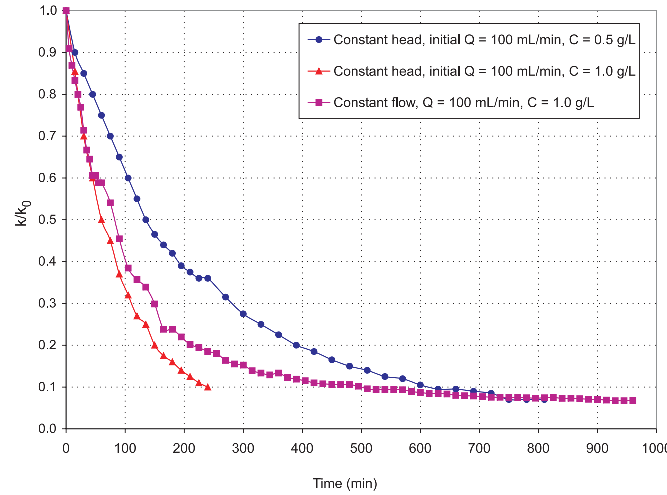


Table 1. List of parameters used in the model.

Parameter	Value
Unit weight of pore fluid, γ (N/mL)	9.8105×10^{-3}
Dynamic viscosity of pore fluid, μ ($N \cdot s/cm^2$)	1.005×10^{-7}
Characteristic pore length, α^* (cm)	0.9
Ionic condition constant, θ_0	3
Critical velocity, v_{cr}, (cm/s)	0.1
Shape factor for cylindrical pores, C_s	1/32

be 0.046 cm/s at a constant head drop of 5 cm (corresponding to a flow rate of 100 mL/min). It can be seen that self-filtration itself can reduce permeability by about 70% to 0.014 cm/s after about 200 pore volumes. For the same initial flow rate (Q_0), the condition of constant head resulted in a greater permeability reduction than the condition of constant flow rate. This is due to the fact that particles, once transported and settled in pores, remain there under constant head conditions, whereas under constant flow rate conditions they might undergo re-entrainment. The extent of the reduction in permeability due to self-filtration revealed in these experiments has important implications for the current methods used to determine hydraulic conductivity of granular soils. The flow rates or gradients used in the experiments and duration used for permeation govern the hydraulic conductivity measurements, particularly in the case of graded soils.

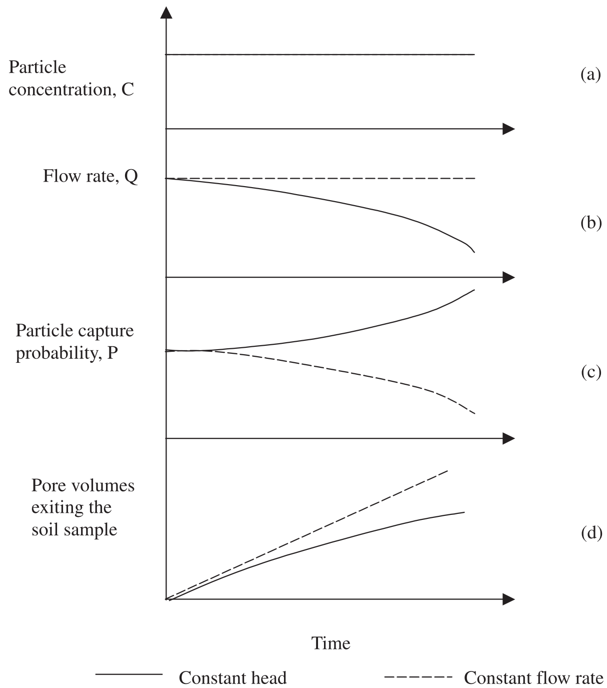
For experiments using particle suspensions in influent, Darcy's law was used to calculate the permeability of the soil sample for both the condition of constant flow rate and the condition of constant head :

$$[12] \quad Q = k\left(\frac{h}{l}\right)A$$

where Q is the flow rate; k is the permeability; h is the pressure head drop across the sample (unit is length); l is the length of the soil sample; and A is the cross-sectional area of the soil sample. For the experiments under the condition of constant flow rate, Q is kept constant throughout the experiment. The pressure head drop across the soil sample increases with time, as a result of gradual particle accumulation. Permeability, k, therefore, decreases with the increase in head drop, h. Under a condition of constant head, a decrease in permeability is accompanied by a decrease in flow rate.

Figure 5 shows the comparison between results from constant flow rate and results from constant head with filter sand and different concentrations of kaolinite suspensions. Each experiment was initially run using tap water only for the self-filtration test. After the permeability was stabilized, particle suspensions at a specified concentration (0.5 or 1.0 g/L) were introduced into the sample. The initial permeability, k_0, in each particle clogging experiment was the sample permeability at the end of the self-filtration test and before kaolinite or polymer microspheres were introduced. Under the constant head condition, pore velocities decrease with time, as a result of particle clogging, leading to a greater likelihood of particle deposition. Thus, one would expect a greater extent of particle deposition in the case of constant head than in the case of the constant flow rate. When the experimental results for the condition of constant flow and those for the condition of constant head, tested in this study under particle concentration of 1.0 g/L, are compared, it is seen that both experimental conditions yielded a similar permeability reduction during most of the experimental period. An explanation for this similarity is as follows. As stated in eq. [1], the rate at which particles accumulate in a given pore is governed by the product of three terms: (*i*) particle concentration in pore fluid; (*ii*) flow rate in the pore; and (*iii*) probability of particle capture in the pore. The variation of each of these terms is shown schematically in Fig. 6, for both constant flow rate and constant head. The particle concentration in pore fluid remains constant for both constant head and constant flow rate (Fig. 6*a*). Under the condition of constant flow rate, the pore flow rate remains constant (Fig. 6*b*); however, the probability of particle capture decreases with the progressive accumulation of particles, as a result of velocities increasing in progressively

Fig. 6. Schematic variation of particle concentration, flow rate, particle capture probability, and pore volumes with time.



smaller pore tubes (Fig. 6*c*). Thus, under constant flow rate, particles accumulate at a progressively smaller rate. Under the condition of constant head, pore flow rate reduces with time, as a result of permeability reductions occurring under constant gradients (Fig. 6*b*). The probability of capture is influenced not only by pore size but also by pore velocity (eqs. [3] and [4]). A reduction in pore size, along with a reduced pore velocity, causes progressively increasing probabilities of captures (Fig. 6*c*). However, the increase in capture probabilities is offset by a decreasing pore flow rate. Thus, the magnitudes of the rate of particle accumulation tend to follow similar trends in both cases, despite the differences in the probability of particle capture.

Figure 6*d* shows the variation in the number of pore volumes exiting the filter sample under constant head and under constant flow rate. As a result of decreasing flow rates under constant head, the number of pore volumes exiting the filter sample gets progressively smaller with time. Because of this, the same experimental data shown in Fig. 5 reveal a useful aspect when the normalized permeability is plotted against number of pore volumes exiting the filter sample (Fig. 7). Permeability reduction occurred over much fewer pore volumes under constant head than under constant flow rate for the experimental conditions used. In other words, for situations requiring significant amounts of drainage from filters, subjecting the filters to a constant flow rate via pumping should be preferred over relying on constant hydraulic gradients alone. Consistent with the results reported by Reddi et al. (2000) for constant flow rate conditions, the permeability reduction was faster when the particle concentration was increased to 1.0 g/L from 0.5 g/L. This is also revealed by eq. [1], in which higher particle concentration in pore fluid results in a higher particle deposition rate, which causes faster permeability reduction. It should also be noted that although permeability reduction with *time* under the condition of constant head is similar to that under the condition of constant flow rate, as shown in Fig. 5, permeability reduction with *pore volumes* under a condition of constant head is different from that under a condition of constant flow rate (Fig. 7). When permeability reductions under the two test conditions are compared, one needs to specify whether the comparison is made against *time* or *pore volume*.

Fig. 7. Permeability reduction versus pore volumes under constant flow rate and constant head conditions.

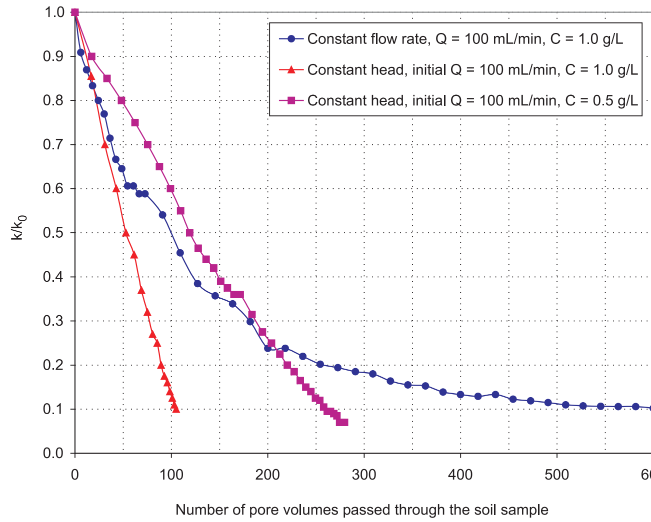
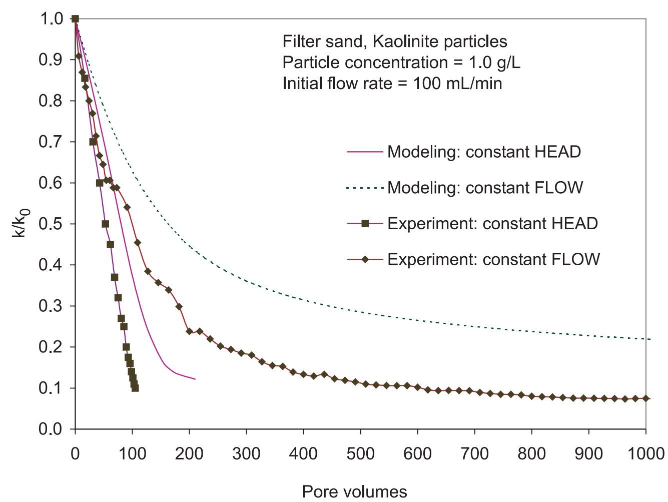


Fig. 8. Comparison between modeling and experimental results under constant head and constant flow rate conditions.



Results from the modeling exercise showed the same trends as in experiments. Figure 8 shows a comparison between the modeling and experimental results under conditions of constant flow rate and constant head. The initial flow rate in both cases was 100 mL/min. The model predicted faster reductions in permeability in the case of constant head than in the case of constant flow rate, and this was consistent with experimental observations. However, it underestimated the permeability reduction in both cases. This was clearly due to the self-filtration occurring in the experiments, and the self-filtration was not accounted for in

Fig. 9. Experimental and modeling results under constant head conditions for two different particle concentrations.

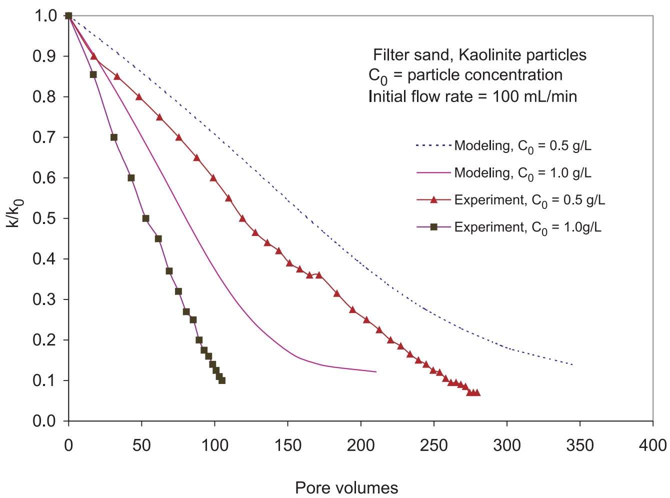
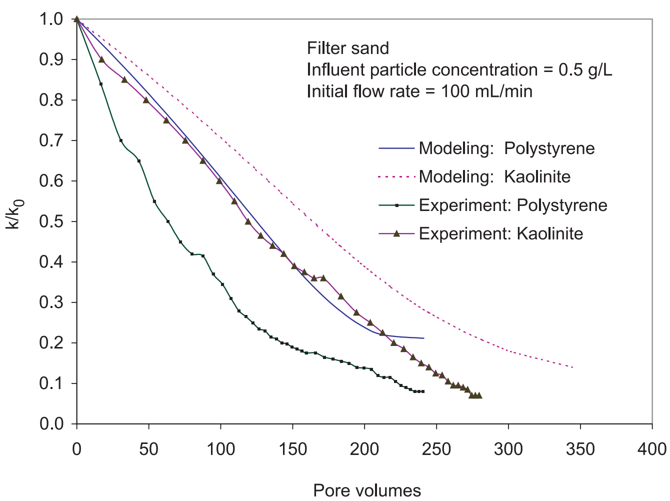


Fig. 10. Experimental and modeling results under constant head conditions for kaolinite and polymer microspheres particles.



the modeling. When tap water was run through the soil filter without particle suspensions, some fine particles of the sandy soil in the upstream section became detached and were entrained in the downstream section and settled at locations where deposition conditions were favorable. As shown in Fig. 4, self-filtration resulted in as much as a 70% permeability reduction. In general, both modeling and experimental results revealed that larger concentrations of fine particles in the influent result in steeper reductions in permeability (Fig. 9), a result that is consistent with the observations in constant flow rate experiments documented in Reddi et al. (2000). Despite the considerable differences in the sizes of kaolinite particles and polymer microspheres, the extent of permeability reduction and the pore volumes within which this reduction occurred were similar in both cases (Fig. 10). This was due to the flocculation occurring in the case of kaolinite particles. Unlike polymer microspheres, kaolinite particles exhibit surface chemistry and flocculate to the extent allowed by the ionic strength of suspension. The sizes of flocs have a direct bearing on permeability reduction. The relative effects of floc size and the ionic strength of influent on permeability reduction were the focus of a parallel study by the authors (Hajra et al. 2002). For instance, depending on the type of ions present in the suspension (tap water, NaOH, or KCl), the median size of influent kaolinite flocs ranged from 3.6 to 14.0 μm. Kaolinite flocs as large as 45 μm were observed in that study (Hajra et al. 2002). Earlier results from constant flow rate experiments revealed that flocculation of a mere 5% of particles is sufficient to cause permeability reductions comparable to those occurring with polymer microspheres (Reddi et al. 2000).

The model developed in this study can be improved by the following:

(*i*) Simulating self-filtration—Self-filtration results in heterogeneity of the porous medium; that is, upstream pore sizes increase as a result of particle detachment, and downstream pore sizes decrease as a result of particle deposition. Therefore, upstream and downstream pore size distributions are different, and measuring them is difficult.

(*ii*) Simulating particle re-entrainment—After initial deposition, it is likely that a particle might be detached because of increased pore flow rate and re-entrained downstream. This model assumes that all particles after deposition remain deposited.

Conclusions

This study allowed an assessment of the differences in soil filter clogging under the condition of constant flow rate and under the condition of constant head. The experiments revealed similar permeability reductions with respect to time in both cases; however, the permeability reduction under constant head occurred in much fewer pore volumes than under constant flow rate. Self-filtration was observed to be greater under the condition of constant head. The physical clogging model, developed earlier for conditions of constant flow rate, showed good qualitative agreement with experimental observations after modifications for conditions of constant head, although the model underpredicted the permeability reductions because self-filtration was not accounted for.

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