CHAPTER 14

FILTRATION

Sand filtration was thought for some time to be the treatment for rendering seawater drinkable (Baker, 1981). Although this theory was debunked, filtration is the bulwark of water treatment. This is reflected by water treatment plants being commonly called filtration plants or simply "the filters." Filtration, especially when joined with chemical coagulation, produces clear water very low in turbidity. Significant removal of bacteria and other microbes also occurs in filtration. Craun (1988) concludes that in all but exceptional situations, effective filtration of surface waters must be provided to minimize waterborne disease outbreak. Another application of filtration in water treatment is preliminary treatment of a raw water with high suspended solids content. Filters with very coarse media, known as roughing filters, are used. Filtration is also used for polishing wastewaters, particularly effluents from stabilization pond systems.

Removal in a filter is accomplished by a number of mechanisms (Tchobanoglous and Eliassen, 1970). Straining, sedimentation, flocculation, and nine other chemical and physical mechanisms have been identified; some are indicated in Fig. 14.1. It is generally accepted that under the conditions of water filtration the dominant mechanisms are diffusion and sedimentation (Amirtharajah, 1988). Biological growth in a filter can significantly affect its performance and influence the predominant removal mechanisms.

Sand is the most common medium; however, other media such as crushed anthracite (hard coals), crushed magnetite, and garnet, besides inert synthetic media are used. The medium size and the pore openings to which it gives rise are important characteristics influencing removal. These characteristics also determine to a large degree the hydraulic performance of the filter.

Removal in a filter is highly dependent on the surface area of the media particles. The surface area of media available in a given volume of filter is large. Consider a 1 m^3 volume of filter with a medium that has a typical porosity of 0.40. Sand particles in filters often have a nominal diameter near 0.50 mm. Using this information and assuming that the medium particles are spheres, the number of particles per cubic meter of filter is 9.17×10^9 and the gross surface area of the particles is $7.20 \times 10^3\ m^2/m^3$. The effective surface area is less than this value because particles are shielded by each other. Even assuming that only 1% of the surface area is effective yields a substantial increase in the available surface area compared to the same volume devoid of media.

14.1 SLOW SAND FILTERS AND RAPID FILTERS

The first filtration operations (dating from 1829 in England) were designed simply to pass water through a bed of sand without any chemical or mechanical assists to the process. The process is similar to withdrawing water from an infiltration gallery placed in the sand bed of a river. The flow rates per unit surface area in these filters are low

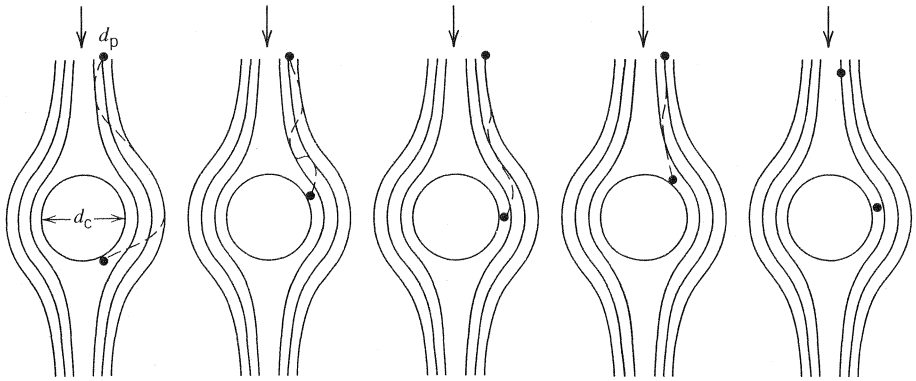

Figure 14.1 Transport and removal mechanisms of filtration. From Ives (1982).

compared to later developments, and they have become known as "slow sand filters." Many cities in Europe incorporated filtration designed along these lines into their water treatment processes and they are still in use today. Slow sand filters are still designed — they are a viable alternative to more recent rapid filters in many circumstances.

Slow sand filters go through a ripening phase of a few weeks after their startup. During this phase a dense microbial zoogleal or gelatinous growth establishes itself in the upper layers of the filter. It is in this layer that most of the removal of suspended and colloidal particles occurs. After a period of time, headloss increases to the cutoff point and a small layer of medium is scraped off the top of the filter. The biological growth extends below the layer that was removed and filter performance is not impaired. This cycle is repeated until a minimum depth of medium remains in the filter. At this time the discarded medium is washed and returned to the filter.

Rapid filters (Fig. 14.2) were conceived in North America as an alternative to slow sand filters. Slow sand filters concentrate removals in the upper layers of the filter, and the rapid filter was designed to utilize the entire depth of a filter bed more fully to attain a higher throughput of water for a given surface area. A higher loading rate produces more rapid headloss development. This and the deeper penetration of solids into the bed limits the only feasible means of rejuvenating the filter media to backwashing the filter. During backwash, water is forced through the filter in the upward direction at a velocity sufficient to expand the media. Cleansing occurs by scour caused by hydraulic shear forces on the media and by abrasive scour resulting from particles rubbing against each other. The former is the more important cleansing mechanism (Amirtharajah, 1978a).

As the expanded media settles after backwashing is terminated, larger particles tend to settle towards the bottom of the filter. Larger void spaces are associated with larger particles of media and the filter becomes a reverse graded sieve, with smaller openings at the top and larger openings at the bottom. The fine media particles will accumulate at the top of the media resulting in clogging at the top layer and little use of the whole filter depth. This leads to a requirement for more uniform media in a rapid filter.

The higher throughput of water and the lack of significant biological growth necessitates the addition of chemical coagulating agents to the influent to a rapid filter.

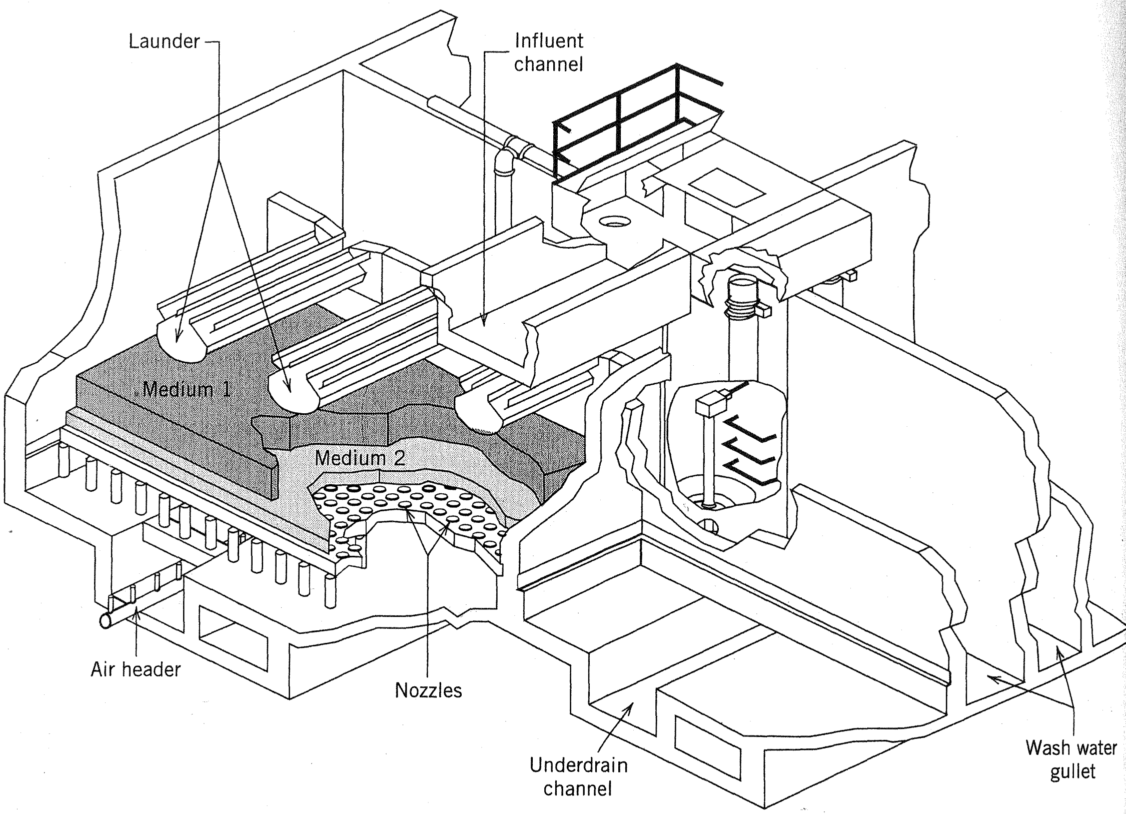

Figure 14.2 Typical rapid multimedia filter waterway. Courtesy of Eimco Process Equipment.

The above characteristics and other features of slow and rapid sand filters are compared in Table 14.1.

14.2 FILTERING MATERIALS

The porosity, e, of the filter depends on how well the particles fit together. As the particles become less spherical, the porosity of a given volume increases. Also, as particles become less spherical, their surface area increases, which has a beneficial effect on removal mechanisms that depend on surface area. The measure of shape is sphericity, ψ, defined as the ratio of the surface area of the equivalent volume sphere to the actual surface area of the particle.

$$\psi = \frac{(\text{surface area of a sphere})/V_{\text{sphere}}}{(\text{surface area of a particle})/V_{\text{particle}}} \qquad V_{\text{sphere}} = V_{\text{particle}}$$

The volume (V_s) and surface area (A_s) of a sphere are calculated from

$$V_s = \frac{\pi d^3}{6} \qquad A_s = \pi d^2$$

where

d is the diameter of the sphere

TABLE 14.1 General Features of Construction and Operation of Slow and Rapid Sand Filters[a]

Item	Slow sand filters[b]	Rapid sand filters[b]
Rate of filtration	1 to $\underline{4}$ to 8 $m^3/m^2/d$ (25 to $\underline{100}$ to 200 gal/ft^2/d)	100–475 $m^3/m^2/d$ (2 500–11 650 gal/ft^2/d)
Depth of bed	0.3 m (1 ft) of gravel 1.0–1.5 m (3.3–5 ft) of sand	0.5 m (1.6 ft) of gravel 0.75 m (2.5 ft) of sand
Size of sand	Effective size: 0.15 to $\underline{0.3}$ to 0.35 mm Uniformity coefficient: 2 to $\underline{2.5}$ to 3 (unstratified)	Effective size: 0.45 mm and higher Uniformity coefficient: 1.5 and lower (stratified)
Length of run	20 to $\underline{30}$ to 120 d	12 to $\underline{24}$ to 72 h
Penetration of suspended matter	Superficial (only the top layer is cleaned)	Deep (whole bed is washed)
Preparatory treatment of water	Generally aeration, but flocculation and sedimentation can be included.	Flocculation and sedimentation are essential
Method of cleaning	(1) Scraping off surface layer of sand and washing removed sand (2) Washing surface sand in place by traveling washer	Scour by mechanical rakes, air or water and removal of dislodged material by upward backwash flow
Costs		
Construction	Higher	Lower
Operation	Lower	Higher
Depreciation	Lower	Higher
Amount of wash water	0.2–0.6% of water filtered	1–6% of the water filtered

[a]Adapted in part from G. M. Fair, J. C. Geyer, and D. A. Okun (1968), *Water Purification and Wastewater Treatment and Disposal,* vol. 2, copyright © 1968 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

[b]Average values are underlined.

Denoting the surface area and volume of an irregularly shaped particle as A_p and V_p, respectively, ψ is

$$\psi = \frac{6\pi d^2}{\pi d^3}\frac{V_p}{A_p} \quad (V_p = V_s) \qquad \text{and} \qquad \frac{A_p}{V_p} = \frac{6}{\psi d} \tag{14.1}$$

(Many authors define $A_p/V_p = S$, which appears throughout their equations). The surface area of a sphere is smaller than for any other particle geometry; therefore ψ is always less than 1.

Table 14.2 gives a classification of media shapes and porosities. Media commonly used for filtration are shown in Table 14.3 along with their properties.

14.2.1 Grain Size and Distribution

The effective sizes (defined in Eq. 14.2) of available media may be too coarse or too fine and they may not be of the required uniformity. Grain size distribution in the medium is determined from a sieve analysis. Size openings of the United States sieve

TABLE 14.2 Particle Sphericity and Porosity

Description	Sphericity (ψ)	Typical porosity (e)
Spherical	1.00	0.38
Rounded	0.98	0.38
Worn	0.94	0.39
Sharp	0.81	0.40
Angular	0.78	0.43
Crushed	0.70	0.48

series are given in Table 14.4. The cumulative percent weight of medium passing a given sieve size is plotted on either log-normal or arithmetic probability paper, whichever gives the best straight line relation. The mean size and standard deviation of the medium can be read or calculated from the 50th and 16th percentile values, respectively.

The size–frequency parameters that are used to characterize the media for filtration are the effective size, ES, which is the 10th percentile value, P_{10}, and the uniformity coefficient, U, which is the ratio of the P_{60} to P_{10} value.

$$\text{Effective size} = \text{ES} = P_{10} \tag{14.2}$$

$$\text{Uniformity coefficient} = U = P_{60}/P_{10} \tag{14.3}$$

where

P represents the percent by weight equal to or less than the size

Note that as the uniformity coefficient increases, the medium is less uniform.

The 10th percentile value is chosen for the ES because it has been found by Hazen (1892) that the hydraulic resistance of sand beds is relatively unaffected by size variation (up to a uniformity coefficient of 5.0) as long as the 10th percentile sand size remains unchanged. The uniformity coefficient describes 50% of the sand relative to the ES.

The d_{10}, d_{60}, and d_{90} values are used in various equations describing filter behavior. These are the diameters of the 10th, 60th, and 90th percentile sand sizes, respectively. If the media sizes are assumed to have a log-normal distribution, the following equation describes the relation between the d_{90} and d_{10} sizes:

$$d_{90} = d_{10}U^{1.67} \tag{14.4}$$

TABLE 14.3 Filter Media Characteristics

Material	Shape	Sphericity	Relative density	Porosity %	Effective size mm
Silica sand	Rounded	0.82	2.65	42	0.4–1.0
Silica sand	Angular	0.73	2.65	53	0.4–1.0
Ottawa sand	Spherical	0.95	2.65	40	0.4–1.0
Silica gravel	Rounded		2.65	40	1.0–50
Garnet			3.1–4.3		0.2–0.4
Crushed anthracite	Angular	0.72	1.50–1.75	55	0.4–1.4
Plastic			Any characteristics of choice		

TABLE 14.4 United States Standard Sieve Size Openings

Sieve designation number[a]	Size of opening mm	Sieve designation number[a]	Size of opening mm
200	0.074	30	0.59
140	0.105	25	0.71
100	0.149	20	0.84
80	0.178	18	1.00
70	0.210	16	1.19
60	0.249	14	1.41
50	0.297	12	1.68
45	0.350	8	2.36
40	0.419	6	3.36
35	0.500	4	4.76

[a]Approximately the number of meshes per inch.

The sand or other media available may not meet ES and uniformity coefficient specifications and so will have to be tailored to requirements. A uniformity coefficient below 1.3 is generally not achievable by manufacturers and a value of 1.5 will have a cost premium associated with it. For less expensive, less uniform media, the fines may be washed out and the coarse particles can be screened out. An outline of the procedure (Fair, et al., 1968) to determine the sizes above and below which media should be discarded, is given next.

From a medium that does not meet size specifications there is a usable portion, P_{use}, a portion that is too fine, P_f, and a portion that is too coarse, P_c. Sand is used as the stock medium here.

Therefore,

$$P_{use} + P_f + P_c = 100 \tag{14.5}$$

All of the sand that lies between the specified sizes of the 10th percentile value and the 60th percentile value is usable.

$$d_{60} = Ud_{10}$$

For the stock sand, P_{st10} and P_{st60} are defined as the percentages of stock sand that are less than the specified P_{10} and P_{60} sizes, respectively.

The amount of sand that lies between the P_{10} and P_{60} sizes comprises 50% of the specified sand. The total usable sand is

$$P_{use} = 2(P_{st60} - P_{st10}) \tag{14.6}$$

Ten percent of the usable sand can be below the specified P_{10} size. For the portion of the stock sand that is smaller than the specified P_{10} size, it would be most desirable to remove the smaller sizes and retain the larger sizes in the amounts required. The percentage of usable stock sand below the P_{10} size is equal to $0.1P_{use}$. The percentage of stock sand that is too fine is

$$P_f = P_{st10} - 0.1P_{use} = P_{st10} - 0.2(P_{st60} - P_{st10}) \tag{14.7}$$

The percentage of stock sand that is too coarse is the remaining portion determined from Eq. (14.5).

$$
\begin{aligned}
P_c &= 100 - P_f - P_{use} \\
&= 100 - P_{st10} + 0.2(P_{st60} - P_{st10}) - 2(P_{st60} - P_{st10}) \\
&= 100 - P_{st10} - 1.8(P_{st60} - P_{st10})
\end{aligned}
\qquad (14.8)
$$

Again it is desirable to retain sand as close as possible to the P_{st60} size. The coarse portion P_c, to be removed, will consist of the largest grain sizes.

■ Example 14.1 Determination of Usable Sand from a Stock Sand

Figure 14.3 and the following table give the size distribution by weight of a local sand that has an effective size of 0.031 cm and a uniformity coefficient of 2.3. A log-normal distribution satisfactorily describes the medium's size variation as observed from the plot. The filter sand specifications are an ES (d_{10}) of 0.050 cm and a uniformity coefficient of 1.4.

The d_{60} size is

$$d_{60} = Ud_{10} = 1.4(0.05\ \text{cm}) = 0.70\ \text{cm}$$

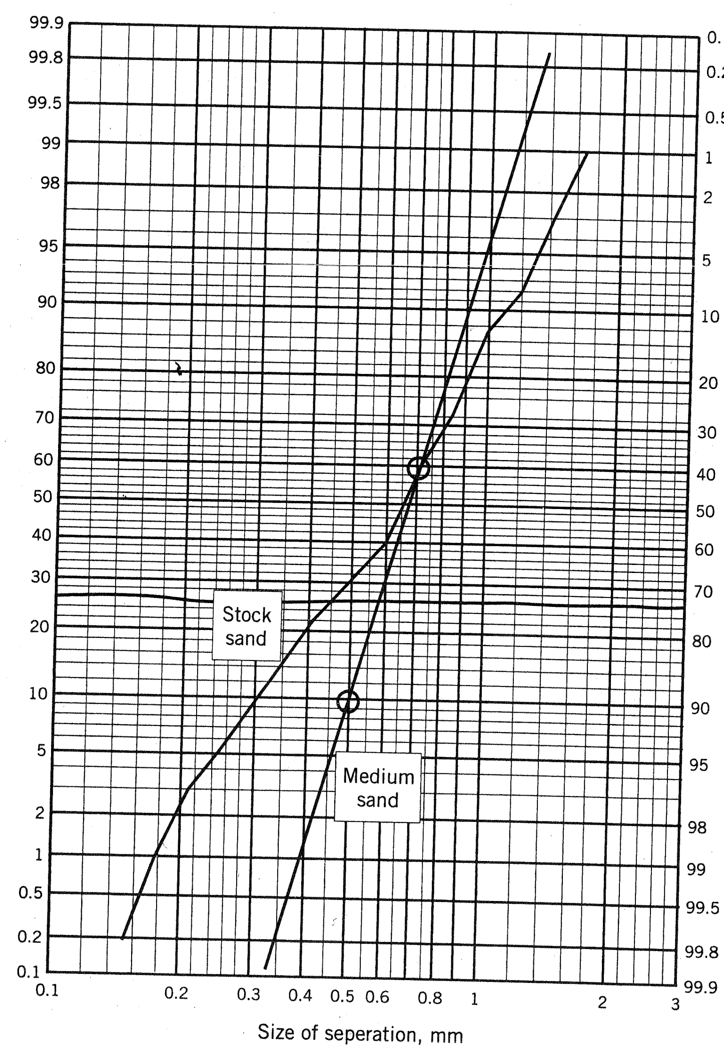

Figure 14.3 Sand grain size distribution for example.

Size of opening mm
0.149
0.178
0.210
0.249
0.297
0.350
0.419
0.500

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14.3

Headl
solids

Size of opening mm	Cumulative weight %	Size of opening mm	Cumulative weight %
0.149	0.2	0.59	40
0.178	1.0	0.71	60
0.210	3.0	0.84	72
0.249	5.1	1.00	85
0.297	8.9	1.19	92
0.350	15	1.41	97
0.419	22	1.68	99
0.500	30		

I.e. Between 0.7mm AND 0.5mm

Stoc 30% of sand is between 0.5–0.7 mm Add that is 50% of total, so X2.

From Eq. (14.6), the proportion of usable stock sand is

$$2(60\% - 30\%) = 60\% = P_{us}$$

The percentage of sand that is too fine from Eq. (14.7) is

$$30\% - 0.2(60\% - 30.0\%) = 24\%$$

<0.50 mm ? 10% of useable

Therefore, it is desired to remove the smallest 24% of the stock sand, which is all sand below the 0.044 cm size. From Eq. (14.8) the percentage of the local sand that is too coarse is

Tot − P_f − P_{use}

$$100\% - 24\% - 60\% = 16\%$$ ← Too coars

The largest 16% of stock sand or sand with a diameter above 0.085 cm is to be discarded.

Media that are too coarse must be removed by sieving. Fines may be removed from a medium by passing water in an upflow direction through the medium. The water velocity required to transport a particle will be just beyond the settling velocity of the particle. A particle moving upward at a constant velocity, v_u, will not be accelerating and the drag force on the particle will depend on the relative velocity of the fluid with respect to the particle. The force balance is as given by Eq. (11.8) using the relative velocity in the drag force term.

$$(\rho_p - \rho)gV_p = \tfrac{1}{2}\rho C_D A_p v_r^2$$

where

v_r is the relative velocity of the fluid with respect to the particle. $v_r = v_f - v_u$ (v_f is the fluid velocity).

Other terms are as defined for Eq. (11.8).

This equation is solved using the drag coefficient relations, Eqs. (11.12a)–(11.12c); however, the Reynold's number should be calculated with the relative velocity of the particle with respect to the fluid.

14.3 HEADLOSS IN FILTERS

Headloss in a filter is a complex function of flow rate, pressure, influent suspended solids concentration, and characteristics of the suspended solids and filter media.

It continuously varies with time and position in the bed. A classical equation was developed by Carman (1937) to describe overall headloss in porous media. The derivation can be started from the Darcy–Weisbach equation for pressurized flow in a closed conduit because flow through a filter is pressurized flow.

$$h_L = f\frac{L}{D}\frac{v^2}{2g} \tag{14.9}$$

where

h_L = headloss
L = length of travel (bed depth)
f = friction factor
D = pipe diameter
v = velocity of flow

Laminar flow conditions are assumed. The problem is to adapt this equation to a filter (porous media), incorporating relevant filter characteristics that are easily determined.

1a. The diameter, D, is replaced by the hydraulic radius, R, of a noncircular section.

$$R = \frac{\text{area}}{\text{wetted perimeter}}$$

For a circular section,

$$R = \frac{\pi D^2}{4\pi D} = \frac{D}{4}$$

Substituting the hydraulic radius into the equation results in:

$$h_L = f\frac{Lv^2}{8gR}$$

1b. The hydraulic radius of a filter is not well defined but a reasonable working definition of the hydraulic radius can be obtained by considering the definition of the hydraulic radius. The hydraulic radius relates the area of flow to the wetted perimeter providing resistance to flow. If the numerator and denominator are multiplied by suitable length parameters, dimensionality of the hydraulic radius is maintained.

$$R = \frac{\text{area}}{\text{wetted perimeter}} \approx \frac{\text{volume (available for flow)}}{\text{total surface area of the particles}}$$

The hydraulic radius can now be related to media characteristics. Defining the number of particles as N and the volume and surface area of a single particle as V_p and A_p, respectively, the total volume, V_{Tp} and total surface area, A_{Tp} of the particles are

$$V_{Tp} = NV_p \qquad A_{Tp} = NA_p$$

The volume available for flow can be calculated using the porosity, e, of the filter bed.

$$e = \frac{\text{void volume}}{\text{filter volume}} \qquad 1 - e = \frac{\text{volume of particles}}{\text{filter volume}}$$

Using the above two relations,

$$V = \frac{NV_p}{1 - e} \quad (14.10a) \qquad V_v = \frac{eNV_p}{1 - e} \quad (14.10b)$$

where
V is the volume of the filter bed
V_v is the void volume

Substituting the appropriate relations into the modified definition of the hydraulic radius,

$$R = \frac{\dfrac{eNV_p}{1 - e}}{NA_p} = \left(\frac{e}{1 - e}\right)\frac{V_p}{A_p} \quad (14.11)$$

2. The volume and surface area of the particles are related by their sphericity. From Eq. (14.1),

For a spherical particle $\dfrac{V_p}{A_p} = \dfrac{d}{6}$ For an irregularly shaped particle $\dfrac{V_p}{A_p} = \psi\dfrac{d}{6}$

3. Substituting for the hydraulic radius and V_p/A_p the equation for headloss becomes

$$\frac{h_L}{L} = f\frac{3v^2(1 - e)}{4g\psi de}$$

The velocity, v, in the above equation is the average velocity in the pores. The superficial velocity, v_s, or velocity related to the surface area of the filter is normally used in the final equation. The superficial velocity is also referred to as the surface loading rate.

$$v_s = \frac{Q}{A_s} \qquad \text{and} \qquad v_s = ev$$

where
Q is the volumetric flow rate
A_s is the surface area of the filter

The resulting equation, known as the Carman–Kozeny equation, is

$$\frac{h_L}{L} = f\frac{3(1 - e)v_s^2}{4g\psi de^3} = f_f\left(\frac{1 - e}{e^3}\right)\frac{v_s^2}{\psi dg} \quad (14.12)$$

where
the numerical constants are incorporated into the filter friction factor, f_f

The friction factor, f_f, is a function of the Reynold's number.

$$\text{Re} = \frac{\rho v_s d}{\mu} \quad (14.13a) \qquad \text{Re} = \frac{\rho v_s \psi d}{\mu} \quad (14.13b)$$

The definition used for Re may vary from author to author. Using the definition in Eq. (14.13b), a commonly used relation for f_f is (Ergun, 1952),

$$f_f = 150\frac{1 - e}{\text{Re}} + k \quad (14.14)$$

where

k is a constant

Ergun originally reported the value of k to be 1.75 and this value has been commonly accepted. Substituting Eq. (14.14) into Eq. (14.12):

$$\frac{h_L}{L} = \frac{150\mu}{\rho g}\frac{(1-e)^2}{e^3}\frac{v_s}{(\psi d)^2} + k\frac{1-e}{e^3}\frac{v_s^2}{\psi d g} \tag{14.15}$$

The first term in Eq. (14.15) is due to losses under laminar flow conditions and the second term applies to losses caused by turbulent conditions. Camp (1964) found that laminar flow conditions applied up to a Re number (based on Eq. 14.13a) of 6. When Re is less than 6, the second term of Eq. (14.15) can be ignored.

Equation (14.15) applies to the clean bed. As time passes, the porosity in the filter decreases because of the accumulation of solids and, concomitantly, the friction factor increases because of the restricted paths available for flow. It is difficult to model the change in the rate of headloss as the run progresses. Practically, filters are operated until a terminal headloss of 1.5 to 2 m (5 to 6.5 ft) is reached.

Metcalf and Eddy (1991) present a semi-empirical approach based on the work of Tchobanoglous and Eliassen (1970) for modeling headloss development in wastewater filters.

14.3.1 Grain Size Distribution and Headloss

A more refined estimate of the initial headloss can be made by taking into account the size distribution of the medium. From a sieve analysis the percentage by weight of each size of particles will be known. The medium can be separated into fractions by weight, x_i, of particles of mean nominal diameter, d_i.

$$\Sigma x_i = 1.0$$

Assuming that the porosity is the same throughout the entire bed, the length, l_i, associated with particles of size d_i is

$$l_i = x_i L$$

The headloss in each depth l_i will vary because d_i changes; the friction factor (Eq. 14.14) also changes because of the variation in d_i.

$$h_{Li} = \left(\frac{1-e}{e^3}\right)\frac{v_s^2}{\psi g} f_{fi}\frac{l_i}{d_i}$$

where

h_{Li} is the headloss of the ith layer

$$h_L = \sum h_{Li} = \left(\frac{1-e}{e^3}\right)\frac{v_s^2}{\psi g} L \sum f_{fi}\frac{x_i}{d_i} \tag{14.16}$$

If a functional relation is developed between x_i and d_i and f_f is relatively constant:

$$h_L = \left(\frac{1-e}{e^3}\right)\frac{v_s^2}{\psi g}Lf_{fi}\int_0^1 \frac{dx}{d} \tag{14.17}$$

In a multimedia filter the above equations are applied separately to each medium if porosity varies from one medium to another.

■ Example 14.2 Initial Headloss in a Dual-Media Filter

Calculate the initial headloss in a dual-media filter containing anthracite and Ottawa sand with depths of 0.45 and 0.30 m (1.5 and 1 ft), respectively. The effective size and uniformity coefficient of the anthracite are 0.85 mm and 1.5, respectively. The corresponding characteristics for the Ottawa sand are 0.55 mm and 1.35, respectively. The sphericities of the sand and anthracite are 0.95 and 0.72, respectively. Use a s.g. of 1.5 for anthracite and information in Table 14.3 for other media characteristics. Perform the calculation for a temperature of 10°C and surface velocity of 175 $m^3/m^2/d$ (4 290 $gal/ft^2/d$).

The P_{60} sizes for the media are

anthracite: $d_{60} = Ud_{10} = 1.5(0.85 \text{ mm}) = 1.27 \text{ mm}$

sand: $d_{60} = Ud_{10} = 1.35(0.55 \text{ mm}) = 0.74 \text{ mm}$

The media size distributions are obtained by plotting the P_{10} and P_{60} sizes for each medium on probability paper and drawing a straight line through them. The media size distribution data obtained from these plots are tabulated below.

Size Distribution of Media

Percentiles (by weight) of media	d_1 mm	d_2 mm	Mean size[a] mm
Anthracite			
5–20[b]	0.72	1.00	0.85
20–40	1.00	1.18	1.09
40–60	1.18	1.27	1.22
60–80	1.27	1.53	1.39
80–95[b]	1.53	1.81	1.66
Sand			
5–20[b]	0.51	0.61	0.56
20–40	0.61	0.68	0.64
40–60	0.68	0.74	0.71
60–80	0.74	0.82	0.74
80–95[b]	0.82	0.93	0.87

[a]The mean size is the geometric mean size because a probability plot is used. $d = \sqrt{d_1 d_2}$.
[b]The 5th and 95th percentile sizes were chosen to represent the extreme sizes.

The headloss calculations are performed by calculating f_{fi} (Eq. 14.14) and then calculating the term after the summation sign in Eq. (14.16) to each layer. The results are shown for each layer in the table of media sizes. Then Eq. (14.16) is applied to each medium.

Headloss Calculations

Percentiles (by weight) of media	Mean size mm	Re[a]	f_{fi}	x_i/d_i[b] mm^{-1}	$f_{fi}\dfrac{x_i}{d_i}$ mm^{-1}
Anthracite					
5–20	0.85	1.02	67.6	0.236	15.9
20–40	1.09	1.31	53.2	0.184	9.8
40–60	1.22	1.48	47.4	0.163	7.7
60–80	1.39	1.68	41.8	0.143	6.0
80–95	1.66	2.01	35.3	0.120	4.2
				Total	43.7
Sand					
5–20	0.56	0.82	111.5	0.359	40.0
20–40	0.64	0.95	96.8	0.311	30.0
40–60	0.71	1.04	88.0	0.282	24.8
60–80	0.74	1.15	80.3	0.257	20.6
80–95	0.87	1.28	71.8	0.229	16.4
				Total	131.9

[a]The porosities of anthracite and sand are 0.55 and 0.40, respectively, from Table 14.3.

[b]The fractions, x_i, were taken to be 0.20 for each mean size.

$$h_L = h_{La} + h_{Ls} = \left(\frac{1-e_a}{e_a^3}\right)\frac{v_s^2}{\psi_a g}L_a\sum\left(f_{fai}\frac{x_{ai}}{d_{ai}}\right) + \left(\frac{1-e_s}{e_s^3}\right)\frac{v_s^2}{\psi_s g}L_s\sum\left(f_{fsi}\frac{x_{si}}{d_{si}}\right)$$

$$h_L = \frac{(1-0.55)\left[175\,\frac{\text{m}}{\text{d}}\left(\frac{1\text{ d}}{86\,400\text{ s}}\right)\right]^2(0.45\text{ m})}{(0.55)^3(0.72)(9.81\text{ m/s}^2)}(43.7\times10^3\text{ m}^{-1})$$

$$+\frac{(1-0.40)\left[175\,\frac{\text{m}}{\text{d}}\left(\frac{1\text{ d}}{86\,400\text{ s}}\right)\right]^2(0.30\text{ m})}{(0.40)^3(0.95)(9.81\text{ m/s}^2)}(131.9\times10^3\text{ m}^{-1})$$

$$= 0.032\text{ m} + 0.163\text{ m} = 0.195\text{ m}$$

In U.S. units:

$$h_L = \frac{(1-0.55)\left[4\,290\,\frac{\text{gal}}{\text{ft}^2\text{-d}}\left(\frac{1\text{ ft}^3}{7.48\text{ gal}}\right)\left(\frac{1\text{ d}}{86\,400\text{ s}}\right)\right]^2(1.5\text{ ft})}{(0.55)^3(0.72)(32.2\text{ ft/s}^2)}(43.7\times10^3\text{ m}^{-1})\left(\frac{1\text{ m}}{3.28\text{ ft}}\right)$$

$$+\frac{(1-0.40)\left[4\,290\,\frac{\text{gal}}{\text{ft}^2\text{-d}}\left(\frac{1\text{ ft}^3}{7.48\text{ gal}}\right)\left(\frac{1\text{ d}}{86\,400\text{ s}}\right)\right]^2(1\text{ ft})}{(0.40)^3(0.95)(32.2\text{ ft/s}^2)}(131.9\times10^3\text{ m}^{-1})\left(\frac{1\text{ m}}{3.28\text{ ft}}\right)$$

$$= 0.102\text{ ft} + 0.543\text{ ft} = 0.645\text{ ft}$$

14.4 BACKWASHING FILTERS

When headloss through the filter reaches a set value, a rapid filter is backwashed to remove the accumulated solid matter (Fig. 14.4). At many plants water sent to a filter

A backwash velocity of 32 m/h (105 ft/h) will achieve an overall bed expansion of 10%. Note that the two layers are expanded to different degrees. The minimum fluidization velocity does not achieve any expansion of the anthracite layer; however, the backwash rate of 26.8 m/h ($1.3v_{mf}$ for the anthracite; 88 ft/h in U.S. units) is near the velocity predicted by Eqs. (14.31) and (14.32) to achieve expansion of this layer. On the other hand, the minimum fluidization velocity for the sand layer is near the minimum fluidization velocity calculated with Dharmarajah and Cleasby's approach. With the 1.3 adjustment factor (resulting in a velocity of 27.3 m/h), the expansion of the sand layer is about 10% which should be sufficient to expand the largest particles.

The results in the table are reasonably close to the observations recorded in Table 14.5. Bed fluidization is a complex hydrodynamic phenomenon dependent on a number of fluid and medium characteristics as well as the manner in which the flow is introduced into the bed. A more accurate estimate of the expansion could be obtained by applying the equations to individual layers within each medium.

14.5 SUPPORT MEDIA AND UNDERDRAINS IN SAND FILTERS

The media in a filter are supported by graded gravel layers that prevent the media from reaching and clogging the water collection underdrains. Figure 14.6 shows two commonly used gravel layer gradations. Headloss through the gravel layers is strictly a function of the filtration velocity. The gravel layers do not clog as filtration progresses. The Ergun equation (Eq. 14.15) can be used to calculate the headloss through each layer in a manner similar to the procedure in Example 14.2. Porosities in gravel vary from 0.18 to 0.35 for coarse to fine gravel (Reed et al., 1988).

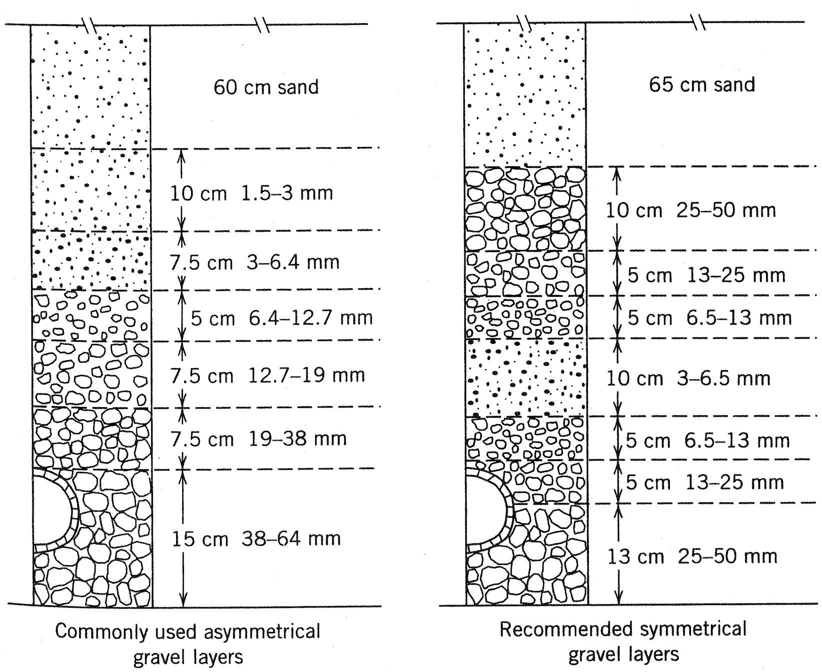

Figure 14.6 Supporting gravel layers for sand filters. From G. M. Fair, J. C. Geyer, and D. A. Okun (1968), *Water Purification and Wastewater Treatment and Disposal*, vol. 2, copyright © 1968 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

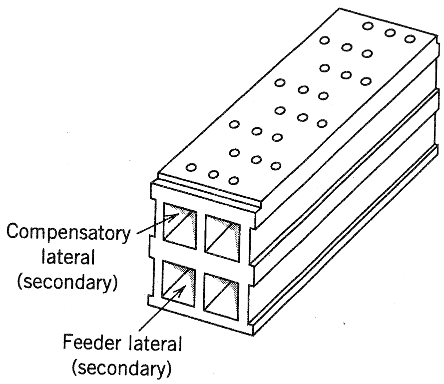

Figure 14.7 Underdrain block for filters.

Backwash velocities will not be sufficient to expand the supporting gravel layers. The backwash velocity should be slowly increased over a period of at least 30 s to avoid disturbing the supporting gravel layers. Headloss through the gravel layers during backwash is also calculated with the Ergun equation.

Underdrain systems serve both collection of the filtered water and uniform distribution of the backwash water. There are a variety of underdrain designs. Figure 14.7 shows one type of underdrain that may be made of plastic or vitrified clay. These blocks are laid across the filter bottom to form continuous channels that feed into larger collection pipes. Some companies supply strainer underdrains, as shown in Fig. 14.8, that do not require a supporting gravel layer.

Headloss through the underdrain openings during filtration or backwash is calculated with an orifice equation. The flow is distributed through all of the orifices to calculate the velocity through each orifice.

$$h_L = \Sigma h_l = nC_d \frac{v^2}{2g} \tag{14.36}$$

where

- h_L is the total headloss through the orifices
- h_l is the headloss through an individual orifice
- C_d is the discharge coefficient for the orifice
- n is the number of orifices

The orifices drain into channels that feed into larger collection channels. Flow in all underdrains is pressurized and the Darcy-Weisbach equation is suitable for calculating headloss in these channels. Headlosses through valves, bends, and other appurtenances

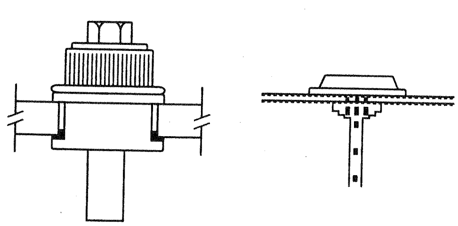

Figure 14.8 Strainers used in false-bottom underdrains without gravel.

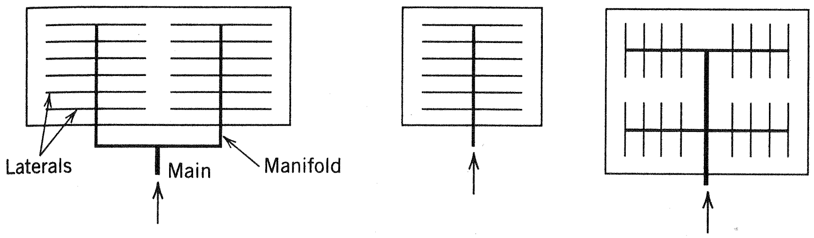

Figure 14.9 Filter underdrain systems. From G. M. Fair, J. C. Geyer, and D. A. Okun (1968), *Water Purification and Wastewater Treatment and Disposal,* vol. 2, copyright © 1968 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

in the channels and pipes from the filter are proportional to the velocity head through the device.

Pipe lateral systems can also be used for underdrains as shown in Fig. 14.9. Laterals are perforated with orifices placed directly downward or in two rows at 45° to the vertical. Orifice diameters range between 6 and 20 mm (0.2–0.8 in.) and are spaced at 6.5–30 cm (2.5–12 in.). Spacing of laterals is equal to the spacing of orifices. High headloss and poor distribution of backwash have been encountered with these systems which has led to a decline in their use in recent times (ASCE and AWWA, 1990).

Other Design Features of Filters

Conduits in filters are designed for velocities near the following ranges:

Influent pipe to filters	0.6–1.8 m/s (2–6.0 ft/s)
Effluent pipe carrying filtered water	0.9–1.8 m/s (3.0–6.0 ft/s)
Drains carrying spent backwash water	1.2–2.4 m/s (4–8 ft/s)
Wash water line (influent)	2.4–3.7 m/s (8–12 ft/s)
Filter to waste drain	3.7–4.8 m/s (12–15.5 ft/s)

Auxiliary Wash Systems

The primary scouring mechanism to cleanse the media during backwashing is hydrodynamic shear (Amirtharajah, 1978a); abrasive scour caused by particle collisions is not significant. Auxiliary wash systems are incorporated into filter systems to promote particle collisions and improve backwashing performance. Two types of auxiliary wash systems are used: surface wash and air scour. The latter is commonly used in Europe.

A surface wash system supplies jets of water from nozzles located 2.5–5 cm (1–2 in.) above the fixed bed surface. The nozzles are directed 15 to 45° below the horizontal. Operating pressures are typically in the range of 350–520 kPa (50–75 psi) (Cleasby, 1990). Either a fixed pipe grid or rotating pipes are used. The nozzle orifice sizes are 2 to 3 mm (0.08–0.12 in.). The surface wash is initiated 1 or 2 min before the backwash flow is started and continued until 2 or 3 min before the backwash flow is terminated. Surface wash systems affect only the upper layers of the expanded bed.

Air scour systems deliver air across the entire area of the filter. The air is introduced at the bottom of the filter and causes particle contact to occur throughout the entire depth of media. These systems are more effective than surface wash systems.

There are two alternatives for applying air scour: air scour applied before the fluid backwash and simultaneous air scour and fluid backwash. In the former case air is supplied for 2 to 5 min before the fluid backwash. The water level in the filter is lowered because the entrained air adds volume to the filter. The expanded volume must not be allowed to rise above the overflow weirs and cause loss of media. After the air scour is terminated, the bed is backwashed at a velocity sufficient to fluidize the bed. Entrained air will be removed from the filter during backwash.

14.6 FILTER BEDS FOR WATER AND WASTEWATER TREATMENT

The design of a rapid filter system for water treatment depends on the treatment objectives and the pre-treatment that has been applied to the filter influent. Design information for filter beds for various applications is given in Table 14.7. The minimum number of individual filter beds is two. When only two beds are installed, a single bed must be capable of meeting water demands during periods of shutdown of either filter for maintenance and backwashing. In medium to large installations (flow greater than 35 000 m^3/d or 10 Mgal/d) at least four beds should be installed (ASCE and AWWA, 1990). The practical maximum area of an individual bed is approximately 150 m^2 (1 600 ft^2) (Kawamura, 1991).

Filtration of wastewater is becoming a more common practice to enhance suspended solids removal. More stringent wastewater treatment standards promote the practice. Typical design information for wastewater treatment filter beds is given in Table 14.8. For wastewater, monomedium filters are more common than dual-media filters. The maximum area of an individual filter for wastewater treatment is the same as for a water treatment filter (Culp et al., 1978).

For intermittent filtration of effluent from stabilization ponds, there are two basic configurations: single-stage intermittent filters or intermittent filters in series USEPA (1983). Single-stage intermittent filters use sand medium with a small effective size in the range of 0.20–0.30 mm. Uniformity coefficients are high, ranging from 5 to 10 for

TABLE 14.7 Design Features of Filter Beds for Water Treatment[a]

	Effective size	Total depth	
	mm	m	ft
A. *Common U.S. practice after coagulation and settling*			
1. Sand alone	0.45–0.55	0.6–0.7	2–2.3
2. Dual media Add anthracite (0.1 to 0.7 of bed)	0.9–1.1	0.6–0.9	2–3
3. Triple media Add 0.1 m (0.3 ft) garnet	0.2–0.3	0.7–1.0	2.3–3.3
B. *U.S. practice for direct filtration*			
Practice not well established. With seasonal diatom blooms, use coarser top size. Dual media coal, 1.5 mm effective size			
C. *U.S. Practice for Fe and Mn filtration*			
1. Dual media similar to A-2			
2. Single medium	<0.8	0.6–0.9	2–3
D. *Coarse single-medium filters washed with air and water simultaneously*			
1. For coagulated and settled water	0.9–1.0	0.9–1.2	3–3.9
2. For direct filtration	1.4–1.6	1–2	3.3–6.6
3. For Fe and Mn removal	1–2	1.5–3	4.9–9.8

[a]From J. L. Cleasby, (1990), "Filtration," in *Water Quality and Treatment*, 4th ed., F. W. Pontius, ed., McGraw-Hill, Toronto, reproduced with permission of McGraw-Hill, Inc.

TABLE 14.8 Design Features of Monomedium Filter Beds for Wastewater Treatment[a]

Characteristic	Value	
	Range	Typical
Shallow bed (stratified)		
Sand		
Depth, cm (in.)	25–30 (10–12)	28 (11)
Effective size, mm	0.35–0.6	0.45
Uniformity coefficient	1.2–1.6	1.5
Filtration rate, m/h (gal/ft^2/min)	5–15 (2–6)	7 (3)
Anthracite		
Depth, cm (in.)	30–50 (12–20)	40 (16)
Effective size, mm	0.8–1.5	1.3
Uniformity coefficient	1.3–1.8	1.6
Filtration rate, m/h (gal/ft^2/min)	5–15 (2–6)	7 (3)
Conventional (stratified)		
Sand		
Depth, cm (in.)	50–76 (20–30)	60 (24)
Effective size, mm	0.4–0.8	0.65
Uniformity coefficient	1.2–1.6	1.5
Filtration rate, m/h (gal/ft^2/min)	5–15 (2–6)	7 (3)
Anthracite		
Depth, cm (in.)	60–90 (24–36)	76 (30)
Effective size, mm	0.8–2.0	1.3
Uniformity coefficient	1.3–1.8	1.6
Filtration rate, m/h (gal/ft^2/min)	5–20 (2–8)	10 (4)
Deep bed (unstratified)		
Sand		
Depth, cm (in.)	90–180 (36–72)	120 (48)
Effective size, mm	2–3	2.5
Uniformity coefficient	1.2–1.6	1.5
Filtration rate, m/h (gal/ft^2/min)	5–24 (2–10)	12 (5)
Anthracite		
Depth, cm (in.)	90–215 (36–84)	150 (60)
Effective size, mm	2–4	2.75
Uniformity coefficient	1.3–1.8	1.6
Filtration rate, m/h (gal/ft^2/min)	5–24 (2–10)	12 (5)

[a]Metcalf and Eddy (1991), *Wastewater Engineering: Treatment, Disposal, Reuse,* 3rd ed., G. Tchobanoglous and F. L. Burton, eds., McGraw-Hill, Toronto, reproduced with permission of McGraw-Hill, Inc.

a number of installations. The other alternative is to employ two or more intermittent filters in series. Coarser media with an effective size of 0.60–0.70 mm are used in the first filter whereas the subsequent filters use media with smaller effective sizes in the range of 0.15–0.40 mm.

14.7 AIR BINDING OF FILTERS

A major problem of filter operation is air binding or the formation of gas bubbles in the filter. The release of dissolved gases can dislodge accumulated solids from the media, driving them deeper into the media and increasing the possibility of their escape

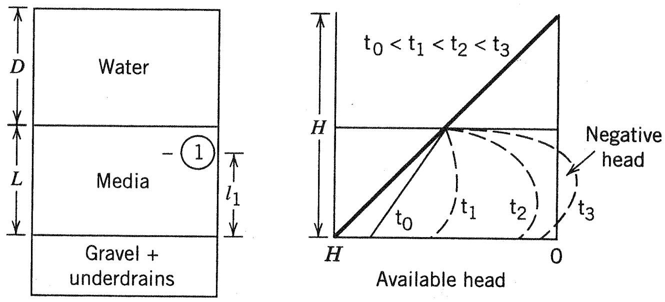

Figure 14.10 Headloss development in a filter.

in the filter effluent. Also, the entrapped gases take a portion of the media out of service, which leads to more rapid deterioration of the effluent quality and shortens run time. Air binding of a filter may result from:

1. Reduction of pressure in the filter to a value less than atmospheric pressure (negative head)
2. Increase of water temperature in the filter
3. Release of oxygen by algae growing in the filter

Measures can be taken to eliminate or reduce the possibility of occurrence of pressures less than atmospheric pressure in the filter. An examination of headloss and pressure variation within the filter is required; calculating the total headloss through the filter will not illustrate the problem. Figure 14.10 shows the progression of headloss development over time in a filter measured by inserting piezometer taps at various depths in the filter. The depth of water over the filter may be constant or vary with time but the headloss progression is similar in any case. The effluent for the filter depicted in Fig. 14.10 is discharged over a weir located at the same elevation as the bottom of the media in the filter. The pressure distribution for a filter with no water flowing simply follows a 45° line.

The available head for frictional dissipation of energy is the piezometric head minus the elevation head when the effluent weir from the filter is located at the same elevation as the bottom of the filter. As water flows downward for a distance it gains available head because of the additional depth of water above it; however, available head is decreased because of losses incurred in traveling the distance.

Once water begins to flow through a clean filter, the initial headloss follows the Carman–Kozeny equation and headloss is a linear function of depth (time t_0). Even in a multimedia filter, solids removal will be greatest in the upper layers and headloss will also be largest in these layers. Therefore, as time goes on, the available head curve becomes skewed to the right in the upper layers. As the water descends from the layer of maximum headloss, it recovers head. Flow is in accord with Darcy's law (see Problem 18). The available head at the bottom of the media must always be equal to the sum of the headlosses through the gravel, underdrains, and other appurtenances in line before the effluent overflow weir.

Extended operation of the filter before backwashing can produce a situation where the available head drops to a negative value in the upper layers (time t_3 in Fig. 14.10). This may cause air binding of the filter. This situation can occur when the overflow weir is located at the same elevation as the bottom of media in the filter and will be examined first. Consider a location (1) within the media (Fig. 14.10). Choosing a datum at the bottom of the media and applying Bernoulli's equation, the absolute pressure at this point may be determined. The energy at location 1 will be related to the energy at the water surface above the media (location 0). The velocity head is insignificant above and in the filter.

$$p_{\text{atm}} + \rho g H = p_1 + \rho g l_1 + \rho g h_{\text{L},0\text{-}1} \tag{14.37}$$

where

$h_{\text{L},0\text{-}1}$ is the headloss between the water surface and location 1
p_{atm} is atmospheric pressure
l_1 is the elevation of point 1 above the datum

Solving for p_1, the pressure at point 1 above the datum,

$$p_1 = p_{\text{atm}} + \rho g H - \rho g l_1 - \rho g h_{\text{L},0\text{-}1} \tag{14.38}$$

It is seen that if $l_1 + h_{\text{L},0\text{-}1}$ is greater than H, then the pressure decreases below atmospheric pressure. It is possible for this to occur in the upper layers given the higher removal of solids and the associated greater headlosses there. Similar results can be obtained by writing Bernoulli's equation between location 1 and the effluent weir located at the same level as the bottom of the filter media.

This situation may be remedied by increasing the elevation of the effluent weir (Fig. 14.11). If the effluent weir is located at the same elevation as the top of the media, the possibility of negative head never arises. Again, considering point 1 within the media and relating the total energy at this location to the total energy at the weir (now located at a height, L, above the datum) through Bernoulli's equation:

$$p_1 + \rho g l_1 = p_{\text{atm}} + \rho g L + \rho g h_{\text{L},1\text{-w}} \tag{14.39}$$

where

$h_{\text{L},1\text{-w}}$ is the headloss from location 1 to the top of the weir

Solving for p_1,

$$p_1 = p_{\text{atm}} + \rho g L - \rho g l_1 + \rho g h_{\text{L},1\text{-w}} \tag{14.40}$$

Because $L \geq l_1$, Eq. (14.40) shows that $p_1 > p_{\text{atm}}$. When the effluent weir is located at the height of the surface of the media, available head within the media is a monotonically decreasing function of depth below the media surface (Fig. 14.11b). The head at the bottom of the media is equal to the headlosses in the gravel, underdrain system, and other appurtenances in the filter effluent line. No available head is gained below the surface of the media.

This solution solves the problem, but as Monk (1984) advises, the safety factor involved is excessive. Increasing the elevation of the weir will require an increase in

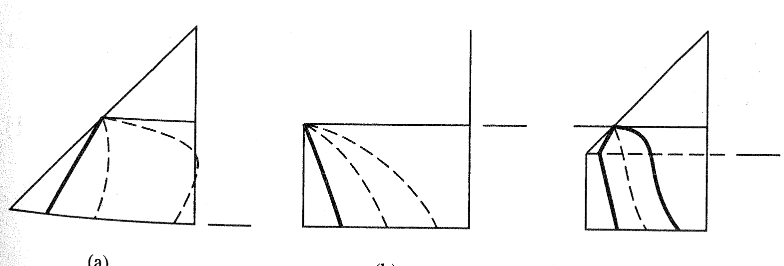

Figure 14.11 Available head development with different weir locations. Weir located at (a) bottom of filter; (b) top of media; (c) an intermediate depth.

depth of water over the media to achieve the same throughput of water. Some economies can be gained by lab studies on the headloss performance of the filter under probable operating conditions and the weir can be lowered and still provide a reasonable safety factor against the development of negative head. The available head curve for this situation is shown in Fig. 14.11(c). It is a combination of curves applying to the cases where the weir is located at the bottom (Fig. 14.11a) or top (Fig. 14.11b) of the filter.

14.8 RAPID FILTRATION ALTERNATIVES

There are some features that apply to all rapid filtration operations. It is essential that the influent to a rapid filter be coagulated. There will be a slight deterioration in filtrate quality after backwashing. Sudden increases in the rate of filtration also deteriorate the filter effluent quality. The bed must not be allowed to dewater.

Single-Medium and Multimedia Filters

Most of the solids removal in single-medium (sand) filters occurs in the top layers of the filter. The full depth of the medium is not effectively used and headloss increases rapidly. Replacing the upper depth of sand with coarser anthracite medium definitely retards the rate of headloss development and increases the length of a filter run. A dual-media filter does not necessarily improve the quality of the filtrate (Cleasby, 1990) but there is no deterioration of the filtrate over the longer run times.

The extension of the dual-media filter is the triple-media (multimedia) filter, which uses garnet or ilmenite media of finer size than sand in the bottom layer of the filter. As a result of a finer grain size being included in the filter, the initial clean bed headloss is larger for triple-media filters compared to single- or dual-media filters. But triple-media filters outperform single-medium filters for headloss development (Cleasby, 1990). The incorporation of finer sized media is expected to improve effluent quality. Cleasby (1990), in a review of multimedia filter studies, was unable to conclude that triple-media filters resulted in superior effluent quality compared to dual-media filters. Some studies were poorly designed and there was a lack of studies comparing dual- and triple-media filters. A comparison of dual-media and mixed media (anthracite, sand, and two size ranges of garnet) performance at a full-scale plant installation showed no significant differences in performance (Barnett et al., 1992).

Granular activated carbon (GAC) can be used as the top medium for taste and odor control and to adsorb organic compounds. GAC has a lower density than anthracite, which affects backwashing requirements.

Constant- and Declining-Rate Filtration

The rate of water throughput in a filter is a function of the headloss through the filter system and the driving head of water over the filter.

$$Q = f(h_D - h_c - h_u - h_f - h_v) \qquad (14.41)$$

where

h_D is the driving head (depth of water over the filter)
h_c is the clean bed headloss
h_u is the headloss through the gravel and underdrain system
h_f is the friction headloss, resulting from solids accumulation in the filter