Wastewater Filtration

Design Considerations

EPA Technology Transfer Seminar Publication

EPA-625/4-74-007a

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1991

ENVIRONMENTAL PROTECTION AGENCY • Technology Transfer

July 1974

ACKNOWLEDGEMENTS

This seminar publication contains materials prepared for the U.S. Environmental Protection Agency Technology Transfer Program and has been presented at Technology Transfer design seminars throughout the United States.

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Revised July 1977

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Chapter I

INTRODUCTION

Wastewater filtration is but one of the design engineer's alternatives which can be considered in waste treatment flow schemes to meet specified effluent quality objectives. He should consider it along with other alternatives, finally reaching a decision as to which of the several alternatives is cost effective. This publication presents the questions which must be asked in wastewater filtration, the alternatives available in answering the questions, and the design procedures involved in those alternatives.

It is presumed that the reader is familiar with granular media filtration from potable water experience or from study of textbook sources[1]. Therefore, this publication stresses special aspects related to wastewater filtration.

The first and most important question the designer must ask is whether filtration can meet the specified effluent quality goals. If the goal is to upgrade the effluent of an existing secondary treatment works, one must first evaluate the present performance and the reasons for that performance. For example, what portions of the present effluent BOD are of soluble and suspended origin? The filter can remove only a portion of the suspended BOD. If the effluent contains highly soluble BOD, the only solution may be to upgrade the secondary treatment. If the effluent contains primarily suspended BOD, effluent filtration or upgrading the secondary settling will be possible alternative solutions.

The expected performance of the granular filters can be estimated from performance at similar plants elsewhere, or by pilot studies at the plant in question. Appendix A presents a summary of operating results at a number of plants in the U.S. and the U.K. Similar compilations with more data from U.S. activated sludge plants are available in recent EPA design manuals[2,3]. The mean and range of the performance data from these two sources is summarized in table I-1.

The data in table I-1 and the sources from which it was derived indicate that a marginal secondary effluent could easily be upgraded to a 30-30 standard, and probably to a 20-20 standard, by tertiary filtration, i.e., without chemical treatment. A good secondary effluent which already meets the 30-30 standard may approach a 10-10 goal by tertiary filtration. If the effluent quality goal is less than 10-10, some form of chemical treatment will be needed in the secondary or in a tertiary stage prior to filtration.

After considering the effluent quality goals and the ability of granular filtration to achieve those goals, and if filtration is still one viable alternative, the following design questions must be considered in arriving at a successful installation:

- What are the appropriate flow schemes?
- What minimum filter run length is acceptable?
- What filter configurations are appropriate for wastewater?
- Is pilot scale testing needed, and if so, how should it be conducted?

- What filtration rate and terminal headloss should be provided?
- What filter media size and depth should be provided?
- Should gravity or pressure filters be provided?
- What system of flow control should be used?
- What backwash provisions are needed for each filter media alternative being considered to ensure effective backwashing in the long term?
- What underdrain system is appropriate for the media and backwash regime intended?

Each of these questions will be discussed in the following sections. But first, some contrasts between wastewater and potable water filtration will be presented.

Table I-1.—*Median and range of performance of wastewater filters, combined data from Appendix A and Reference 2. The data below give the range of mean values and the median of the means including all filtration rates from 2 to 6 gpm/sq ft (inclusive) and media sizes ≥ 1mm effective size*

Filter influent type	Suspended solids (mg/l)				BOD_5 (mg/l)			
	Influent		Effluent		Influent		Effluent	
	Range	Median	Range	Median	Range	Median	Range	Median
Tertiary filtration of trickling filter plant effluent	20-51	29	5-13	9	23-35	30	10-14	12
n = number of observations	*n* = 31		*n* = 31		*n* = 6		*n* = 6	
Tertiary filtration of activated sludge plant effluent	7-55	16	2-10	6	No data		No data	
	n = 23		*n* = 23					
Filtration after chemical treatment of secondary effluent	6-16	10	1-8	1.5	No data		No data	
	n = 7		*n* = 6					

Chapter II

CONTRASTS WITH POTABLE WATER EXPERIENCE

Most of the published information on the design and operation of granular media filters has been derived from experiences in potable water filtration. Such filters have been used in potable water production for the removal of solids present in surface waters pretreated by coagulation and sedimentation, for the removal of precipitates resulting from lime or lime/soda-ash softening, and for the removal of iron or manganese found in many underground water supplies. The design and operating experience in these applications has been carried over into wastewater applications, sometimes disastrously. In evaluating waterworks filtration experience for application in wastewater-filtration situations, several very important differences need to be emphasized.

With built-in raw and filtered water-storage capacity, water filters can be and generally are operated at constant filtration rates for long periods, and steady operating conditions will prevail. Thus, plant design can be based on the maximum day demand, not on hourly demand. In wastewater filtration, however, the plant must be designed to handle a continuously varying and highly unpredictable rate of flow, with variations from a nighttime dry weather flow to peak hourly flows in stormwater runoff periods. As a result, the potential effects of peak flow rates must be considered in the filter-plant design.

In waterworks experience, the water filtered is much more consistent in both the level of solids present and in their filtering characteristics (for example, iron-removal plants). Even with pretreated surface water, the solids to be removed by filtration consist of low levels (usually under 5-10 FTU) of floc carryover, with some attached colloidal solids contributing to the original raw water turbidity. The filtering characteristics of solids that are mainly inorganic are more predictable than the filtering characteristics of the inorganic-organic solids found in typical wastewaters to be treated. Even in pretreated surface water filtration, the daily variations in solids levels are less than those encountered in wastewater. Considerable data are available to demonstrate that in raw wastewater, the suspended solids levels will vary directly with the flow. Furthermore, treatment processes are least efficient under peak-load operating conditions. Therefore, high suspended solids may leave the final settling tanks of secondary treatment during peak-flow periods and be delivered to tertiary filters. Thus, applied to filters, the wastewater presents its highest solids concentration to be removed during the highest flow-rate periods. Even in well-operated plants, suspended solids loadings to tertiary treatment filters during peak-flow periods can reach 30-50 mg/l (15-25 FTU). Such loadings contribute to high headloss and a consequent potential for very short filter runs. Thus, the critical design condition to be considered occurs under peak-flow operating conditions.

With more uniform suspended solids levels and filtering characteristics, water-treatment filter efficiency is a function mainly of the filtration rate and the influent suspended solids concentration. In wastewater filtration, however, filtrate quality is less dependent on rate and influent suspended solids levels.

As explained previously, the operation of secondary biological-treatment plants results in wide variations in solids levels carried over to tertiary filters. The filtering characteristics of such solids are affected by the solids retention times maintained in the biological reactors. As a result, filtering characteristics are highly variable. Such solids, however, are much more "sticky" than water-plant solids, and are much more difficult to remove effectively during filter backwashing. If chemical treatment of the wastewaters is practiced, the floc carryover to the filters assumes more of the character of the chemical floc. Thus, a wastewater which has received a high degree of chemical pretreatment will filter more like the comparable floc of potable water experience.

Chapter III

FILTER DESIGN – GENERAL CONSIDERATIONS

WASTEWATER FILTRATION FLOW SCHEMES

In advanced wastewater treatment, filtration (figure III-1) is used for:

- Removal of residual biological floc in settled effluents from secondary treatment by trickling filters or activated-sludge processes—the primary emphasis of this presentation. It will be referred to as "tertiary filtration."

- Removal of precipitates resulting from alum, iron, or lime precipitation of phosphates in secondary effluents from trickling filters or activated-sludge processes. The suspended solids to be filtered can be substantially different from those in normal secondary effluent.

- Removal of solids remaining after the chemical coagulation of wastewaters in physical-chemical waste-treatment processes, i.e., following lime treatment of raw wastewater and before adsorption removal of soluble organics in carbon columns. Again, the solids to be filtered can be substantially different from normal secondary effluent solids.

Filters may be used as the final process of wastewater treatment (polishing secondary or tertiary effluents) or as an intermediate process to prepare wastewater for further treatment (for example, before downflow carbon adsorption columns or clinoptilolite ammonia exchange columns). In either case, the required filters should be designed to provide a quality of filtrate equal to or better than the desired effluent-quality goal at all times. Achieving this quality may require a pilot study to evaluate the flow characteristics and solids characteristics of the water to be filtered. In the absence of a pilot study, the design must be based on experience with similar filter influent waters at other installations.

The problem of potential short filter cycles during peak-load periods requires that a flow equalization tank be considered as part of the plant flow scheme. It may be installed near the plant entrance to benefit the entire treatment operation, or it could be installed just prior to the filters. Figure III-1 indicates that at this point, wastewater quality is such as to present no odor or mixing problems. Fifteen to twenty percent of mean daily-flow storage capacity would permit constant-rate flow to the filters for a 24-hour period. One hundred percent of mean daily-flow storage capacity would permit constant flow and nearly constant solids loads to the filters. Neither practice is widespread today, and the benefits to filtration alone may not jusitfy the costs for such provisions.

MINIMUM ACCEPTABLE FILTER RUN LENGTH

Since the capital cost of a filter is chiefly a function of the area of filter provided, the use of a high filtration rate is usually preferred. In general, filter design should seek to maximize the net water production per square foot of filter consistent with filter operating feasibility. Useful relationships between net water production and run lengths obtained at different filtration rates are shown in figures III-2 and III-3. There are two alternatives. The first is shown in figure III-2 and it occurs when filtered water is used for backwashing as all potable water filtration and most wastewater filtration plants. The second

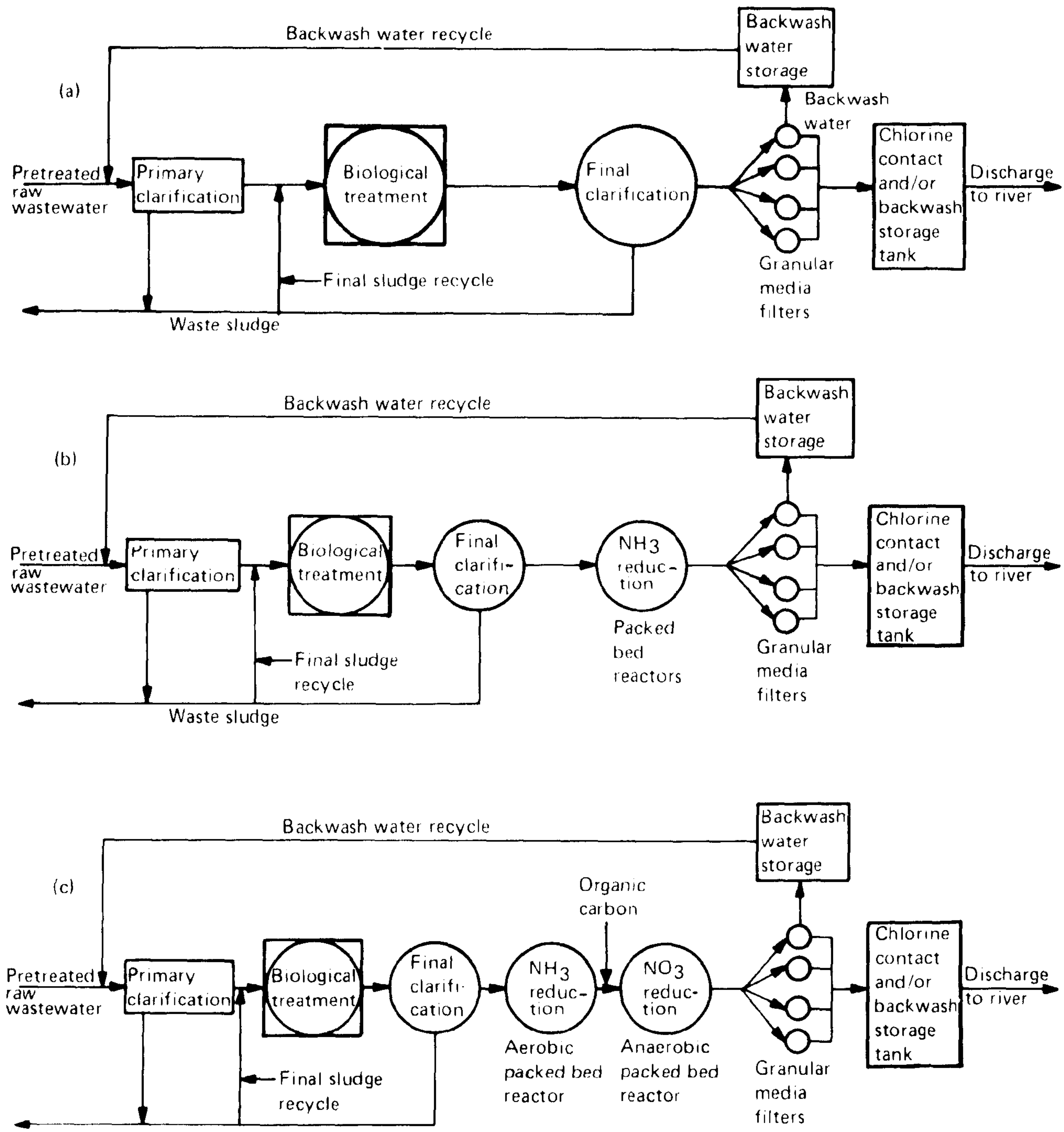

Figure III-1. Granular media filters for tertiary wastewater treatment: (a) following biological secondary treatment for carbonaceous BOD removal; (b) following biological secondary and biological tertiary (packed-bed reactors) treatment for carbonaceous BOD and ammonia reduction; (c) following biological secondary and biological tertiary (packed-bed reactors, both aerobic and anaerobic) for carbonaceous BOD, ammonia, and nitrate reduction. (Phosphorus levels may also be reduced by adding ferric or aluminum salts and a polymer feed to solids contact units located ahead of the granular-media filters.)

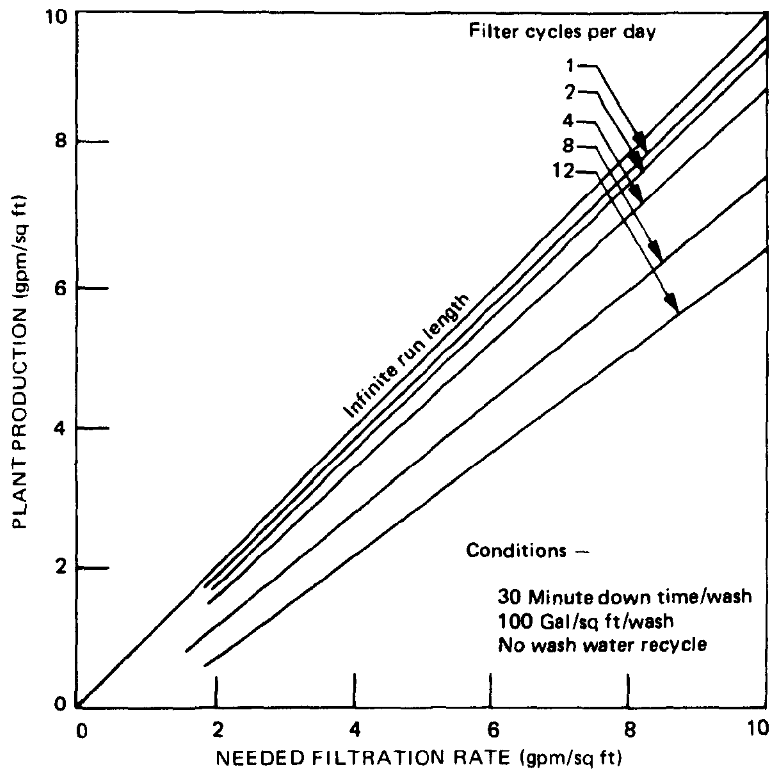

Figure III-2. Effect of number of filter cycles per day on filtrate production when using filtered water for backwashing.

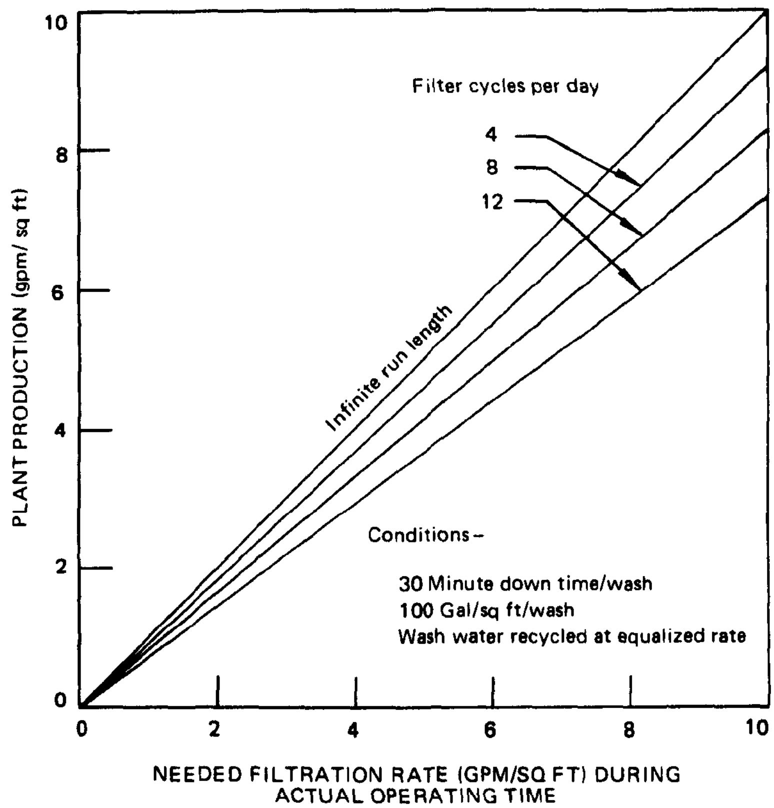

Figure III-3. Effect of number of filter cycles per day on filtrate production when using unfiltered water for backwashing. Plant production is the average plant output over the full 24 hour day.

case, shown in figure III-3, occurs when unfiltered water is used in backwashing. However, it is not generally recommended because of potential clogging of underdrain strainer or orifice openings.

The data for both figures was calculated assuming 30 minutes total down time per backwash to allow for draindown time, auxiliary scour time, actual backwash time and start-up time to reach normal rate. Also, the 100 gal/ft^2 total wash water per backwash is typical of volumes adequate for most filtration situations. In the case of recovered wash water, it is assumed that dirty wash water would pass through a holding tank to permit flow equalization of the recirculated water. A sample calculation for figure III-2 is shown below:

Backwashes per day (6-hour cycles) = 4

Downtime per backwash = 30 min

Actual filtration time (1,440) - (4 x 30) = 1320 min

Plant production = 4 gal/min/ft^2 x 1,440 min/day = 5,760 gpd/ sq ft

Backwash water used = 100 x 4 = 400 gal

Needed filtration rate (5760 gal + 400 gal)/1320 = 4.67 gal/min/ft^2

Backwash water as percent of production = (400/5760) x 100 = 6.9%

Figure III-2, which is appropriate for most wastewater filtration, would indicate little loss of production if the number of backwash cycles per day per filter is limited to four or less, i.e., 6-hour filter cycles or longer. Thus, under the peak-flow and suspended-solids load conditions predicted for the design year, the cycles should be at least 6 hours. Considering typical flow and solids load variations, this should result in 24-hour cycles under average design year loads.

One must keep in mind the conditions selected to construct figures III-2 and III-3. Some filters require more than 30 minutes to complete a backwash cycle, especially if complete gravity draindown is essential or desired. Some require more than 100gal/sq ft/ wash. If the downtime and water use for a particular type of filter are expected to deviate significantly from those used above, then the figures should be reconstructed and the cycle length decision reconsidered. Different backwashing routines are discussed in more detail in a later section of this manual.

FILTER CONFIGURATIONS

A filter configuration must be selected for wastewater filter which is appropriate for the higher influent solids anticipated as discussed in the previous sections. A granular media filter is intended to filter in depth, i.e., it is intended that solids removal take place within the filter, and not primarily at the entering surface.

A number of alternate filter configurations have been developed to accommodate the higher solids loads described above and to encourage filtration in depth. The alternates were developed because the conventional sand filter of United States design is not well-suited to handle high influent solids loads. This is due to the fact that the media is backwashed at a rate sufficient to expand the bed, and stratification of the media occurs—the smaller sand grains tending to collect near the top of the bed and the larger grains at the bottom. Since the conventional sand filter is operated with downward flow, the solids encounter the finest media first, and filtration in depth is not encouraged. The configurations developed to encourage filtration in depth, and thus to achieve longer filter cycles with higher influent solids loads, are illustrated in figure III-4. The most common method in the United States is the double-layer filter bed composed of a coarse layer of crushed anthracite coal over a layer of finer sand. Coal has a lighter specific gravity than sand, and when the proper size, it remains on top during backwashing if the backwash rate is sufficient to achieve full fluidization and some expansion of the bed. The coarse coal removes the major portion of the sediment, allowing significant depth of penetration, while the fine sand layer polishes the water to provide an effluent of good quality. The crushed coal is angular in shape and has a higher porosity than sand (about 0.50 versus 0.40). Therefore, the coal has a greater storage capacity for solids removed by filtration. One manufacturer carries the concept one step further with another layer of finer and heavier garnet or illmenite under the silica sand. The three layers are sized to encourage some intermixing between the layers.

European practice has included deeper filters of coarse sand[4] (hereinafter referred to as single media unstratified filters) backwashed without expansion using air and water simultaneously so that stratification does not occur.

European practice also includes upflow and middle outlet sand filters. The upflow filter achieves its benefit because of the graded gravel support at the bottom which acts as a roughing filter prior to the deep bed of unstratified coarse sand above. However, there is a danger of lifting part of the filter bed during the filtration cycle, which is seriously detrimental to the filtrate quality. This is overcome with a retaining grid of steel bars on 4- to 6-inch centers which resists the lifting tendency during the filtration cycle. This type of filter has received considerable attention in Great Britain for wastewater filtration because of its high solids capacity[5,6]. Several full scale plants are in service. The lifting problem is overcome by a different approach in the center outlet "Bi-Flow" filter which has an inlet at both top and bottom. The flow automatically distributes itself so that the upward and downward forces balance each other, eliminating the uplift danger of the upflow filter. The extent of full-scale use of the "Bi-Flow" design is not known to the authors.

The advantages and disadvantages of these various configurations are partly related to the backwashing requirements and are discussed in a general way below. Further details on backwash design are presented later.

The advantages of the dual media filter include the fact that the practice is well established in U.S. potable water practice, and the increased solids handling capacity of the coal layer is achieved with a relatively shallow media because the sand layer is provided to protect the filtrate quality. The shallow media reduces structural costs. One disadvantage is that full fluidization is essential during the water backwash to ensure that the coal will remain on top after backwash. This requirement may dictate excessive wash rates if a coarse coal is selected to lengthen filter cycles. Another disadvantage is incurred if media-retaining underdrain strainers are provided to eliminate the use of supporting graded gravel. The fine sand of the lower layer requires very fine openings in the strainers, and clogging problems may result.

The triple media filter has similar requirements, advantages and disadvantages as the dual media filter. It is a proprietary design and less experience is available than with dual media. In some applications, the third media may not improve filtrate quality as will be shown later. These filters are usually supported on graded gravel, thus avoiding the dangers of strainers with fine openings.

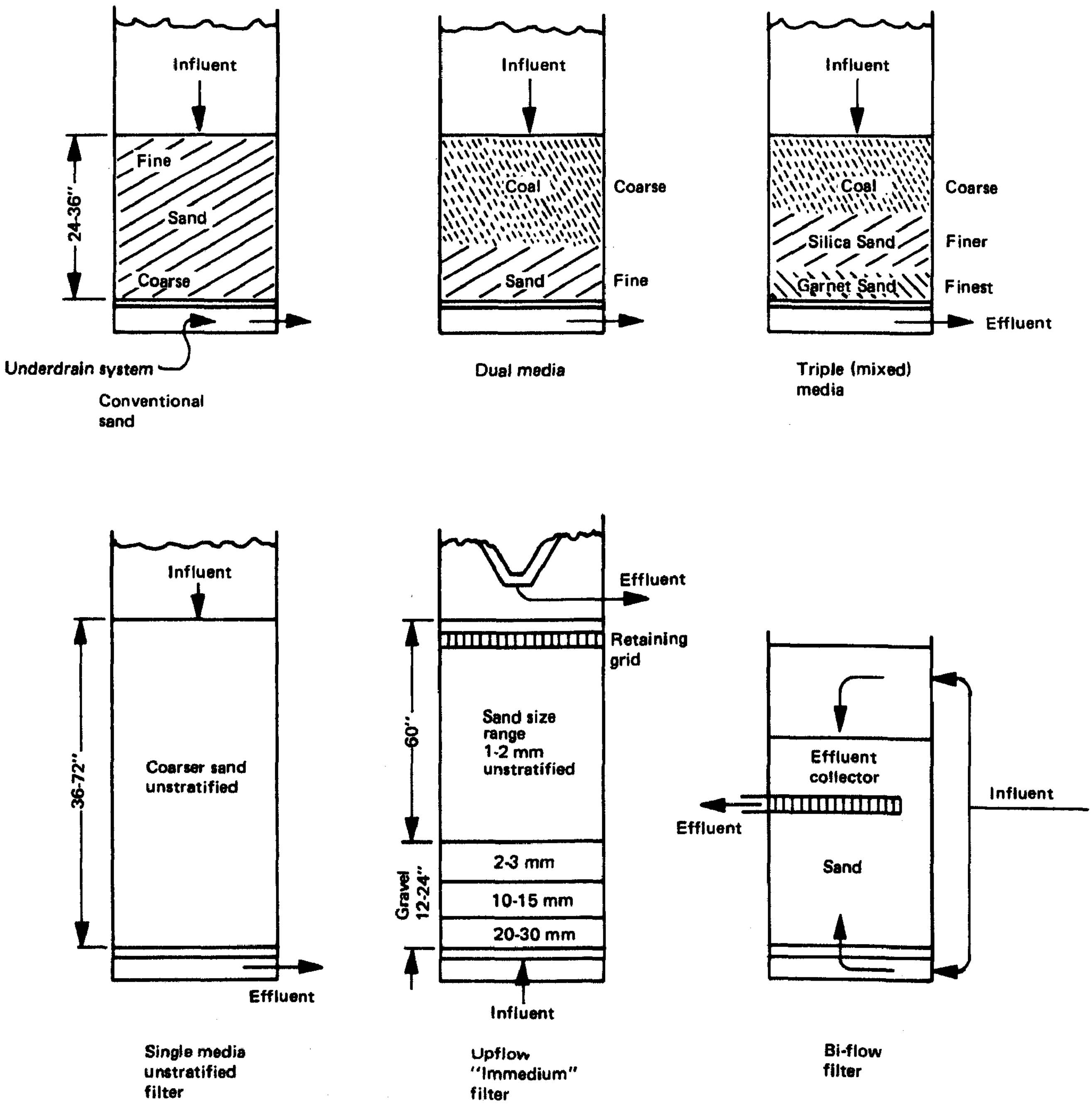

Figure III-4. Schematic diagrams of filter configurations for granular media filtration.

The advantages of the single media unstratified filter are:

- The simplicity of a single media.
- The ready availability of the media.
- That coarser media may be used to achieve longer filter cycles without excessive backwash rates since these filters are backwashed with air and water simultaneously without bed expansion (i.e., below the water fluidization velocity).
- That larger underdrain strainer openings may be used and thus, less danger of strainer clogging.
- That transport of heavy solids released from the filter during backwash to the overflow is more effective with simultaneous air and water wash than with water alone.

The disadvantages of the single media unstratified filter are:

- That a deeper bed is needed to try to compensate for the coarser media resulting in higher structural costs.
- That the deeper bed may not necessarily ensure equivalent filtrate quality compared with a dual or triple media bed as will be shown later.
- Precautions are necessary to ensure that filter media is not lost during the air plus water backwashing operation.

The advantages of the upflow filter are:

- Unfiltered water is usually used for backwashing without danger of strainer clogging since strainers are not used in the underdrain.
- There is no need to provide substantial water depth above the media for head development as is desired in downflow gravity filters to avoid negative head development.
- Low backwash water rates are used because part of the wash cycle is with air and water simultaneously.

The disadvantages of the upflow filter include:

- The total bed depth is large adding to the structural costs.
- Media loss during the air and water backwash has exposed the hold down grid in some plants, leading to uplift difficulties at high headlosses.
- The backwash routine including draindown is fairly long, about 45-60 minutes depending on the details.
- The initial filtrate after backwash can be no better than the backwash supply quality which can be quite poor at times.

The uplift difficulty can be solved by retopping the filter with sand to keep the grid submerged below the fixed bed sand surface, but such details are sometimes neglected.

In addition to the configurations shown in figure III-4, there are various proprietary filter configurations which are in use in wastewater filtration which depend on other methods to overcome the high headloss characteristic of wastewater filtration. They are not described here because of their proprietary nature and because they have been described in another EPA publication[2]. Their absence herein represents neither a criticism nor an endorsement of those filters.

HEADLOSS DEVELOPMENT

Figure III-5 shows several examples of headloss development during solids separation by filtration. Granular media filters remove suspended solids in one of the following ways:

- By removal of the suspended solids at the surface by the finer media at the top of the filter, which forms a relatively thin layer of deposited solids at the surface.

- By depth removal of the suspended solids within the voids of the porous media—the better the distribution of the solids throughout the depth of the filter media, the better the use of the head available.

- By a combination of surface removal and depth removal, which is the usual case in filtration of secondary effluents.

Solids removal may be predominantly at the surface if the filter media is too small or if the filtration rate is too low. Surface removal of a compressible solid results in a headloss curve that is exponential (fig. III-5a). Increasing the terminal headloss does not increase production per filter run significantly with this type of headloss pattern. With surface-cake filtration of this type, the filtration is dominantly achieved by the cake itself, and filtrate quality is constant throughout the run.

On the other hand, if removal occurs entirely within the filter media, a headloss pattern such as that in figure III-5b will result. Increasing the filtration rate increases the initial headloss. Since the headloss curves are essentially parallel, increasing the filtration rate slightly decreases production to any particular terminal headloss. Increasing the terminal headloss increases both the run length and the production per run since the curves are nearly linear. Depth removal of this type may be experienced using larger-size surface media and the various filter designs that provide coarse-to-fine filtration. It is the most common pattern in potable water filtration and is observed in some wastewater filtration.

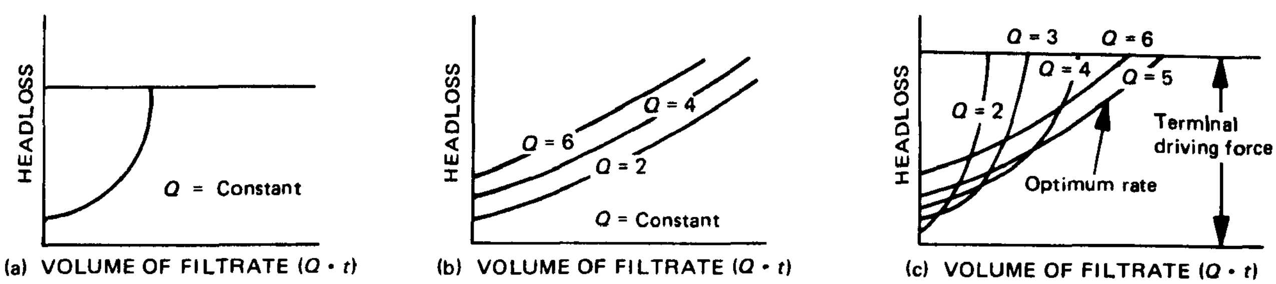

Figure III-5. Headloss development during filtration: (a) surface removal of compressible solids; (b) depth removal of suspended solids; (c) depth removal of suspended solids with surface cake.

When the solids are partly removed on the surface and partly in the depth of the filter, surface removal will predominate at low filtration rates, and the headloss generated is characteristic of surface removal headloss (e.g., fig. III-5a at the lower rates). With higher rates, the solids are carried deeper into the granular medium and more filtrate is produced before the surface cake forms (Q = 3 and 4, fig. III-5c). The rate may become high enough to prevent surface cake formation and the headloss will then be controlled only by depth filtration. In figure III-5c, therefore, a flow rate of 5 would be the optimum production rate since it produces the most filtrate per run. No substantial surface cake forms with filtration rates of 5 or higher, resulting in parallel headloss curves above that rate. Filtration of secondary effluents tends to involve both surface and depth filtration, and may thus behave as in figure III-5c[7]. Nearly linear headloss curves have been observed in filtration of alum-treated secondary effluent[8].

Thus, if an exponential headloss curve is observed, production per filter run can be increased by using higher filtration rates, or alternatively, the top media size can be enlarged by skimming or replacement of the coal media if necessary.

In the operation of tertiary wastewater filters, plots of headloss versus time or volume of filtrate can yield valuable information on the design of the filter media or the choice of a filtration rate.

Chapter IV

FILTER DESIGN — DETAILED CONSIDERATIONS

The objective of filtration is to produce the desired quality and quantity of filtrate at the least cost per unit of filtrate produced. The designer must choose between the various pretreatment alternatives and various performance variables discussed below in reaching a final design. The various alternatives must be tested against the basic objective.

The variables which affect performance fall in two categories: (1) the influent suspended solids variable, such as the type, amount and filtrability of the solids, and (2) the physical filtration variables such as the rate of filtration, terminal headloss provided, and the size, depth and type of filter media.

PILOT SCALE TESTING

When new types of waters are to be filtered containing solids of unfamiliar filtrability, pilot testing may be necessary to arrive at the proper design. Pilot testing on various wastewaters has become increasingly common as such filters are needed in process flow schemes.

The pilot filtration apparatus should have three or more filters which can be run in parallel. This is necessary because the influent solids may change from day to day (even hour to hour) so that various design or operating variables must be compared in parallel rather than sequentially. The three pilot filters can be operated in a series of experiments to evaluate the effect of media size, media depth, media type (single, dual or multi media) and filtration rate on filtrate quality and headloss generation. The filters should be equipped with pressure taps at intervals through the depth so that the extent of depth filtration can be ascertained. The influent and effluent should be monitored for suspended solids, turbidity and other parameters of interest so that the ability to achieve filtrate quality goals can be determined, and the relation of solids load to headloss development can be approximated. The pilot experiments should cover the full range of the variables that may be used in the plant design, e.g., filtration rates of 2 to 8 gal/min/ft^2 and terminal headlosses to 30 feet. A good deal about expected performance can be learned by studying the results of other investigators who have filtered similar influent solids. Substantial data of this type is presented for wastewater filtration in Appendix A and in other sources[2,3].

Data for particular media can be presented as shown in figure IV-1, assuming that filtrate quality goals are achieved for all variables presented. On such curves, it will be noted that the influent solids load is the product of influent concentration (C_o) and the run length (t) and filtration rate (v). The three curves can be normalized roughly into a single curve by plotting the average product of $v \cdot t$ versus C_o. The shape of the curve is approximately hyperbolic indicating an equation of the form $x \cdot y$ = constant, i.e., $C_o \cdot t \cdot v$ = constant. This fact has led to an approximate approach for predicting headloss on the basis of solids capture per unit headloss increase for a particular media size and type and concentration of influent solids. Some typical data of this type are presented in table IV-1.

While the data in table IV-1 are limited, especially for activated sludge, several things should be noted:

- There is substantial variability from plant to plant illustrating the uncertainty of using data from one plant in the design of another.

- There is little variation in solids capture over the common range of filtration rates, 2 to 6 gal/min/ft^2, but variation is evident at extremely high rates as evidenced by the Cleveland data[9], and some Ames data at 8 gal/min/ft^2 [15].

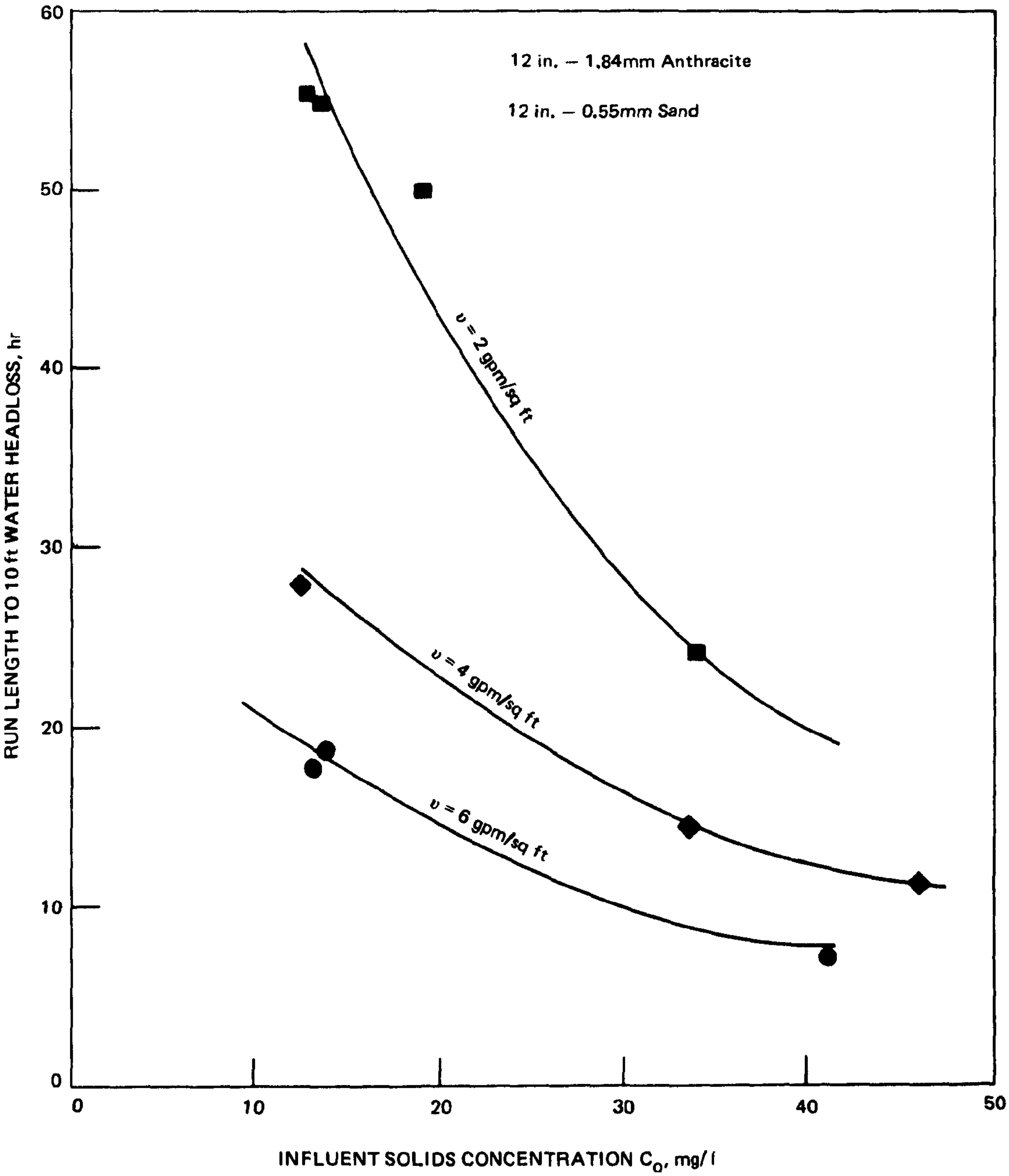

Figure IV-1. Run length vs influent SS concentration at various flow rates[7].

Table IV-1.—*Solids capture per foot of headloss increase in direct filtration of secondary effluents*

Secondary effluent type[a]	Filtration rate gal/min/ft^2	Mode of operation[b]	Media size mm	Solids capture lb/ft^2/ft headloss increase	Location	Reference
TF (Full scale sand)	2.5 - 4	C	0.85-1.7[c]	.04-.05	Luton, Eng.	10 & 11
TF (Pilot dual media)	2 - 6	C	1.84 ES[d]	0.07	Ames, Iowa	7
TF (Pilot sand)	2 - 6	C	0.55 ES	.06-.07	Ames, Iowa	12
TF (Full scale sand)	2.3	–	0.55 ES	.03	Pretoria	13
TF (Pilot dual media)	2.1	C	1.03 ES	.08	Ames, Ia. Parallel operation	8
TF (Pilot sand)	2.1	C	2.0-3.6[c]	.16		
TF (Pilot dual media)	3.2	C	1.03 ES	.04	Ames, Ia. Parallel operation	8
TF (Pilot sand)	3.2	C	2.0-3.6[c]	.14		
TF (Pilot sand)	3.2 - 3.8	–	1-2.06[c]	0.29	Finham, Eng.	14
TF (Pilot sand)	2	C	2.31 ES[e]	0.23	Ames, Iowa Parallel operation	15
TF (Pilot sand)	2	C	1.82 ES[e]	0.19		
TF (Pilot sand)	2	C	1.49 ES[e]	0.11		
TF (Pilot sand)	2	C	0.97 ES[e]	0.06		
TF (Pilot sand)	4	C	2.31 ES[e]	0.26	Ames, Iowa Parallel operation	15
TF (Pilot sand)	4	C	1.82 ES[e]	0.21		
TF (Pilot sand)	4	C	1.49 ES[e]	0.12		
TF (Pilot sand)	4	C	0.97 ES[e]	0.07		
TF (Pilot sand)	8	C	2.31 ES[e]	0.31	Ames, Iowa Parallel operation	15
TF (Pilot sand)	8	C	1.82 ES[e]	0.26		
TF (Pilot sand)	8	C	1.49 ES[e]	0.15		
TF (Pilot sand)	8	C	0.97 ES[e]	0.10		
AS (Pilot dual media)	16	C	1.78 ES	.35	Cleveland, Oh. Parallel operation	9
AS (Pilot dual media)	24	C	1.78 ES	.093		
AS (Pilot dual media)	32	C	1.78 ES	.093		
AS (Pilot dual media)	16	D	1.78 ES	.23	Cleveland, Oh. Parallel operation	9
AS (Pilot dual media)	22.2	D	1.78 ES	.21		
AS (Pilot dual media)	27.6	D	1.78 ES	.12		
AS (Pilot upflow)	2-5	C	1-2[c]	.26	W. Hertfordshire, Eng.	16
AS (Pilot dual media)	5.1	C	1.08 ES	.24		17
AS (Pilot dual media)	5.1	C	1.45 ES	.34		17
TF (Pilot dual media)	4.24	D	1.28 ES	0.07-0.10	Nevada, Ia.	18
AS (Pilot dual media)	4.24	D	1.28 ES	0.01-0.04	Marshalltown, Ia.	18

[a]TF = trickling-filter-plant final effluent; AS = activated-sludge-plant final effluent
[b]C = constant rate; D = declining rate
[c]Media size range, unstratified due to backwash provisions
[d]Effective size of media. In dual media, only the ES of the top coal layer is presented
[e]Unstratified due to backwash provisions

- The solids capture is quite low for media sizes about 1 mm and below, about 0.04 to 0.08 lb/sq ft/ft of headloss increase.

- Substantially higher solids capture is possible with larger media as illustrated by the parallel studies at Ames[8, 15].

- The low solids capture value with the coarse dual media[7] is not in harmony with other observations at Ames for media of this size for unknown reasons.

Additional data of this type would be desirable, especially for tertiary filters at activated sludge plants, and for filters at plants practicing various types of chemical treatment. Until such data are collected, if one must design without the benefit of pilot studies, one must select conservative solids capture values of about 0.04 form media of 1 mm ES and perhaps 0.12 for media of 2 mm ES with gradual variation for intermediate sizes. These values should be appropriate in the common range of filtration rates from 2 to 6 gal/min/ft^2.

Chemical treatment produces weaker floc, and somewhat higher solids capture values can be expected although the authors have data from only one study to present. A solids capture value of about 0.04 lb/sq ft/ft headloss increase was calculated for a chemically treated secondary effluent[8]. In this case, trickling filter plant effluent at Ames, Iowa, was treated with 200 mg/l of alum for phosphorous reduction, flocculated and settled before filtration. The filtration rate was 2 gpm/sq ft and the dual media pilot filters had 0.94 mm ES coal. The low value indicates that the media size was too small.

In view of the low solids capture for media of 1 mm ES or below, such sizes are not recommended for wastewater filtration. A size of at least 1.2 mm ES is suggested as a minimum, and coarser sizes are preferred if appropriate backwashing is provided.

OPTIMIZATION CONSIDERATIONS

When one considers physical filtration variables, it is apparent that the engineer has a large number of filter design options. Many combinations of filtration rate, media size and depth, and terminal headloss may achieve the desired filtrate quality with acceptable filter run length. Higher rates of filtration will require deeper beds and/or finer media for equal filtrate quality. Coarser filter media will require deeper beds and/or lower filtration rates.

The various feasible design alternatives which will meet filtrate quality goals can be compared on a capital and operating cost basis. A particular combination of the physical variables may result in the filter effluent quality reaching its upper limit of acceptability at the same time that the total headloss reaches a selected limit. Such a combination constitutes an optimum[19], or more precisely an operational optimum[20]. A number of operational optimums are possible with a given influent water and filtrate quality goal, but only one would yield water at least cost, i.e., at the *economic optimum*. In recognition of these concepts, attempts are being made to optimize filter design[20,21,22].

As one considers the available options, it should be cautioned that real life suspensions to be filtered do not always fit idealized mathematical models of filtration. For example, it is not always possible to offset higher rates of filtration, or coarser media sizes, with increased bed depth.

A good example of this occurs in direct filtration of secondary effluent wastewater. It appears that a portion of the influent suspended solids are easily filtered from water regardless of the media size, depth, or rate of filtration within the normal ranges of these variables. Another portion is not removed regardless of the choices made for these variables. This may be due to the bimodal distribution of particle sizes observed in secondary effluent of an activated sludge process [23] with one group of very small particles (0 to 10 μm with mode at about 5 μm) and another group of much larger particles (20 to 160 μm with the mode at about 80 μm).

In recognition of the many options available and the limitations discussed in the preceding paragraphs, current practice with regard to these physical filtration variables will be discussed. This is done with recognition of the possibility that current practice does not necessarily reflect optimum design and that other designs may be equally good.

SELECTION OF FILTRATION RATE AND TERMINAL HEADLOSS

Wastewater solids may generate rapid headloss development due to the high solids concentrations in the filter influent and the strong surface removal tendency of the solids. This is especially true in the tertiary filtration of secondary effluents where filtrate quality is not appreciably deteriorated by filtration rates as high as 5 or 6 gal/min/ft^2 using media with effective sizes up to about 2 mm with media depths appropriate to the size. Nevertheless, average rates of 2 to 3 gal/min/ft^2 and peak rates of 5 gal/min/ft^2 are common to achieve run length objectives[2]. Thus, in wastewater filtration, the rate of filtration is dictated more by run length considerations than by filtrate quality considerations.

Modeling of the filtration process has not yet progressed to the point where it is possible to determine precisely what economic filtration rate and terminal headloss should be provided for a granular-media filter. Huang and Buamman [21] found that the most economic terminal headloss for filtration of iron on unisized-sand filters ranged between 8 and 11 feet at all filtration rates from 2 to 6 gal/min/ft^2. Normal American water treatment practice would use a terminal headloss of 8 to 10 feet when using gravity filters. The filtration rate and terminal head should not be so high as to result in failure of the filtration process by solids breakthrough. However, solids breakthrough does not generally occur in the filtration of secondary effluents. A fraction of the solids pass through the filter during the entire run, but further deterioration does not usually occur as the run progresses.

Studies indicate that pressure drops of as much as 30 feet of water could be used in filtration of trickling filter effluents[7,24] and in activated sludge effluents[9,18] through dual-media filters without solids breakthrough. Economic considerations, however, may dictate pressure filters if such terminal headlosses are to be provided.

The selection of the filtration rate and terminal headloss to be provided in design involves consideration of a number of interrelated questions.

- What are the desired minimum and maximum filter run lengths? As discussed earlier, run length should be at least 6 to 8 hours to avoid excessive backwash water use, but less than about 36 to 48 hours to reduce anaerobic decomposition within the filter and possible detriment to the effluent BOD. The desired run length can be achieved by selecting either the terminal headloss or the filtration rate or both.

- Will the backwash operation be automated to avoid manpower costs if short filter runs occur? Automatic backwash is commonly provided in wastewater filtration plants.

- Is pressurized discharge desired to a subsequent treatment unit or to an effluent force main? Pressurized discharge would tend to favor the use of pressure filters. In such cases, higher rates and/or higher terminal headlosses may be economically feasible where they would not be with gravity filters.

- Is the hydraulic profile of the existing secondary plant such that tertiary filters could be added without repumping by limiting the terminal headloss?

- What is the size of the plant, the capital available, and the space available for tertiary filters? A large plant with adequate capital resources may prefer multiple gravity filters, at lower filtration rates and lower terminal headloss, using a more-or-less conventional water plant design. A smaller plant, or one with limited capital or space, may prefer pressure filters operated at higher filtration rates.

- Are there any regulatory agency policies which require gravity filters or prohibit pressure filters, or does the client insist upon gravity filters for easier maintenance?

- What variations in influent flow rate and suspended solids concentration are expected, and how will they be handled? If influent flow equalization is provided, this concern is partially eliminated. If 24-hour minimum filter runs are the goal, the hourly variations in load will balance out over the day and become of less concern. On the other hand, if 6-hour minimum cycles are selected, peak 6-hour loads would be of concern.

To answer these questions rationally, some method of predicting run length as a function of filtration rate, terminal loss, media size, and influent suspended solids is needed. As discussed earlier, pilot plant studies at the plant in question yield the most reliable prediction. In their absence, the designs can be based on a conservative value of solids capture per unit headloss.

If pilot plant data such as figure IV-1 are collected for different filter media and different terminal headlosses, the data can be used to select several alternative design combinations of media, filtration rate, and terminal headloss. These can be compared on the basis of capital and operating costs. Furthermore, if the flow and solids load variations are predicted, the operational consequences of those variations can be analyzed. One must be sure to limit the design alternatives to those that have been shown to produce acceptable filtrate quality.

To illustrate the use of figure IV-1, assume that the minimum desired run length is selected to be 8 hours, the maximum 8-hour influent solids concentration is estimated to be 40 mg/l, and the terminal headloss is limited to 10 feet by one of the factors discussed above. From figure IV-1, the peak 8-hour filtration rate must then be limited to 6 gal/min/ft^2. If the average annual flow rate is one third the peak, and the average influent solids is predicted to be 20 mg/l, then from figure IV-1, the average run length could be 42 hours. It would be desirable under such loads to wash on a maximum 24-hour override to prevent anaerobic conditions in the filter.

If the design must be based on an assumed solids capture per unit headloss, then alternative designs can be selected as illustrated below.

Assume that a value of 0.07 lb/sq ft/ft headloss has been estimated for a trickling filter effluent and a media size of 1.2 mm ES from table IV-1. This value can be used to estimate the terminal headloss that must be provided to achieve a desired filter run length using an estimated secondary effluent suspended solids concentration. For example, find the needed terminal headloss to achieve 24-hour average filter runs under the following conditions:

Average filtration rate = 3 gal/min/ft^2, with range of 2 to 4.5 during the day

Average secondary effluent suspended solids = 30 mg/l

Average effluent suspended solids = 5 mg/l

Average suspended solids capture = 25 mg/l

Top media size = 1.2 mm

Calculate solids capture per square foot per run:

$$25 \text{ mg/l removed} \times 3 \text{ gal/min/ft}^2 \times 1{,}440 \text{ min/filter run} \times \frac{8.33}{10^6} = 0.90 \text{ lb/ft}^2\text{/run}$$

$$\text{Terminal headloss increase} = \frac{0.90 \text{ lb/ft}^2\text{/run}}{0.07 \text{ lb/ft}^2\text{/ft headloss}} = 13 \text{ ft/run}$$

Thus, a terminal headloss increase of about 13 feet would be required to meet the 24-hour filter run. The initial headloss must be added to this figure to obtain the total terminal headloss. This total is above the normal headloss provided for gravity filters and suggests either that pressure filters be considered, or that the average filtration rate be reduced to 2 gal/min/ft^2.

The filter runs could become substantially shorter during periods of poorer secondary treatment plant performance. For example, if the secondary effluent suspended solids climbed to 50 mg/l, the run length would drop to 13.3 hours, other conditions being unchanged. Peak flows could prevail for such a run length, further accentuating the solids load and reducing the run to 8.9 hours. When filter cycles get this short, the backwash water being returned through the plant becomes substantial and further increases the load on the filters shortening the filter runs.

SELECTION OF FILTER MEDIA

The selection of the size and depth of filter media and the appropriate filtration rate are interrelated. In general, filtrate quality is improved by the use of finer media, greater media depth or lower filtration rates. Similarly, headloss generation rate is increased by finer media, greater media depth and higher filtration rates. With some influent suspensions, these generalizations are not demonstrated significantly. For example, in filtration of secondary effluents, filtration rate has little effect upon filtrate quality over the usual range of rates employed—2 to 5 gal/min/ft^2—and increased media depth may not compensate for coarser media in achieving filtrate quality. As evidence, tables IV-2a and b show that a dual media and a triple media filter provided slightly better filtrate quality than an unsatratified coarse sand filter of 46-inch depth. Furthermore, table IV-2c shows that changing the depth of the unstratified coarse sand filter had little effect on performance at the filtration rate of 3 gal/min/ft^2. However, greater depth is of benefit in maintaining filtrate quality at higher filtration rates[15].

Table IV-2a.—*Performance of a dual media, triple media and unstratified coarse sand filter when filtering secondary effluent from the trickling filter plant at Ames, Iowa[8]. Results are the mean values from periodic composite samples collected during 8 weeks of operation in 1974 at 2.1 gal/min/ft^2*

	Influent	Filtered effluent		
		Dual media[a]	Triple media[b]	Unstratified media[c]
Suspended solids (mg/l)	37.49	6.84	6.31	7.92
n = 14[d]	σ = 12.03[e]	σ = 3.23	σ = 3.87	σ = 5.80
Turbidity (FTU)	16.38	2.38	2.20	2.89
n = 16	σ = 4.31	σ = 0.97	σ = 0.56	σ = 1.10
BOD_5 (mg/l)	14.61	3.73	4.11	4.73
n = 13	σ = 6.00	σ = 1.72	σ = 2.03	σ = 2.56
Soluble BOD_5 (mg/l)	3.88	1.97	2.20	2.34
n = 10	σ = 1.79	σ = 0.96	σ = 0.99	σ = 1.11

[a]Dual media: 15 in. of 1.03 mm ES coal, 1.57 UC
9 in. of 0.49 mm ES sand, 1.41 UC
[b]Triple media: Same as above plus 3 in. garnet with 0.27 ES and 1.55 UC
[c]Unstratified sand: 46 in. of 2.0 mm ES sand (2 to 3.6 mm size range, 1.52 UC)
[d]n = number of composites averaged, each representing one filter run
[e]σ = standard deviation

The selection of the filter media also determines the required backwash regime, and thus, the backwash requirements become an integral part of the media decision. Those requirements have been discussed briefly in the prior section on filter configuration and will be discussed in more detail later in the section on backwashing.

Granular filter media commonly used in water and wastewater filtration include silica sand, garnet sand, and anthracite coal. These media can be purchased in a broad range of effective sizes and uniformity coefficients. (The term "effective size" indicates the size of grain (in millimeters) such that 10 percent (by weight) of the particles are smaller and 90 percent larger than itself. "Uniformity coefficient" designates the ratio of the size of grain which has 60 percent of the sample finer than itself, to the effective size which has 10 percent finer than itself.) The media have specific gravities approximately as follows:

- Anthracite coal, 1.35 to 1.75; most U.S. anthracite, 1.6 to 1.75; U.K. anthracite, 1.35 to 1.45.
- Silica sand, 2.65.
- Garnet sand, 4 to 4.2.

The detrimental effects of the strong surface removal tendency previously discussed for wastewater filtration must be counteracted by selecting a media size where the flow enters the media which will ensure that the bulk of the suspended solids removal does not occur at the

Table IV-2b.—*Performance of a dual media, triple media and unstratified coarse sand filter when filtering secondary effluent from the trickling filter plant at Ames, Iowa[8]. Results are the mean values from periodic composite samples collected during 9 weeks of operation in 1974 at 3.2 gal/min/ft²*

	Influent	Filtered effluent[a]		
		Dual media	Triple media	Unstratified media
Suspended solids (mg/l)	34.08	7.05	6.82	9.46
n = 14[b]	σ = 16.87[c]	σ = 4.27	σ = 3.10	σ = 4.53
Turbidity (FTU)	17.60	4.80	6.78	4.66
n = 15	σ = 6.18	σ = 2.28	σ = 3.01	σ = 2.12
BOD_5 (mg/l)	30.38	12.68	12.99	14.46
n = 15	σ = 14.52	σ = 6.88	σ = 6.82	σ = 6.56
Soluble BOD_5 (mg/l)	9.67	7.21	7.27	7.78
n = 15	σ = 3.76	σ = 3.72	σ = 3.61	σ = 3.57

[a]Filter media same as in table IV-2a except coal depth in dual and mixed media increased to 17 in.
[b]n = number of composites averaged, each representing one filter run
[c]σ = standard deviation

entering surface. Pilot testing of different media is desired if time and budgets permit. If it is not feasible, the following information will assist in selecting the media size or sizes.

For the tertiary filtration of secondary effluents, media size of at least 1.2 mm ES is required, and coarser media is preferred if appropriate backwash is provided. Benefits to filter run length accrue at least up to 2.3 mm ES as shown in the prior solids capture data in table IV-1.

For the filtration of chemically treated secondary effluents, a media size of not less than 1.0 mm has been suggested[2]. However, benefits of coarser media should also occur here, and the sparsity of data makes pilot testing even more important.

Once the size of the media at the entering surface has been selected, the rest of the media specification is dependent thereon. For example, the uniformity coefficients, the size of the sand in dual media, and the depth of each media must be selected.

Low uniformity coefficients (UC) are desired to achieve easier backwashing. This is especially true where fluidization of the media is required during backwashing as with dual and triple media filters. This is true for dual and triple media because the entire media should be fluidized to achieve restratification; therefore, the greater the UC (i.e., less uniform size range), the larger the backwash rate required to fluidize the coarser grains thus provided. A UC of less than 1.3 is not generally practical because of the sieving capabilities of commercial suppliers. A UC of less than 1.5 can be obtained at a cost premium and is recommended.

A UC of less than 1.5 has the advantage that it will ensure that the coarser grain size in the media (such as the 90 percent finer size, d_{90}) is not excessively large, requiring a large backwash rate. Sieve analyses of filter media will usually plot linearly on either log-probability or

Table IV-2c.—*Performance of three unstratified coarse sand filters of different depth when filtering secondary effluent from the trickling filter plant at Ames, Iowa*[8]. *Results are the mean values from periodic composite samples collected during 5 weeks of operation in 1975. Sand size was 2.5 to 3.7 mm size range*

	Filter influent	Filtered effluent[a]		
		24 in. depth[b]	47 in. depth	60 in. depth
Suspended solids (mg/l)	31.3	5.9	6.4	5.7
$\sigma(n = 11)$[c]	9.7	2.1	2.3	1.8
Turbidty (FTU)	12.6	3.30	3.38	3.14
$\sigma\ (n = 11)$	3.14	1.21	1.14	1.14
BOD_5 (mg/l)	15.6	6.5	7.1	6.6
$\sigma\ (n = 11)$	4.7	2.8	2.5	2.5
Soluble BOD_5 (mg/l)	5.3	3.9	4.0	3.8
$\sigma\ (n = 11)$	1.7	1.3	1.3	1.3

[a]Filtration rate, 3.0 gal/min/ft^2
[b]Filtrate from 24 inches of sand and 12 inches of supporting gravel
[c]σ = standard deviation
n = number of composites averaged, each representing one filter run

arithmetic-probability paper. The ratios of d_{90}/d_{10} for media with a UC of 1.5 is 2.0 for the log probability distribution and 1.83 for the arithmetic probability distribution. These ratios are useful in estimating the d_{90} grain size which can then be used to determine the needed backwash rate.

An alternate method of specifying filter media which is used in the U.K. is to specify the range of size within which the media must fall. For example, a 1.4 to 2.4 mm size range would fall between a U.S. standard 14-mesh and 8-mesh sieve. Some tolerance must be allowed at either end to allow for the sieving capabilities of the suppliers. A 10 percent tolerance at each end is suggested, i.e., 10 percent by weight could be smaller than 1.4 mm and 10 percent coarser than 2.4 mm. This system of specification has the advantages that the effective size could be no smaller than the lower end of the range, and the coarser media is more precisely limited which is of importance in selecting the needed backwash rate.

DUAL MEDIA

For dual media filters, the sizes of the sand layer must be selected to be compatible with the coal which has been selected. The bottom sand (e.g., the 90 percent finer size) should have approximately the same or a somewhat lower flow rate required for fluidization than the bottom coal to ensure that the entire bed fluidizes at the selected backwash rate.

To assist in the selection of the required backwash rate, and to assess the compatibility question raised above, empirical data on the minimum fluidization velocity of coal, sand and garnet sand at 25°C are presented in table IV-3, as well as empirical correction factors to be applied for other water temperatures. The temperature correction factors agree substantially with data presented by Camp[25]

Table IV-3.—*Minimum fluidization velocities for various uniform sized media to achieve 10 percent expansion at 25° C, observed empirically*[8]

Between U.S. standard sieves			Mean size mm	Flow rate to achieve 10% expansion at 25°C, gpm/ft^2		
Passing	(mm)	Retained		Coal	Sand	Garnet
7	2.83	8	2.59	37		
8	2.38	10	2.18	30		
10	2.00	12	1.84	24	41	
12	1.68	14	1.54	20	33	
14	1.41	16	1.30	15.7	27	49
16	1.19	18	1.09	12.5	21	40
18	1.00	20	0.92	9.9	16.4	32
20	0.841	25	0.78	8.4	12.6	27
25	0.707	30	0.65	7.0	9.0	22
30	0.595	35	0.55		6.3	18.0
35	0.500	40	0.46		5.4	13.7
40	0.420	45	0.38		4.0	11.3
50	0.297	60 (0.25mm)	0.27			6.3
Specific gravity				1.7	2.65	4.1

Temperature correction – The following are approximate correction factors to be applied for temperatures other than 25°C.

Temperature C°	Multiply 25° value by
30	1.09
25	1.00
20	0.91
15	0.83
10	0.75
5	0.68

The effective size of the sand for a dual media filter should be selected to achieve the goal of coarse-to-fine filtration without causing excessive media intermixing. If the coal density is in the typical range of 1.65 to 1.75 g/cm^3, a ratio of the 90 percent finer coal size to the 10 percent finer sand size equal to about 3 will result in a few inches of media intermixing at the interface[25]. A ratio of these sizes of 4 will result in substantial media intermixing, whereas a ratio of 2 to 2.5 will cause a sharp interface. Choosing media sizes to achieve a sharp interface will mean that the benefits of coarse-to-fine filtration will be partly lost. Therefore, a size ratio of about 3 is recommended.

The use of table IV-3 and the foregoing recommendation can be illustrated with an example. Assume a coal of 1.2 mm ES has been selected with a UC less than 1.5 (size range of 1.2 to 2.2 mm, 8- to 16-mesh range). The sand should have an effective size of about 0.7 mm to be one third of the coarse coal size. A sand size range of 0.7 to 1.4 mm could be specified (14- to 25-mesh range), or one with an ES of 0.7 mm. The backwash rate for the coarse end of the coal (2.38 mm) is 30 gal/min/ft^2 at 25°C and the coarse sand (1.4 mm) is 27 gal/min/ft^2. Thus, they are compatible. If the peak expected operating temperature is expected to be 15°C, the required backwash rate would be 30 X 0.83 = 25 gal/min/ft^2.

It should be noted that no harm would be done if the coarser sand grains were smaller than 1.4 mm. They would merely reach fluidization before the coarser coal gains. There is no danger of inversion of the coal and sand layers during backwashing or complete intermixing as there is with sand and garnet sand. The intermixing behavior of coal and sand, and sand and garnet sand has been experimentally demonstrated[26].

In addition to specifying the gradation of filter media used, the depth of media must be established. At present, there is no reasonable method – other than pilot-plant operation — that can be used to determine the optimum depth of filter media. Huang[7,24] established that, for filtration of tricking filter plant effluent, a depth of at least 15 inches of 1.84 mm ES coal was desirable. Theoretical considerations would indicate that media depths should increase with media size. For practical designs based on a minimum of available information, the following minimum media depths are recommended for dual media filters:

- Anthracite coal, 15 inches minimum to 20 inches.

- Silica sand, 12 inches minimum to 15 inches.

It should be emphasized that the media design illustrated by the foregoing example is one appropriate design for tertiary filtration but it is not the only possibility. A coarser coal would yield longer filter runs but require higher backwash rates. Nor is the example media design necessarily best for chemically pretreated wastewaters, or where polyelectrolytes are to be used as filter aids. In the latter case, a coarser top size may be desired (1.2 to 1.5 mm).

In dual or triple media filters, after each media layer is installed in the filter, it should be backwashed and skimmed to remove unwanted fine sand before installing the next layer. This step can be important, for example, because the sand may collect a low density coating after a number of filter cycles. In one case, using alum coagulation of secondary effluent, these coatings caused the fine sand to migrate to the coal surface where it formed a blinding surface layer[27].

UNSTRATIFIED SINGLE MEDIA

Single media filter beds comprised of unstratified coarse sand are also being used for wastewater filtration. Sand depths of 4 to 5 feet and size ranges of 1.5 to 2.5 mm, 2 to 3 mm, and 2 to 4 mm are being used. These filters offer the advantage of using a coarser media size and thus achieve greater solids capture per unit headloss as shown previously in table IV-1. However, the provision of adequate backwashing is essential.

Because of the coarse sand sizes, backwash by fluidization and bed expansion in the usual U.S. fashion would require excessive wash rates and is not feasible. Therefore, these filters are backwashed with air and water simultaneously at rates just sufficient to cause a pulsing and a slow circulation of the sand in the bed. This is followed by a short water wash at a rate below fluidization to expel some air from the bed.

The overflow level during backwashing must be high enough above the sand surface to prevent excessive loss of sand during the simultaneous air-water backwash. Even though the bed is not fluidized, grains of sand are thrown above the fixed bed surface by the violence of the combined air-water action. A vertical distance to overflow of 24 inches is recommended for the sand sizes mentioned above based on laboratory, pilot and plant scale observations[8]. The common wash routines for these sand sizes are presented in table IV-4.

METHODS OF FILTER FLOW CONTROL

If the design decisions made thus far are favorable to the use of gravity filters, the authors would urge that the control system for the filters be achieved without an effluent flow controller. Two preferable alternatives are presented on the following pages.

Before discussing these alternatives, a fundamental fact about filtration should be recalled, namely, that in any filtration operation, the rate of flow through a filter may be expressed as

$$\text{rate of flow} = \frac{\text{driving force}}{\text{filter resistance}} \qquad \textit{Equation 1}$$

The rate of flow through a water filter is usually expressed in gallons per minute per square foot. The driving force refers to the pressure drop across the filter, which is available to force the water through the filter. At the start of a filter run, the filter is clean and the driving force need only overcome the resistance of the clean filter medium. As filtration continues, the suspended solids removed by the filter collect on the filter surface, or in the filter medium, or both, and the driving force must overcome the combined resistance of the filter medium and the solids removed by the filter.

The filter resistance refers to the resistance of the filter medium and the solids removed by the filter medium to the passage of water. The filter resistance increases during a filter run because of the accumulation of the solids removed by the filter. The filter resistance also increases as the pressure drop across the cake increases, because the solids already removed compress and become more resistant to flow. Hence, as the filter resistance increases, the driving force across the filter must increase proportionally to maintain a constant rate of flow.

There are three basic methods of operating filters that differ primarily in the way that the driving force is applied across the filter. These methods are referred to as "constant pressure filtration", "constant rate filtration", and "variable declining rate filtration".

CONSTANT PRESSURE FILTRATION

In true constant pressure filtration the total available driving force is applied across the filter throughout the filter run. At the beginning of the filter run, the filter resistance is low and the rate of filtration is very high. (High driving force/low filter resistance = high rate of flow.) As the filter clogs with solids, filter resistance increases, and, because the driving force remains constant, the flow rate decreases. This method provides true declining rate filtration. Some pressure filters are operated using this mode of operation.

Table IV-4.—*Unstratified sand filter designs for wastewater with appropriate backwash routines*

Media		Simultaneous wash			Water wash		Source[a]
Size range	Depth	Air rate	Water rate	Dur.	Rate	Dur.	
(mm)	(ft)	(cfm/ ft^2)	(gal/min/ ft^2)	(min)	(gal/ min/ft^2)	(min)	
1.5-2.5	3	2.7	5.5	10	11	5	1
2-3	4-6	6-8	6-8	15	8	5	2
2.5-3.7	4 & 5	7	15	10	15	3	3

[a]1 — Successfully operating full scale plant in tertiary filtration at activated sludge plant in England observed by authors
2 — Manufacturers suggested media and wash routine in the U.S. for 2-3 mm and 2-4 mm sand. Provided acceptable wash of 2-3.6 mm sand in tertiary filtration study at Ames, Iowa[8]
3 — Successful wash routine in pilot scale study at Ames, Iowa, in tertiary filtration of trickling filter plant effluent[8]

CONSTANT RATE AND CONSTANT WATER LEVEL FILTRATION

The constant pressure method of filtration is seldom used with water or wastewater gravity filters because it requires a relatively large volume of water storage on the upstream side of the filter. Current practice, therefore, has tended to the use of constant rate or constant water level for gravity filters. The constant rate method is equally appropriate for pressure or gravity filters. In constant rate and constant water level filtration, a constant pressure is supplied across the filter system and the filtration rate or water level is then held constant by the action of a manually operated or automatic effluent flow control valve. At the beginning of the filter run, the filter is clean and has little resistance. If the full driving force were applied across the filter only, the flow rate would be very high. To maintain a constant flow rate or water level, some of the available driving force is consumed by an effluent flow control valve.

At the start of the filter run, the flow control valve is nearly closed in providing the additional resistance needed to maintain the desired flow rate or water level. As filtration continues, the filter becomes clogged with solids and the flow control valve opens slowly. When the valve is fully open the run must be terminated, since any further increase in filter resistance will not be balanced by a corresponding decrease in the resistance of the flow control valve. Thus, the ratio of driving force to filter resistance (*Equation 1*) will decrease, and flow rate will decrease (water level also increases on gravity filters).

The disadvantages of effluent control constant rate operation include the following:

- The initial and maintenance costs of the fairly complex rate control system are high.
- The filtered water quality is not as good using gravity granular media filters as that obtained using declining rate filter operation in potable water filtration[28,29]. This disadvantage, however, is not important in wastewater filtration.
- The rate or level control systems frequently do not function properly causing sudden changes in rate which can have a bad effect in filtrate quality. In many existing works, the control systems are non functional.

INFLUENT FLOW SPLITTING

A number of other alternative methods of flow control are coming into use that will supplant the effluent flow control valve[28] for gravity filters. For example, some plants have been constructed so as to split the flow nearly equally (influent flow splitting) to all the operating filters, usually by means of an influent weir box on each filter. A schematic diagram of such a gravity filter is shown in figure IV-2. The advantages of this system include the following:

- Constant rate filtration is achieved without rate controllers if the total plant flow remains constant.

- When a filter is taken out of service for backwashing or returned to service after backwashing, the water level gradually rises or descends in the operating filters until sufficient head is achieved to handle the flow. Thus, the rate changes are made slowly and smoothly without the abrupt effects associated with automatic or manual control equipment, causing the least harmful effect to filtered water quality in potable filtration experience[30,31]. The importance of this factor in wastewater filtration has not been studied. It would tend to be most important when filtering wastewaters pretreated with alum or iron salts for coagulation or phosphate reduction.

- The headloss for a particular filter is evidenced by the water level in the filter box. When the water reaches a desired maximum level (the desired terminal headloss), backwashing of that filter is required.

- The effluent control weir must be located above the sand to prevent accidental dewatering of the filter bed. This arrangement eliminates completely the possibility of negative head in the filter and the well-known and undesirable problems (air binding due to gases coming out of solution) that sometimes result from it.

The only disadvantage of the influent flow splitting system is that additional depth of filter box is required owing to the raising of the effluent control weir. The depth of filter box above the effluent weir must be high enough to provide the design terminal headloss.

VARIABLE DECLINING RATE FILTRATION

Variable declining rate operation is similar to influent flow splitting, and is another desirable method of operation for gravity filters. Variable declining rate operation achieves all the influent flow splitting advantages and some additional ones, without any of the disadvantages. Despite the merits of this method, however, it has not received enough explanation or attention[28].

Figure IV-3a illustrates the desirable arrangement for new plants designed for variable declining rate operation. Great similarity exists between figures IV-2a and IV-3a, the principal differences being the location and type of influent arrangement and the provision of less available headloss.

The method of operation is similar to that described for figure IV-2a, with the following exceptions. Figure IV-3b illustrates the typical water level variation and headloss variation observed with this mode of operation. The filter influent enters below the wash trough level of the filters. When the water level in the filters is below the level of the wash trough, the installation operates as an influent flow splitting constant rate plant. When the water level is above the level of the wash trough, the installation operates as a variable declining rate plant. In general, the only time the filter water level will be below the wash trough level will be when all filters are backwashed in rapid sequence or after the total plant has been shut down, with no influent, so that the water level drops below the wash trough. In most cases, the clean filter headloss through the piping,

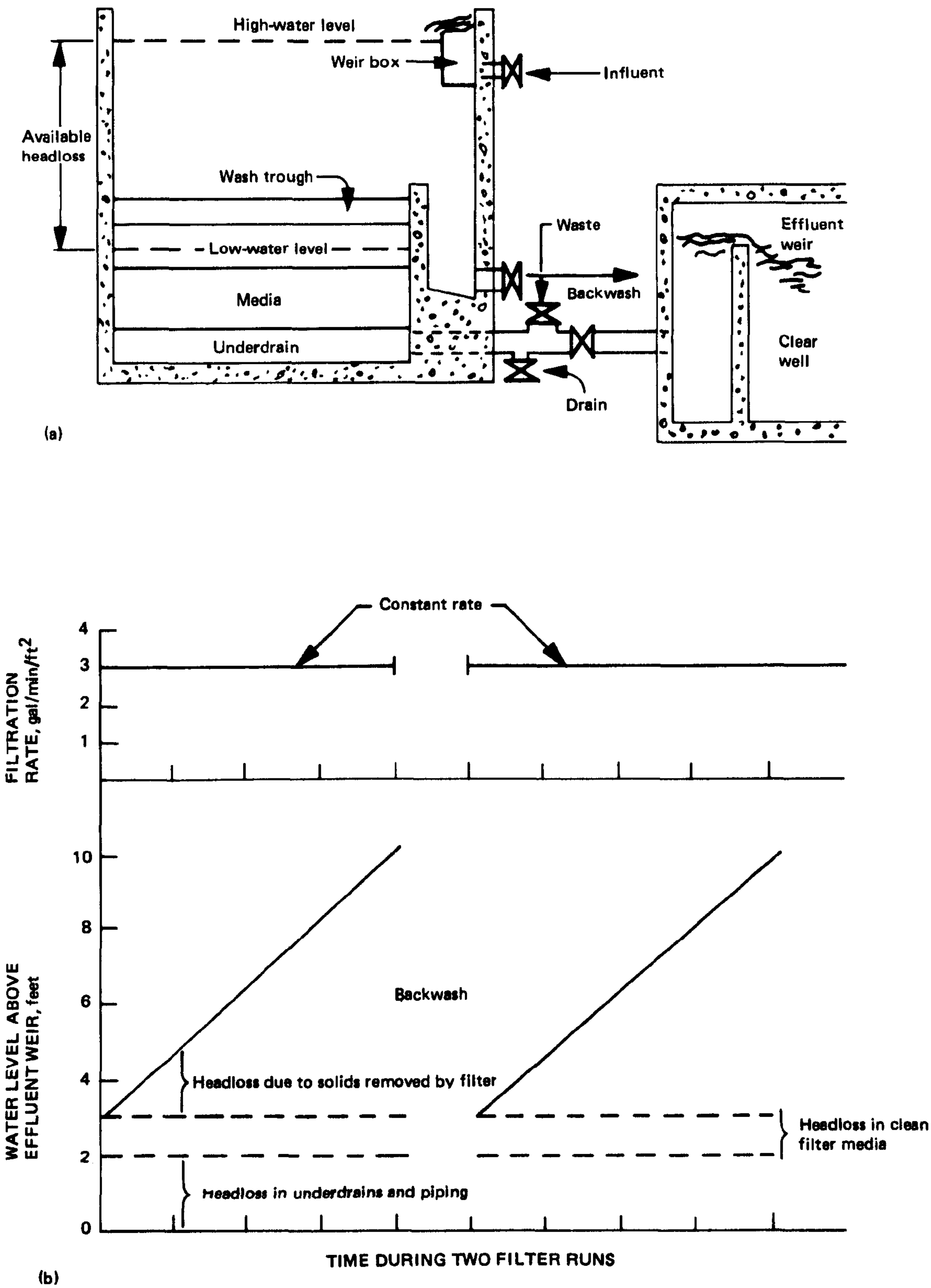

Figure IV–2. Influent flow splitting filtration: (a) typical filter and clear well arrangement; (b) filtration rate, water level, and headloss during two filter runs.

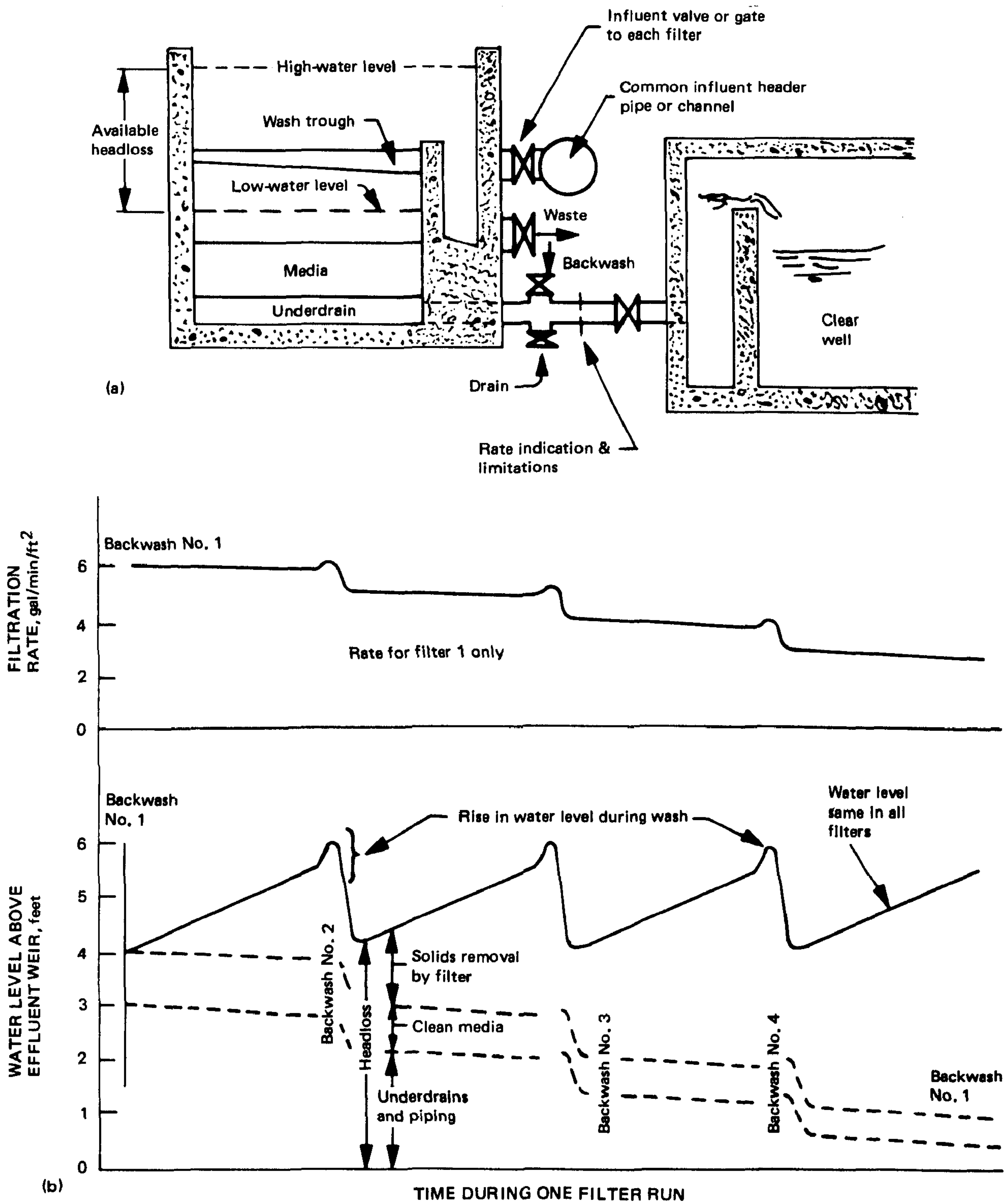

Figure IV-3. Variable declining rate filtration: (a) typical filter and clear well arrangement; (b) filtration rate, headloss, and water level during one filter run in a plant having four filters.

media, and underdrains will range from 3 to 4 feet and keep the actual low water level above the wash trough. The water level is essentially the same in all operating filters at all times; this is achieved by providing a relatively large influent header (pipe or channel) to serve all the filters, and a relatively large influent valve or gate to each individual filter. Thus, headlosses along the header or through the influent valve are small and do not restrict the flow to each filter. The header and influent valve will be able to deliver whatever flow each individual filter is capable of taking at the moment. A flow restricting orifice or valve is recommended in the effluent pipe to prevent excessively high filtration rates when the filter is clean and to indicate the approximate clean bed filtration rate.

Each filter will accept at any time that proportion of the total flow that the common water level above all filters will permit it to handle. As filtration continues, the flow through the dirtiest filter tends to decrease the most rapidly, causing the flow to redistribute itself automatically so that the cleaner filters pick up the capacity lost by the dirtier filters. The water level rises slightly in the redistribution of flow to provide the additional head needed by the cleaner filters to pick up the decreased flow of the dirtier filters. The cleanest filter accepts the greatest flow increase in this redistribution. As the water level rises, it partly offsets the decreased flow through the dirtier filters; as a result, the flow rate does not decrease as much or as rapidly as expected.

This method of operation causes a gradually declining rate toward the end of a filter run. Filter effluent quality is affected adversely by abrupt increases in the rate of flow — here, the rate increases occur in the cleaner filters where they have the least effect on filter effluent quality[30]. Rate changes throughout the day due to changes in total plant flow, both upward and downward (in all of the filters, dirty or clean), occur gradually and smoothly without any automatic control equipment.

The advantages of declining rate operation over constant rate operation are as follows[28,29].

- For waters that show effluent degradation toward the end of the run, the method provides significantly better filter effluent quality than that obtained with constant rate (or constant water level) filter operation.

- Less available headloss is needed compared with that required for constant rate operation because the flow rate through the filter decreases toward the end of the filter run. The headloss in the underdrain and effluent piping system therefore decreases (with the square of the flow rate) and becomes available to sustain the run for a longer period than would be possible under constant rate operation with the same available head. Similarly, the head dissipated through the clogged portions of the filter media decreases linearly with the decreasing flow rate.

For the foregoing reasons, declining rate filters are considered to be the most desirable type of gravity filter operation, unless the design terminal headloss is quite high (e.g., greater than 10 feet). Then constant level control or pressure filters may be a more economical choice. A bank of pressure filters can also operate using variable declining rate filtration; however, any rate changes imposed on the plant cause sudden changes in filtration rates with pressure filters.

Some of the concerns and questions raised about variable declining rate filtration are as follows:

- It appears to be an uncontrolled system with little available operator manipulation. This is, in fact, an attribute which prevents operational abuse of the delicate filtration mechanisms.

- If the rate limiting device is sized for design year peak loads, it will permit higher than necessary filtration rates in the early plant life. This is true unless one limits the headloss utilized during the early plant life, i.e., backwashes at lower water levels.

- What is the total available headloss to be provided? This is a difficult question but no more difficult than it has been in the past for constant rate filtration plants. It is best guided by past experience at the plant in question, or by pilot testing. In the absence of these, one must resort to an assumed solids capture per unit headloss design as discussed previously to select terminal headloss, and make adjustments downward for the headloss recovery discussed previously.

Surprisingly, the water level fluctuation in plants operating on this system is not as great as anticipated. Typical variations of 1.5 to 2 feet (0.5 to 0.7 m) have been reported in potable water plants[32,33].

Chapter V

BACKWASHING OF WASTEWATER FILTERS

The principal problems in filter operation are associated with maintaining the filter bed in good condition. Inadequate cleaning leaves a thin layer of compressible dirt or floc around each grain of the media. As pressure drop across the filter media increases during the subsequent filter run, the grains are squeezed together and cracks may form in the surface of the media, usually along the walls first.

The heavier deposits of solids near the surface of the media break into pieces during the backwash. These pieces, called mudballs, may not disintegrate during the backwash. If small enough and of low density, they float on the surface of the fluidized media. If larger or heavier, they may sink into the filter, to the bottom, or to the sand-coal interface in dual media filters. Ultimately, they must be broken up or removed from the filter or they reduce filtration effectiveness, or cause shorter filter runs by dissipating available headloss.

In wastewater filters, slimes can reduce the average density of the filter grains and can cause more loss of filter media during backwashing, or migration of fine sands in dual media higher into the coal layer. Filamentous growths can cause blinding of the surface layers which shorten filter runs.

Dirty filter media may be chemically cleaned in place as a temporary expedient short of rebuilding the filter bed. Various chemicals have been used, including chlorine, copper sulfate, acids and alkalies. Chlorine may be used where the material to be removed includes living and dead organisms or their metabolites. Copper sulfate is effective in killing algae growing on the walls or medium. Alkalies can be effective on greasy deposits on the filter grains.

However, rather than attempting to correct dirty filter problems after they occur, the backwashing system should be designed to prevent them from the onset.

Potable water filter backwashing practice in the U.S. has used the high velocity wash with substantial bed expansion (20 to 50%). This method does not solve all problems with dirty filters, and it has created problems with shifting of the finer supporting gravel layers. The provision of a surface wash system which introduces high velocity water jets before and during the backwash has largely solved the problem of dirty filter media for potable water filters, but has not solved the problem of shifting gravel. The growing use of wastewater filtration has further demonstrated the weakness of water fluidization backwash. Backwashing is substantially more difficult and problems of agglomerates and filter cracks are prominent.

The problem of shifting gravel and the more difficult backwashing of a wastewater filter has stimulated renewed interest in the air scour method of auxiliary agitation, which has continued in use in European practice. There is also interest in the use of underdrain systems with fine strainers that do not require gravel, a system which was abandoned in the U.S. in the early twentieth century due to clogging and corrosion problems.

SURFACE WASH TO ASSIST BACKWASHING

Evidence of the benefits of surface wash led to its widespread adoption for potable water filters in the United States. Surface wash is introduced at pressures of 45 to 75 psig through orifices on a fixed piping grid or on a rotating arm, located 1 to 2 inches above the fixed bed. Surface wash flow rates are about 1 in/min for the rotary type and 3 to 6 in/min for the fixed nozzle type. The desired operating sequence involves draining the filter to the wash trough level or below, applying the surface wash flow with no concurrent backwash flow for 1 to 2 minutes to break up surface layers on top of the media, then continuing the surface wash with concurrent backwash flow for several minutes until the backwash water begins to clear up. The concurrent application may be at two rates, first a low rate to barely immerse the surface wash jets in the media followed by a period with normal bed expansion. The surface wash is then terminated and water fluidization backwash alone follows for 1 to 2 minutes to stratify the bed, a provision which is only important in dual or triple media filters.

AIR SCOUR TO ASSIST BACKWASHING

Air scour consists of the distribution of air over the entire filter area at the bottom of the filter media so that it flows upward through the media. It is used in a number of fashions to improve the effectiveness of backwashing, and/or to permit the use of lower backwash water flow rates. The air may be used prior to the water backwash or concurrently with the water backwash. When used concurrently during backwash overflow, there is legitimate concern over potential loss of filter media to the overflow due to the violent agitation created by the air scour. When air is used alone, the water level is lowered 6 to 8 inches below the overflow level to prevent loss of filter media during the air scour.

Air scour may be introduced to the filter through a pipe system which is completely separate from the backwash water system, or it may be added through the use of a common system of nozzles (strainers) which distribute both the air and water, either sequentially or simultaneously. In either method of distribution, if the air is introduced below graded gravel supporting the filter media. there is concern over the movement of the finer gravel. This may occur as air is expelled by the water at the onset of the water wash, or especially by air and water used concurrently by intention or accident. This concern has led to the use of media retaining strainers in some filters which eliminate the need for graded support gravel in the filter. However, these strainers may clog with time causing decreased backwash flow capability or, possibly, structural failure of the underdrain system. Such failures have occurred[27]. This clogging may be due to fine sand or coal, which leaks through the strainer during downflow filtration and later is lodged forcefully in the strainer slots during water backwash. The underdrain plenum below the strainers must be scrupulously cleaned before the strainers are installed to prevent construction dirt or debris from later clogging the strainers during backwash.

In view of the concerns expressed above and the renewed interest in air scour in the United States, a summary of European air scour practice (in potable water treatment primarily) may be worthwhile because it has been used there since the beginning of rapid filtration.

Current potable water practice in the U.K. uses air first followed by water backwash. Plastic strainers with 3 mm slots are used in the underdrains covered by 2 or more layers of gravel to support the media. Single media sand filters are common with size range of 0.6 to 1.2 mm, but dual media filters are becoming more common since about 1970. For single media sand filters, the water wash rate is intended to just reach minimum fluidization velocity with only 1 to 2 percent bed expansion. Air is introduced for 3 to 5 minutes through the gravel layers at rates of 1 to 1.5 scfm/sq ft*, sometimes up to 2 scfm/sq ft, followed by water at 12 in/min (7.5 gal/min/ft^2). Problems with gravel movement have occurred but only in a few cases. The absence of such problems must be attributed to the low water and air flow rates which are presumably not sufficient to move the fine gravel, and the fact that air and water are not used simultaneously. The preference for strainers with 3 mm slots and gravel is based on prior experience with clogging of strainers with 0.5 mm slots.

*Standard cubic feet per minute (at 70°F and 14.7 psia in U.S. practice).

The backwashing practice on the European continent as described below is obtained from four sources[4, 34, 35, 36], and therefore, probably does not reflect the diversity of the continental practice. The sources describe several points about the continental practice which differ substantially from both U.S. and U.K. practice.

- Deeper beds of coarser sand are used and the backwash of these sands is at low rates, with little or no expansion of the bed.

- Backwash is with air and water simultaneously at low water rates followed by water alone. The air rate is 2 to 4 cfm/sq ft and the water rate is 10 in/min (6.2 gal/min/ft^2) for the smaller sand sizes (1 to 2 mm size range) and 6 to 8 cfm/sq ft air and 10 to 12 in/min (6.2 to 7.5 gal/min/ft^2) water for the coarser sizes with size ranges such as 2 to 3 mm, 2 to 4 mm. In fact, the use of air and water simultaneously is considered absolutely essential for proper backwashing. The danger of media loss is acknowledged if the simultaneous air-water backwash is continued during overflow. It is suggested that if loss is observed, the backwash water rate be reduced during the simultaneous air water backwash[35]. Mudballs are unknown in Europe using this type of bed design and backwash system.

- Supporting gravel is sometimes used but is of the double reverse graded gravel arrangement, i.e., coarse to fine to coarse in gradation[36]. Otherwise, media retaining strainers are used but the dangers of clogging and underdrain failure are acknowledged[34].

The renewed use of air scour in the United States has been patterned more after the British practice of using air scour alone first, followed by water backwash. U.S. air rates have been typically 3 to 5 scfm/sq ft for 3 to 5 minutes, and the subsequent water wash is above fluidization velocity to expand and restratify the dual media bed, typically 24 to 36 in/min (15 to 22.5 gal/min/ft^2). The filters are usually equipped with media retaining underdrain strainers without graded gravel support for the media. Because of the fine sand media used in the dual media beds (0.5 mm to 0.6 mm effective size), the strainer openings are very small (0.25 to 0.5 mm) and the strainer clogging problems and some underdrain failures have occurred therefrom. Because of this problem, a reconsideration of the U.S. air scour design practice may be appropriate.

BACKWASHING RECOMMENDATIONS

In view of the difficulty of backwashing wastewater filters, and the various filter media and backwash routines available, a research study was conducted to compare the various alternatives as applied to wastewater filtration. Various granular media filters were studied including single, dual and triple media. Various methods of backwashing were compared including:

- Water fluidization only

- Air scour followed by water fluidization

- Surface wash and subsurface wash before and during water fluidization backwash

- Surface wash and subsurface wash before and during water fluidization backwash

- Simultaneous air scour and subfluidization water backwash.

Some of the conclusions of that study are important to the design of wastewater filters and are, therefore, quoted below[8]:

- The cleaning of granular media filters by water backwash alone to fluidize the filter bed is inherently a weak cleaning method because particle collisions do not occur in a fluidized bed and thus abrasion between the filter grains is negligible.

- The weakness of water fluidization backwash alone was clearly demonstrated during wastewater filtration studies where a dual media filter which was washed by water fluidization alone developed serious dirty filter problems such as floating mud balls, agglomerates at the walls and surface cracks. These problems were observed when filtering either secondary effluent or secondary effluent which had been treated with alum for phosphorous reduction.

- The heavy mud ball and agglomerate accumulations caused higher initial headlosses and shorter filter cycles. They may also cause poorer filtrate quality in some cases, although such detriment was not demonstrated in this research.

- Simultaneous air scour and subfluidization backwash of unstratified coarse sand filters proved to be the most effective method of backwash. However, this method should not be used for finer filter media such as the coals and sands of the typical sizes used in dual and triple media filters because loss of media will occur during backwash overflow. The choice of simultaneous air and water flow rates must be appropriate for the sand being used and should result in some circulation of the sand for effective backwashing.

- The other two methods of improving backwashing, namely air scour followed by water fluidization backwash, and surface (and subsurface) wash before and during water fluidization backwash, proved to be comparable methods of backwash which can be applied to single, dual and triple media filters. These two methods did not completely eliminate all dirty filter problems, but both auxiliaries reduced the problems to acceptable levels so that filter performance was not impaired.

- The use of some form of air scour auxiliary, or some form of surface wash auxiliary, is essential to the satisfactory functioning of wastewater filters comprised of deep beds (2 to 5 ft) of granular material which are backwashed after several feet of headloss development. The auxiliary and the backwash routine must be appropriate to the filter media. For example, subfluidization wash is limited to single media filters because stratification is not essential (or even desired) for such filters. Fluidization capability is essential for dual or triple media filters to permit restratification of the layers in their desired positions at the end of the backwash. Air scour and water backwash simultaneously during overflow is primarily useful on coarse sand filters because finer media will be lost due to the violence of the combined air and water action. However, the simultaneous use of air and water can be useful on dual and triple media filters prior to the onset of backwash overflow. The above conclusion is not intended to apply to all types of wastewater filters such as the various proprietary filters with their special backwashing provisions. Such filters and provisions were not studied.

- The use of graded gravel to support the filter media is not recommended where the simultaneous flow of air scour and backwash water can pass through the gravel by intention, or by accident, due to the danger of moving the gravel and thus upsetting the desired size stratification of the gravel.

- Media retaining underdrain strainers with openings of less than 1 mm are not recommended for wastewater filters due to the danger of progressive clogging.

- The filter influent feedwater (e.g., secondary effluent) is not recommended as a backwash water source because of the danger of progressive clogging of underdrain strainers and/or gravel. The advantages of using feedwater do not justify the risks that result therefrom.

- Air scour is compatible with dual or triple media filters from the standpoint of minimal abrasive loss of the anthracite coal media. However, the backwash routine must be concluded with a period of fluidization and bed expansion to restratify the media layers after the air scour.

The authors urge you to use the foregoing conclusions as design guides. In addition, the following design suggestions concerning the backwashing provisions should also be considered.

First, consider the use of air scour as applied to dual or triple media filters backwashed with fluidization capability. In this case:

- Provide operational flexibility in the period of air scour between 2 and 10 minutes so the operator can select the period he deems most appropriate.

- If supporting gravel is not used, provide the capability for simultaneous air and water backwash. This technique requires provisions to allow for rapid draining of the filter to near the filter media surface, followed by the brief simultaneous air and water backwash until the water reaches within 6 to 8 inches of the wash troughs. The simultaneous wash is then stopped, and either air alone or water alone may be continued. The water rate during the simultaneous air water wash should be below fluidization velocity to extend the time duration of that action to the maximum.

- Provide a backwash water volume of at least 100 gal/ft^2 of filter per wash. This is based on the observation that when backwashing at rates above the fluidization velocity for the media, the total wash water required for effective cleaning is about the same regardless of the backwash rate—about 75 to 100 gal/ft^2 of filter. This observation is for typical U.S. wash trough spacing with the trough edges about 3 feet above the surface of the filter media. Larger spacing between troughs, or greater height of trough above the media, would increase the wash water requirements. No economy of total wash water use is achieved by adopting lower backwash rates (above fluidization), because the length of required backwash must be increased proportionately.

Second, consider the use of air and water backwash simultaneously without fluidization capability. In this case:

- Provide a backwash water volume of about 150 to 200 gal/ft^2 of filter per wash. This is larger than the prior case because less experience is available.

- Because of the effective solids transport capability of air and water used simultaneously, wash troughs can be eliminated in favor of a single overflow trough along the length of the filter if the transport distance is limited to about 12 feet.

Third, consider the use of surface wash auxiliary in dual or triple media filters backwashed with fluidization capability.

- Provide a subsurface washer (as well as the surface washer) to attack the mudballs that sink to the interface between the coal and the sand. The subsurface jets should be located at the expected depth of the expanded interface. The writers have no information on the ability of full scale rotary subsurface washers to remain operational in the long term due to the greater drag they encounter, and the hostile environment. The pilot rotary subsurface washer used in the foregoing research was not a good model of a full scale unit, and considerable difficulty was encountered in keeping it operational.

Two additional backwashing problems are of importance in wastewater filter plant design.

- Where does one get the water for backwashing?
- What is done with the dirty backwash water?

The best source of water for backwashing will be the effluent from the filters. If disinfection of the plant effluent is practiced, the chlorine or ozone contact tank should provide sufficient capacity to permit drawing backwash water from this tank. If disinfection is not provided, then a special backwash storage tank should be provided, through which all filter effluent should be directed before final discharge. The backwash water storage tank should normally have sufficient capacity to store all the water needed to backwash at least three filters in succession with the volumes suggested above.

The dirty backwash water must be returned to the plant influent for further treatment. Because of the nonuniform scheduling of filter backwashing, the backwash water presents a significant sludge load on the primary or secondary treatment facilities if returned to them at the rate of backwashing. For that reason, dirty backwash water should be sent to a dirty backwash storage tank and delivered from there at a nearly constant rate to the plant influent or secondary influent. If flow equalization is not being practiced at the plant, it would be desirable to return the backwash wastewater during the low flow period of the day. This would entail a larger wastewater storage tank and return pumping capability, but it would assist in flow balancing.

Chapter VI

SUMMARY

The key questions involved in the proper design of granular filters for wastewater filtration have been discussed in the foregoing sections, and design recommendations have been presented. These recommendations are summarized as follows:

- The variable hydraulic and suspended solids load in secondary effluents must be considered in the design to avoid short filter runs and excessive backwash water requirements.

- A filter that allows penetration of suspended solids is essential to obtain reasonable filter run lengths. The filter media on the influent side should be at least 1.2 mm for tertiary filtration, and preferably larger if appropriate backwash is provided.

- The filtration rate and terminal headloss should be selected to achieve a minimum filter run length of 6 to 8 hours if flow equalization is not provided. Estimates of headloss development and filtrate quality preferably should be based on pilot scale observations at the particular installation. If such studies are not feasible, headloss development should be based on past experience on the suspended solids capture per foot of headloss increase from other similar installations.

- The effect of recycling of used backwash water through the plant on the filtration rate and filter operation must be considered in predicting peak loads on the filters and resulting run lengths.

- High filtration rates (3 gal/min/ft^2 or higher at average load) and/or high influent suspended solids to the filters (30 mg/l or higher at average load) will cause high terminal headlosses and may favor the use of pressure filters over gravity filters, especially for smaller plants with limited capital resources.

- Lower filtration rates or lower influent suspended solids may permit the economical use of gravity filters, especially in larger plants where multiple filters will be needed. At least two, and preferably four, filters should be provided. If only two filters are provided, each should be capable of handling peak design flows to allow for one filter to be out of service for backwashing or repair. If four or more gravity filters are provided, the variable declining rate method of operation is strongly recommended.

- The success of the wastewater filtration plant depends upon the provision of an effective backwash system which is appropriate for the media selected. Details of backwashing requirements for dual and triple media filters and for unstratified coarse sand filters are presented.

REFERENCES

[1] W. J. Weber, Jr., *Physicochemical Processes for Water Quality Control,* Wiley Interscience, New York, N.Y., 1972.

[2] "Process Design Manual for Suspended Solids Removal," U.S. Environmental Protection Agency, EPA-625/1-75-003a, Washington, D.C., Jan. 75.

[3] "Process Design Manual for Upgrading Existing Wastewater Treatment Plants," U.S. Environmental Protection Agency, Washington, D.C., Oct. 1974.

[4] H. Jung and E.S. Savage "Deep Bed Filtration," *J. Amer. Water Works Assoc.,* 66:73-78, 1974.

[5] J. R. Joslin and G. Greene, "Sand Filter Experiments at Derby," *Water Pollution Control,* 69:611-622, 1970.

[6] A. E. Naylor, S. C. Evans, and K. M. Dunscombe, "Recent Developments on the Rapid Sand Filters at Luton," *Water Pollution Control (British),* pp. 309-320, 1967.

[7] J. Y. C. Huang, "Granular Filters for Tertiary Wastewater Treatment," unpublished doctoral thesis, Iowa State University Library, Ames, Ia., 1972.

[8] J. L. Cleasby, "Backwash of Granular Filters Used in Wastewater Filtration," Final Report, EPA Project R802140, Jan. 1976. (ISU-ERI-Ames 76114.)

[9] R. Nebolsine, I. Pouschine, Jr., and C. Y. Fan, "Ultra High Rate Filtration of Activated Sludge Plant Effluent," U.S. Environmental Protection Agency, Office of Research and Monitoring, EPA-R2-73-222, Apr. 1973.

[10] S. C. Evans and F. W. Roberts, "Twelve Months' Operation of Sand Filtration and Micro-Straining Plant at Luton," *J. Proc., Inst. Sewage Purif.,* pt 4:333-341, 1952.

[11] S. C. Evans, "Ten Years of Operation and Development at Luton Sewage Treatment Works," *Water Sewage Works,* 104:214-219, 1957.

[12] K. J. Merry, "Tertiary Treatment of Domestic Wastewater by Rapid Sand Filtration," unpublished master's thesis, Iowa State University Library, Ames, Ia., 1965.

[13] N. P. Nicolle, "Humus Tank Performance, Microstraining and Sand Filtration," *J. Proc., Inst. Sewage Purif.,* pt 1:19, 1955.

[14] A. E. S. Pettet, W. F. Collett, and T. H. Summers, "Mechanical Filtration of Sewage Effluents. I. Removal of Humus," *J. Proc., Inst. Sewage Purif.,* pt 4:399-411, 1949.

[15] M. F. Dahab, "Single-Media, Unstratified-Bed Filtration of Secondary Effluent," unpublished master's thesis, Iowa State University Library, Ames, Ia., 1976.

[16] R. Wood, W. S. Smith, and J. K. Murray, "An Investigation Into Upflow Filtration," *Water Pollution Control (British)*, 67:421-426, 1968.

[17] G. Tchobanoglous, "Filtration Techniques in Tertiary Treatment," *J. Water Pollution Control Fed.*, 42:604-623, 1970.

[18] D. A. Willis, "Variable Declining-Rate Filtration of Secondary Effluent," unpublished master's thesis, Iowa State University Library, Ames, Ia., 1976.

[19] J. Y. C. Huang, "Least Cost Sand Filter Design for Iron Removal," unpublished master's thesis, Iowa State University Library, Ames, Ia., 1969.

[20] E. R. Baumann, *Least Cost Design—Optimization of Deep Bed Filters, The Scientific Basis of Filtration*, K. J. Ives, Editor, Noordoof International Publishing, Leyden, The Netherlands, 1975.

[21] J. Y. C. Huang and E. R. Baumann, "Least Cost Sand Filter Design for Iron Removal," *J. Sanit. Engr. Div., Amer. Soc. Civil Engr.*, 97:SA2, 171-190, Apr. 1971.

[22] K. J. Ives, "Optimization of Deep Bed Filters," *Proceedings, First Pac. Chem. Engr. Cong., Soc. Chem. Engr., Japan*, pt. 1:99-107, Oct. 1972.

[23] G. Tchobanoglous and R. Eliassen, "Filtration of Treated Sewage Effluent," *J. Sanit. Engr. Div., Amer. Soc. Civil Engr.*, 96:243-265, 1970.

[24] E. R. Baumann and J. Y. C. Huang, "Granular Filters for Tertiary Wastewater Treatment," *J. Water Pollution Control Fed.*, 46:8, 1958-1973, Aug. 1974.

[25] T. F. Camp, "Discussion-Experience with Anthracite Filters," *J. Amer. Water Works Assoc.*, 53:1478-1483, 1961.

[26] J. L. Cleasby and C. F. Woods, "Intermixing of Dual Media and Multi-Media Granular Filters," *J. Amer. Water Works Assoc.*, 67:4, 197-203, Apr. 1975.

[27] J. L. Cleasby, E. W. Stangl, and G. A. Rice, "Developments in Backwashing Granular Filters," *J. Env. Eng. Div., Amer. Soc. Civil Engr.*, 101:EE5, 713-727, 1975.

[28] J. L. Cleasby, "Filter Rate Control Without Rate Controllers," *J. Amer. Water Works Assoc.*, 61:4, 181-185, Apr. 1969.

[29] H. E. Hudson, Jr., "Declining Rate Filtration," *J. Amer. Water Works Assoc.*, 51:11, 1455, Nov. 1959.

[30] J. L. Cleasby, M. M. Williamson, and E. R. Baumann, "Effect of Filtration Rate Changes on Quality," *J. Amer. Water Works Assoc.*, 55:7, 869-880, July 1963.

[31] J. Tuepker, "Filter Performance Under Varying Operating Conditions," *Proceedings of Conference on Water Filtration*, University of Missouri, Rolla, Mo., 1965.

[32] J. Arboleda, Jr., "Hydraulic Control Systems of Constant and Declining Rate in Filtration," *J. Amer. Water Works Assoc.*, 66:87-94, 1974.

[33] J. L. Cleasby, "New Ideas in Filter Control Systems," In the Proceedings of a Symposium "Procesos Modernos De Tratamiento De Agua" *XIII Congreso Interamericano de Ingenieria Sanitaria*, Published by the Pan American Health Organization, Aug. 1972.

[34] L. Huisman, "Rapid Filtration, Part 1," Delft University of Technology, Department of Civil Engineering, mimeographed book of lectures in English, 1974.

[35] S. A. Degremont, *Water Treatment Handbook*, Third English Edition, distributed by Hugh K. Elliot, Ltd., 2a Russell Gardens Mews, Kensington, London W14, England, 1965.

[36] E. S. Savage (Dravo Corp.), personal communication, February 19, 1975, describing media and backwash provisions used by Pintsch Bamag (Germany) and Dravo (U.S.), 1975.

Appendix A

PERFORMANCE DATA
FOR
WASTEWATER FILTRATION
FROM
THE LITERATURE

Table A-1.—*Reported Efficiencies For Direct Filtration Of Trickling Filter Plant Effluents*[a]

Location	Source	Media			Filter Rate gpm/sq ft (U.S.)	*N*[c]	Suspended solids (mg/l)				BOD$_5$ (mg/l)				
		Type	Size[b] (mm)	Depth (in.)			Influent		Effluent		Influent		Effluent		%
							Range	Avg	Range	Avg	Range	Avg	Range	Avg	Removal
Luton, England (1949)	1														
Lab study		sand	.85-2.0	21	2.0	5	34-77	53	1-20	6					
		coal	1.0-2.0	21	2.0	7	41-67	51	4-13	7					
		sand	0.5-0.85	21	2.0	2	40-59	50	1-2	2					
		sand	0.5-1.0	21	2.0	2	49	49	0	0					
Pilot study		sand	0.85-1.7	24	2.33	3*m*	20-37	28	2-5	3	24-40	30	8-15	11	
		sand	0.85-1.7	24	2.66	1*m*	20	20	3	3	31	31	15	15	
		sand	0.85-1.7	24	2.83	5*m*	15-25	19	2-5	3	17-28	21	6-9	7	
		coal	0.85-2	18	1.7	2*m*	20-37	29	3-5	4	27-40	34	10-14	12	
			2-5	10	2.8	7*m*	15-28	21	2-5	3	17-28	23	6-13	8	
Finham, England	1	sand	1.0-2.0	24	1.6	*m*	32		5		26		10		
Pilot study		sand	1.0-2.0	24	1.9-2.4	*m*	40		6		31		10		
(1949)		sand	1.0-2.0	24	2.9-3.2	*m*	35-36		7-8		22-25		10-13		
		sand	1.0-2.0	24	3.2-3.8	*m*	34-35		7-9		27-38		11-14		
		coal	1.0-2.0	24	1.4-1.9	*m*	43		10		32		14		
			2-5	3	2.0-2.3	*m*	38		5		27		10		
					2.4-2.8	*m*	32		11		35		14		
					2.9-3.2	*m*	34		8		28		13		
					3.2-3.7	*m*	34-35		7-8		30-37		14		
Luton, England															
Full scale (1957)	2	sand			2.5	12*m*	7-18	13	1-7		6-15	9	2.8	4	
Full scale (1967)	3	sand	0.85-1.7	36	3.4	3*m*	28-35		9-10		9-10		3-4		
		sand	0.85-1.7	36	5	3*m*	13		8		5		3		
Pilot scale Upflow		sand	0.85-1.7	60	3.4	3*m*	29-35		5		9-10		3		
		sand	0.85-1.7	60	4.8	3*m*	13		6		5		3		
Derby, England	4	sand	1.2-2.3	24	2.0	4	25-35	29	7-14	10					
Pilot study (1970)					3.0	2	29-31	30	10-13	12					34-45
					4.0	5	27-31	29	12-14	13					11-39
					8.0	2	24-29	27	16-18	17					
		sand	1.2-1.7	24	2.0	4	26-35	29	7-15	10					45
					3.0	2	29-31	30	9-13	11					15-47
					4.0	5	24-32	28	11-14	13					
					8.0	2	23-29	26	16-18	17					

Table A-1.—*Reported Efficiencies for Direct Filtration Of Trickling Filter Plant Effluents*[a] (continued)

Location	Source	Media			Filter Rate gpm/sq ft (U.S.)	N[c]	Suspended solids (mg/l)				BOD_5 (mg/l)				
		Type	Size[b] (mm)	Depth (in.)			Influent		Effluent		Influent		Effluent		%
							Range	Avg	Range	Avg	Range	Avg	Range	Avg	Removal
Triple media "a"		coal	1.4-2.3	8	2.0	2	25-35	29	6-12	8					
		sand	1.2-1.4	8	4.0	4	27-28	27	11-13	12					33
		garnet	0.7-0.85	8	8.0	3	23-29	26	15	15					
Upflow sand "a"		sand	0.7-2.3	24	2.0	2	28-37	33	9-15	12					
					4.0	3	27-30	27	13-14	13					
					8.0	2	23-29	26	16-17	17					
Triple media "b"		coal	1.4-2.3	8	3.0	2	29-31	30	7-11	9					38
		sand	0.85-1.0	8	4.0	2	29-32	31	10	10					38
		garnet	0.7-0.85	8											
Upflow sand "b"		sand	0.85-2.3	24	3.0	2	29-31	30	10-14	12					
					4.0	2	28-32	30	11-14	13					
Derby, England	4	sand	1.2-2.3	24	3.0	2	22-25	24	9-10	10					31
Pilot study after					4.0	5	20-24	22	7-10	9					50-64
trickling filter					6.0	2	19-26	23	10-11	11					35
improvement															
		sand	1.2-1.7	24	3.0	2	22-25	24	9-10	10					54
					4.0	4	21-24	23	8-10	9					53-65
Triple media "b"		triple (see above)			3.0	1	22	22	8	8					
					4.0	5	20-24	22	6-9	8					
					6.0	2	19-26	23	9	9					
Upflow sand "b"		sand (see above)			3.0	2	22-25	24	8-10	9					69
					4.0	4	21-24	23	8-11	10					59-71
Upflow sand		sand	1.2-2.3	36	6.0	4	19-26	23	9-11	10					43-67
Ames, Iowa															
Pilot study	5	sand	0.55 ES	24	2.0	15	11-49	20	1-15	6	38-115	56	13-49	24	
(1965)			2.36 UC		4.0	15	10-58	19	1-24	6	29-130	53	15-65	23	
					6.0	15	8-60	18	2-27	6	25-132	50	13-74	24	

Table A-1.—*Reported Efficiencies for Direct Filtration Of Trickling Filter Plant Effluents*[a] (continued)

Location	Source	Media			Filter Rate	*N*[c]	Suspended solids (mg/l)				BOD_5 (mg/l)				
		Type	Size[b] (mm)	Depth (in.)	gpm/sq ft (U.S.)		Influent		Effluent		Influent		Effluent		%
							Range	Avg	Range	Avg	Range	Avg	Range	Avg	Removal
Pilot study (1973)		dual media													
unpublished		coal	0.9 ES	12	1.7	12	21-75	33	1-8	4	39-85	57	6-19	13	
		sand	0.4 ES	12											

Prepared by Gary A. Rice and John L. Cleasby, Iowa State University, March, 1974.

[a]Blank spaces in table due to data missing or not presented in manner needed for table, e.g., for averaging. All mg/l values rounded to nearest 1 mg/l.

[b]Range in size given, British practice, or ES (effective size) and UC (uniformity coefficient), U.S. practice.

[c]*N* = number of values reported in the range and average presented. *N* generally represents individual filter runs unless followed by the letter *m* which indicates the number of average monthly values presented. *m* without numeral means average of several months data (unspecified duration).

Table A-2.—*Reported Efficiencies For Direct Filtration Of Activated Sludge Plant Effluents*[a]

Location	Source	Media			Filter Rate gpm/sq ft (U.S.)	*N*[c]	Suspended solids (mg/l)				BOD_5 (mg/l)				
							Influent		Effluent		Influent		Effluent		%
		Type	Size[b] (mm)	Depth (in.)			Range	Avg	Range	Avg	Range	Avg	Range	Avg	Removal
West Hertfordshire,															
England (1968)	6	gravel	40-50	6	2.16		10-89	44	1-2	22		58		3.9	
		gravel	8-12	10	4.0		9-70	37	2-7	3.7		53		4.6	
Pilot scale "Boby" upflow		gravel	2-3	10	5.0		12-128	55	1-17	7.1		42		5.6	
filter		sand	1-2	60	6.0		5-97	37	2-22	9.9		35		4.7	
Letchworth, England	7														
(1968)		gravel	20-30	4	3-4	81	10-26	22	1-12	5.5					
		gravel	10-15	4	4-6	66	10-28	16	2-15	6.7					
Pilot scale "Boby" upflow		gravel	2-3	4	6-8	65	7-24	14	6-14	8.8					
filter		sand	1-2	60											
Los Angeles, Calif. (1961)	8	sand	0.95 ES	11	2	5	19-34	27	7-21	15	6-15	10	2.8	4	
Preliminary tests			1.6 UC												
Philowith, Ore. (1967)	9	mixed	–	30	5		30-2180	59	1-20	4.6	17-36	26	1-4	2.5	
Extended aeration AS)		media													
Peoria, Ill. (1964)	10	sand			1.1			35		8		45		17	
(High rate AS)															
Cleveland, Ohio	11	dual media													
		coal	1.78 ES	60	8	1	20		5		19		14		
			1.63 UC		16	1	27		8		9		5		
		sand	0.95 ES	24	24	2	22-23		9-11		9-10		4-6		
			1.41 UC		32	1	29		14		9		7		
Declining rate filters at		dual media													
Cleveland. (Avg filter		coal	4.0 ES	60	8	1	13		4		7		5		
rate presented)			1.5 UC		16	1	13		4		7		5		
		sand	2.0 ES	24	24	1	13		6		7		5		
			1.32 UC												

Prepared by Gary A. Rice and John L. Cleasby, Iowa State University, March, 1974.

[a]Blank spaces in table due to data missing or not presented in manner needed for table, e.g., for averaging. All mg/l values rounded to nearest 1 mg/l.

[b]Range in size given, British practice, or ES (effective size) and UC (uniformity coefficient), U.S. practice.

[c]*N* = number of values reported in the range and average presented. *N* generally represents individual filter runs unless followed by the letter *m* which indicates the number of average monthly values presented. *m* without numeral means average of several months data (unspecified duration).

SOURCES

[1] A. E. J. Pettet, W. F. Collett, and T. H. Summers "Mechanical Filtration of Sewage Effluents. I. Removal of Humus," *J. Proc., Inst. Sewage Purif.*, Part 4:399-411, 1949.

[2] S. C. Evans, "Ten Years of Operation and Development at Luton Sewage Treatment Works," *Water and Sewage Works*, 104:214-219, 1957.

[3] A. A. Naylor, S. C. Evans, and K. M. Dunscombe, "Recent Developments on the Rapid Sand Filters at Luton," *Water Pollution Control (British)*, 66:309-315, 1967.

[4] J. R. Joslin and G. Greene, "Sand Filter Experiments at Derby," *Water Pollution Control (British)*, 69:611-622, 1970.

[5] K. J. Merry, "Tertiary Treatment of Domestic Wastewater by Rapid Sand Filtration," unpublished master's thesis, Iowa State University Library, Ames, Ia., 1965.

[6] R. Wood, W. S. Smith, and J. K. Murray, "An Investigation into Upward Flow Filtration," *Water Pollution Control (British)*, 67:421-426, 1968.

[7] G. A. Truesdale and A. E. Birkbeck, "Tertiary Treatment of Activated Sludge Effluents," *Water Pollution Control (British)*, 67:483-492, 1968.

[8] F. B. Laverty, L. A. Meyerson, and R. Stone, "Reclaiming Hyperion Effluent," *J. Sanit. Engr. Div., Amer. Soc. Civil Engr.*, 87:1-40, 1961.

[9] G. L. Culp and S. P. Hansen, "Extended Aeration Effluent Polishing by Mixing Media Filtration," *Water and Sewage Works*, 114:46-51, 1967.

[10] E. B. Fall, Jr. and L. S. Kraus, "Tertiary Treatment for High Rate Activated Sludge Effluent," unpublished paper presented at the 37th Central States Water Pollution Control Association meeting, June 1964.

[11] R. Nebolsine, I. Pouschine, and C. Y. Fan, "Ultra-High Rate Filtration of Activated Sludge Plant Effluent," U.S. Environmental Protection Agency, Office of Research and Monitoring, EPA-R2-73-222, Apr. 1973.

 ☆ U. S. GOVERNMENT PRINTING OFFICE: 1977-757-140/6569 Region No. 5-11

METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Length	meter	m	*Basic SI unit*	39.37 *m* = 3.281 ft = 1.094 yd
	kilometer	km		0.6214 mi
	millimeter	mm		0.03937 in
	micrometer or *micron*	μm or μ		3.937×10^{-5} in = 1×10^4 Å
Area	square meter	m^2		10.76 sq ft = 1.196 sq yd
	square kilometer	km^2		*0.3861 sq mi = 247.1 acres*
	square millimeter	mm^2		0.001550 sq in
	hectare	ha	The hectare (10,000 m^2) is a recognized multiple unit and will remain in international use.	2.471 acres
Volume	cubic meter	m^3		35.31 cu ft = 1.308 cu yd
	litre	l		1.057 qt = 0.2642 gal = *0.8107×10^{-4} acre ft*
Mass	kilogram	kg	*Basic SI unit*	2.205 lb
	gram	g		0.03527 oz = 15.43 gr
	milligram	mg		0.01543 gr
	tonne	*t*	1 *tonne* = 1,000 kg	0.9842 ton (long) = 1.102 ton (short)
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 in a mass of 1 kg.	0.2248 lb = 7.233 poundals
Moment or torque	newton meter	N-m	The meter is measured perpendicular to the line of action of the force N. Not a joule.	0.7375 lb-ft 23.73 poundal-ft
Flow (volumetric)	cubic meter *per second*	m^3/s		15.850 gpm = *2,119 cfm*
	liter per second	l/s		15.85 gpm

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Velocity linear	meter per second	m/s		3.281 fps
	millimeter per second	mm/s		*0.003281 fps*
	kilometers per second	km/s		2,237 mph
angular	radians per second	rad/s		9.549 rpm
Viscosity	pascal second	Pa-s		*0.6722 poundal(s)/sq ft*
	centipoise	Z		1.450×10^{-7} Reyn (μ)
Pressure or *stress*	newton per *square meter* or pascal	N/m^2 or Pa		0.0001450 lb/sq in
	kilonewton per square meter or kilopascal	kN/m^2 or kPa		*0.14507 lb/sq in*
	bar	bar		14.50 lb/sq in
Temperature	Celsius (centigrade)	°C		(°F−32)/1.8
	Kelvin (abs.)	°K		°C + 273.2
Work, energy, quantity of heat	joule	J	1 joule = 1 N-m where meters are measured along *the line of action* of force N.	2.778×10^{-7} kw-hr = 3.725×10^{-7} *hp-hr* = 0.7376 ft-lb = 9.478×10^{-4} Btu
	kilojoule	kJ		2.778×10^{-4} kw-hr
Power	watt	W	1 watt = 1 J/s	44.25 ft-lbs/min
	kilowatt	kW		1.341 hp
	joule per second	J/s		3.412 Btu/hr

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Precipitation, run-off, evaporation	millimeter	mm	For meteorological purposes, it may be convenient to measure precipitation in *terms of mass/unit* area (kg/m^2). 1 mm of rain = 1 kg/m^2	
Flow	cubic meter per second	m^3/s		35.31 cfs
	liter per second	l/s		15.85 gpm
Discharges or abstractions, yields	cubic meter per day	m^3/d	1 l/s = 86.4 m^3/d	0.1835 gpm
	cubic meter per year	m^3/year		264.2 gal/year
Usage of water	liter per person per day	l/person/day		0.2642 gcpd

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Density	kilogram per cubic meter	kg/m^3	The density of water under standard conditions is 1,000 kg/m^3 or 1,000 g/l or 1 g/ml.	0.06242 lb/cu ft
Concentration	milligram per liter (water)	mg/l		1 ppm
BOD loading	kilogram per cubic meter per day	$kg/m^3/d$		0.06242 lb/cu ft/day
Hydraulic load per unit area, *e.g., filtration* rates	cubic meter per square meter *per day*	$m^3/m^2/d$	If this is converted to a velocity, it should be expressed in mm/s (1mm/s = 86.4 m^3/m^2/day).	3.281 cu ft/sq ft/day
Air supply	cubic meter or liter of free air per second	m^3/s l/s		
Optical units	*lumen per* square meter	*lumen/m^2*		0.09294 ft candle/sq ft

*Miles are U.S. statute, qt and gal are U.S. liquid, and oz and lb are avoirdupois.