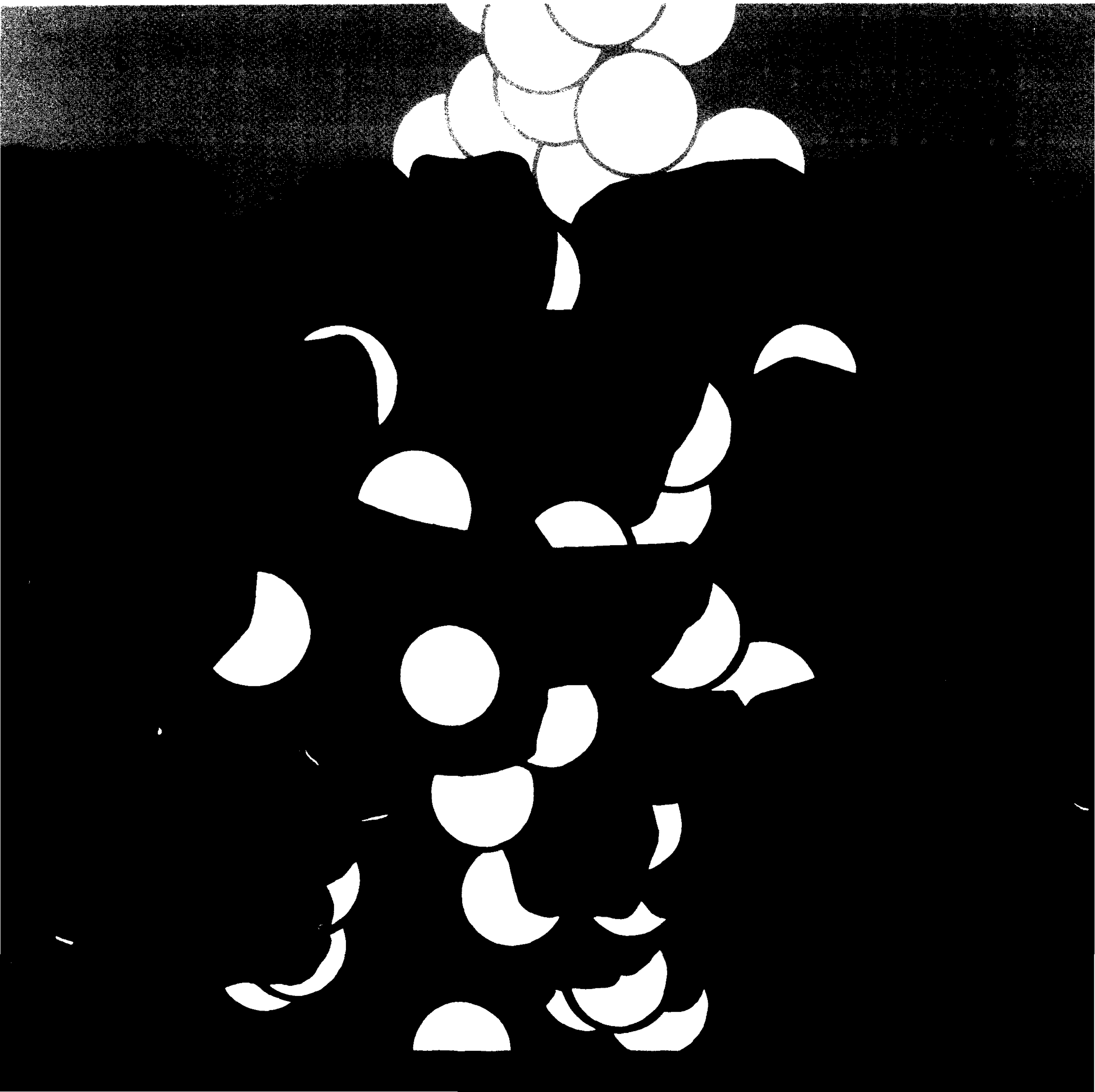
Wastewater Filtration
Design Considerations

EPA Technology Transfer Seminar Publication



WASTEWATER FILTRATION

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INTRODUCTION

If water containing suspended solids is passed through a layer of porous media, some of the suspended and colloidal materials are removed. This process is called "filtration," and its efficiency and cost are a function of

- The concentration and characteristics of the solids in suspension (particle-size distribution, surface characteristics, organic versus inorganic, etc.)
- The characteristics of the filter media and other filtering aids used (particle-size distribution, surface characteristics, etc.)
- The characteristics of the solids in solution in the water filtered
- The characteristics of the filter and its method of operation

The criteria that must be considered in design involve finding

- The *operational optimum* filter-design characteristics
- The *economic optimum* filter design, the primary goal sought in engineering design

During a filter run, the headloss across the filter media will increase owing to the accumulation of solids within the filter media. When this head reaches the limit set by the hydraulic conditions of the design, the filter run must be stopped. Figure 1 indicates that, as the headloss increases during a filter run, the filtrate quality also changes, the effluent solids level tending to rise in value as time proceeds. Although the filter could be designed to produce a satisfactory filtrate quality during the early stages of the run, the time may come when the filtrate quality will become unsatisfactory and the filter run will have to be terminated because of solids breakthrough above the maximum permissible concentration of suspended solids (C_c). A filter operational optimum condition occurs when both the headloss and effluent quality reach their respective critical values (H_c and C_c) *at the same time.*

To achieve an operational optimum, many alternative designs are possible that can produce "equivalent performance." Two or more filters may be said to provide equivalent performance when they produce the same quality and quantity of filtered water from the same water source during the same time period. Of the equivalent-performance filters, however, only one will produce water at the least total cost per 1,000 gallons. Current trends indicate that the time is approaching when it will be possible to design filters to provide *both* operational and economic (least cost) optimums.

Operational optimum design of a filter to remove a particular suspended solid requires that the designer first select the type of chemical pretreatment required (if any) to achieve the desired water quality. With the filter influent water quality then established, the filter design requires selection of

- Media sizes
- Media depths

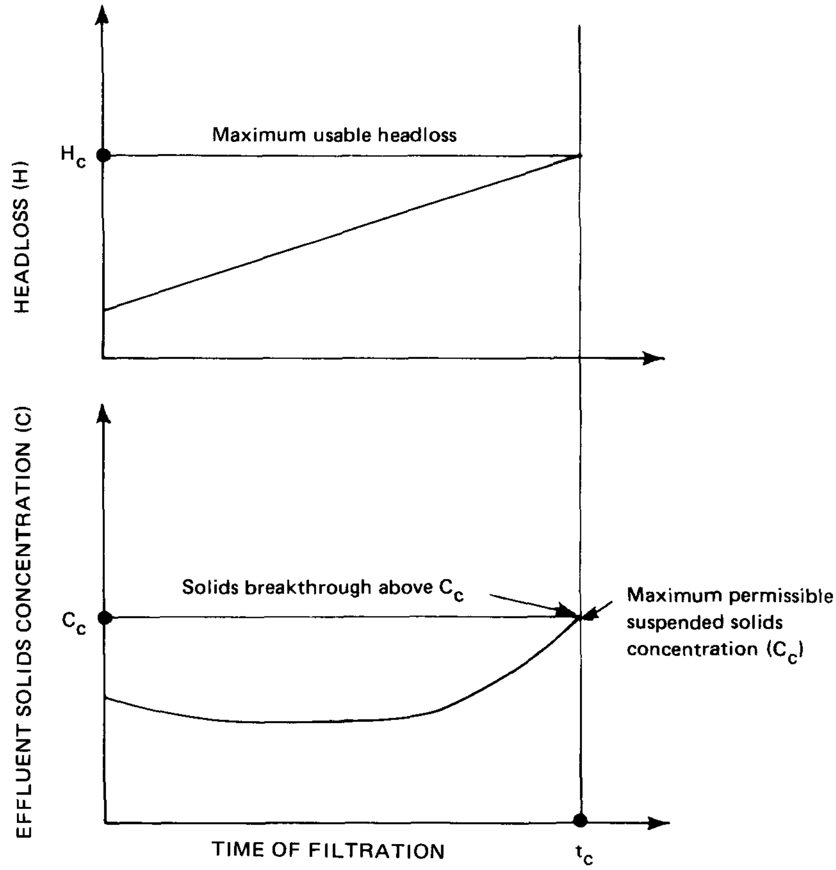

Figure 1. Operational optimum filter design.

- Filtration rate
- Terminal headloss

In a least cost optimum design, the cost effects of the filter structure, hydraulic appurtenances, energy, labor and maintenance requirements, and total filter plant size also have to be considered. Ives[1] and Huang and Baumann[2] have recently considered in detail the data needed for design of operational and economic or least cost optimum designs for granular media filters suitable for water and wastewater-treatment applications. To accomplish this purpose, three elements are necessary.

- A method (mathematical model) to predict the performance of the filters, to determine combinations of the filter-design variables that will provide the same quality of effluent under operational optimum conditions
- A filtration system whose first cost, operating cost, and maintenance cost can be predicted with reasonable accuracy
- A computer program that can provide a list of operational optimum filter designs and predict the one design which will provide the treated water at least cost

To date, there is no mathematical model available for consideration in wastewater-filtration applications; nor is there even carefully documented filter operating experience with wastewater.

Accordingly, it would appear that before detail design of filters for wastewater treatment the required operational optimum least cost design should be developed by

- Running pilot-plant studies using a water pretreated as proposed in the tertiary filtration application

- Building the complete treatment plant based on pilot-plant design, with enough filter flexibility for perfecting the operation, using full-scale plant filters and the actual waste to be treated

The analysis of the data collected to produce a satisfactory model could be accomplished using the techniques of Hsiung and Cleasby.[3] Until such data are available, the design requirements of tertiary filtration plants must be based on the wastewater-filtration experience of others.

Chapter I

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 Jtu) 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. That is, if in a 24-hour period the flow peak is twice the average daily flow, the raw suspended solids level will also be about twice the average daily suspended solids level. Because all wastewater-treatment processes are least efficient under their peak-load operating conditions, the high suspended solids level in raw wastewaters under peak flows will be carried over 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 Jtu). 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 the 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.

In wastewater filtration, the operation of secondary biological-treatment plants are subject to wide variations in solids levels carried over to tertiary filters. The filtering characteristics of such

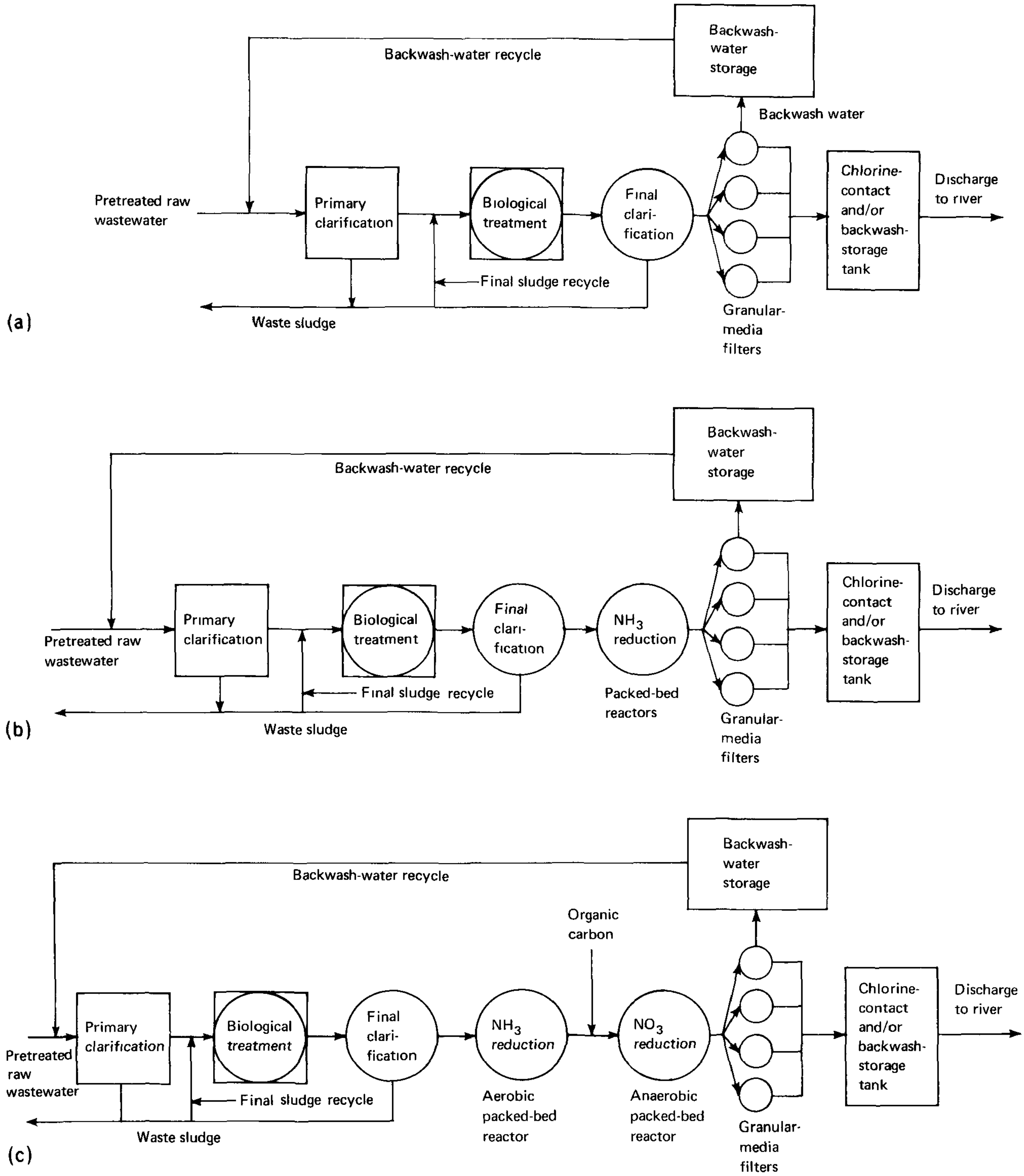

Figure I-1. Granular media filters for tertiary wastewater treatment: (a) following biological secondary treatment for carbonaceous BOD removal; (b) following biological secondary and biological tertiary (packed-bed reactors) treatment for carbonaceous BOD and ammonia reduction; (c) following biological 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.)

solids are affected by the solids retention times maintained in the biological reactors and 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.

In advanced wastewater treatment, filtration (fig. I-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 will require a pilot study to evaluate the flow characteristics and solids characteristics of the water to be filtered. Some wastewater filters are being built merely because they are expected to improve effluent quality, without prior evaluation of their operating characteristics. In such cases, reliance is placed on designing for a given filter-operating experience (a given length of run at the design filtration rate), with acceptance of the effluent quality that results.

The problems encountered in designing filters for wastewater treatment *require* that the following be considered:

- A completely mixed flow equalization basin ahead of granular media filters should be provided. Figure I-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 justify the costs for such provisions.

- The higher solids loadings to wastewater filters require better distribution of solids throughout the filter bed. This improvement can be accomplished by

  — using coarser top media, requiring a higher backwash rate

  — using dual- or triple-media or upflow filters to achieve coarse-to-fine filtration

  — using coarse, deep bed, nearly unisize media filters

  — providing higher terminal headlosses

Chapter II

FILTER-DESIGN CONSIDERATIONS

GENERAL

The design of a filter for a given application requires that selection of the following be considered:

- Filter configuration
- Method of flow control
- Terminal headloss, feet of water
- Filtration rate, gallons per minute per square foot
- Filter media, sizes and depths
- Backwashing requirements

Because the capital cost of a filter is chiefly a function of the area of filter provided, a high filtration rate usually is preferred. In general, the filter design should seek to maximize the net water production (total water filtered less water used for backwashing) per square foot of filter *consistent with filter-operating feasibility.* A most useful relationship between net water production and run lengths obtained at different filtration rates is shown in figure II-1. This figure was constructed by calculating the net useful water production at various filtration rates when run lengths (hours) of 1, 2, 3, 5, 10, 20, 30, 50, and infinite duration are obtained. Backwash is assumed to require 30 minutes, including 3 minutes of air scour at a rate of 3 $ft^3/min/ft^2$ followed by a water wash of 5 minutes at 20 $gal/min/ft^2$ with wash water *not* recycled through the plant. Figure II-1 shows that there exists an upper limit of net water production at each filtration rate. The maximum net water production that can be obtained in a day is 2,880, 5,760, or 8,640 $gal/d/ft^2$ at filtration rates of 2, 4, or 6 $gal/min/ft^2$, respectively. An assumed desirable net water production of 3,500 $gal/d/ft^2$ can be obtained at a filtration rate of 2.45 $gal/min/ft^2$ if an infinite run length can be obtained. At 2.5 $gal/min/ft^2$ a 50-hour run is needed. At 3 $gal/min/ft^2$ a run length of 5 hours is needed. With a 1-hour run length, a 5.3-$gal/min/ft^2$ filtration rate could be used.

In summary, figure II-1 indicates that any run length greater than 1 hour at 5.3 $gal/min/ft^2$ would produce more net water production than an infinite run length at 2.45 $gal/min/ft^2$. Figure II-1 should be interpreted as proof that both filtration rate *and* run length are very important in their effect on filtration economy. Run lengths longer than 24 hours at any filtration rate do not increase net water production significantly. Run lengths shorter than about 10-12 hours do affect net water production and present a second major effect.

In normal practice, at least two and usually four filters are provided in a tertiary wastewater-treatment plant. Even with four filters, it is unusual when all are in operation simultaneously. In fact, with 1-hour runs,

- All four filters can never be in operation simultaneously.

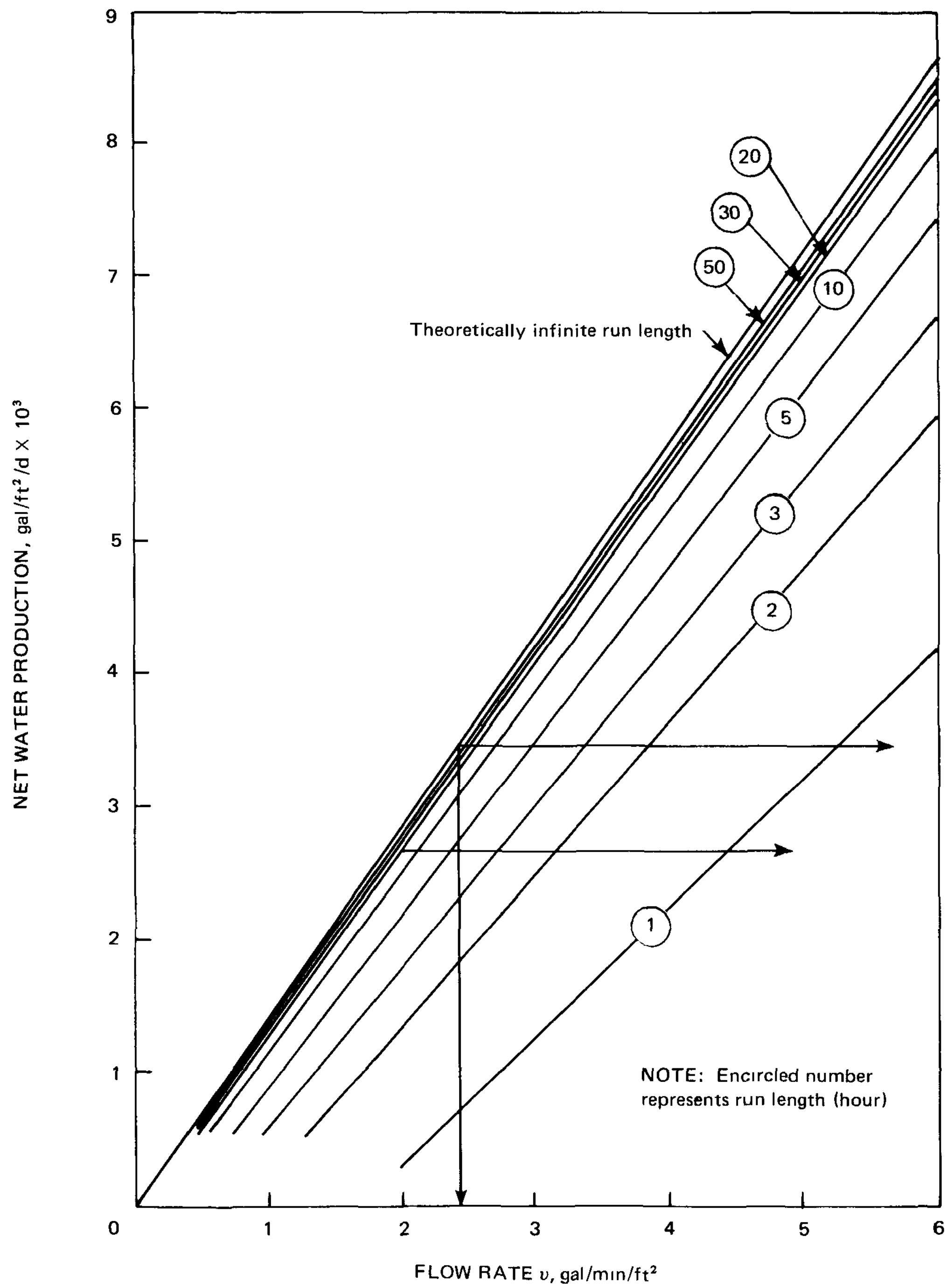

Figure II-1. Net water production versus flow rate at various run lengths (30-minute backwashing period assumed).

- At most, three filters will be in operation simultaneously only two-thirds of the time.
- In general, only two filter cells are in operation for one-third of the time.

To have at least three filters in operation all the time, run lengths must exceed 1½ hours. Figure II-2 shows the percentage of time that all four filter cells will be in operation as a function of run length.

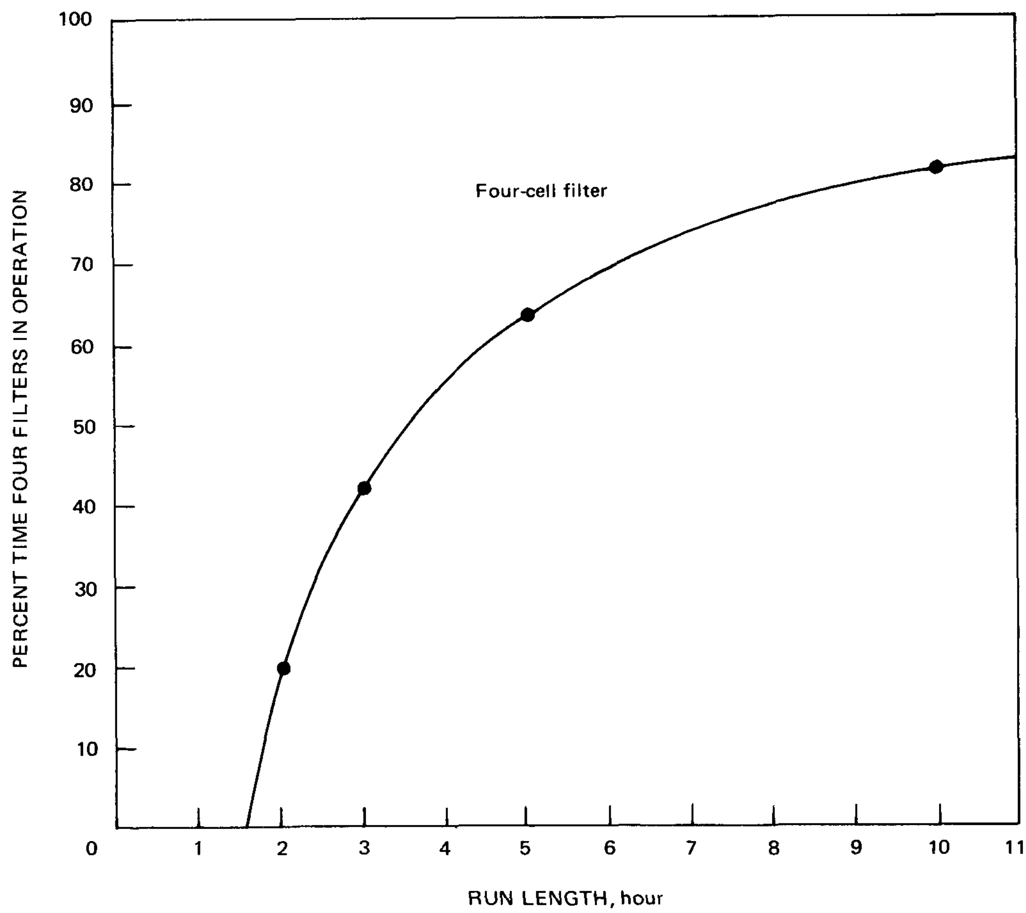

Figure II-2. Percentage of time four filter cells in operation versus run length.

This figure says, in effect, that when run lengths are only 9.25 hours, the filtration rate on the filters will be 1.33 times the design filtration rate for *20 percent of the time.* If run lengths are shorter, the percentage of time of rate overload increases significantly. Practically, this figure says that run lengths under peak operating conditions should not be less than 10-12 hours and, even then, there is reason to increase filter area about 20 percent to be able to meet the peak operating conditions successfully.

FILTER CONFIGURATIONS

Figure II-3 shows several filter configurations that have been used in water and wastewater filtration. The granular-media filter originally was used for potable water filtration with single media (effective size = 0.5 mm, 1.5-1.8 uniformity coefficient) and downward filtration (first through the finest media, which collected at the top of the filter during backwash) at low filtration rates (2 gal/min/ft^2 (fig. II-3a). In recent years, design trends are toward use of higher filtration rates (2-5 gal/min/ft^2), deeper media, and coarser media. To distribute solids better within the filter media, several filter configurations are in use.

- Upflow filtration through a relatively deep, coarse filter media (fig. II-3b). This concept only recently has been promoted, and limited American experience is available.

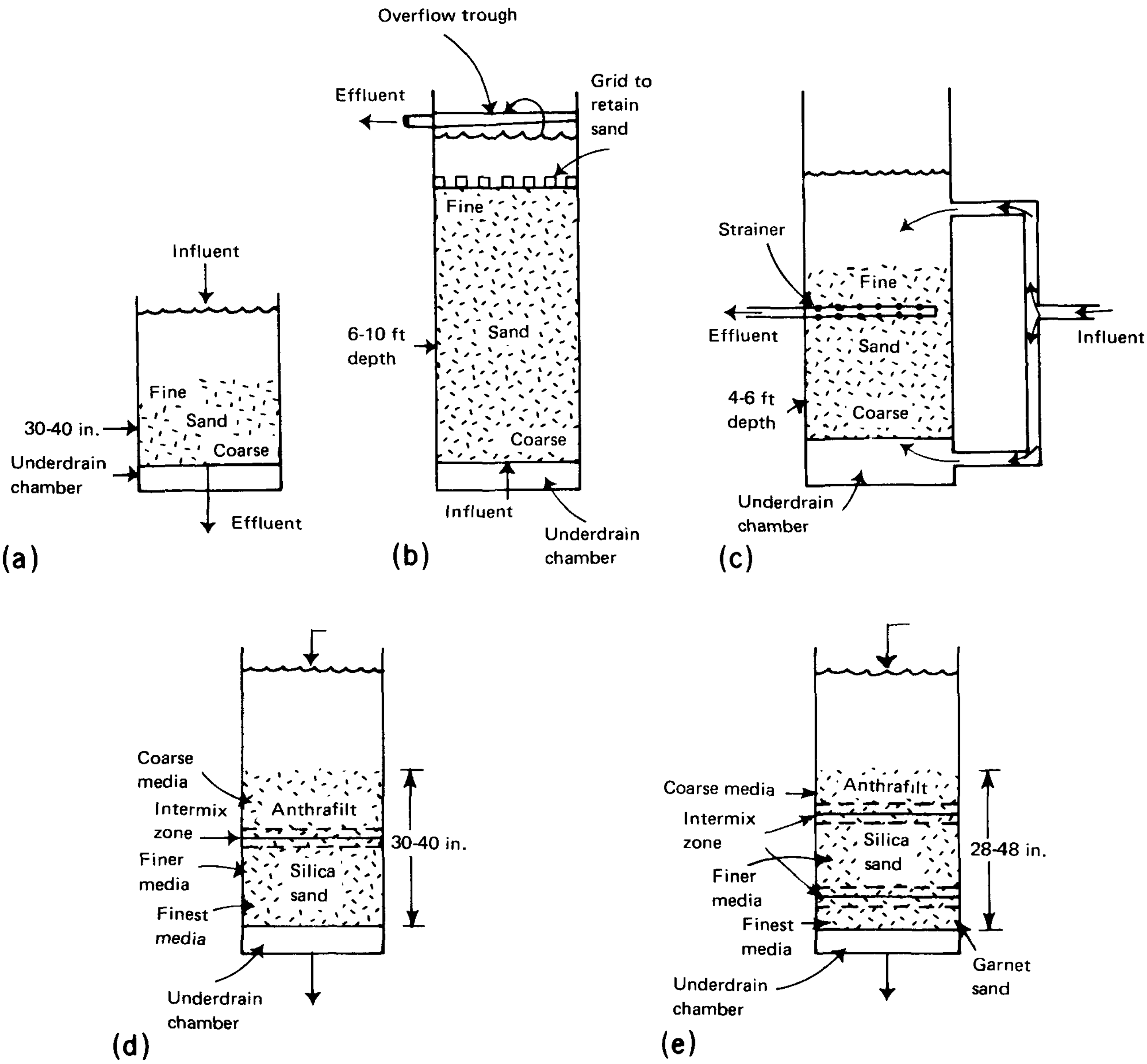

Figure II-3. Filter configurations: (a) single-media conventional filter; (b) single-media upflow filter; (c) single-media biflow filter; (d) dual-media filter; (e) mixed-media (triple-media) filter.

- Using a filtered-water collection device within the filter media and bringing water in from both the bottom and top of the filter media (fig. II-3c). This concept was introduced in Europe, and only limited experience is recorded.

- Dual or mixed media (trimedia), with downward flow as in conventional filters (fig. II-3d, e).

British practice in tertiary treatment tends toward the figure II-3b configuration,[4] which has several advantages.

- Filtration can take place from coarse to fine media, using a single media for better solids distribution. Therefore, a coarser effective size with a larger uniformity coefficient media can be used.

- Effective backwash time is less because draindown time is significantly less.

- Raw water is used for backwashing, reducing somewhat the amount of water that must be filtered twice.

- There is no need for high walls on the gravity-flow filter tank to build up the static head required for filtration.

The chief disadvantage of this configuration is the need for a bar-screen configuration within the sand near the surface to retain the media in place against the upward force exerted during filtration.

American practice tends toward two or three media, as in figure II-3d and e. Usually a coarse anthrafilt (specific gravity = 1.35-1.75) is used on top of a finer silica sand (specific gravity = 2.65). Occasionally, a still finer garnet sand (specific gravity = ±4.1) is used at the bottom. Dual- or triple-media filters have the following advantages:

- Filter design follows current American practice and permits production of the desired quality of effluent with reasonable run lengths.

- No filter-media-restraining grids are required.

These filters have the following disadvantages:

- Dual- or triple-media filters require relatively deep filter cells to provide the required filtering head without creating negative head conditions in the filter (unless pressure filters are used).

- Filtering-down time in preparation for backwashing is significant. It extends the out-of-service time for backwashing to about 34 minutes at a filtration rate of 3 gal/min/ft^2 and 24 minutes at 6 gal/min/ft^2 (unless the water above the media is dumped to the wastewater gullet and returned to head end of the plant).

- Use of dual- or triple-media filters creates a need for more care in media selection and backwash design to prevent loss of media or excessive intermixing of the media.

Although configurations shown in figure II-3b, d, and e all have applications in tertiary wastewater filtration, only the dual-media filter configuration will be considered for further discussion because it can provide effective operation in tertiary filtration applications and is nonproprietary.

METHODS OF FLOW CONTROL

In any filtration operation, the rate of flow through a filter may be expressed as

$$\text{rate of flow} = \frac{\text{driving force}}{\text{filter resistance}} \tag{1}$$

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.

Constant-Rate Filtration 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 operating 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.[5,6] This disadvantage, however, is not as important in wastewater filtration.

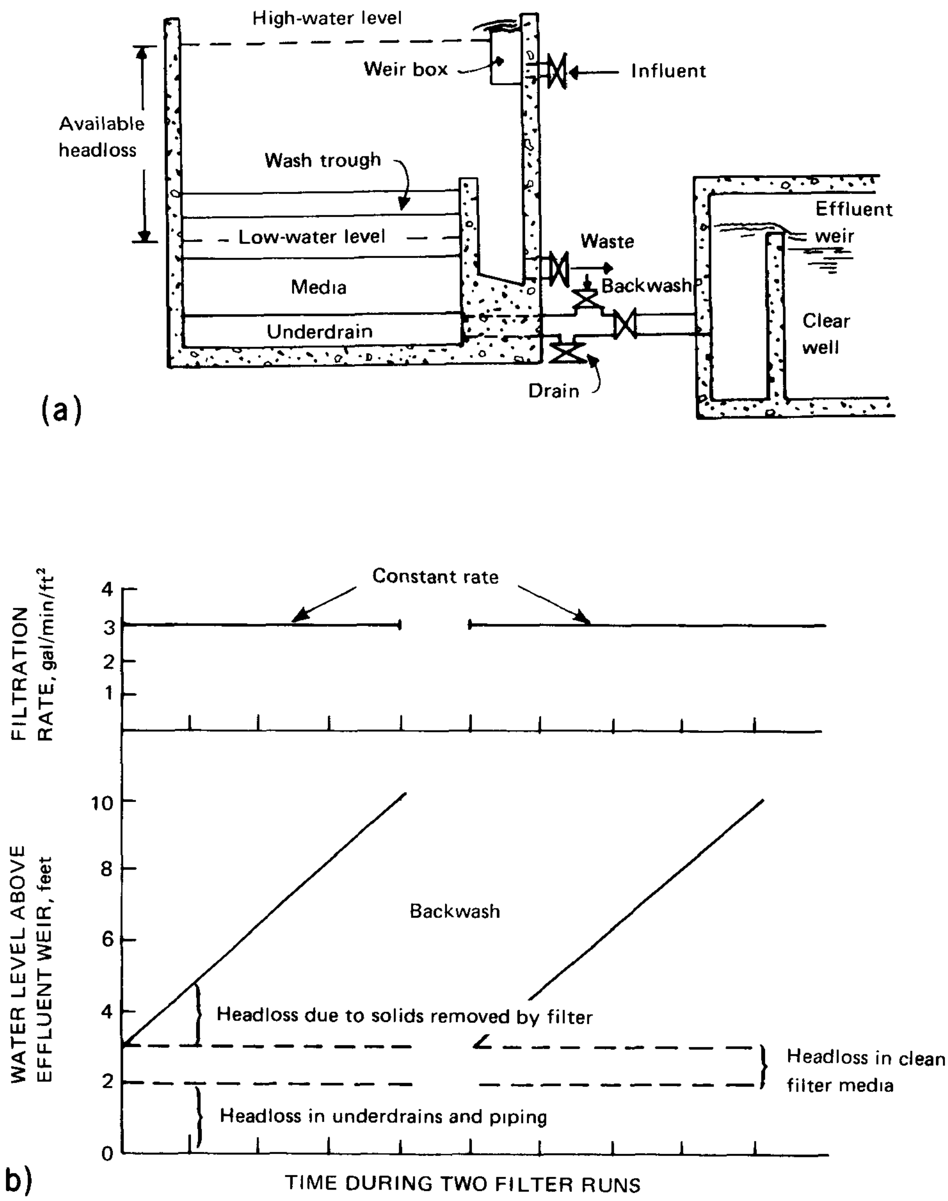

Figure II-4. Influent-flow-splitting filtration: (a) typical filter and clear well arrangement; (b) filtration rate, water level, and headloss during two filter runs.

Rate control for granular-media filters has been achieved for many years by such effluent-control systems. The valves used as rate controllers frequently do not function properly and require an excessive amount of maintenance. A number of other alternative methods of flow control are coming into use that will supplant the effluent-flow-control valve[5] 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 II-4. 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 lowers 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.[7,8] 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.[5]

Figure II-5a illustrates the desirable arrangement for new plants designed for variable declining-rate operation. Great similarity exists between figures II-4a and II-5a, 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 II-4a, with the following exceptions. Figure II-5a 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, 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

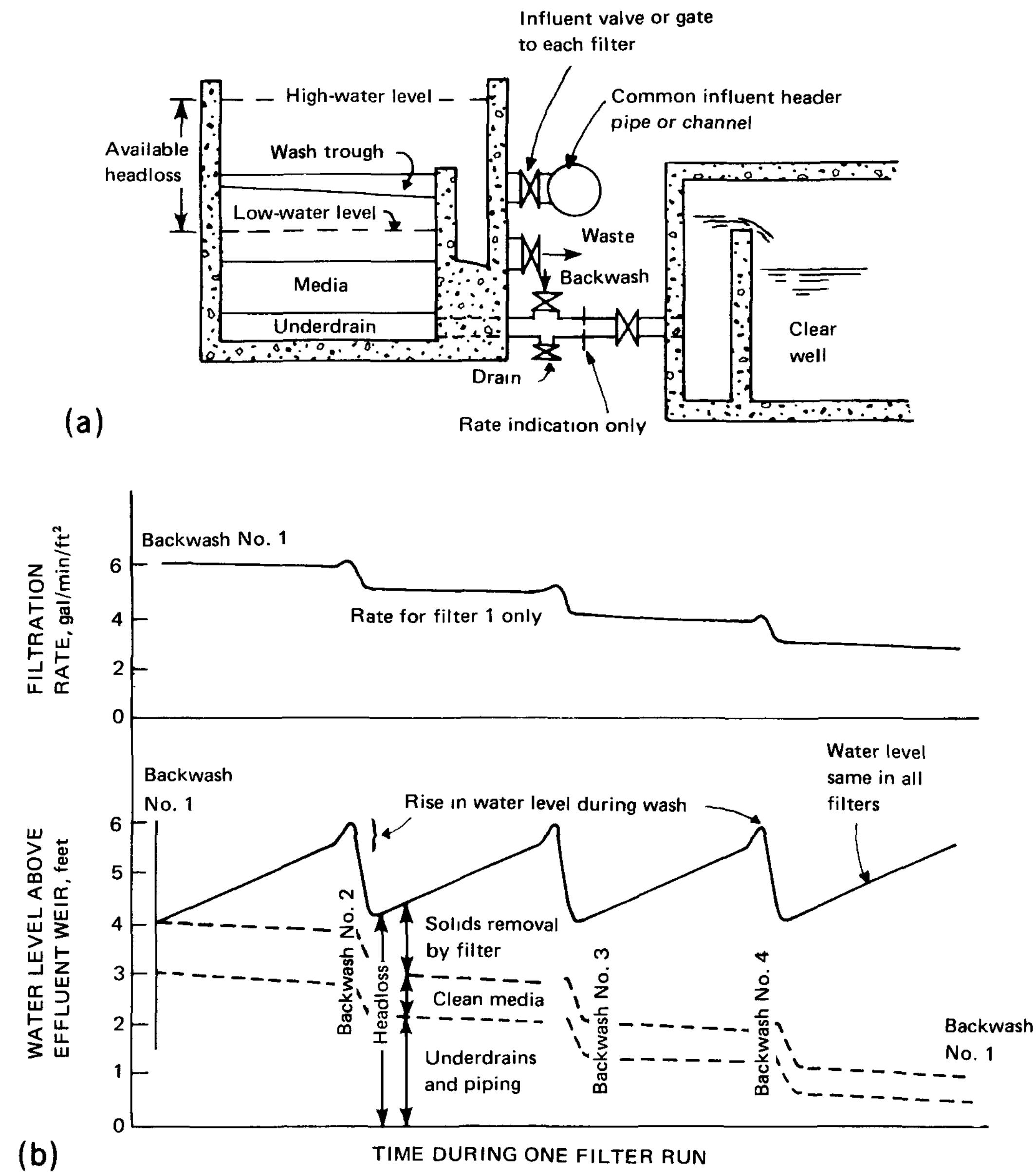

Figure II-5. Variable declining-rate filtration: (a) typical filter and clear well arrangement; (b) filtration rate, headloss, and water level during one filter run in a plant having four filters.

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.

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.[7] 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:[5,6]

- 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.

FILTER MEDIA

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 millimetres) 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, 1.35-1.75

- Silica sand, 2.65

- Garnet sand, 4-4.2

Filters using two or three media of these specific gravities can be used so that after backwashing the media will be arranged with the coarse, lighter coal on top, the finer, heavier garnet sand on the bottom and the middle-sized silica sand between the two. The actual distribution of media after backwashing will depend, of course, on whether single, dual, or mixed media are used, and on the relative particle-size distributions and specific gravities of each of the media.

The literature in filtration of secondary effluents makes it abundantly clear that some of the biological floc carryover to the filters tends to be strongly removed at the top surface and in the upper layers of the filter media, causing rapid headloss development, short filter runs, and excessive backwash requirements. Furthermore, the removal of solids is less affected by filtration rate, influent suspended solids concentration, or media size than is typical for potable water filtration. It is believed that this factor derives from the bimodal distribution of particle sizes in secondary effluents reported by Tchobanoglous.[9,10]

The detrimental effects of this strong surface-removal tendency can be counteracted by several alternative design choices.

- A coarser top media size
- Dual- or mixed- (triple-) media filters to protect the filtrate quality when using coarse top-sized coal
- Deep beds of near unisized coarse filter sand
- A coarse-to-fine flow direction (upflow) in a graded media filter
- Higher filtration rates, which cause greater penetration of solids into the media
- Higher terminal headloss capability to achieve acceptable run length, even though high headloss occurs

These design choices are not all mutually compatible and have secondary implications that must be considered. Each layer will consist of fine media at the top and coarser media at the bottom. A coarse top media size in the anthracite layer means a still coarser bottom size. The coarse bottom size dictates the minimum backwash rate required to fluidize the bed. Use of too coarse a media may require a backwash rate that is abnormally high, causing extra costs for backwash pumps, water storage, piping, and appurtenances. The coarse bottom size can be reduced by specifying a more uniform media, which may also have some minor benefit to filtrate quality. Such specification, however, increases the media cost.

The use of dual- or triple-media filters with a coarser top-sized coal to prevent surface cake formation will provide additional protection to the filtrate quality. This practice, however, does complicate bed design, as each of the media must be specified to achieve the desired degree of intermixing or nonintermixing, and the coarser sizes of all three media should be fluidized at about the same minimum fluidization velocity to insure that all media are adequately fluidized during the backwash operation.

One other possible advantage of dual-media over single-media filters in tertiary filtration is that any mud balls formed in the filter form at the surface and, when large enough, they sink to the sand-coal interface. There they remain exposed to auxiliary washing agitation rather than sinking to the bottom of the filter where they tend to accumulate. Of course, adequate backwashing is desired to attempt to prevent mud-ball formation in the first place, as will be discussed later.

The use of deep beds of near-unisized coarse filter media attempts to achieve depth filtration, filtrate quality, and solids storage comparable to that achieved by coarse-to-fine filtration in a shallower depth, dual- or triple-media filter. Because media size has only a small effect on filtrate quality in tertiary filtration, this concept may be valid; however, published data to support it are unavailable. The very coarse size of the sand media used (2-3 mm) would dictate extremely high backwash rates if bed fluidization were to be achieved. However, the promoters of this concept claim success with subfluidization water backwash and simultaneous air scour.

Upflow graded-sand filters using a restraining grid to prevent uplifting of the media during the filtration cycle are being promoted by at least two U.S. companies. A deeper bed of sand (4-6 feet) is used so that the added weight of the sand also resists the uplift forces. The upflow filter may not be compatible with higher filtration rates, especially those occurring during peak-flow periods.

Higher filtration rates result in deeper suspended solids penetration and better bed utilization, but also cause higher rates of headloss development. Excessively high filtration rates (20-30 gal/min/ft^2) can even cause most of the solids to pass through the coal layer and be removed in the sand layer.[11] Provision of such higher filtration rates, possibly using pressure filters to achieve higher terminal headloss, may yield optimum operating and capital economy. Some operating flexibility is needed, however, so that the operator can select the numbers of filters in service to achieve optimum operation for varying flow rates and influent suspended solids levels.

Pressure filters may be necessary in an optimum economic system, but their drawbacks are well known. Observation, inspection, or replacement of the media is difficult unless something better than the usual access manhole is provided. If pressurized discharge is desired, there may be danger to the underdrain plate if the operator inadvertently allows excessive pressure drop to occur through the media, merely because a high influent pressure is available. A bank of pressure filters fed by an influent pumping system through a common influent header can function in the declining rate mode of operation if influent flow equalization is provided. If one filter is removed from service for backwashing, however, the other filters pick up the load instantaneously, which may cause some temporary detriment to filtrate quality.

Since use of dual-media filters is the most common method of achieving adequate solids storage and filter run length, and since several of the other filter-design alternatives are at least partly proprietary in nature, specific design discussion (and recommendations for the filter media) will be limited to consideration of dual-media filters. The points that follow are particularly important.

The top size of the coal should be between 1 and 2 mm. The coarser size in this range would permit more solids storage but require higher backwash rates. An effective size of at least 1 to 1.2 mm should be specified. After placement in the filter the coal should be backwashed two or three times, and 1-inch layers of the fine surface material should be skimmed off after each wash. The benefits of two or three such skimmings, compared with two or three backwashes and a single skimming, have not been evaluated. Skimming is *essential* by one of these two procedures to remove unwanted fine coal. This technique should achieve a top surface media size of at least 1.1 to 1.2 mm.

As nearly uniform coal as practicable (low uniformity coefficient) should be specified to minimize the bottom coal size and the backwash rate required. The minimum fluidization velocities (V_{mf}) of coal, sand, and garnet sand of various *uniform* sizes is shown in figure II-6, and the typical effect of temperature in figure II-7. Figure II-6 agrees substantially with one presented earlier by Camp.[12] The minimum uniformity coefficient commercially available is about 1.3. More uniform media can be achieved by specifying that all coal lie between adjacent U.S. sieve sizes (e.g., -14 +16) passing 14 mesh and retained on 16 mesh) or, more practically, between alternate adjacent sieve sizes (e.g., -12 +16) (table II-1). The latter type of specification is preferred because it does not encourage the manufacturer to combine two different size media to achieve a specified uniformity coefficient. Naturally, some tolerance must be allowed on either end of the specification, because large-scale machine sieving is never complete or accurate. For example, 10-15 percent by weight may be allowed finer than the specified smaller sieve and 10-15 percent coarser than the larger sized sieve. Surface skimming of the backwashed media after placement in the filter can then be used to remove the finer media.

A sand specification compatible with the specified coal should be selected. The bottom sand size (e.g., the 90-percent finer size) should have the same V_{mf} as the bottom coal to insure that the entire bed fluidizes at about the same backwash rate. The effective size of sand should be such as to

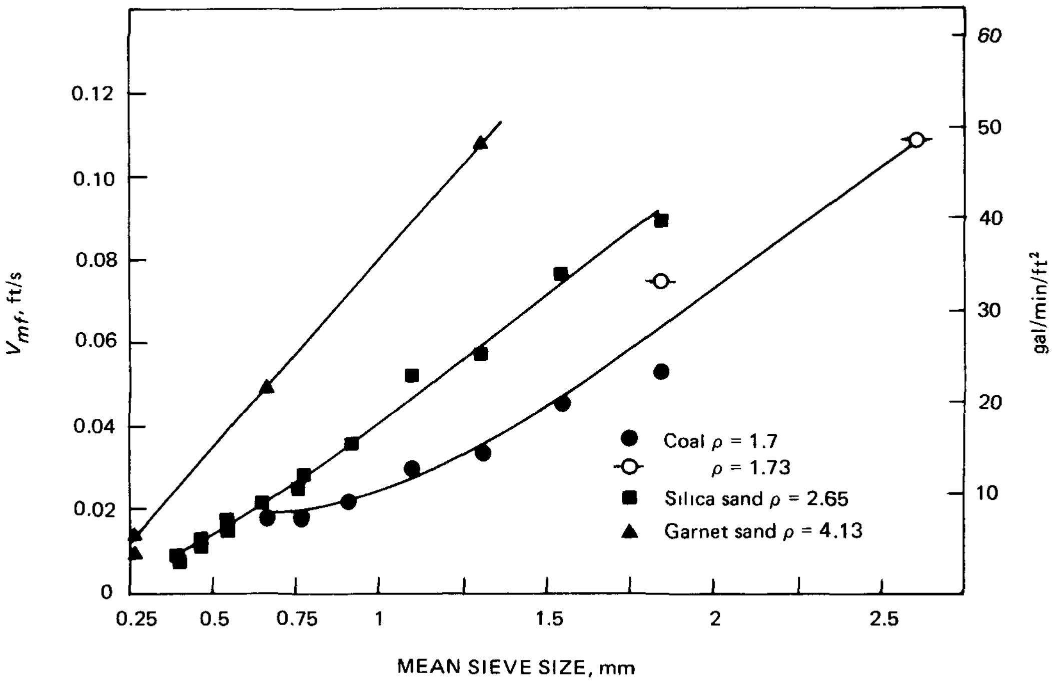

Figure II-6. Minimum fluidization velocity (V_{mf}) needed to achieve 10 percent bed expansion at 25° C.

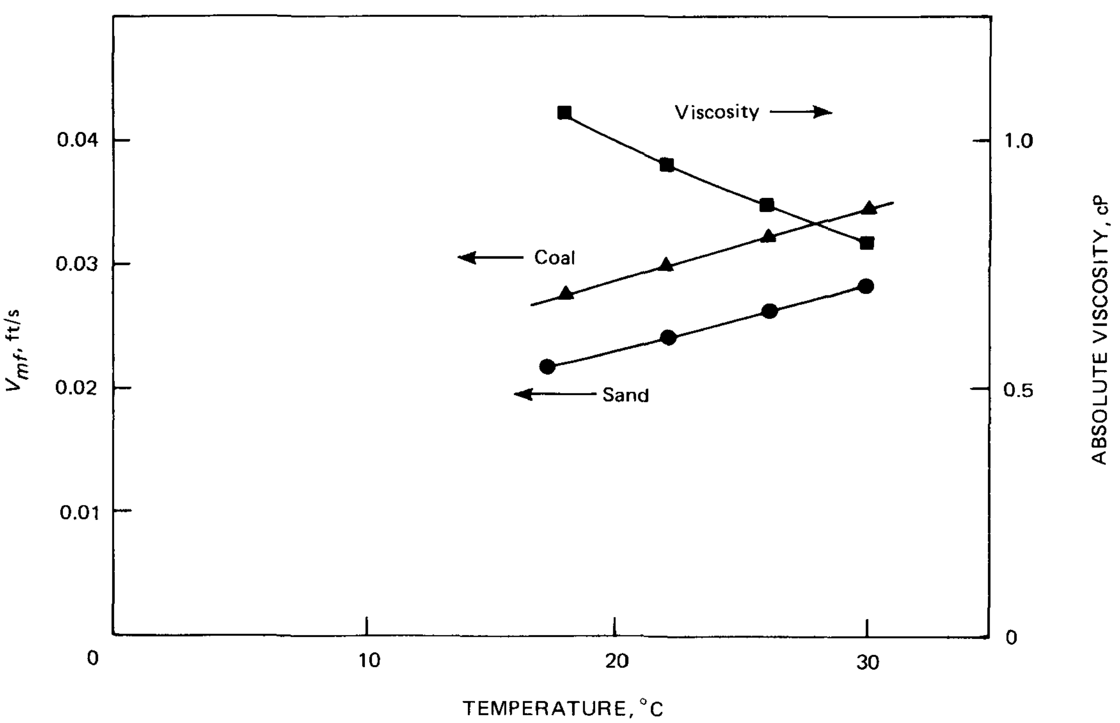

Figure II-7. Effect of water temperature on V_{mf} of sand and coal, and on absolute viscosity of water.

Table II-1.—*Size range of unisized media*

Geometric mean size, mm	Sieve opening		U.S. sieve No.	
	Passing, mm	Retained, mm	Passing	Retained
0.46	0.50	0.42	35	40
.55	.60	.50	30	35
.65	.71	.60	25	30
.77	.84	.71	20	25
.92	1.00	.84	18	20
1.09	1.19	1.00	16	18
1.30	1.41	1.19	14	16
1.54	1.68	1.41	12	14
1.84	2.00	1.68	10	12

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.[12] 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. After selecting the 90-percent finer sand size and the effective size, the uniformity coefficient (or other specification) should be determined graphically to insure that the desired top and bottom media sizes can be achieved.

After the sand is installed in the filter, the media should be backwashed and skimmed one or two times to remove any unwanted, excessively fine material before installing the coal. This step is important 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.[13]

The backwash rate selected should be appropriate to the media specified at the maximum seasonal backwash water temperature anticipated. Use figure II-6 for the 90-percent finer sizes of the media and figure II-7 for the temperature adjustment.

Design example: Select the dual-media sizes (and backwash rate) to obtain a top coal size of 1.2 mm with maximum wash water temperature of 20° C.

Coal specification, −12 +16 mesh U.S. standard sieve, with 10 percent tolerance beyond either end of the specification (table II-1). Therefore, the 90-percent finer size is 1.68 mm and the 10-percent finer size is 1.19 mm.

Purchase excess coal depth of 3 inches to permit three backwashes and three skimmings of 1-inch depth to try to remove the fines smaller than 1.2 mm.

The 90-percent finer sand size with equal V_{mf} to the 90-percent finer coal would have a size of about 1.25 mm (fig. II-6).

The effective sand size should be about 0.55 mm to achieve a ratio of bottom coal size to top sand size of 3.

Assuming log normal size distribution for the sand, the 90-percent and 10-percent sand size can be plotted on log probability paper. The 60-percent finer size is found to be 0.9 mm and the uniformity coefficient is 1.63. Thus, a uniformity coefficient of less than 1.65 should be specified.

The required backwash rate from figure II-6 is 0.056 ft/s or 25 gal/min/ft^2 at 25° C.

Adjust backwash rate to 20° C using figure II-7. Backwash rate at 20° C = backwash rate at 25° C × (V_{mf} at 20° C/V_{mf} at 25° C). Backwash rate at 20° C = 25 × (0.9/1) = 22.5 gal/min/ft^2.

It should be emphasized that this design—while appropriate for direct tertiary filtration and for alum-treated secondary effluents—would not necessarily be best for all chemically pretreated wastewaters, or where polyelectrolytes are to be used as filter aids. In the latter case, a coarser top size may be desired (1.2-1.5 mm).

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[14,15] established that, for filtration of trickling-filter-plant effluent, a depth of at least 15 inches of 1.84-mm 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:

- Anthracite coal, 15 inches minimum to 20 inches
- Silica sand, 12 inches minimum to 15 inches

BACKWASHING REQUIREMENTS

The backwashing of deep granular filters is one of the neglected areas of filtration research. At a time when such filters are assuming an increasingly important role in wastewater filtration, many questions concerning backwashing remain unanswered. Two factors prevent the ready transfer of backwash technology from water-treatment practice to wastewater-treatment practice.

- Wastewater filters receiving secondary or tertiary treated wastewaters receive heavier solids loads, and these solids adhere more tenaceously to the filter media.
- Some wastewater filters are being designed using new filter media sizes and size gradations.

It has been accepted generally that auxiliary scour devices are essential to obtain adequate cleaning of wastewater filters. Before discussing the benefits of air scour or surface wash, however, the weaknesses of backwashing by the upward flow of water alone and some of the conclusions related to optimum backwash in this manner should be reviewed.[16,17]

- The cleaning of granular filters by the upward flow of water alone to fluidize the filter bed is inherently a weak cleaning method, because particle collisions do not occur in a fluidized bed. Therefore, abrasion between the filter grains is negligible.
- The cleaning that results in a water-fluidized bed is due to the hydrodynamic shear at the water-filter grain interfaces. A simple mathematical model has been developed to calculate

the porosity at which maximum hydrodynamic shear occurs in a fluidized bed. This porosity is between 0.68 to 0.71 for sand sizes normally used in filtration. Optimum cleaning of filter media at this porosity has been demonstrated experimentally.

- When backwashing by water fluidization alone, a slight economy in total wash water used results from expanding the bed to the optimum porosity. Lower wash rates (anywhere above the rate for minimum fluidization) will result in nearly the same terminal wash water turbidity, but proportionately longer backwash times will be required to reach this terminal wash water turbidity level. Therefore, no economy of total backwash water use is achieved by low backwash rates.

Owing to the inherent weakness of water backwashing, auxiliary means of improving filter-bed cleaning are generally desirable. A study of the benefits of air scour or surface wash has been partly completed.[16,17] Two phases of the study are complete and the third is underway.

Laboratory, pilot, and plant-scale studies were made to evaluate the relative effectiveness of three methods of backwashing filters used for the filtration of various types of waters. Typical potable water and wastewater solids were filtered in the studies. The methods of backwashing studied included

- Water fluidization only
- Air scour followed by water fluidization
- Surface wash before and during water fluidization

The following conclusions were reported:[13]

- The wastewaters filtered in the study caused more difficult backwashing problems than the potable waters.

- Filtration of secondary effluent from a trickling-filter plant gave the most difficult backwashing problems. None of the three cleaning methods was able to maintain the filter beds completely free of mud balls or surface-cracking problems during 10 weeks of experimental operation. However, both air scour and surface wash resulted in substantially fewer mud-ball accumulations. It should be emphasized that no significant differences in filtrate quality were observed between the air-scoured and surface-washed filters. These minor accumulations of mud balls may not be a serious detriment; full-scale tertiary filters have been in service in England for over 10 years (and in the United States for a lesser period) without reported process failure resulting from this problem.

- Filtration of secondary effluent that had been treated with alum for phosphorus reduction provided the second most difficult filter to backwash. Water-fluidization backwash alone resulted in heavy mud-ball accumulations and large surface cracks during the filter cycles. Air scour could eliminate mud balls, but could not eliminate completely the surface-cracking problem. Studies with an auxiliary surface wash were not of sufficient duration to present conclusions on this cleaning method.

- The most powerful filter-cleaning method used was a concurrent wash with air and water above fluidization velocities, followed by a normal air scour and subsequent water-fluidization wash. By this method it was possible to eliminate accumulated mud balls that the other three methods had not been able to eliminate in routine operation.

- Heavy mud-ball accumulations are undesirable because they contribute to higher initial headlosses and higher headlosses during the filter cycle. They may also lower filtrate quality in some cases.

- Filter cracking, which is a sign of compressible coatings on the filter media, allows deeper penetration of solids into the filter and may lower filtrate quality. The cracking and deeper penetration of solids reduces the rate of headloss development in the surface layer, but increases it in the deeper layers of the filter.

The study leaves unanswered some important questions.

- What is the minimum degree of bed expansion necessary to insure adequate cleaning of filter media over a long period of service?

- Can coarser media filters be successfully cleaned over a long period by air scour followed by (or concurrent with) water backwash at rates below the level required to fluidize the bed?

Several manufacturers are promoting wastewater filters with various media combinations and sizes, using air-scour and water-backwash rates well below the fluidization velocity. In view of the difficulty of cleaning more conventionally designed wastewater filters,[13] the design of filters using subfluidization water backwash is subject to serious question. It is anticipated that large portions of such beds will become agglomerated and inactive in the filtration function. These agglomerates will contribute to higher interstitial velocities, higher rates of headloss development, and may lower filtrate quality. Further work is needed to verify or refute these concerns.

Other concerns need to be emphasized with regard to the design, construction, and operation of air-scour facilities used in filter cleaning. The use of fine-media-retaining strainers to eliminate the need for supporting gravel may cause difficulty. The strainers may gradually or suddenly clog, resulting in excessive pressure drop across the underdrain plate or slab causing excessive uplift pressures and potential uplift failure. Such failures have occurred.[13] 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.

These concerns about air-scour systems suggest that the air should be introduced to the filter through a separate pipe manifold above the underdrain water supply orifices. A typical water underdrain system with graded gravel should be used for the water backwash. If the air is introduced below the gravel, care must be taken when beginning the water backwash so that the gravel is not upset by the sudden introduction of water. The water backwash valve should be opened just a crack, or a special small valve should be opened first, until all the air has been expelled from the gravel before the water valve is opened gradually to achieve a full backwash rate. On the other hand, if the air is introduced above the gravel layer, provisions must be made to prevent entrance of the filter media to the air manifold through the air supply orifices.

The following tentative design recommendations are based on the foregoing observations, studies, and concerns:

Air scour and surface wash (possibly subsurface wash) are essential auxiliary cleaning devices and should be provided for all wastewater filters. The air-scour rate currently common in U.S. practice is 3 to 5 scfm[a]/ft^2 for a duration 3 to 5 minutes. Most of the beneficial action of air scour

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[a]Standard cubic feet per minute at 70° F and 14.7 psia.

appears to occur in the first 2 minutes, so extension of air scour beyond 2 minutes is of questionable value (based on visual observations). However, operational flexibility in the period of air scour between, let us say, 2 and 10 minutes could be provided easily, so the operator could select the period he deems most appropriate.

If an auxiliary air scour is provided, the capability for simultaneous air and water backwash should also be provided. This technique requires provisions to allow for rapid draining 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 just above minimum fluidization velocity to extend the time duration of that action to the maximum.

Water backwash capability should be adequate to fluidize all the media. The coarser media sizes and warmest expected water temperatures will dictate the minimum backwash rates required. The 90-percent finer sizes of coal, sand, and garnet are suggested as the practical sizes to be used to select the required backwash rate.

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 requirement. No economy of total wash water use is achieved by adopting lower backwash rates (above fluidization), because the length of backwash must be increased proportionately.

If an auxiliary surface wash is provided, it may prove necessary to use a subsurface washer to attack the mud balls 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.

At this time, it is not possible to report whether the air scour or surface (subsurface) wash is most advantageous.

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

- Where do we get the water for backwashing?
- What do we do with the dirty backwash water?

The best source of water for backwashing will be the effluent from the tertiary 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 once. This volume will approximate 75-100 gal/ft^2 of filter for the area of three filters.

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 slug load on the primary and 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 during the low-flow period of the day. Thus, the dirty-backwash-water storage tank should have sufficient storage capacity to store all the backwash water generated during the peak 12-hour flow period during the day, and sufficient discharge capacity to empty this volume plus the additional volume generated during the low-flow period of, let us say, 8-10 hours.

The need for backwashing filters and the return of the backwash water to the head of the plant increases the hydraulic load on both the filters and the rest of the plant. This effect is accentuated at the lower filtration rates and with the shorter filter runs, as shown in table II-2.

The data in table II-2 were generated by assuming that the daily backwash is returned to the plant inflow at a uniform rate over a 24-hour period. For example,

Backwashes per day (6-hour runs) = 4

Downtime per backwash = 20 min

Actual filtration time (1,440 - 4 X 20) = 1,360 min

Nominal filtration rate, Q = 4 gal/min/ft^2

Backwash water used = 150 X 4 = 600 gal

Filtered water produced = 1,360 min X 4 gal/min/ft^2 = 5,440 gal

Needed filtration rate = 5,440 gal + 600 gal/1,360 = 4.44 gal/min/ft^2

Backwash water as percent of Q = q/Q X 100 = 0.44/4.00 X 100 = 11.0%

where

Q = plant wastewater flow rate, gallons per minute per square foot of filter

q = average backwash-water flow rate, gallons per minute per square foot of filter equalized over 24 hours.

and backwash is 15 gal/min/ft^2 for 10 minutes, or 150 gal/ft^2. (See also fig. II-8.)

Table II-2.—*Gross rate of filtration* (Q + q) *per unit filter area as related to net filtration rate* (Q) *and the number of backwash cycles per day*

Net daily filtration rate, gal/min/ft^2	Gross filtration rate (Q + q)/backwash water as a percent of Q (q X 100/Q), number of backwash cycles per day				
	6	4	2	1	0.5
1	1.68/68.0	1.44/44.0	1.21/21.4	1.11/10.5	1.05/5.2
2	2.68/34.0	2.44/22.0	2.21/10.7	2.11/5.3	2.05/2.6
3	3.68/22.7	3.44/14.7	3.21/7.1	3.11/3.5	3.05/1.7
4	4.68/17.0	4.44/11.0	4.21/5.4	4.11/2.6	4.05/1.3
6	6.68/10.4	6.44/7.3	6.21/3.6	6.11/1.8	6.05/0.9
8	8.68/8.5	8.44/5.5	8.21/2.7	8.11/1.3	8.05/0.7

Note.—See figure II-8.

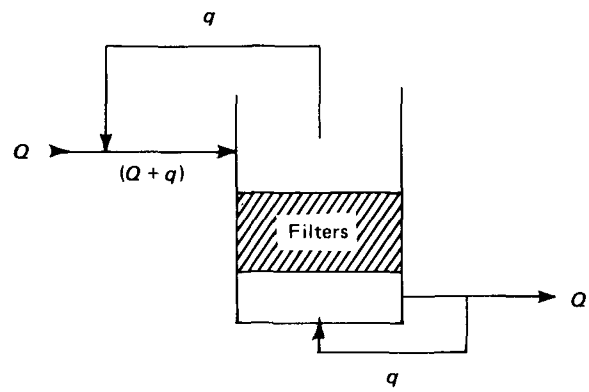

Figure II-8. Diagram showing gross rate of filtration.
(See table II-2.)

The results in table II-2 indicate that the actual filtration rate needed to filter the plant wastewater must be increased as the runs become shorter and the nominal filtration rate becomes smaller.

HEADLOSS DEVELOPMENT

Figure II-9 shows several examples of headloss development during solids separation by filtration. Granular 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. II-9a). 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 II-9b 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-sized 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.

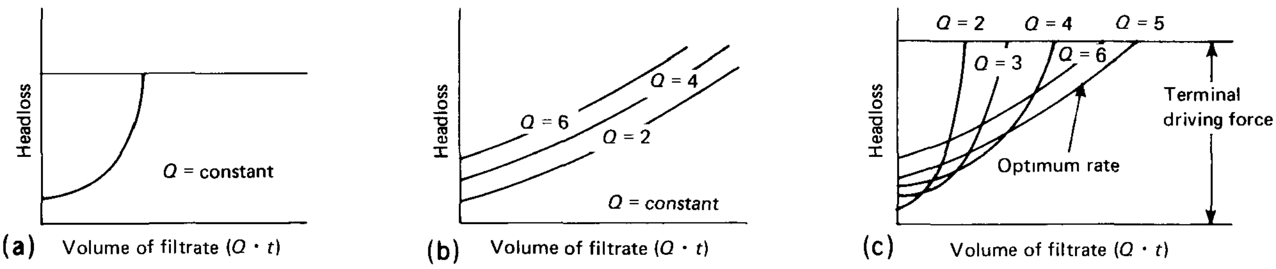

Figure II-9. Headloss development during filtration: (a) surface removal of compressible solids; (b) depth removal of suspended solids; (c) depth removal of suspended solids with surface cake.

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 headlosses (e.g., fig. II-9c 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. II-9c). The rate may become high enough to prevent surface cake formation and the headloss will then be controlled only by depth filtration. In figure II-9c, 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 II-9c.[15] Nearly linear headloss curves have been observed in filtration of alum-treated secondary effluent.[13]

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.

SELECTION OF FILTRATION RATE AND TERMINAL HEADLOSS

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 Baumann[2] 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 so as to result in failure of the filtration process by solids breakthrough (fig. II-1). 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.

A recent study indicates that pressure drops of as much as 30 feet of water could be used in plain filtration of trickling-filter effluents[14,15] and in activated-sludge effluents[11] 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 6-8 hours to avoid excessive backwash water use, but less than about 36-48 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 excessive manpower costs if short filter runs occur?

- 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.

- 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. Figure II-10 shows the relationship between run length, filtration rate, and suspended solids levels observed in pilot-plant studies with trickling-filter-plant effluents.[14,15] An analysis of other available data for direct filtration of secondary effluents—both from activated-sludge and trickling-filter plants—indicates that the dominant factor controlling headloss development is the suspended solids capture (influent minus effluent suspended solids) as a function of time. Media size does not have a substantial effect on solids capture, provided the top size is greater than about 1 mm. Filtration rate has only a minor effect, at least in the range from 2 to 6 gal/min/ft^2.[14,15,18] Data on solids capture per unit increase in headloss are presented in table II-3.

An extensive survey of the literature on wastewater filtration[15] was consulted in compiling table II-3. Unfortunately, not all references presented sufficient information to calculate the solids capture. Therefore, particularly for trickling-filter-plant final effluent filtration, the data presented are from only two locations. This sample is limited and additional data are needed, especially on trickling-filter plants. There is remarkable agreement between the results from three studies at Ames, Iowa. Until additional data are collected at other plants, an average solids capture value of 0.07

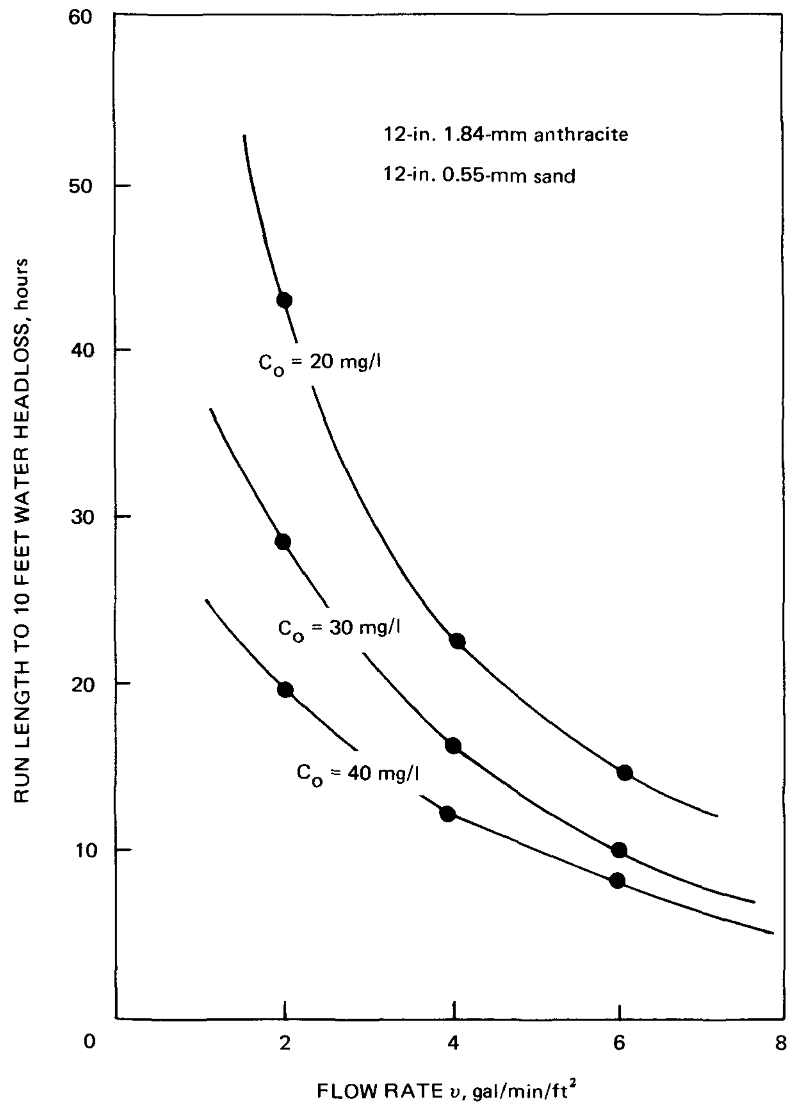

Figure II-10. Run length observed with trickling-filter-plant effluent at various filtration rates and suspended solids levels.

lb/ft^2 headloss increase is suggested for the design of filters for filtration of trickling-filter final effluent. 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

Table II-3.—*Solids capture per foot of headloss increase in direct filtration of secondary effluents*

Secondary effluent type[1]	Filtration rate, gal/min/ft^2	Mode of operation[2]	Top media size, 10% finer, mm	Solids capture, lb/ft^2/ft headloss increase	Reference
TF	3.33	-	0.85	0.052	19
TF	2	-	.85	.074	20
TF	3	-	.59	.035	21
TF	2	C	1.84	.064	15
TF	4	C	1.84	.070	15
TF	6	C	1.84	.079	15
TF	2	C	.42	.065	18
TF	4	C	.42	.070	18
TF	6	C	.42	.073	18
TF	7	C	.92	.078	([3])
AS	16	C	1.78	.35	11
AS	24	C	1.78	.093	11
AS	32	C	1.78	.093	11
AS	16	D	1.78	.23	11
AS	22.2	D	1.78	.21	11
AS	27.6	D	1.78	.12	11
AS	2-5	C	Upflow	.26	22
AS	5.1	C	1.08	.24	9
AS	5.1	C	1.45	.34	9

[1]TF = trickling-filter-plant final effluent; AS = activated-sludge-plant final effluent.
[2]C = constant rate; D = declining rate.
[3]Gary Sejkora, private communication, 1973.

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 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.

The effect of the number of backwashes of each filter per day on the gross filter rate, or the percent of recycled flow due to backwash, can be presented in a number of ways. Table II-2 is one method of presentation.

SUMMARY

The key questions involved in the proper design of granular filters for the filtration of secondary effluents 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 (e.g., a coarse-to-fine filtration system) is essential to obtain reasonable filter run lengths. The filter media on the influent side should be at least 1 to 1.2 mm.

- The backwash-water flow rate should be large enough to fluidize the coarser-sized grains of each component of the filter media. More uniform media sizes will reduce the backwash flow rate required and are thus desired, even though the cost of the media will be increased.

- Auxiliary agitation of the media is essential to proper backwashing. Either air scour or surface (and possibly subsurface) washers should be installed.

- 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.

- The filtration rate and terminal headloss should be selected to achieve a minimum filter run length of 6 to 8 hours during peak-load conditions. This requirement will mean an average run length of about 24 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.

- 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.

This publication does not attempt to present detailed discussion of all elements of filter design that have been well established in water-treatment practice and are presented in various text books.[23] Rather, the emphasis is on the differences between water-treatment practice and wastewater-treatment practice, which must be considered for successful design of wastewater filters.

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METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents
Length	metre	m	*Basic SI unit*	39.37 in.=3.28 ft= 1.09 yd
	kilometre	km		0.62 mi
	millimetre	mm		0.03937 in.
	micrometre	µm.		3.937 X 10^{-3}=10^{3}A
Area	square metre	m^2		10.764 sq ft = 1.196 sq yd
	square kilometre	km^2		6.384 sq mi = 247 acres
	square millimetre	mm^2		0.00155 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 metre	m^3		35.314 cu ft = 1.3079 cu yd
	litre	l	The litre is now recognized as the special name for the cubic decimetre.	1.057 qt = 0.264 gal = 0.81 X 10^{-4} acre-ft
Mass	kilogram	kg	*Basic SI unit*	2.205 lb
	gram	g		0.035 oz = 15.43 gr
	milligram	mg		0.01543 gr
	tonne or megagram	t Mg	1 tonne = 1 000 kg 1 Mg = 1 000 kg	0.984 ton (long) = 1.1023 ton (short)
Time	second	s	*Basic SI unit*	
	day	d	Neither the day nor the year is an SI unit but both are important.	
	year	year		
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 in a mass of 1 kg.	0.22481 lb (weight) = 7.233 poundals
Moment or torque	newton metre	N·m	The metre is measured perpendicular to the line of action of the force N. Not a joule.	0.7375 ft-lbf
Stress	pascal	Pa		0.02089 lbf/sq ft
	kilopascal	kPa		0.14465 lbf/sq in

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents
Velocity linear	metre per second	m/s		3.28 fps
	millimetre per second	mm/s		0.00328 fps
	kilometres per second	km/s		2.230 mph
angular	radians per second	rad/s		
Flow (volumetric)	cubic metre per second	m^3/s	Commonly called the cumec	15,850 gpm = 2.120 cfm
	litre per second	l/s		15.85 gpm
Viscosity	pascal second	Pa·s		0.00672 poundals/sq ft
Pressure	newton per square metre or pascal	N/m^2 Pa		0.000145 lb/sq in
	kilometre per square metre or kilopascal	kN/m^2 kPa		0.145 lb/sq in.
	bar	bar		14.5 b/sq in.
Temperature	Kelvin	K	*Basic SI unit*	
	degree Celsius	C	The Kelvin and Celsius degrees are identical. The use of the Celsius scale is recommended as it is the former centigrade scale.	$\frac{5F}{9} - 17.77$
Work, energy, quantity of heat	joule	J	1 joule = 1 N·m where metres are measured along the line of action of force N.	2.778 X 10^{-7} kw hr = 3.725 X 10^{-7} hp-hr = 0.73756 ft-lb = 9.48 X 10^{-4} Btu
	kilojoule	kJ		2.778 kw-hr
Power	watt	W	1 watt = 1 J/s	
	kilowatt	kW		
	joule per second	J/s		

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents
Precipitation, run-off, evaporation	millimetre	mm	For meteorological purposes it may be convenient to measure precipitation in terms of mass/unit area (kg/m^3). 1 mm of rain = 1 kg/m^2	
River flow	cubic metre per second	m^3/s	Commonly called the cumec	35.314 cfs
Flow in pipes, conduits, channels, over weirs, pumping	cubic metre per second	m^3/s		
	litre per second	l/s		15.85 gpm
Discharges or abstractions, yields	cubic metre per day	m^3/d	1 l/s = 86.4 m^3/d	1.83 X 10^{-3} gpm
	cubic metre per year	m^3/year		
Usage of water	litre per person per day	l/person day		0.264 gcpd
Density	kilogram per cubic metre	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.	0.0624 lb/cu ft

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents
Concentration	milligram per litre	mg/t		1 ppm
BOD loading	kilogram per cubic metre per day	kg/m^3d		0.0624 lb/cu-ft day
Hydraulic load per unit area; e.g. filtration rates	cubic metre per square metre per day	m^3/m^2d	If this is converted to a velocity, it should be expressed in mm/s (1 mm/s = 86.4 m^3/m^2 day).	3.28 cu ft/sq ft
Hydraulic load per unit volume; e.g., biological filters, lagoons	cubic metre per cubic metre per day	m^3/m^3d		
Air supply	cubic metre or litre of free air per second	m^3/s l/s		
Pipes diameter	millimetre	mm		0.03937 in.
length	metre	m		39.37 in. = 3.28 ft
Optical units	lumen per square metre	lumen/m^2		0.092 ft candle/sq ft

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