4. Filter design and construction

The essential parts of a slow sand filter are:

1. the supernatant water reservoir,
2. the filter-bed,
3. the filter bottom and under-drainage system,
4. the filter box, containing (1), (2), and (3) above,
5. the filter control system.

In addition, a filter covering structure may be required in certain circumstances.

Before designing these constituent parts it will be necessary to consider the treatment plant as a whole, to decide on the methods of delivery of the raw water and collection of the treated effluent, to determine whether and to what extent pretreatment is called for, and to select the number and size of the filters required. Consideration should also be given to the likelihood of future extensions being required so that provision for additional filters may be made in the original layout. It should be possible for such extensions to be made without interrupting the working of the original plant. Thus, access for construction plant and materials to future sites should be allowed for, and pipework should be of adequate capacity to deal with future loading and should be provided with suitable T-junctons or blank ends to facilitate the connexion of the new filters when built.

The first decisions will concern the quantity of water to be treated—that is to say, the average quantity per day, the average quantity per hour, and the peak flows.

The average quantity per day that must be provided to a public water supply system comprises:

(1) the number of consumers multiplied by the average consumption per head,
(2) water for industrial, commercial, or other special purposes,
(3) wastage.

These items will be considered in reverse order.

Wastage. This must obviously be kept to a minimum by ensuring that the mains, fittings, service pipes, and other parts of the distribution system
are watertight. However, even with new installations that have been subject to the most careful supervision during construction and to regular servicing afterwards, some wastage inevitably occurs. In older systems, where earth movements and corrosion have affected mains, or where inadequate precautions are taken against dripping taps, leaking washers, or careless use by consumers (in areas where consumption is not metered), wastage may account for an appreciable part of the daily demand. Actual figures for water “unaccounted for” amounting to 25–30% of the total quantity treated are not unknown. Such losses constitute an expensive drain on the financial resources of a water undertaking and an unforeseen load on the treatment plant. The designer will be wise to include an allowance for wastage depending on the complexities of the system and on his own judgement of the quality of care and supervision that will go into the construction and maintenance of the system as a whole.

Special purposes. Where industrial premises are to be served from the public supply it will be necessary to estimate the rate of withdrawal on a daily basis under the worst possible conditions. It is sometimes possible to smooth a fluctuating load by making arrangements with the industrial consumer; for example, a plant working 5 days a week may be required to install a storage tank and take only a specified maximum quantity daily. Commercial consumption may similarly fluctuate. In a small installation, especially, such demands as the washing down of a market should be anticipated if stores of treated water are not to be depleted. In larger communities the various demands may well smooth themselves into a fairly average rate of withdrawal, but they should be assessed in advance and provision made in the capacity of the treatment plant. Allowance should also be made for sudden heavy demands for water such as occur when fire hydrants are used or the streets are cleansed by the municipal authorities.

Domestic use. The demand for domestic water is usually by far the largest demand on a supply authority, and it consequently needs the greatest study to ensure that the treatment works will have an appropriate capacity. Two factors have to be considered—the number of consumers and the daily requirements of each. The first of these should be relatively easy, but allowance must be made for future population growth over the design period, say 7–10 years, since it will usually be a most unpopular move politically if extensions are called for soon after the works are completed. On the other hand, a design that attempts to accommodate an uncertain population increase that continues into the distant future may be economically inexcusable.

The average demand per head of the population is more difficult to forecast unless experience in similar circumstances is available. In a community having piped water for the first time, individual consumption may start slowly, but familiarity with piped water will soon stimulate
demand. In an industrialized society the number of domestic fittings may be expected to increase, and once the supply is shown to be ample and reliable such appliances as washing machines, air conditioning units, and hoses for car washing and garden watering will be connected. In developing countries, street standpipes will give way to individual house connexion, waterborne sewerage may be installed, fittings such as baths and lavatories will proliferate, and mains water will be used for the watering of domestic animals and garden crops.

**Hourly consumption.** It has been shown in earlier chapters that a slow sand filter is at its most efficient when a constant filtration rate is maintained, i.e., when the daily quantity to be treated is equally divided among the 24 hours. Continuous operation is no problem as far as the filter itself is concerned, but the pumping schedules may call for three-shift working (unless automatic devices such as float- or electrode-operated pumps are installed) and the raw water and treated water reservoirs will have to be adapted to continuous operation. Some additional expenditure may be involved in such items as external lighting for the safety of night shifts.

On the other hand, the hourly quantity to be treated will be correspondingly reduced and the area of the filter-beds will be smaller. Except perhaps for the very smallest installations, the capital cost of a continuously working plant is likely to compare very favourably with one designed for intermittent operation.

It should be made clear that the term “intermittent operation” refers to intermittent filter operation and not to interrupted delivery to consumers. Delivery through the public mains must always be continuous; the importance of this cannot be overstressed. Intermittent supply to consumers can never be justified, because it carries grave health hazards and in the long run saves neither water nor costs. Interrupted supplies to consumers must always be regarded as a serious reflection upon the designers or the operators of a water system.

**Peak flows.** Peak flows may be of several types daily, weekly, seasonal, or occasional. Daily peaks are inevitable and are accommodated by having an adequate storage capacity of treated water. To a great extent weekly peaks (e.g., the Monday-morning wash-day customary in many communities) may be accommodated in the same way, as may certain occasional peaks such as might arise in an emergency such as a fire or a burst water main.

Seasonal fluctuations are often more difficult to cater for. In most communities maximum demand takes place at the worst time of the year from the waterworks operator’s point of view, i.e., in the hottest season when the source water may be at its scarcest. In northern latitudes another seasonal peak may occur during heavy frost, when wastage from burst pipes and fittings and from taps deliberately left dripping to avoid freezing coincides with reduced biological activity in the filters. To some extent such
peak demands may be met by forethought in operation—for instance, by arranging that filter cleaning is carried out in advance of a period of excessive demand so that the full filtration area is available during a seasonal emergency.

*Total filter area.* After taking all the above factors into account, the designer will be able to determine his first criterion—the hourly quantity of water to be treated, $Q$. The other key factor is the filtration rate $v$, and the method of arriving at this has been discussed in previous chapters. The total filtration area $A$ must clearly be equal to or greater than $Q/v$. The next step will be to choose the number, shape, and dimensions of the filters to provide this total filtration area.

Filters need periodic cleaning, which, when carried out by hand, should be completed in 1–2 days. However, it is usually necessary to allow a further period for ripening, if the effluent quality is to reach its former standard. To avoid overloading the remaining filters, with consequent clogging and reduced flow rates, at least one additional filter (and in large installations two or more filters) should be provided as a reserve. Thus the number of filters $n$ may be obtained by dividing the total filtration area by the area of each filter $A$ and adding the chosen number of reserve filters.

Certain further factors will influence the relationship between the number of filters and the surface area of each.

1. The weakest part of a filter (from the point of view of effluent quality) is the edge of the filter-bed, where the raw water may leak past the sand if care is not taken in design and operation. In order to minimize this weakness, filters should not be too small. The minimum workable size is usually considered to be 100 m$^2$, and filters of twice this area are to be preferred.

2. Larger units have a lower initial cost per square metre than do smaller units.

3. Over a certain size the risk of cracks due to subsidence or temperature stresses increases greatly, the dangers depending on such factors as the structure of the subsoil and climatic conditions. A maximum size, which might be between 2000 m$^2$ and 5000 m$^2$, should be decided upon.

4. No treatment plant for public supply, however small, should have less than two filters, and four is a much more suitable minimum number.

5. More flexible operation is possible with a larger number of filters, and smaller filters are more easily cleaned, with consequent saving in the number of labourers required.

Only after all these factors have been weighed can the final decision be made, but a useful first approximation can be made from the formula:

$$n = \frac{1}{4} \sqrt{Q}$$

in which $n$ is never less than 2 and $Q$ is expressed in m$^3$/h.
With regard to the shape and spacing of individual filters, the following considerations have to be borne in mind.

(1) Easy access to every filter is desirable to facilitate cleaning by hand, particularly when the removal of large quantities of algae may be expected. (2) The construction of filters of random size to make the best use of available land may effect initial economies, but if mechanical cleaning is contemplated it will be preferable to make the beds of equal size and rectangular in shape (Fig. 7).

(3) If filters are to be covered, the quantity of material to be removed at each cleaning will be greatly reduced, since little or no algae will be present. A few access points for cleaning will suffice, and filters may be built adjacent to each other. Rectangular filter boxes of standard size will then permit the use of prefabricated construction elements.

(4) For maximum economy, the position of influent and effluent pipes should be planned at an early stage.

**Supernatant water reservoir**

Having decided on the number and surface area of the filters to be constructed, the designer will have to consider the component parts of each.
The supernatant water reservoir consists essentially of an upward extension of the walls of the filter box from the sand-bed surface. The reservoir so formed serves two purposes: it provides a waiting period of some hours for the raw water, during which sedimentation, particle agglomeration, and oxidation occur, and it provides a head of water sufficient to overcome the resistance of the filter-bed, thereby inducing downward flow through the filter.

It is this second factor that determines the vertical dimension of the reservoir. Since, as has been shown, the resistance varies from a minimum $H_0$ when the bed has been newly cleaned to a maximum $H_{max}$ at the end of the filter run, the water level in the reservoir may theoretically be allowed to vary within these limits, but in practice it is preferable to maintain a constant depth of water, equal to or greater than $H_{max}$, by imposing an artificial resistance with the aid of a regulating valve on the effluent pipe. A constant depth reduces the dangers of disturbing the *schmutzdecke* as it forms, enables floating impurities to be removed from the reservoir through fixed scum outlets, and prevents the deep penetration of sunlight, which might encourage the growth of rooted aquatic plants in the filter surface.

The depth of water in the reservoir will be determined according to the maximum resistance anticipated. In practice, a head of between 1.0 m and 1.5 m is usually selected. The figure may, exceptionally, be as high as 2.0 m, but rarely more than this.

Above the water level in the supernatant reservoir the walls must be carried up to form a freeboard about 20-30 cm high.

**Filter-bed**

The medium through which the water is passed is normally a selected sand, though other granular substances such as crushed coral or burnt rice husk have been used in the absence of suitable sand. As the amounts required are large it is usual to employ ungraded material as excavated from natural deposits. In rapid sand filtration ungraded media would not be satisfactory, since the backwashing process would result in stratification of the material. Slow filtration does not suffer from this disadvantage, and the relatively heavy expense of careful grading is avoided.

Some degree of uniformity is, however, desirable in order to ensure that pore sizes are reasonably regular and that there is sufficient porosity. Accordingly sand having a coefficient of uniformity (see page 40) of less than 3 should always be chosen. A coefficient of less than 2 is preferable, but there is little advantage, in terms of porosity and permeability, in sand having a coefficient below 1.5 if additional cost is thereby incurred.

Filter media should be composed of hard and durable grains, preferably rounded, free from clay, loam, and organic matter. If necessary the sand
should be washed—a process that will also remove the finest grains, thus lowering the coefficient of uniformity and raising the average particle diameter. To prevent cavities developing in the filter-bed through attack by water that has a high content of carbon dioxide, the sand should not contain more than 2% of calcium and magnesium, calculated as carbonates.

Ideally the effective diameter of the sand, \( d_{50} \), should be just small enough to ensure a good quality effluent and to prevent penetration of clogging matter to such a depth that it cannot be removed by surface scraping. This effective diameter usually lies in the range 0.15–0.35 mm and is determined by experiment. Both finer and coarser materials have been found to work satisfactorily in practice, and the final selection will usually have to depend on the locally available materials. It is possible to combine two or more types of stock sand to bring the effective diameter of the mixture closer to the ideal (Fig. 8). Mixing must be carried out very thoroughly, preferably in a concrete mixer. The resulting medium will have a higher coefficient of uniformity than either of its components.

The factors governing the thickness of the filter-bed have been discussed in earlier chapters. Since much depends on local circumstances, the thickness can be more effectively determined by experiment than by any other means. In general, three important considerations must be kept in mind:

1. Immediately below the filter-skin lies the zone in which purifying bacteria abound. The thickness of this zone is usually between 0.3 m and 0.4 m, the higher figure being appropriate when the sand grains are relatively coarse and the filtration rate reasonably high.
(2) Below this depth, chemical reactions take place in what may be described as the “mineral oxidation” zone, within which the organic materials liberated by the bacterial life-cycle in the upper sand layer are chemically degraded. The thickness of this zone may be between 0.4 m and 0.5 m, the higher figure applying when the raw water has a high organic content. Therefore under no circumstances should the total bed thickness, (1) plus (2), be less than 0.7 m.

(3) Except when hydraulic processes are used, a filter is cleaned by skimming off the top 1–2 cm of material. This material is not immediately replaced, and on restarting the filter the whole filtration process takes place at the same depth below the new surface, i.e., 1–2 cm lower in the same bed. Only after the filter has been operating in this way for some years will the bed surface be brought back to its former level by the addition of new material. Provision must therefore be made in the original thickness to allow for successive cleanings during this period. In a filter having an average run of 2 months between cleanings some 9–10 cm will be removed each year, and an allowance of an additional 0.5 m of thickness will allow for 5 years of operation before resanding becomes necessary.

These three considerations, taken together, lead to a total of 1.2–1.4 m for the filter-bed thickness to be provided initially. It may be somewhat less if the raw water is reasonably clear and filter runs are consequently longer than average.

When an effluent of particular high quality is desired, a layer of activated carbon is sometimes incorporated into the filter-bed. This layer, about 0.1 m in thickness, is usually placed near the bottom of the filter-bed, thus permitting it to adsorb any last traces of taste- and odour-producing substances that have passed through the filtration process. It normally has to be removed and replaced during resanding operations, by which time the carbon will have become saturated with the impurities. In a similar way a layer of crushed shells is sometimes included to correct the pH of a naturally aggressive water.

Under-drainage system

The under-drainage system, though unseen, plays an important part in the efficient operation of a filter. It serves the dual purpose of supporting the filter medium and of providing an unobstructed passage-way for the treated water to leave the underside of the filter. Once the filter bed has been laid, the under-drainage system cannot usually be inspected, cleaned, or repaired in any way without a major disturbance to the bed as a whole, and it is therefore important that it should be so designed and constructed that it cannot become choked by the entry of granular material
from above, that it does not become disturbed during the laying and spreading of the filter medium, and that at all times it collects the treated water evenly over the entire bed area so that all parts of the filter perform the same degree of work.

The simplest form of under-drainage consists of a system of main and lateral drains (Fig. 9). Although improved methods and materials have been introduced at various times to increase the efficiency of the system, the hydraulic principles remain much the same.

In the simple, piped system, the lateral drains consist of porous or perforated unglazed drainage tiles, glazed pipes laid with open joints, or perforated pipes of asbestos cement or polyvinylchloride, covered with layers of gravel of successively diminishing grain size to prevent the intrusion of filtering medium. In small filters the main drain may also be constructed of pipes, but in larger filters it is more commonly made of concrete, frequently recessed into the floor of the filter box.

The flow of water through such a system is accompanied by head losses due to friction, turbulence, and the conversion of static head into velocity head, resulting in a lowering of the piezometric level. The head of the filtered water is thus not constant, but varies from point to point over the area of the filter-bed, while the raw water head has the same value at all points of the area. The resulting variation in filtration rate cannot be entirely prevented but it can be minimized by good design, which should ensure that the differences in filtered water head remain small in relation to the total head loss encountered by the water in its downward percolation through the filter.

The maximum variation in the piezometric level of the filtered water in a typical under-drainage system of the kind shown in Fig. 9a is calculated in Annex 1. The total loss in the under-drainage system should not usually
exceed 10% of the resistance of the filter-bed when at its lowest (i.e., when the sand is clean and the bed is at its minimum thickness after repeated scrapings) so that the variation over the area of the filter may be kept within an acceptable limit. In the example given in Annex 1 the consequences of this constraint are examined in detail.

Fig. 10 shows a number of arrangements for the construction of the filter bottom, and Annex 2 gives the relevant calculations of loss in piezometric level for a filter bottom constructed with standard bricks, as shown in Fig. 10a.

Between the under-drainage system proper and the filter-bed itself there should be some layers of gravel to prevent the filtering medium from entering and choking the drainage waterways and to ensure a uniform abstraction of the filtered water when a limited number of drains are provided. This supporting gravel system is built up of various layers, ranging from fine at the top to coarse at the bottom, each layer composed of carefully graded grains (i.e., the 10% and 90% passing diameters should differ by a factor of not more than $\sqrt{2} = 1.41$).
The grains of the bottom layer of gravel should have an effective diameter of at least twice the size of the openings into the drainage system (e.g., the spacings between bricks or between open jointed pipes). Each successive layer should be graded so that its smaller \(d_{10}\) particle diameters are not more than four times smaller than those of the layer immediately below.

The uppermost layer of gravel must be selected with a \(d_{10}\) value more than four times greater than the \(d_{15}\) value of the coarsest filtration sand and less than four times greater than the \(d_{50}\) value of the finest filtration sand taken from natural deposits, which will vary in grain size from one spot to another. Hence, when supporting a filter medium for which \(d_{15} = 0.18\) mm and \(d_{50} = 0.3\) mm, the uppermost layer of the supporting gravel should have a \(d_{10}\) value between 0.7 mm \((4 \times 0.18)\) and 1.2 mm \((4 \times 0.3)\).

A gravel with a \(d_{10}\) value of 1.0 mm, and a \(d_{90}\) value of 1.4 mm \((d_{10} \times \sqrt{2})\) would therefore be suitable. The layer immediately below could have equivalent values of 4.0 mm and 5.6 mm, and the third layer 16 mm and 23 mm respectively. If the joints in the under-drainage system are 8 mm or less in width, these three layers will suffice.

Where it is too difficult or expensive to grade the gravel within a layer to the recommended ratio of \(1 : \sqrt{2}\), the requirement may be relaxed to a factor of \(1 : 2\), but in this case the layers should have their \(d_{10}\) values restricted to three times that of the layer above. In the example just quoted, four layers would then be required with gradings of 0.7–1.4 mm 2–4 mm, 6–12 mm, and 18–36 mm. Some engineers even prefer the \(d_{10}\) ratio between layers to be restricted to a factor of 2, which in the case quoted would call for six layers with gradings of 0.6–1.2 mm, 1.2–2.4 mm, 2.4–4.8 mm, 4.8–10 mm, 10–20 mm, and 20–40 mm.

Gravel for slow sand filters should conform to similar specifications to those applied to the filtering medium itself. The stones should be hard, preferably rounded, with a specific gravity of at least 2.5, and should be free from sand, clay, loam, dirt, and organic impurities of all kinds. If necessary the gravel should be washed to ensure its cleanliness. Not more than 5% by weight should be lost after immersion for 24 hours in warm concentrated hydrochloric acid. The thickness of each layer should be at least three times the diameter of the largest stones in its grading, but for practical purposes the minimum thickness of the layers is usually increased to 5–7 cm for the finer material and to 8–12 cm for the coarser gravel.

A number of such layers amounts to an appreciable depth of gravel, which has little effect on the quality of the effluent but involves expense in the construction of the filter box as well as in the provision of the gravel itself. To reduce this expense to the minimum, the openings into the under-drainage system are kept as small as possible, often by the use of porous concrete or similar material having fine openings. A floor of such material,
as illustrated in Fig. 10d, made of porous concrete using 5–10 mm aggregate, would require only one layer of 1.2–2.4 mm gravel to support a filter sand with an effective size of 0.3 mm or more.

Because of the high permeability of gravel, the resistance to downward flow is negligible. As an example, with a filtration rate of 0.5 m/h, the total resistance of the four-layer filter support referred to above would be made up as follows:

1. Top layer, 6 cm thick, 0.7–1.4 mm gravel, \( k = 25 \text{ m/h} \):
   
   \[
   \text{resistance} = 0.5 \times 0.06/25 \text{ m} = 1.20 \text{ mm}.
   \]

**FIG. 11. UNDER-DRAIN SYSTEM AND SUPPORTING GRAVEL LAYERS IN A SLOW FILTER**

Municipal Waterworks, Amsterdam
(2) Second layer, 6 cm thick, 2-4 mm gravel, \( k = 200 \) m/h:
resistance = \( 0.5 \times 0.06/200 \) m = 0.15 mm.

(3) Third layer, 6 cm thick, 6-12 mm gravel, \( k = 1800 \) m/h:
resistance = \( 0.5 \times 0.06/1800 \) m = 0.02 mm.

(4) Bottom layer, 12 cm thick, 18-36 mm gravel, \( k = 16000 \) m/h:
resistance = \( 0.5 \times 0.12/16000 \) m = 0.00 mm.
Total resistance for filter support 30 cm thick = 1.37 mm.

The resistance of a filter bottom made of bricks or concrete slabs is also extremely small. One constructed of slabs (Fig. 10b) with 15 cm between supports and 5 mm open joints has a resistance (when the flow rate is 0.5 m/h) of only 0.1 mm.

The gravel layers must be carefully placed (Fig. 11), since subsequent movement may disturb the filter sand above and either lead to chocking of the under-drainage system or produce cavities through which the water may pass with insufficient treatment. The larger sizes of gravel may even be hand packed.

Filter box

Most filter boxes are today built with vertical or near-vertical walls (in which case the horizontal dimensions will be those of the filter-bed surface) of a depth sufficient to accommodate the constituent parts just described. In the example given, the internal depth of the box would be the sum of the following depths, starting from the top:

<table>
<thead>
<tr>
<th>Depth Description</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeboard above supernatant water level</td>
<td>0.25</td>
</tr>
<tr>
<td>Supernatant water</td>
<td>1.25</td>
</tr>
<tr>
<td>Filter medium (initially)</td>
<td>1.25</td>
</tr>
<tr>
<td>Four-layer gravel support</td>
<td>0.30</td>
</tr>
<tr>
<td>Brick filter bottom</td>
<td>0.16</td>
</tr>
<tr>
<td>Total</td>
<td>3.21</td>
</tr>
</tbody>
</table>

Other conditions may call for depths of between 2.5 m and 4 m. The most common constructional materials are concrete for the floor and concrete (mass or reinforced), stone, or brick for the walls, according to the materials and skills most readily available. Occasionally, where economy in initial capital cost is of overriding importance, sloping walls are used, thus reducing the structural stresses by making fuller use of the bearing capacity of the ground. However, this technique calls for a greater land area and may present problems of aquatic growth at the edges and of watertightness. Reliance has sometimes been placed on puddled clay as a waterproof bottom and as a backing for brickwork or masonry sides.
(Fig. 12), but unless it is extremely well prepared and laid it can prove very inefficient.

In the past, filters have been constructed to curved plans and random areas to conform to the contours of the site or in irregular geometrical shapes to take maximum advantage of the available area (Fig. 13). Some have even been laid out to give the appearance of ornamental gardens.

Nowadays, except for the very smallest installations, when circular filters (occasionally encased in steel tanks) may be used (Fig. 14 and 15),
FIG. 14. CIRCULAR FILTERS FOR RURAL WATER SUPPLIES, SURINAM

FIG. 15. PILOT PLANT IN SURINAM USING SMALL CIRCULAR FILTERS

Note the shell filters for pH correction after filtration.
filters are generally rectangular in plan and all the units in a plant are of similar size.

Filter boxes should be watertight, not merely to prevent loss of treatment water but to preclude ingress of groundwater, which might contaminate the treated effluent. For the same reason it is a wise precaution to ensure that the floor is above the highest water table.

FIG. 16. VILLAGE WATER TREATMENT INSTALLATION, S URINAM

Both for heat insulation and for easy access, the top of the walls should be only a short distance above finished ground level, the surrounding area being made up or graded to accomplish this. It is, however, prudent to allow part of the wall to protrude above ground level, to about the same
height as that of the internal freeboard element, in order to trap wind-blown dust and debris and to reduce the chances of small animals falling into the filter.

In climates where low temperatures are not a problem, it may be more convenient to build small filters above ground. This may be so where the groundwater table is high or where excavation in hard rock would be expensive. Fig. 16 shows one of the treatment units constructed in Surinam to serve villages of up to 500 inhabitants (capacity 25 m³/day). The units are built in pairs to ensure continuity of supply. As will be seen, the whole unit—settlement tank, aerator, filter, clear-water tank and chlorine doser—is built above ground level.

"Short circuiting", or the downward percolation of water along the inner wall face without passing through the filter-bed, endangers the purity of the effluent, and structural precautions must be taken against it. It is no problem with sloping walls, since the sand tends to settle tightly against them, but with vertical walls it may be necessary to incorporate devices such as built-in grooves or artificial roughening of the internal surface (Fig. 17). The most effective precaution is to give the walls a slight outward batter, so as to obtain the advantages of sloping walls, and to use grooved or roughened surfaces in addition. A precaution that was once in common use was to keep the under-drainage some distance from the base of the walls, but this method decreases the effective filtering area and is rarely adopted nowadays. Above and below the area of contact with the sand-bed, all concrete surfaces should be as dense and smooth as possible to reduce fouling by slimes and other aquatic growths.

When designing filter boxes it should be remembered that the special requirements relating to water-retaining structures and the precautions necessary against thermal expansion and contraction, shrinkage of concrete,
uplift of floors, and unequal settlement become more difficult as the area of the structure increases. To ensure watertightness it may be preferable to plan for a larger number of filters of smaller size.

Filter controls

To be certain that the filters will operate successfully in accordance with the calculated hydraulic characteristics already described, it is important that the pipework, valves, and devices used to regulate the operation of the filter should be planned and calculated with the same care.

Basically, means must be available to:

1. deliver raw water into the supernatant water reservoir;
2. remove scum and floating matter from the supernatant water;
3. drain off the supernatant water prior to filter cleaning;
4. lower the water level within the bed;
5. control the rate of filtration and adjust it as bed resistance increases throughout the length of the filter run (a venturi or other type of meter is a very desirable component of the regulating system and is recommended for all but the smallest filters);
6. ensure that negative pressures cannot occur within the bed (the weir is the device usually used for this purpose);
7. convey the filter effluent to the filtered water reservoir;
8. run filtered water to waste or to the inlet side of other filters during the ripening process;

FIG. 18. DIAGRAM OF A SLOW FILTER, SHOWING CONTROL VALVES
(9) fill the sand-bed from below with filtered water (from other filters) after cleaning.

Fig. 18 is a diagram of a slow sand filter on which all these various regulating devices are indicated, and it is pertinent to consider their interrelationship during the normal working of a filter.

The flow of water through a filter-bed depends on three factors (Fig. 19):

- the raw water (or inlet) head $H_1$
- the filtered water (or exit) head $H_2$
- the filtration rate, or velocity of flow, $v_f$.

These factors are related, according to Darcy’s law, by the equation

$$v_f = \alpha (H_1 - H_2)$$

in which the coefficient $\alpha$ depends on the hydraulic characteristics of the complete filter. Some of these characteristics are “built-in” (e.g., the thickness and grain size of the medium and the hydraulic resistance of the under-drainage system), but the most significant is clogging, which increases with the length of the filter run and depresses the value of $\alpha$.

The importance of maintaining a steady rate of filtration throughout the filter run and the advantages of keeping a constant head of water over the bed surface have been discussed. It is usual, therefore, to control the influent raw water to keep $H_1$ constant, and to adjust the exit head $H_2$ to compensate for the decrease in $\alpha$ as the bed resistance builds up.

It would at first sight appear simpler and equally satisfactory to operate a slow sand filter with no control on the outlet by delivering a constant quantity at all times (by the regulation of the delivery pump for example).
Fig. 20 illustrates this arrangement. However, apart from the consequent variations in the level of the supernatant reservoir ($H_1$ continually increasing to compensate for the reducing coefficient $\alpha$), this method of operation is unsatisfactory for the following reason.

It will be obvious that the quantity of the influent raw water $Q_i$ will be equal to the effluent quantity $Q_e$ only when the supernatant water level remains steady. If $H_1$ drops by an amount $\delta H$ in unit time the inlet and outlet quantities will bear the relationship:

$$Q_e = Q_i + \delta H \cdot A$$

where $A$ is the area of the filter surface. Since it is the outflow $Q_e$ that determines the filtration rate $v_f$,

$$v_f = \frac{Q_e}{A} = \frac{Q_i}{A} + \delta H.$$ 

Even under the most careful working conditions it is possible for $\delta H$ to reach a value of 0.1 m/h. In rapid filtration, with $Q_i/A$ in the neighbourhood of 5–10 m/h, such a lowering of the raw water level would increase the filtration rate by a mere 1–2%, which is negligible, but in slow sand filtration, with $Q_i/A$ as low as 0.1–0.2 m/h, it would increase the filtration rate by 50–100%, with a consequent deterioration in effluent quality. It is thus more satisfactory to maintain a constant raw water level and to regulate the filtration rate by a control in the effluent line.

The raw water level may be maintained automatically by means of a float-controlled butterfly valve, as shown in Fig. 21, or by manual operation of the raw water inlet valve or supply pump. Manual control is relatively simple since the water level can be maintained immediately below the lip of the scum outlet troughs (which act as an overflow if the level rises). It has a further advantage in the case of filters supplied by individual pumps, for then the output of the raw and filtered water pumps...
may be balanced if no separate storage reservoir for filtered water has been provided.

Control of the filtration rate by means of a regulating valve on the effluent line may also be effected automatically, using one of the many rate controllers manufactured for the purpose. In rapid filtration, with a fast build-up of filter resistance, automatic control is virtually essential, but with slow filtration the increase in filter resistance and the resulting decrease in filtration rate proceed so gradually that manual control need present no difficulties. In Fig. 21 the gate valve in the effluent line is regulated by hand in such a way as to maintain a constant filtration rate, the value of which can be accurately read from the preceding venturi meter. Where labour is scarce the valve may be automated to operate from the meter, or a remote control system may be installed. While such devices will add to the initial cost, the need for supervision will be reduced, though this in itself may carry dangers if minor defects are not noticed and corrected promptly.

FIG. 22. MANUAL CONTROL OF RAW WATER LEVEL; MANUAL CONTROL OF FILTRATION RATE BY MEANS OF ADJUSTABLE OVERFLOW TUBE
It has been shown earlier that the provision of an effluent weir is a valuable device to prevent negative heads and air binding, and Fig. 21 shows such a weir at the entrance to the clear well. In Fig. 22, on the other hand, the weir and control valve are combined into a single and very simple unit consisting of a pair of telescopic tubes, the inner of which can be raised and lowered to adjust the rate of filtration.

The various controls listed at the beginning of this section are discussed in the following paragraphs.

Raw water delivery. When individual pumps for each filter are provided, control of the quantity supplied may be effected at the pump outlet, but in the more usual case where a common set of pumps supplies a number of filters or the raw water flows by gravity from a single reservoir a regulating valve will be necessary to maintain the supernatant water at a constant level, (valve A in Fig. 18).

The entrance of the raw water into the supernatant water reservoir must be so arranged that the sand-bed below is not disturbed by turbulence. In one arrangement, shown in Fig. 18, a drainage trough is constructed under the inlet to absorb the vertical force of the incoming water at the start of filling operations, before a sufficient layer of water has accumulated to protect the filter surface.

Scum outlet. This should preferably take the form of a trough, and in large filters several should be constructed on different sides of the filter box so that whichever way the wind is blowing floating matter may be removed by the simple method of increasing the rate of inflow very slightly and allowing the supernatant water to spill over the lip of the trough. No valve need be provided on the trough drain, which is led to waste. The scum outlet acts also as a safety overflow to prevent the supernatant water level rising excessively owing to careless operation.

Supernatant water drain. When a filter is due for cleaning it is necessary to remove the supernatant water so that the bed surface is exposed. The resistance of the filter being high, it would take a long time to lower the level of the supernatant water by allowing it to drain through the filter-bed and would cause an excessive delay in restoring the filter to operational use.

A separate drain and emptying trough (B in Fig. 18) is therefore provided through which the supernatant water may be discharged to waste (if water is plentiful) or returned to the raw water pumps or reservoir for treatment through other filters.

It must be remembered that with successive filter cleanings the bed surface will be lowered until after some years it approaches 0.5 m below its original level. If the emptying trough is fixed in height, this depth of water will be left on the filter surface. A common practice is therefore to construct the trough with an adjustable sill along at least part of its length,
so that after each cleaning the lip can be matched to the new surface level. The length of time the filter is idle is reduced by rapid emptying, which depends on the capacity of the discharge pipe and the accompanying losses due to friction and turbulence.

**Bed drainage.** After draining off the supernatant water it is desirable, before cleaning, to lower the level within the bed by a further 10 cm or more so that the *schmutzdecke* and the top layer is relatively dry and easy to handle. Resanding, replacement of an activated charcoal layer, or repairs to the underdrainage system will need more complete drainage of the bed. A valve (D in Fig. 18) should be provided to carry this drainage to waste.

**Effluent control valve.** The use of this, the most important control in filter operation, has already been referred to in some detail earlier. It is shown diagrammatically at E in Fig. 18, immediately downstream of the flow meter.

**Effluent weir.** This may take the form of a fixed weir as shown in Fig. 18. To enable its height to be varied to suit the head loss through the system, this weir should either have an adjustable crest plate or a telescopic outlet, as shown in Fig. 22, in which case it is possible to omit the effluent control valve.

The purpose of the weir is threefold—to prevent negative heads developing in the bed with consequent air binding, to aerate the effluent, thus raising its oxygen content and releasing such dissolved gases as carbon dioxide that could render the water aggressive, and to make the operation of the filter independent of water-level fluctuations in the clear-water reservoir. The structure enclosing the weir must be very well ventilated both to provide oxygenating air and to prevent possible accumulations of gas that could be harmful to operators.

A bypass valve (G in Fig. 18) should be provided to enable the down-stream side of the weir to be emptied through valve F, which is also capable of draining the chamber upstream of the weir.

**Adduction line.** The outlet valve (J in Fig. 18) and pipes leading to the clear water reservoir should, like the raw water pipeline, be of a size that accords with normal hydraulic principles, taking into account losses from friction and turbulence. It should be remembered that, once a filter has been built, the filtering area is fixed but the filtration rate is capable of being changed. An increase in filtration rate might, for example, become possible in the future if the raw water quality were to be improved by pretreatment. If the adduction lines and control valves were inadequate to cope with this increase, the cost of duplication or replacement would be high. It is good practice, therefore, to install pipework capable of carrying, say, 50% more than the load immediately anticipated, since this adds little to the initial cost but could save large sums later.

**Diversion of filtered water.** During the ripening period of a new or
recently cleaned filter it is necessary to divert the effluent to waste, or return it to the raw water reservoir, until the bacterial action of the bed has become established and the effluent quality is satisfactory. Valve H in Fig. 18 is included for this purpose.

**Backfilling.** After the cleaning of a filter-bed (as well as during its initial filling) filtered water is introduced from the bottom to drive out air bubbles from the medium as the water level inside the sand rises. The filtered water is obtained from the clear well or from the outlet side of another filter and is admitted through valve C in Fig. 18.

**Miscellaneous.** Fig. 18 also shows how, by sloping the floor of the filter box, it is possible to drain the filter completely and how vents can be suitably located to prevent any air pockets forming in pipework or valve chambers.

**Covering of filters**

Covering of filters may be necessary for one or more of the following reasons:

1. to prevent deterioration of effluent quality during periods of low temperature, i.e., below 6°C for several months during the year or below 2°C for one month or more;
2. to prevent the expense and operational difficulty of ice removal in places where heavy frosts may occur;
3. to prevent algal growth in the supernatant water, by excluding sunlight;
4. to avoid a deterioration of the raw water quality through windborne contamination or bird droppings.

The first two of these obviously apply to plants constructed in temperate or cold climates, whether due to latitude or to altitude. If conditions are likely to be severe, heat insulation is required, which was at one time usually accomplished by constructing a flat concrete roof and covering it with a layer of soil. This had certain disadvantages. Soil, especially when saturated, is very heavy, necessitating an expensive load-bearing roof, while the insulating properties of wet soil are not particularly high. This method of construction has now been almost entirely superseded by lightweight roofs lined with insulating material such as plastic foam, but precautions must be taken against corrosion of the structure since a high degree of condensation of an aggressive nature can be expected beneath the cover.

The same type of covering can obviously be used to exclude sunlight where both temperature and algal problems are to be expected. In tropical
or subtropical climates, sunlight exclusion alone may be needed, and a much less solid structure will suffice. Indeed, small filters are often roofed with removable frames covered with corrugated iron or even grass matting, placed on bearers immediately above the water level and having small sections easily lifted for inspection.

Permanent structures must have ample working headroom above the filter surface, with sufficient access for the removal of filter scrapings and the introduction of new sand when replacement becomes necessary. With large filters, the ability to drive dump trucks on to and around the surface may save considerable handling costs during resanding operations. If mechanical cleaning is to be adopted, the construction of the roof must obviously allow sufficient clear space free from supports or other structural members and even if mechanical equipment of this nature is not to be immediately installed the possibility of later conversion must be borne in mind if high structural conversion costs are to be avoided in the future.

With both permanent and light temporary structures, allowance must be made for wind pressures and uplift acting on the comparatively large areas of unsupported roof involved in filter covering.