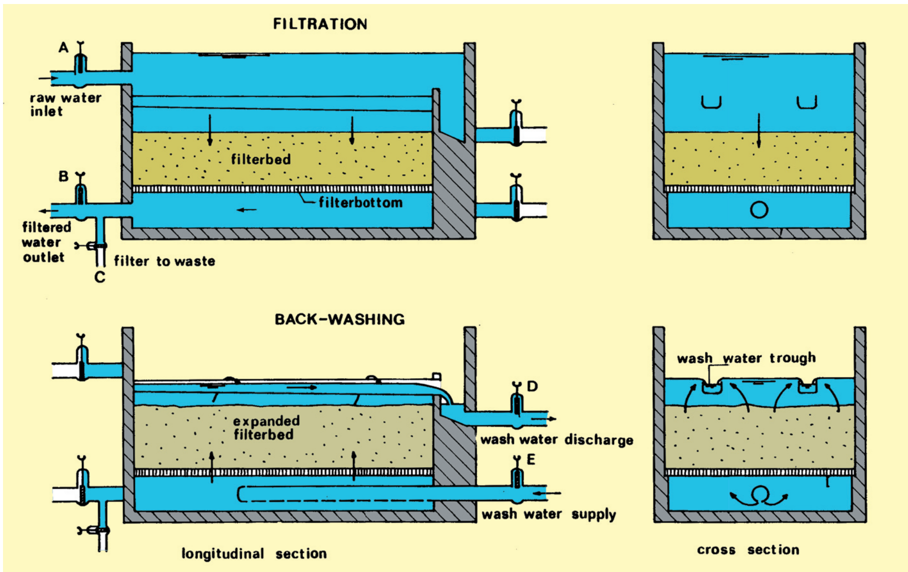
CT 4470

# Rapid filtration

September 2004

Prof.dr.ir. L. Huisman

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## Principal notations

$a, A$ - area ($m^2$)
$b, B$ - width (m)
$c, c_o, c_e$ - concentration of impurities in water ($g/m^3$)
$d, d_o$ - diameter of spherical filter grain (m)
$d_h$ - hydraulic diameter of non-spherical filtering material (m)
$d_s$ - specific diameter of non-uniform filtering material (m)
$d_n$ - diameter which is not reached by n percent of the filtering material (m)
$D$ - inside diameter of pipelines (m), depth of supernatant water (m)
$e$ - base of natural logarithm (2.71828....)
$E$ - percentage increase in filterbedthickness by expansion during backwashing
$F_o$ - optimisation factor in rapid filter design (sec)
$g$ - gravity constant (9.80665 $m/sec^2$)
$h$ - depth of water (m)
$H$ - filter resistance as head loss during filtration (m)
$I, I_o$ - slope of piezometric surface in filterbed (m/m)
$k$ - coefficient of permeability (m/sec)
$l$ - depth or length (m)
$L$ - filterbed thickness (m)
$L_e$ - thickness of expanded filterbed during backwashing (m)
$p, p_o$ - pore space in filterbed
$p_e$ - porosity of the expanded filterbed during backwashing
$Q$ - capacity or discharge ($m^3/sec$)
$s$ - settling velocity (m/sec)
$s$ - clear opening of square woven wire sieves (m)
$S, S_o$ - combined grain surface per unit volume of filtering material ($m^{-1}$)
$t$ - time (sec) or temperature ($^oC$)
$t, t_o$ - pore diameter (m)
$T_q$ - length of filterrun with respect to effluent quality (sec)
$T_r$ - length of filterrun with respect to filter resistance (sec)
$v$ - rate of filtration (m/sec)
$V$ - volume ($m^3$)
$y$ - vertical coordinate (m)
$z$ - head loss during backwashing (m)

| | |
|---|---|
| $\alpha$ | - coefficient in filtration theory ( $sec^{-1}$) |
| $\phi_1$ | - shape factor of non-spherical grains during filtration |
| $\phi$ | - shape factor of non-spherical grains during backwashing |
| $\lambda, \lambda_o$ | - coefficient in filtration theory ($m^{-1}$) |
| $\nu$ | - kinematic viscosity ($m^2/sec$) |
| $\rho$ | - mass density ($kg/m^3$) |
| $\rho_d$ | - mass density of impurities as deposited in the filterbed ($kg/m^3$) |
| $\sigma$ | - gravimetric concentration of impurities in filterbed ($g/m^3$) |
| $\sigma_v$ | - volumetric concentration of impurities in filterbed ($m^3/m^3$) |

Abbreviations

| | |
|---|---|
| A.W.W.A. | - American Water Works Association |
| J.T.U. | - Jackson Turbidity Unit |
| Re | - Reynolds number($\frac{v.d}{\nu}$) |

# 1. INTRODUCTION

## 1.1. Definitions and terms

Filtration is the purification process, whereby the water to be treated is passed through a porous substance. During this passage water quality improves by part removal of suspended and colloïdal matter, by reduction of the number of bacteria and other organisms and by changes in its chemical constituents. In the practice of water purification, the porous substance may be in principle any stable material, as well as a granular bed of sand, crushed stone, anthracite, glass, cinders, etc., as a consolidated layer of porous concrete, stoneware, plastic and so on. In the field of public and larger private water supllies, however, granular beds of sand are almost used exclusively. Such beds allow a penetration of impurities from the raw water without an immediate deterioration of effluent quality. In this way a silt storage capacity is created, by which also more turbid waters can be dealt with. Sand as filtering material has the advantages of availability, relative low cost and the satisfactory experience that it has given. Even when an other granular filtering material as for instance anthracite is applied, this is mostly done in combination with sand to obtain multi-layered filterbeds with a higher capacity for the storage of silt. Filtration incidentally should not be confused with straining, using a fine meshed filter cloth on which a mat of retained material is formed. When to promote mat formation and straining efficiency, particulate matter as for instance diatomaceous earth is added to the raw water, the difference with filtration proper in the meanwhile is almost negligeable.

When during the process of filtration the impurities are removed from the water, they accumulate on the grains and in the openings between the grains of the filterbed, in this way reducing the effective pore space by which the resistance against the flow of water increases and the filtration efficiency drops. After some time, this resistance becomes so high or the quality of the effluent so low, that cleaning the filter is necessary. With regard to the interval between cleanings and the way this cleaning is effected, two groups of filters may be distinguished, slow filters and rapid filters, which filters also differ greatly with respect to the filtration rate, that is the capacity per unit area of filterbed surface.

Slow filters are the oldest type of filters used for public drinking water supplies, going back as far as 1829 when they were first built by James Simpson for the Chelsea Water Company in Londen. In these slow filters, the water is passed by gravity downward through a layer of fine sand at low velocities. For conditions of average daily demand, the filtration rate varies from less than $(0.03)10^{-3}$ to about $(0.15)10^{-3}$ m/sec (that is $m^3$/sec per $m^2$ of filterbed area). This rate is so small, that only after an extended period of service, a few weeks to a few months or more, cleaning is necessary. With the filterbed composed of fine grains, effective diameter between about 0.15 and 0.35 mm, suspended and colloïdal matter from the raw water are retained in the very top of the filterbed. The clogged material here may be removed and the filter restored to its original capacity by scraping off this top layer of dirty sand, to a depth varying from one to a few centimetres.

With rapid filters on the other hand, the water flows down a bed of medium to coarse sand at relatively high velocities. For the normal type of downflow filtration, this sand is carefully graded to a uniform size, varying from one case to another between about 0.5 and 2mm, or larger, while for conditions of average daily demand the filtration rate is commonly in the neighbourhood of $(1.5)10^{-3}$ m/sec. This rate is so high that a rapid clogging of the filterbed occurs, necessitating cleaning every one to a few days. By the use of medium to coarse sand more over, impurities from the raw water penetrate the filterbed to greater depths. Cleaning of a rapid filter is therefore only possible by backwashing, reversing the flow of water which expands the filterbed and scours the grains, carrying the accumulated impurities to waste.

Rapid filters have first been used in 1885 in the U.S.A. at Somerville, New Jersey and in 1895 in Europe, for the municipal water supply of Zürich in Switserland. These filters were built as submerged filters with a free surface passing the water downward by gravity. The majority of the rapid filters built today are still constructed in this way, of which fig. 1.1. shows a modern example. In the past 80 years, however, many other constructions have emerged, as most important of which may be mentioned pressure filters, upflow filters, multi-layered filters and dry filters.

In the gravity type or free-surface filters, the maximum allowable head loss is governed for the greater part by the depth of water on top of the filterbed. When longer filterruns with larger head losses are

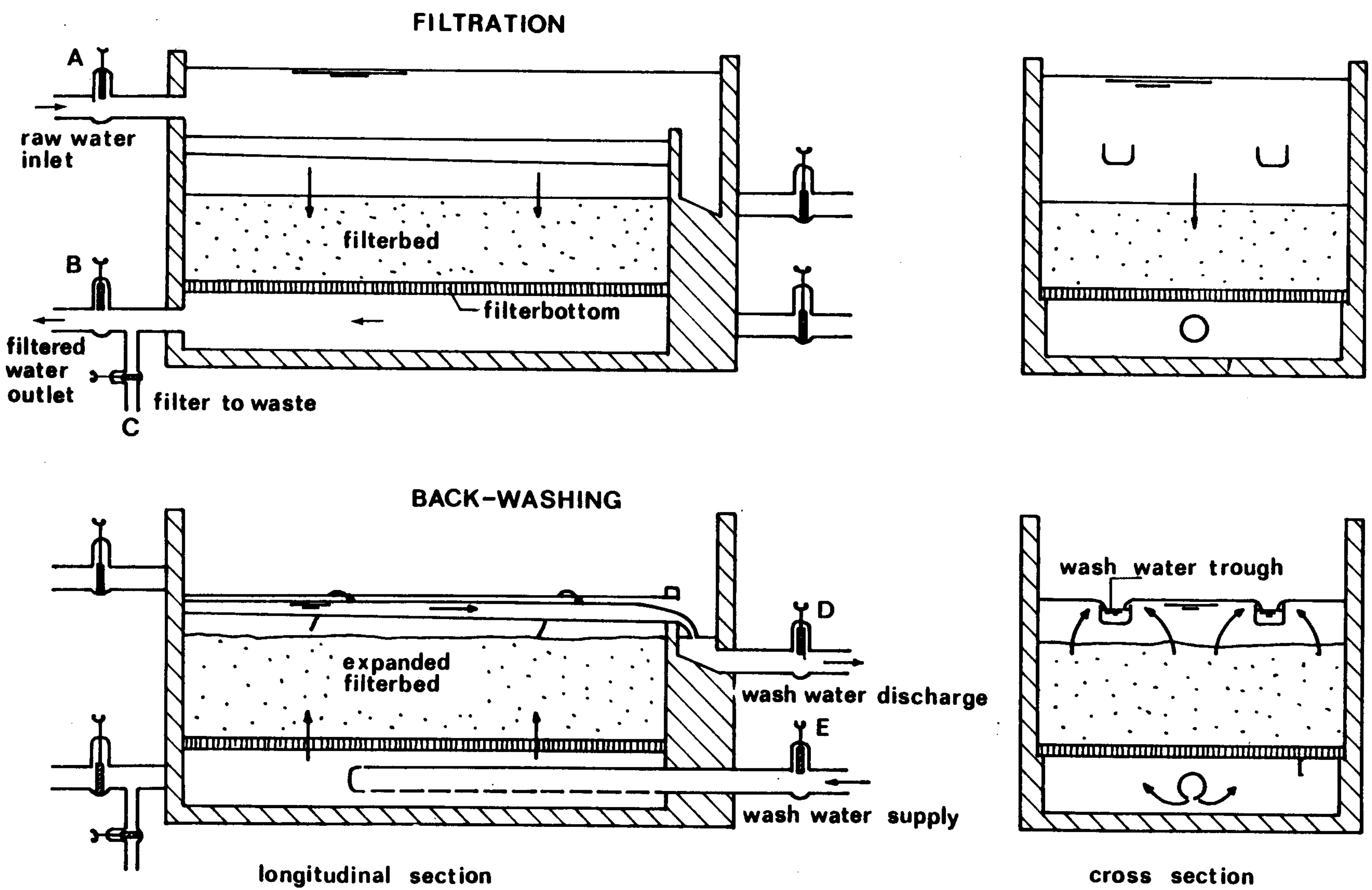



Fig. 1.1 Free-surface rapid gravity filter.

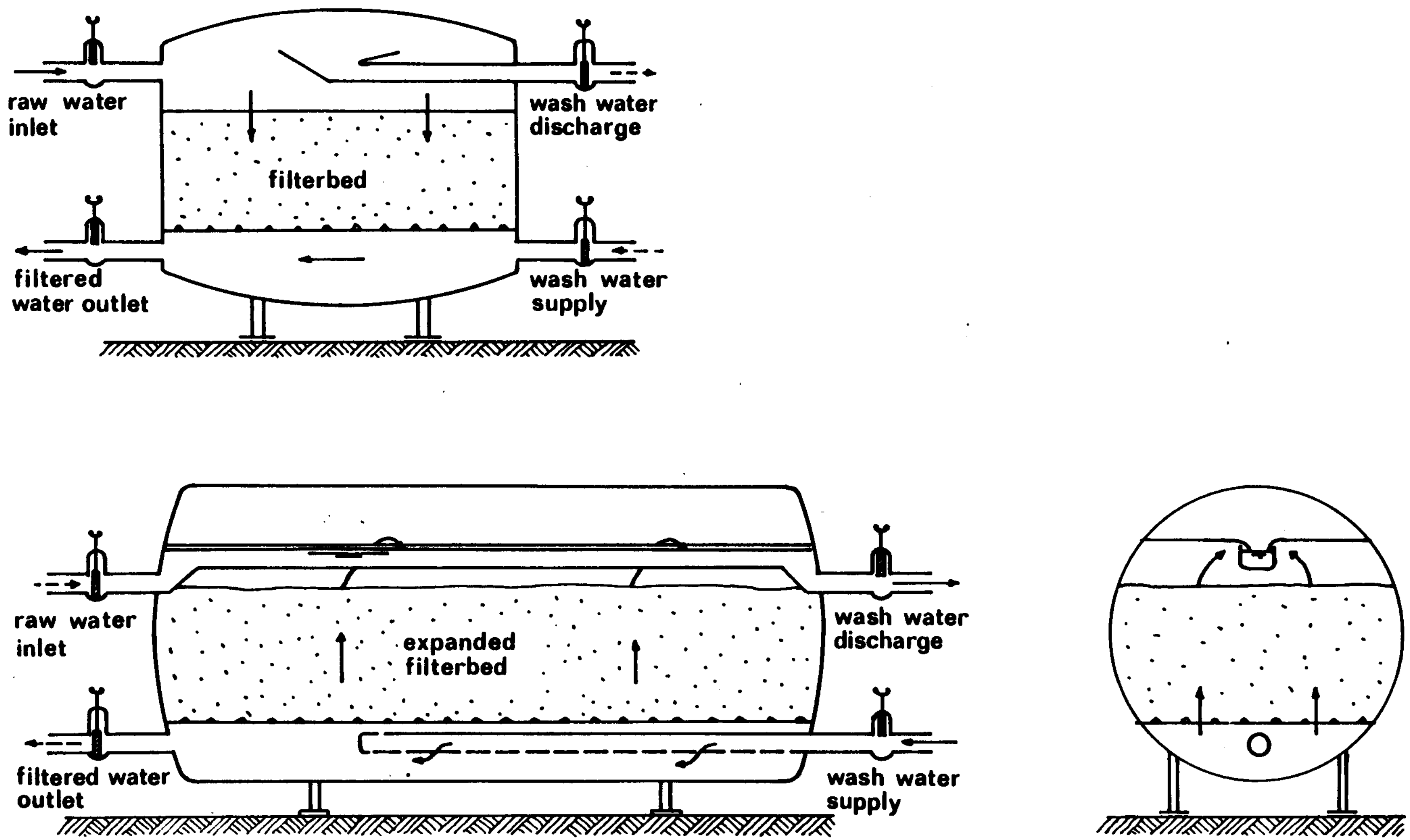



Fig. 1.2 Pressure filters.

desired, this depth could be increased, but this asks for a greater height of the filterbcx, appreciably increasing the cost of construction. In such cases, a more economical solution sometimes may be obtained by enclosing the rapid filter in a water-tight steel cylinder (fig. 1.2.). The driving force is now the difference in water pressure before and after passing the filterbed, which head loss can be augmented at will. By the absence of a free surface, these so-called pressure filters may also be set at any random level, in an odd corner and even outside buildings, very important for industrial water supplies while by the lack of contact with the outside air, no airborne contamination can occur. The filtered water, moreover, appears under pressure and in many cases broken pumping can thus be avoided. Pressure filters may be constructed with the axis of the cylinder vertical or horizontal as shown in fig. 1.2. Vertical filters make a better use of the space available, but forging of the end plates limits their diameter to 4 or 5 m. With filterbed areas in excess of 10 to 20 $m^2$, horizontal filters must therefore be chosen.

As other disadvantage of downflow filters must be mentioned, that backwashing results in a hydraulic grading of the filtering material, bringing the fine grains to the top and the coarse ones to the bottom of the bed. In this way, the raw water to be treated comes first into contact with fine filtering material, which clogs easily with a rapid increase of the filter resistance and shortened filterruns as unavoidable results. This disadvantage can be lessened, but not eliminated altogether, by the use of a very uniform filtering material, with a coefficient of uniformity (ratio between the 60 and 10% grainsize passing) less than 1.2 or 1.3. Such uniform filtering material might be fairly expensive, while the hydraulic classification of non-uniform filtering material on the other hand could be used to advantage by reversing the direction of the flow. In these upflow filters (fig. 1.3.), the turbid raw water first passes the coarser grains of the filtering material, which are able to retain a large part of the suspended load without an appreciable increase in filter resistance. In the upper part of the bed, the more or less clear water is purified by the finer grains, removing the small amount of remaining impurities again without a rapid clogging, in this way providing a better water quality during extended filterruns.

With downflow filtration, the filter resistance is taken up by the underdrainage system, which can be made as strong as required. With upflow

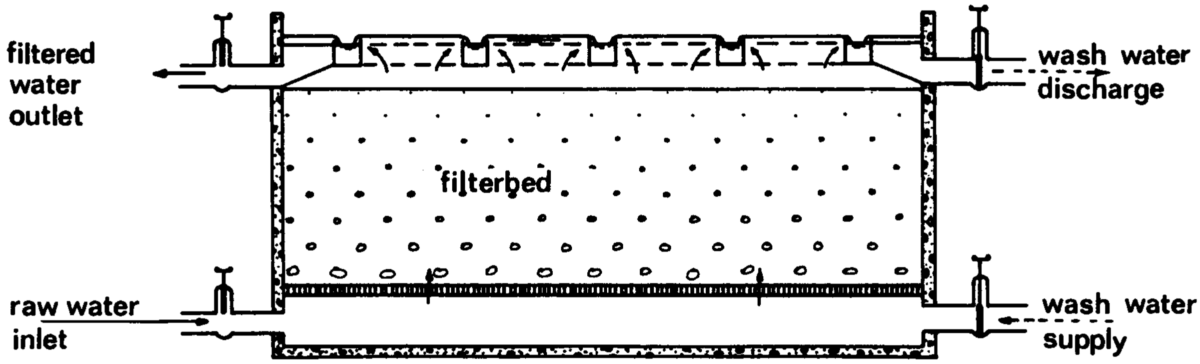
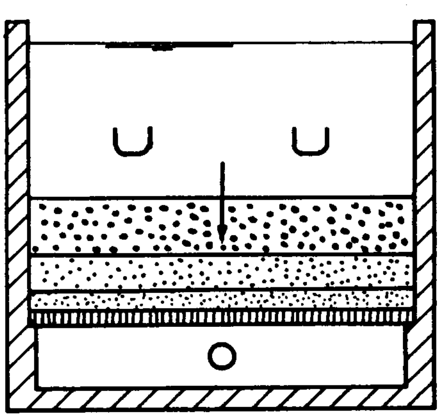



Fig. 1.3 Upflow filtration.

filtration on the other hand, the submerged weight of the filtering material is the counter-acting force, limiting the maximum allowable filter resistance to about the thickness of the filterbed when sand is used (compare section 3.2). Larger values could be allowed by the use of a heavier filtering material such as garnet or magnetite, but garnet in particular is very expensive with the great bed thicknesses commonly applied. On the other hand, the prime purpose of upflow filters, that is filtration from coarse to fine, can also be obtained with ordinary downflow filters by composing the filterbed of different layers with decreasing grain sizes in the direction of flow. To prevent these layers from overturning during back-wash, the decrease in grainsize should be accompanied by an increase in specific gravity, using for instance sand as middle layer with a lighter material such as anthracite on top and a heavier material as magnetite below (fig. 1.4.).



**Filterbed composed of**

0.6 m anthracite, $\phi$1.6mm, $\rho_a/\rho_w$ :1.5
0.4 m sand, $\phi$0.8mm, $\rho_a/\rho_w$ :2.6
0.2 m garnet, $\phi$0.5mm, $\rho_a/\rho_w$ :4.2

Fig. 1.4 Multi-layered filterbed.

A disadvantage inherent to all the rapid filtration processes mentioned above, is the limited amount of oxygen water can carry in solution. Under atmospheric conditions, oxygen saturation values vary from about 14 $g/m^3$ at a water temperature of $0^\circ$ C to 8 $g/m^3$ at $30^\circ$ C. During the process of filtration, this oxygen is consumed in small amounts for the oxidation of iron and manganese in somewhat larger amounts for the degradation of organic matter, but in great quantities for the nitrification of ammonia when present. With 3.6 g of oxygen necessary for the oxydation of 1 g of ammonia, the removal of ammonia by rapid filtration is thus limited to 2 or 3 $g/m^3$. When the raw water has a higher ammonia content, double filtration with aeration in between must consequently be applied. Such high ammonia contents often occur with groundwater, where a secondary filtration is otherwise not required. The same results, but at much lower costs, may now be obtained with dry filtration as shown in fig. 1.5. Here the raw water to be treated percolates downward through the filterbed, accompanied by an equal to a few times smaller or larger amount of air from which the oxygen consumed for nitrification is replenished immediately, allowing complete removal of ammonia contents as high as 5 or 10 $g/m^3$. As other advantage of dry filtration may be mentioned, that the presence of air in the pores of the filterbed increases the actual velocity at which the water moves downward. This means stronger cross-currents and a greater chance for suspended particles to come into contact with the filter grains, the catalytic surface action of which promotes filtration efficiency. This is the reason that dry filtration is also preferred when the presence of organic matter prevents spontaneous deferrisation.

In the following, attention will first be limited to the various aspects of the traditional submerged rapid gravity filter, after which the peculiar features of the other types of rapid filters will be treated in separate chapters. All these filters have in common, that their main purpose is clarification of the water by removal of suspended and colloïdal matter. This is not the case with filtration processes such as taste and odor removal using a bed of granular activated carbon, removal of agressive carbon dioxide with a bed of broken marble or burned dolomite, changing or decreasing the mineral content by ion-exchange, etc. The filtration aspects of these unit operations will be dealt with in chapter 10.

In this publication the International System (SJ) Units will be applied, using the kilogram as unit of mass and the Newton as unit of force.

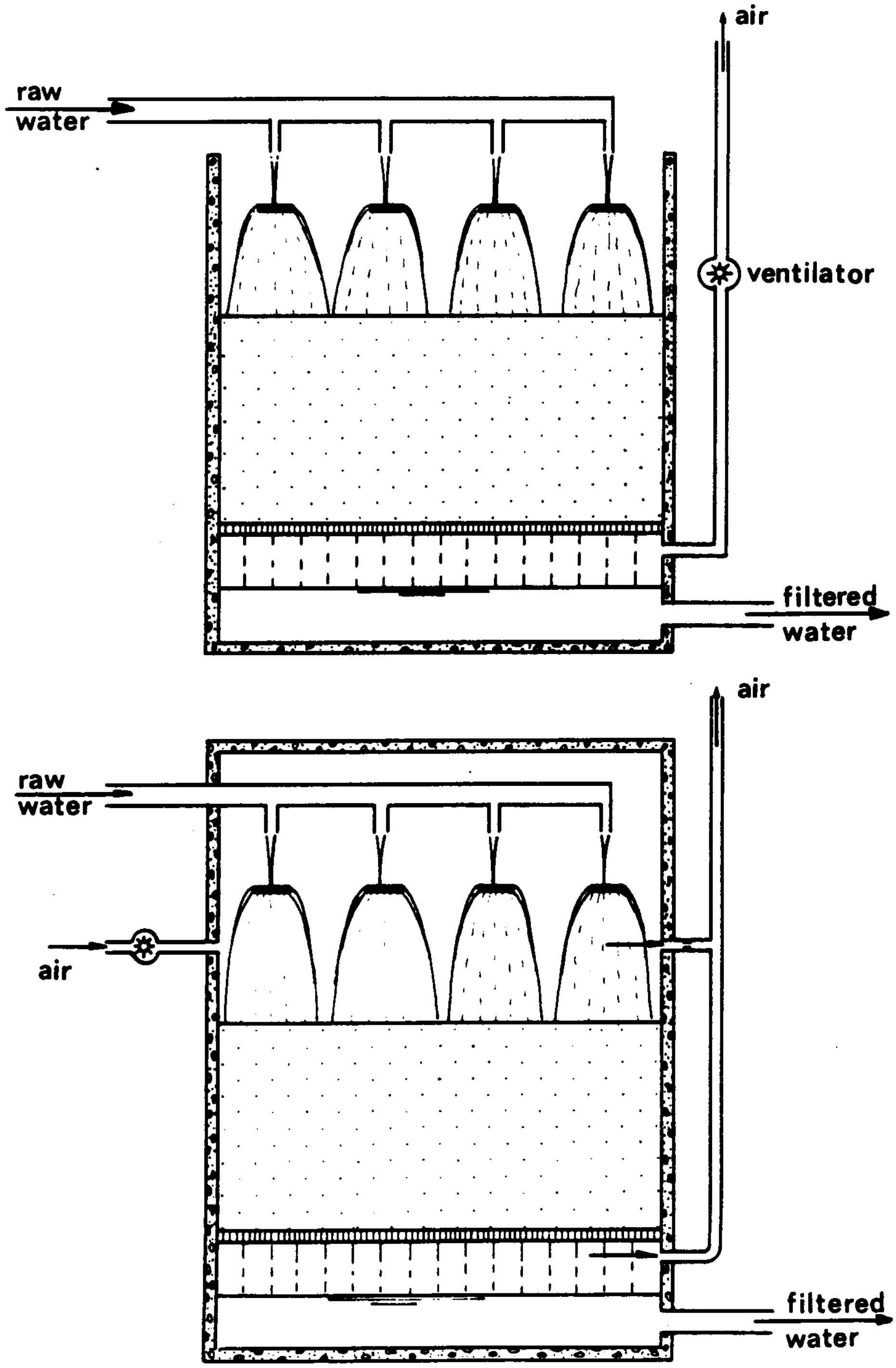



Fig. 1.5 Dry filters.

## 1.2. Elements of a submerged rapid gravity filter

The open downflow type of rapid filters essentially consists of a box, commonly made of reinforced concrete, rectangular in shape and varying in filterbed area between about 15 and 150 $m^2$. This box is filled with a 0.5 to 2 m deep layer of filtering material on top of which the raw water to be treated is present in a depth of 0.25 to 2 m. At the lower side this filterbed is supported by a system of drainage, the so-called filterbottom which at the same time allows the discharge of filtered water and the supply of wash-water. For convenience in drawing only, a porous filterbottom is chosen as underdrainage system of the filters shown in fig. 1.1 and 1.3 to 1.5 inclusive, their use being in reality rather exceptional. During back washing, the wash-water together with the dislodged impurities from the filterbed is carried away with a system of troughs and gulleys at a distance of 0.4 to 0.6 m above the filterbed. The filterbox is finally provided with a number of influent and effluent lines, equipped with valves and with controllers to keep waterlevels and the filtration rate constant. For clarity in presentation, all these lines have been drawn separately in the figures mentioned above, although in practice they are combined and concentrated as much as possible to reduce the cost of construction and operation.

A rapid filtration plant always consists of a number of filtering units, mostly between 4 and 40. These units are commonly situated on one or on both sides of a two level corridor, while a central building houses special equipment such as pumps, compressers and tanks for back-washing with water and air, heating and ventilation equipment for air-conditioning, storeroom, laboratory and offices, etc. In cold climates the filters themselves are housed to prevent freezing in winter time (fig. 1.6), but in hot climates they are built in the open air (fig. 1.7). The saving in cost of construction thus obtained is appreciable and as a consequence this solution is sometimes also applied in moderate climates, although in severe winters some protection may still be necessary (fig. 1.8).

The operation of a rapid gravity filter is shown schematically in fig. 1.1. During filtration the raw water enters the filter through valve A, flows down the filterbed and the underdrainage system and out through valve B, while all other valves are closed. By a gradual clogging of the pores of the filterbed, the resistance against downward water movement

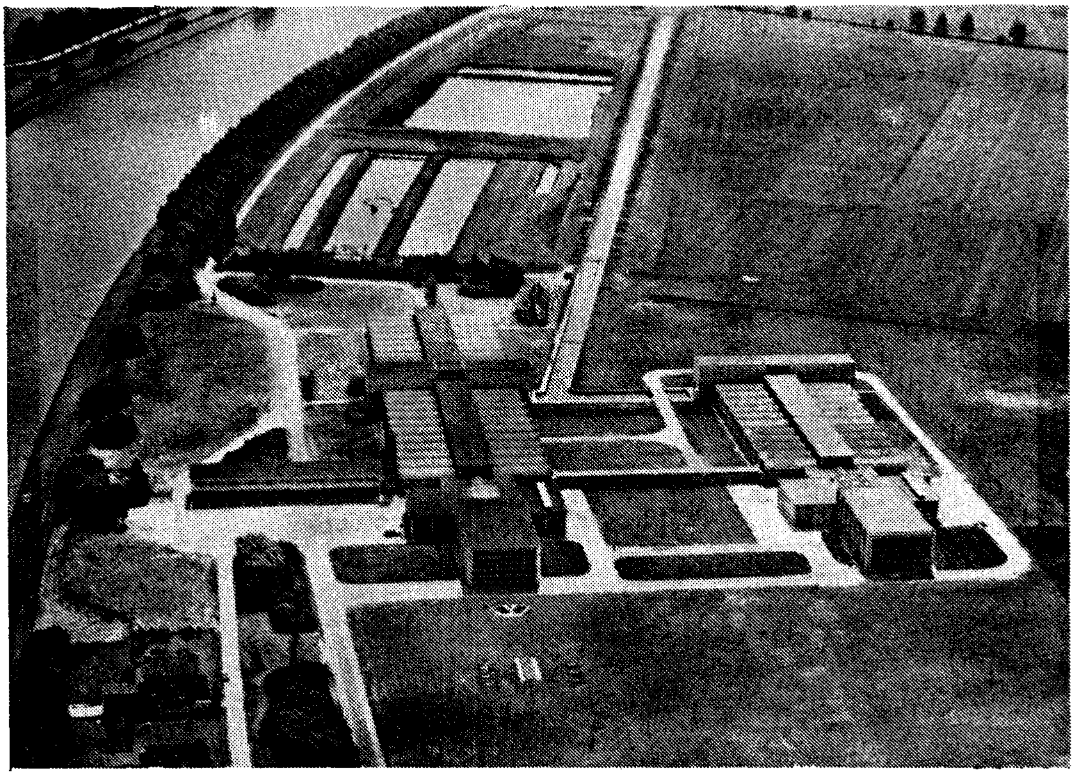


Fig. 1.6 Rapid filtration plant of the N.V. Watertransportmaatschappij Rijn-Kennemerland at Jutfaas, Netherlands.

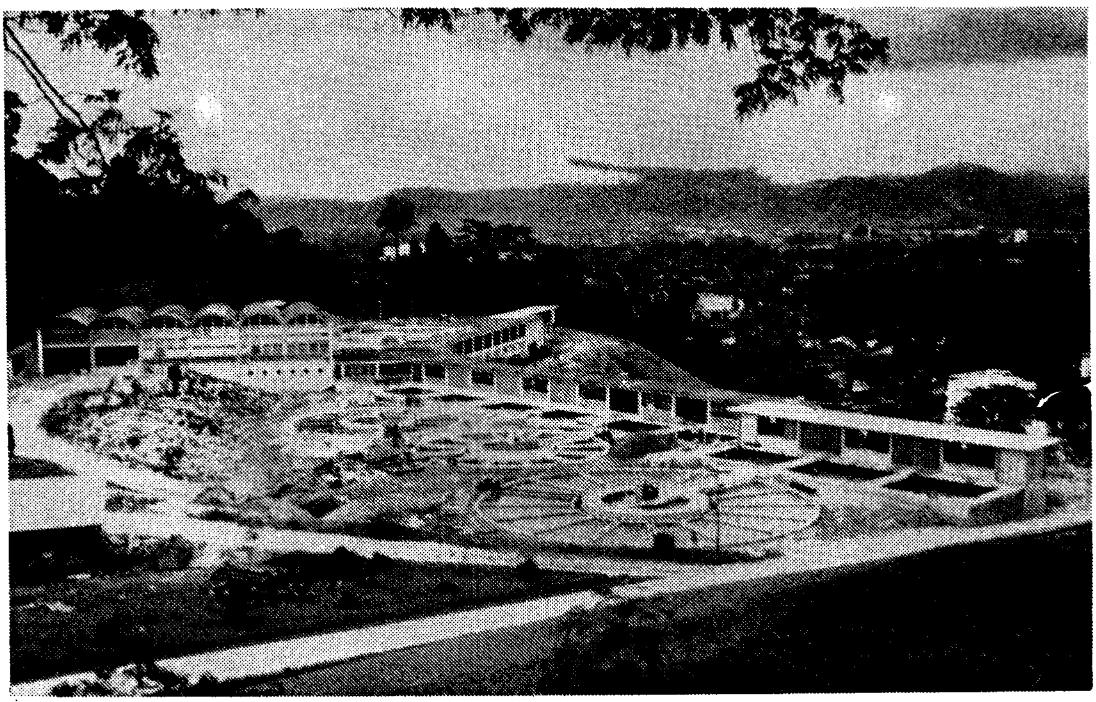


Fig. 1.7 Bukit Nanas treatment plant at Kualalumpur, Malaysia.

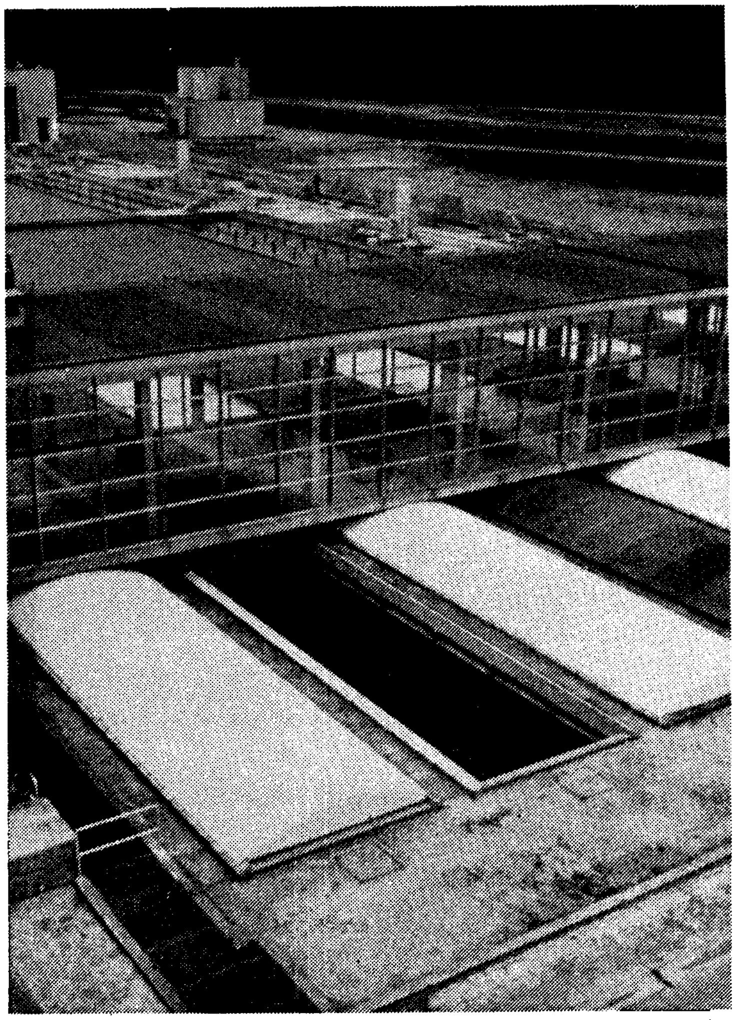


Fig. 1.8 Rapid filtration plant of the Antwerp Waterworks at Oelegem, Belgium.

increases with time. This is compensated by opening the filter rate controller in influent or effluent line in such a way, that the over-all loss of head remains the same and the filtration rate constant. When after some time the filter rate controller is fully opened, a further increase in filter resistance would result in a lowering of the filtration rate and the filter must be taken out of service for back-washing. Valve A is now closed, but when time permits, valve B is kept open for another period to remove the water above the filterbed as much as possible in the normal way. After valve B has been shut, valve D is opened by which the remainder of the supernatant water is drawn off to the upper level of the wash-water troughs. Washwater is now admitted to the space below the porous bottom by opening valve E. The upward flow of washwater expands the filterbed, scours the filtergrains and takes the accumulated clogging with it to above. After passing the filterbed, the dirty washwater is discharged with the help of wash-water troughs into a gulley, from which it is carried through valve D to waste. When backwashing has been completed, valves E and D are closed and valve A opened. To prevent sediment that possibly may be near the bottom of the filterbed from passing into the filtered water reservoir, the effluent is sometimes carried to waste through valve C for the first 10 or 20 minutes. After this period valve C is closed, valve B opened and the cycle as described above starts anew. In some cases, the scour provided by the rising washwater is insufficient to keep filterbeds clean on the long run. An additional scour is now desirable, mostly produced by backwashing with air, complicating the procedure described above.

## 1.3. Application of rapid filtration for public and private water supplies

For the production of drinking and industrial water, rapid filtration may be used in three different ways, as sole treatment, as preliminary treatment to lighten the load on subsequent (slow sand) filters and as final treatment to remove the last traces of impurities which have escaped the preceding process of coagulation and sedimentation.

In drinking water practice, clarification by rapid filtration alone is quite common for the deferrisation and demanganisation of deep groundwaters, which are safe in hygienic respect by virtue of their origin (fig. 1.9). Fairly coarse grains, often above 2 mm and high filtration rates, up to $(15)10^{-3}$ m/sec and more may now be used. The same sole treatment, but now with finer grains and preceded or followed by sterilization with ozone or chlorine may be applied in those exceptional cases that a fair and unsullied surface water is available. In case the surface water at hand is turbid, but the suspended load is small during most of the time, chlorina-

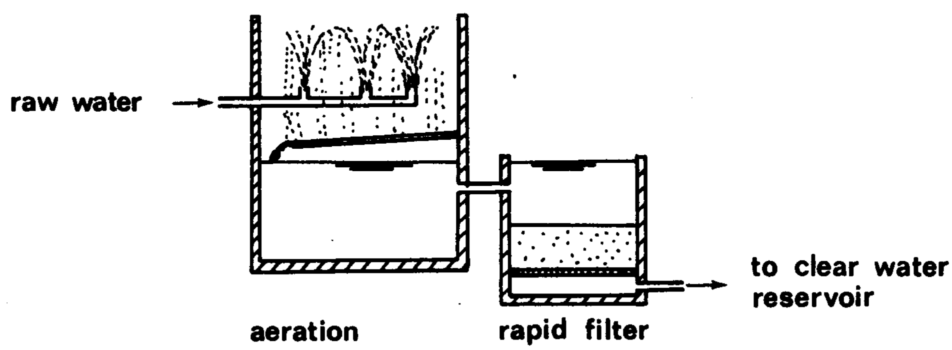



Fig. 1.9 Treatment system for deep anaerobic groundwater.

tion and rapid filtration alone may again be applied when the colloidal matter is brought to combine into larger aggregates with the help of iron or aluminium coagulants and/or by the application of one of the many high-molecular weight flocculants (fig. 1.10). For many an industrial water supply, complete clarification is not required. Rapid filtration as single treatment of surface water is here quite popular, even when this water is rather polluted.

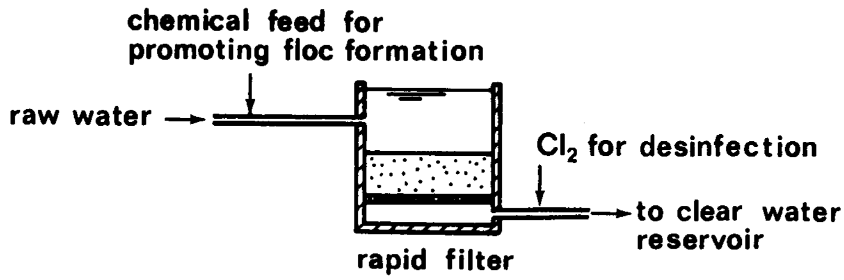



Fig. 1.10 Treatment system for slightly polluted surface water.

As mentioned in the preceding section, filtration of surface water for public water supplies started in 1829 in London, using slow sand filters for the purification of Thames derived river water. These slow filters gave and still give excellent results, not to be surpassed by any other treatment system, provided that the average suspended load of the raw water is small, less than 2 to 10 $g/m^3$ and that the organic matter content including ammonia is not so high as to result in near anaerobie conditions. A higher suspended load will result in a rapid clogging of the filterbed, necessitating filter cleanings at short intervals and asking for lower filtration rates. These disadvantages may be obviated, however, by a pre-treatment of the water, removing the major part of the suspended particles in the raw water. As such pre-treatment, rapid filtration is used on a large scale in Europe (fig. 1.11). The object of these roughing filters is not to produce

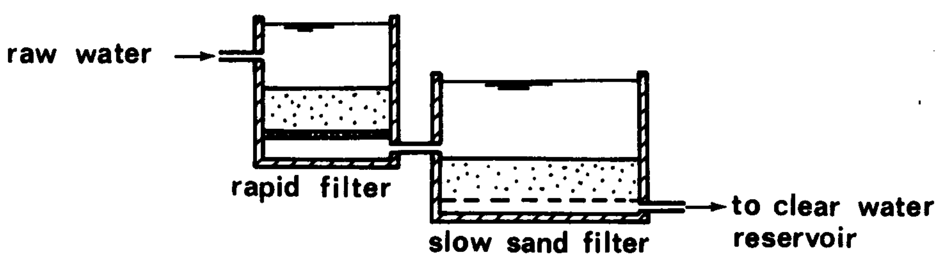



Fig. 1.11 Biological purification of surface water.

drinking water quality, but only to lighten the load on the subsequent slow filters, enabling these slow filters to operate at higher rates for prolonged periods of time. A rapid filter effluent turbidity of 2 to 5 $g/m^3$ is more than sufficient for this purpose. This allows the use of coarse grained filterbeds, average diameters mostly between 1 and 2 mm, with a deep penetration of the impurities from the raw water. Such deep bed filters have a large silt storage capacity and average raw water turbidities of 20 to 50 $g/m^3$ or even more are consequently easily dealt with.

Slow sand filters have been used in the U.S.A., but here they never became popular and as soon as rapid filters developed, they were applied as sole treatment of surface water, in the way as indicated in fig. 1.10. The effluent of these rapid filters has to satisfy drinking water standards and this is only possible by the use of finer grained filterbeds, with average particle diameters between about 0.6 and 1.2 mm. This limits the penetration of impurities from the raw water into the filterbed, reduces the silt storage capacity and asks for a less turbid raw water with suspended loads not exceeding average values of 10 to 20 $g/m^3$, depending on the size distribution. When the turbidity of the raw water is larger or the effluent requirements are stricter, pre-treatment is again required for which coagulation followed by sedimentation has found wide acceptance. With this American system of drinking water production, the rapid filters are used as polishing filters to remove the last traces of flocculated matter and other suspended or dissolved impurities carried over from the settling tanks. (fig. 1.12.) This requires fine grain sizes, 0.5 to 1.0 mm with a limited penetration of impurities from the raw water and surface filtration as unavoidable result. Only the excellent quality of the settled water with suspended loads normally below 2 to 5 $g/m^3$ allows these finishing filters to operate at normal rates with filter runs of acceptable lengths.

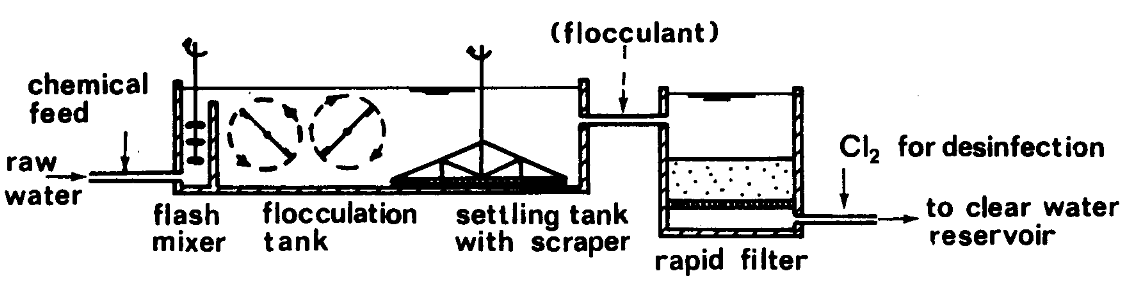



Fig. 1.12 Mechanical purification of surface water.

In contrast with slow filters, rapid filters are not able to produce from surface water sources a water safe in bacteriological respect and for drinking water purposes a separate desinfection is still required. Ozone has been used since the end of last century, but never became popular and the wide acceptance of the American system had to wait till 1908, when chlorine was first applied for this purpose.

# 2. FILTRATION

## 2.1. Mechanisms of filtration

The over-all removal of impurities associated with the process of filtration, is brought about by a combination of different phenomena, the most important of which are (a) mechanical straining, (b) sedimentation, (c) adsorption, (d) chemical and (e) biological activity. For ease in understanding, these actions will be described separately in the next pages. In nature no such partition is present, while the interaction of these processes together with others still partly understood or even fully unknown is of paramount importance. In the field of waterworks engineering, filtration is already used for one and a half century, but still much research is needed to get to the bottom of it.

(a) Mechanical straining is the purifying process most easy to grasp, removing the particles of suspended matter that are too large to pass through the openings between the sand grains. As such it takes place at the surface of the filterbed and is independent of the filtration rate. Even with a grain size of 0.4 mm only, the pores are still a little over 60 μm in diameter (fig. 2.1) and are thus unable to retain colloidal matter (0.001 - 0.1 μm), bacteria (1 - 10 μm) or even small iron or aluminium flocs (say 20 - 50 μm). Some suspended particles may be trapped in the converging

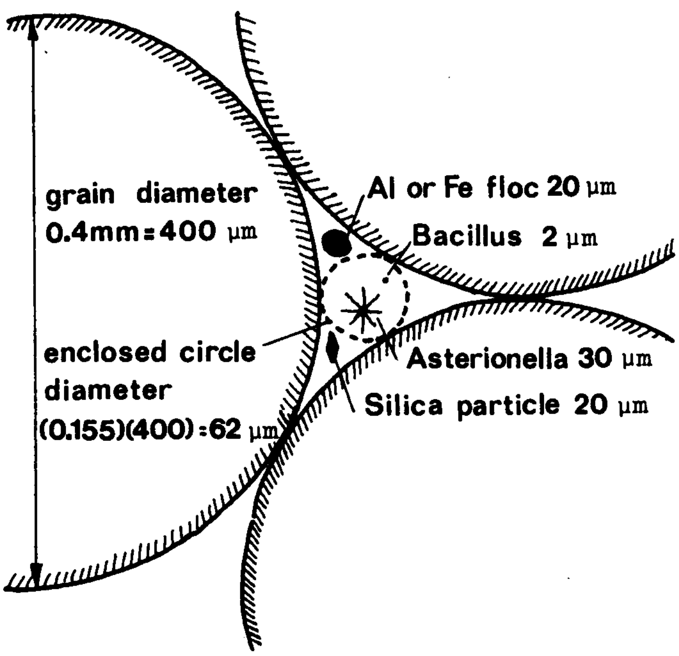



Fig. 2.1 Size of pore openings and suspended particles

spaces between adjoining filter grains (interstitual straining), while the twisting movement of the water through the pores of the filterbed creates velocity gradients, bringing the suspended particles in contact with each other. Some aggregation of finely divided particulate matter will now occur and part of the flocs thus created are again retained at greater depth in the filterbed. Clogging of the filterbed will reduce pore sizes and theoretically at least, straining efficiency will increase with time. In rapid filtration practice, however, straining removes only a negligeable part of the suspended load. When with larger suspended particles, carried by the water in a fast flowing mountain stream for instance, straining would become important, such a rapid increase of filter resistance with time will occur that a coarser grained filterbed must be chosen.

(b) Sedimentation removes particulate suspended matter of finer sizes than the pore openings by precipitation upon the surface of the sand grains, in exactly the same way as in any ordinary settling tank. In such a tank, however, deposits can only form on the bottom, while now in principle the combined surface area of all filtergrains is available. With a pore space p, one $m^3$ of spherical filtergrains with a diameter d has a gross surface area of $\frac{6}{d}$ (1-p) $m^2$. For a normal porosity of 0.4 and a grain diameter of 0.8 mm, this gross area amounts to no less than 4500 $m^2$ per $m^3$ of filtering material and 5400 $m^2$ per $m^2$ of filterbed when a depth of 1.2 m is chosen. Even when only a fraction of this area is effective (facing upward, not in contact with other grains and not exposed to scour) the area of deposition per $m^2$ of filterbed will easily attain a value of 300 $m^2$. The surface loading as quotient of the amount of water to be treated and the area of deposition will now be extremely small, with a filtration rate of $(1.5)10^{-3}$ m/sec not more than $(5)10^{-6}$ m/sec. Sedimentation efficiency is a function of the ratio between this surface loading and the settling velocity s of the suspended particle. For laminar settling Stokes gives

$$s = \frac{1}{18} \frac{g}{\nu} \frac{\Delta\rho}{\rho} d^2$$

in which d is the diameter of the spherical particle, $\rho$ and $\rho + \Delta\rho$ the mass density of water and suspended matter respectively, g the gravity constant (9.81 m/sec$^2$) and $\nu$ the kinematic viscosity of the fluid. For water at 10°C, $\nu = (1.31)10^{-6}$ $m^2$/sec, giving with $\frac{\Delta\rho}{\rho} \simeq 0.1$ for suspended particles containing 95% adsorbed water

$$s = (0.0416)10^6\ d^2.$$

More or less complete removal is obtained for particles with a settling velocity in excess of the overflow rate, in the case under consideration for

$$(0.0416)10^6d^2 > (5)10^{-6} \qquad \text{or}$$

$$d > (11)10^{-6}\ \text{m} = 11\ \mu\text{m}$$

Smaller and lighter particles are only partly removed, although flocculation accompanying downward water movement will increase sedimentation efficiency with depth. Truely colloïdal matter, however, cannot be extracted in this way. As filtration continues and settled out material decreases effective pore openings, the real velocity of downward water movement will increase. This exposes these deposits to scour, either preventing further sedimentation (as found by Ives) or even picking up settled out material and carrying it to greater depth in the filterbed (as advocated by Mintz). As this bed has a limited thickness only, ultimately suspended matter will appear in the effluent. The filter must now be taken out of service for backwashing, to restore its purifying capacity.

(c) Without any doubt, adsorption is the most important purifying action in rapid filtration, removing finely divided suspended matter as well as colloïdal and molecular dissolved impurities. The forces of adsorption, however, exert their influence over extremely short distances only, not more than 0.01 - 1 μm, while the water film surrounding the filter grains has a much greater thickness. In the example quoted above, filtering material of 0.8 mm grainsize and a porosity of 40% was assumed. Spreading the 0.4 $m^3$ of pore water per $m^3$ of filtering material over the combined surface area of the grains at 4500 $m^2$ gives an average film thickness of no less than 90 μm, which value is moreover large compared to the size of the particles to be removed. This means that purification by adsorption is only possible after another mechanism has brought the impurities to be removed in the immediate vicinity of the filtergrain surfaces. Many of these transport mechanisms are present in the flowing interstitial water, as most important of which may be mentioned gravity, inertia, diffusion, hydrodynamic forces and turbulence. Gravity tries to move particles with a greater mass density than water vertically downward. Larger particles are thus able to settle on the filtergrains, while smaller particles may be brought in the immediate vicinity of the grain surfaces, after which the attractive forces of adsorption are

able to extract them from the flowing liquid. Inertia induces particles heavier than water to keep as much as possible their original direction of motion. When now the flowlines curve around the filter grains, this results in a crossing of the flowlines by the particles, bringing them at or near the grain surfaces. This centrifugal action is again more pronounced when the particles are heavier, when the difference between their mass density and that of the surrounding fluid is greater and when particle sizes are larger. Diffusion is the random motion of particles caused by collision with surrounding molecules. When by adsorption a concentration gradient is produced, this Brownian motion transports particles towards the grain surfaces, easier as the particles are of smaller weight, that is when their mass density differs less from that of the surrounding fluid and their sizes are smaller. Particles larger than 2 μm are practically not affected. The movement of water through the pores of a rapid filterbed mostly occurs under streamline flow conditions. Even with laminar flow, however, suspended particles may move across the flowlines when the resultant of the forces exerted by the surrounding water does not pass through their centre of gravity. This transverse movement even reaches large proportions when turbulent flow conditions are present, as sometimes is the case when filtering water at very high rates through beds of coarse, broken material. Again this transport mechanism is more effective as the particles have a smaller submerged weight, by smaller dimensions or by a smaller difference in mass density compared to water.
Adsorption proper has many faces, the simplest of which is interception after the particle has been brought to a distance equal to half its size from the grain surface with subsequent adherence to the sticky gelatinous coating formed on the filtering material by previously deposited bacteria and colloïdal matter. Much more important in the meanwhile is the active promotion of this adsorption by the physical attraction between two particles of matter (London - Van der Waals' forces) and by the electrostatical attraction between opposite electrical charges (Coulomb forces). Mass attraction is present always and everywhere, but its magnitude decreases with the 6th power of the distance between centres, making its influence negligeable at a distance larger than about 0.01 μm from the grain surface. Electrostatical forces are inverse proportional to the second power of the distance and their influence consequently reaches deeper into the body of the passing liquid, up to and sometimes above 1 μm from the surface of the grains. On the other hand, attraction now only occurs when the filter grain and the particle carry unlike potentials. Like potentials result in mutual repulsion, creating a barrier to adhesion which can only be broken through

when the transport mechanism has given the particle sufficient kinetic energy of approach. By the nature of its crystalline structure, clean quartz sand has (at normal pH) a negative charge and is thus able to adsorb positively charged particles, in the form of suspended or colloïdal matter such as crystals of carbonates, flocs of iron- and aluminium oxide hydrates, etc, as well as cations of iron, manganese, aluminium and so on. Colloïdal matter of organic origin, bacteria included, mostly has a negative charge. They are consequently not attracted and indeed when a filter with clean sand is first taken into service, such impurities are practically not removed. When during the process of filtration positively charged particles are attached to the filter grains, however, the over-all potential decreases, allowing adsorption by other mechanisms. So much positive charges may even accumulate on some parts of the filtergrain surface, that here oversaturation occurs, by which locally the charge of the coated particle reverses and becomes positive. After this primary adsorption, secondary adsorption is able to remove negatively charged particles, as well suspended or colloïdal matter of animal and vegetable origin as truely dissolved impurities, anions as $NO_3^-$, $PO_4^{3-}$ and so on. When this secondary adsorption leads to oversaturation, the charge becomes again negative, allowing the adsorption of positive charges and so on. This process of reversing potentials takes place continuously and simultaneously, each area of a single grain surface perpetually changing its electric charge. Every time, however, the magnitude of the charge decreases, lowering the forces of adsorption and the efficiency of filtration. More impurities in the raw water will pass the filterbed, deteriorating effluent quality. Ultimately backwashing of the filter is necessary to restore the purifying capacity of its bed.

In case removal of negatively charged particles is of primary importance, clean sand as may be obtained by breaking solid rock should not be used. Natural sands are now better suited, as these have always picked up some positive charges from the groundwater flowing through them, shortening the breaking-in period after the filter has first been taken into service. If desired, the potential of the sand grain surface may even be reversed from the beginning, by coating the grain with a solid layer of cement or with a liquid layer of cationic polymers. From this description it will be clear that for deferrisation broken material is advantageous, while the potential on the grain surface may further be augmented by a prior application of anionic polymers. Especially with deferrisation in the meanwhile, next to the electrostatical potential mentioned above, the electrokinetical potential is of great importance. This potential is created when with high-rate filtration ions from the sand grain surfaces are dragged away by the flowing liquid, in this way increasing the charge of the particle. In some exceptional cases finally it may be desirable to decrease the rate of adsorption by electrical forces so as to obtain a deeper penetration of the impurities from the water into the filterbed, resulting in a slower increase of filter resistance and longer filterruns. This may be obtained by adding for instance polyphosphates to the water to be treated, raising the potential of the particles to be removed so high that existing particle deposits repel approaching particles, forcing them to travel to greater depths of the filterbed, where clean surfaces are still available for deposition.

(d) Chemical activity is the process by which dissolved impurities are either broken down into simpler, less harmfull substances, or converted into insoluble compounds after which straining, sedimentation and adsorption may remove them from the flowing water. In the presence of oxygen, organic matter can be degraded aerobically. Going out from the average composition, this reaction may qualitatively be represented as

$$C_5\,H_7\,O_2\,N + 5O_2 \longrightarrow H_2O + 4CO_2 + NH_4^{\,+} + HCO_3^{\,-}$$

requiring 1.4 g oxygen and producing 0.16 g of ammonia per g of organic matter. The carbon dioxide thus formed usually stays in solution, to be discharged with the effluent, but the ammonia is further oxydised with the help of bacteria, with **nitrosomonas** to nitrite

$$NH_4^{\,+} + \frac{3}{2}\,O_2 \longrightarrow H_2O + NO_2^{\,-} + 2H^+$$

and with nitrobacter to nitrate

$$NO_2^{\,-} + \frac{1}{2}\,O_2 \longrightarrow NO_3^{\,-}$$

**Together with**

$$H^+ + HCO_3^- \longrightarrow H_2O + CO_2$$

**this gives as over-all reaction**

$$C_5\,H_7\,O_2\,N + 7O_2 \longrightarrow 3\,H_2O + 5\,CO_2 + NO_3^{\,-} + H^+$$

increasing oxygen requirements to 2.0 g per g of organic matter, while for complete oxydation of 1 g ammonia present in the raw water no less than 3.6 g of oxygen is necessary.

Oxygen requirements are much less with deferrisation, converting the soluble ferrous compounds into insoluble ferric oxide-hydrates. When bicartonate is present, as it mostly is, the reactions are

$$4Fe^{++} + O_2 + (2n + 4)\ H_2O \longrightarrow 2\ Fe_2\ O_3\ .\ n\ H_2O + 8H^+$$

$$8H^+ + 8\ HCO_3^- \longrightarrow 8\ H_2O + 8\ CO_2$$

together

$$4Fe^{++} + O_2 + (2n - 4)H_2O + 8\ HCO_3^- \longrightarrow 2\ Fe_2\ O_3\ .\ nH_2O + 8\ CO_2$$

consuming only 0.14 g of oxygen per g of iron.For the removal of manganese the reactions read

$$4Mn^{++} + (2x + y - 2)O_2 + (2y + 4z + 4)H_2O \longrightarrow$$

$$4MnO_x\ (OH)_y\ (H_2O)_z + 8H^+$$

$$8H^+ + 8\ HCO_3^- \longrightarrow 8\ H_2O + 8\ CO_2$$

$$4Mn^{++} + (2x + y - 2)O_2 + (2y + 4z - 4)H_2O + 8\ HCO_3^- \longrightarrow$$

$$4MnO_x\ (OH)_y\ (H_2O)_z + 8\ CO_2$$

With the maximum possible value of $(2x + y)$ equal to 4, the coefficient $(2x + y - 2)$ is never more than 2, limiting oxygen requirements to 0.29 g per g of manganese, corresponding with the reaction

$$2Mn^{++} + O_2 + 4\ HCO_3^- \longrightarrow 2\ MnO_2 + 2\ H_2O + 4\ CO_2$$

by which manganous components are converted into manganese dioxyde.

Pure chemical reactions in the meanwhile are an exception. Some require the catalytic action of previously formed reaction products (e.g. with demanganization) and many the intervention of bacteria (e.g. of Nitrosomonas for the conversion of ammonia to nitrite and of Nitrobacter for the subsequent conversion of nitrite into nitrate). Both circumstances mean, however, that the chemical or bio-chemical reactions only take place on the surface of the filtergrains, where the catalytic agent is present and/or the necessary bacteria abound. Previous adsorption is thus a prerequisite for these removal mechanisms.

(e) Biological activity finally is the action of micro-organisms, living on and in the filterbed. During the breaking-in period, bacteria naturally present in the raw water or purposely added to it, are adsorped on the filtergrains, where they multiply selectively, using as food the inorganic or organic matter deposited here. This food is partly oxydised to provide the energy these bacteria need for their living processes (dissimilation) and partly converted into cell material for their growth (assimilation), thus transforming colloïdal and molecular dissolved impurities into living particulate matter. The dissimilation products are carried on by the water to be used again at greater depths by other bacteria. In this way the organic matter is gradually broken down (e.g. ammonia $\longrightarrow$ nitrite $\longrightarrow$ nitrate) and finally converted into rather innocent inorganic compounds such as water, carbon dioxide, nitrates, phosphates, etc (mineralization), mostly to be discharged with the filter effluent. With the limited amount of food supplied by the inflowing raw water, only a restricted bacterial population can be maintained and the growth (assimilation) mentioned above is therefore accompanied by an equivalent die-away. The deceased bacteria are partly flushed away during backwashing, partly broken down in the same way as described above, by which all degradable organic matter in the raw water is finally converted into mineral constituents. The raw water to be treated in the meanwhile not only brings innocent and useful bacteria to the filter, but may also contain E.coli and even pathogens. Part of these organisms will be transferred from the flowing water to the filtergrain surfaces by straining, sedimentation and adsorption. After adherence, their doom is sealed. For intestinal bacteria, the water environmnent is decidedly an unhealthy place, where the temperature is too low and insufficient organic matter of animal origin is available to suit their living requirements with starvation as ultimate result. Bacteria which escape attachment, however, will pass the filterbed unimpaired, the detention time of a few minutes only being too small for any antagonistic action. Rapid filters are consequently unable to produce a water safe in bacteriological respect, the reduction in E.coli content being a factor of 2 to 10 only, making preceding coagulation, subsequent slow sand filtration or post-chlorination a necessity for this purpose.

## 2.2. Filtration results

The description of the purification processes accompanying rapid filtration, as given in the preceding section, certainly promotes understanding. It fails, however, in giving data about the filtration rate to be applied and about the thickness and grain size distribution of the filterbed to be used, while next to this the increase in filter resistance remains unknown. Such data can only be obtained by operating a pilot plant, actually submitting the raw water available to rapid filtration and really measuring the improvement in water quality and the accompanying clogging of the filterbed that will thus occur. Mostly such a pilot plant is equipped with a number of experimental filters, allowing several investigations (with various grain sizes for instance) to be carried out simultaneously, while with regard to seasonal changes in raw water quality the experiments must be carried out over long periods, varying from 3 months to a full year. Very conscientious research workers even operate these filters in pairs (fig. 2.3) to increase the reliability of the results. Schematically, the construction of the experimental filters is shown in fig. 2.2, mainly consisting of a cylindrical container, usually made of clear plastic (polymethylmetacrylate as for instance perspex made by ICI), with a heigt of 3-4 m and an inner diameter of 0.1 - 0.3 m and sometimes even larger.

Over the length of the container a number of connections are fitted, usually spiraling downwards, by means of which water samples can be taken and water pressures can be measured above and at different depths below the top of the filterbed. At the lower end the cylinder is provided with a perforated or porous plate, acting as filterbottom, above which the filtering material to be applied is present to a certain depth.

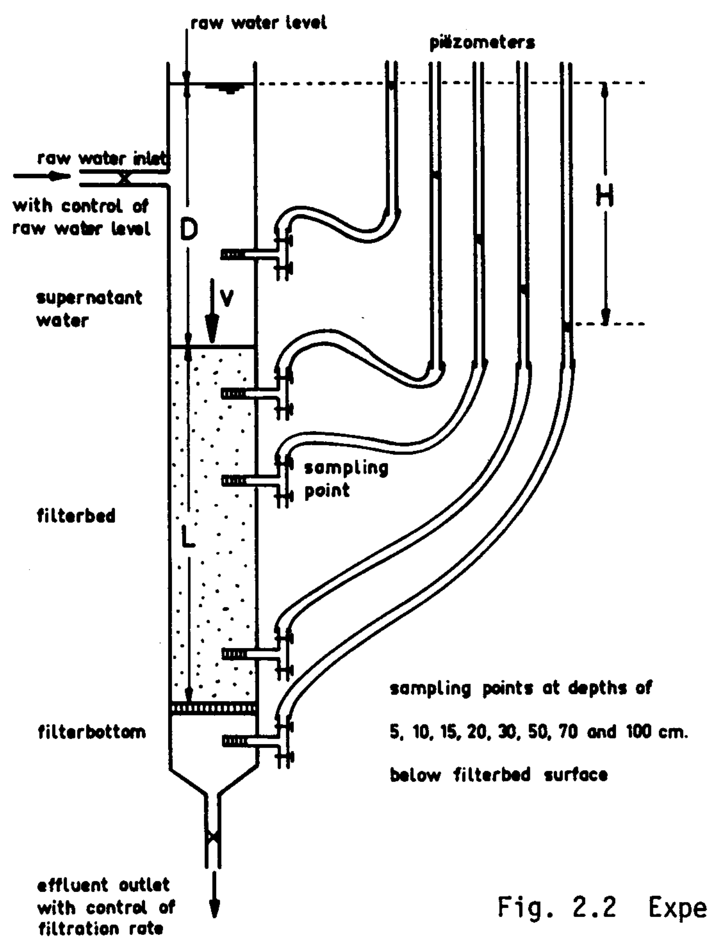



Fig. 2.2 Experimental filter

Testing starts by slowly charging the filter from below with clear water, allowing the air from the pores of the filterbed to escape upwards. By opening the inlet valve, raw water is admitted to the top of the filter, while opening the outlet valve starts the filtration proper. The inlet and the outlet moreover are provided with controls to maintain the desired depth of water on top of the filter and the rate of filtration at the chosen values. As filtration goes on and clogging occurs, the resistance of the filterbed against downward water movement increases. To keep the filtration rate constant, the outlet control gradually opens. When this control is fully open, the filterrun is broken off, the filter cleaned by backwashing and the procedure repeated. As many filterruns are made as is necessary to obtain steady state conditions, without changes in effluent quality by deposits formed on the surface of the filtergrains. With most

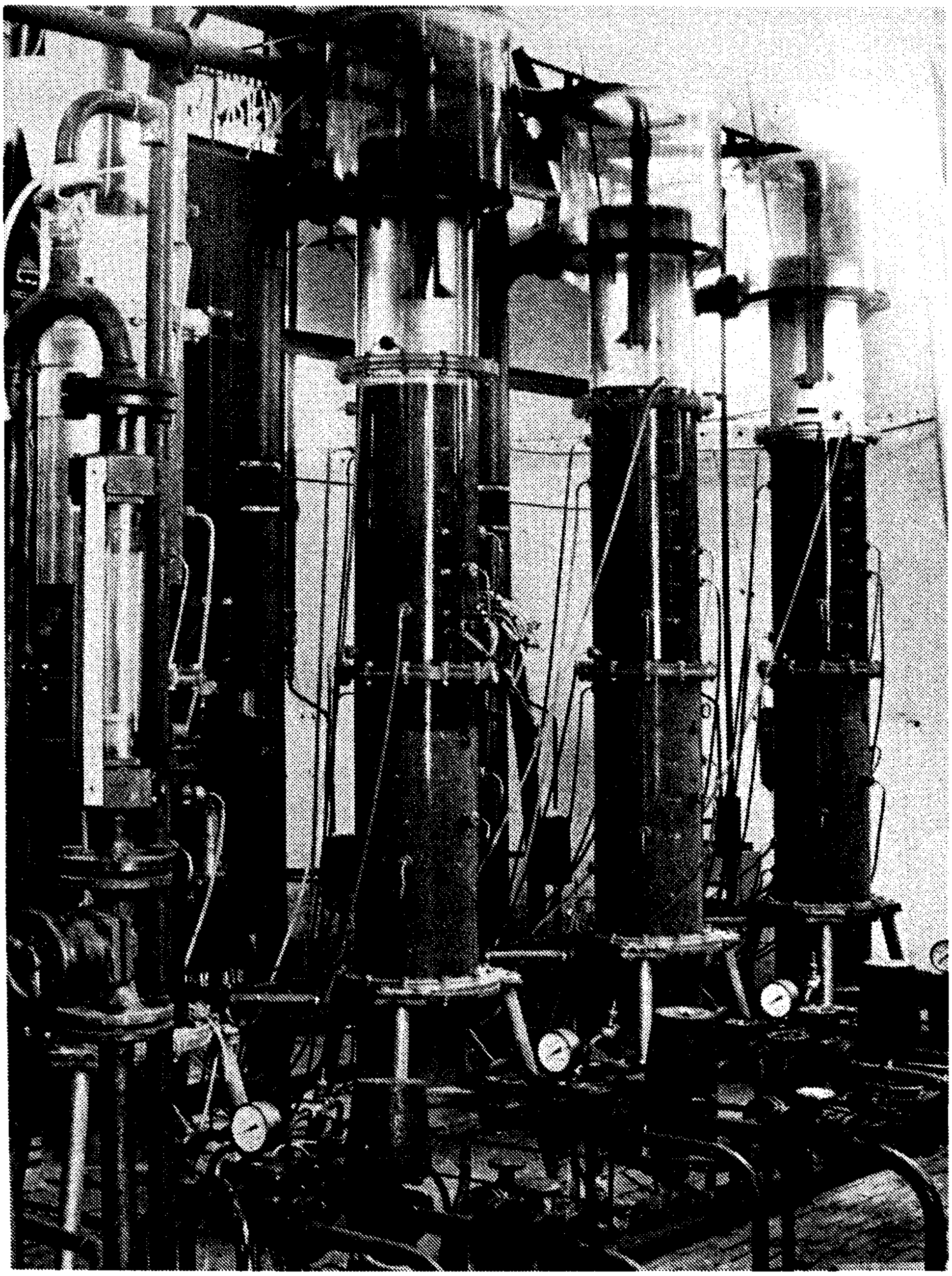


Fig. 2.3 Inside view of a pilot plant

surface water sources, the raw water quality shows a marked seasonal variation, if only with regard to water temperature and the tests must be carried on over a full year to take these fluctuations into account. After the chosen conditions have been fully investigated, a new series of test runs may be initiated, with a different filtration rate, another thickness, grain-size or even grain-size distribution of the filtering material, etc.

During the experiments mentioned above, water pressures are recorded and water samples are taken at various depths. The samples may be analysed for suspended and colloidal matter, turbidity, colour, iron, manganese, aluminium, oxygen, biochemical and chemical oxygen demand, number of bacteria or any other index that is affected by rapid filtration. Generally speaking, the water quality will improve as the water passes deeper into the filterbed and more impurities are removed from it. As filtration goes on, however, deposition of these impurities occur at greater depths in the filterbed, deteriorating water quality at the successive sampling points. As a consequence, water quality depends on two factors, on the depth below the top of the filterbed and on the time elapsed after the filterrun started. The same holds true for the pressure loss, being larger at greater depth and increasing with time. Schematically these time-depth relationships are shown in fig. 2.4, comprising all observations made during testing.

Without any doubt, fig. 2.4 gives the most complete information about the time-dependent results that can be obtained by submitting a raw water of constant quality to rapid filtration at a specified rate through filterbeds of variable thickness but unchanging composition. Especially with regard to water quality, however, the results arrived at are rather unreliable. In order not to disturb the downward water movement too much, only small amounts of water may be withdrawn from the various tapping points and even when sufficient for analysis, the results gained need not to be representative for the time and depth at which the samples are taken. Reproducable results can only be obtained by operating a number of experimental filters in parallel, each provided with a different depth of filtering material, for instance 0.6, 0.8, 1.0 and 1.2 m. Needless to say that this means an enormous amount of work. When next to this the influence of filtration rate, type of filtering material, grain-size and grain-size distribution, etc. needs investigation, the amount of experimental work to be done even increases by some orders of magnitude. This is the reason that in actual practice a rather haphazard way of investigation is followed, using as much as possible the experience and intuition of the operator in charge.

For the method of experimentation mentioned above, it is wise to recall that the various parameters of rapid filtration have different economic impacts. Commonly the choice of a finer or coarser grain size has no in-

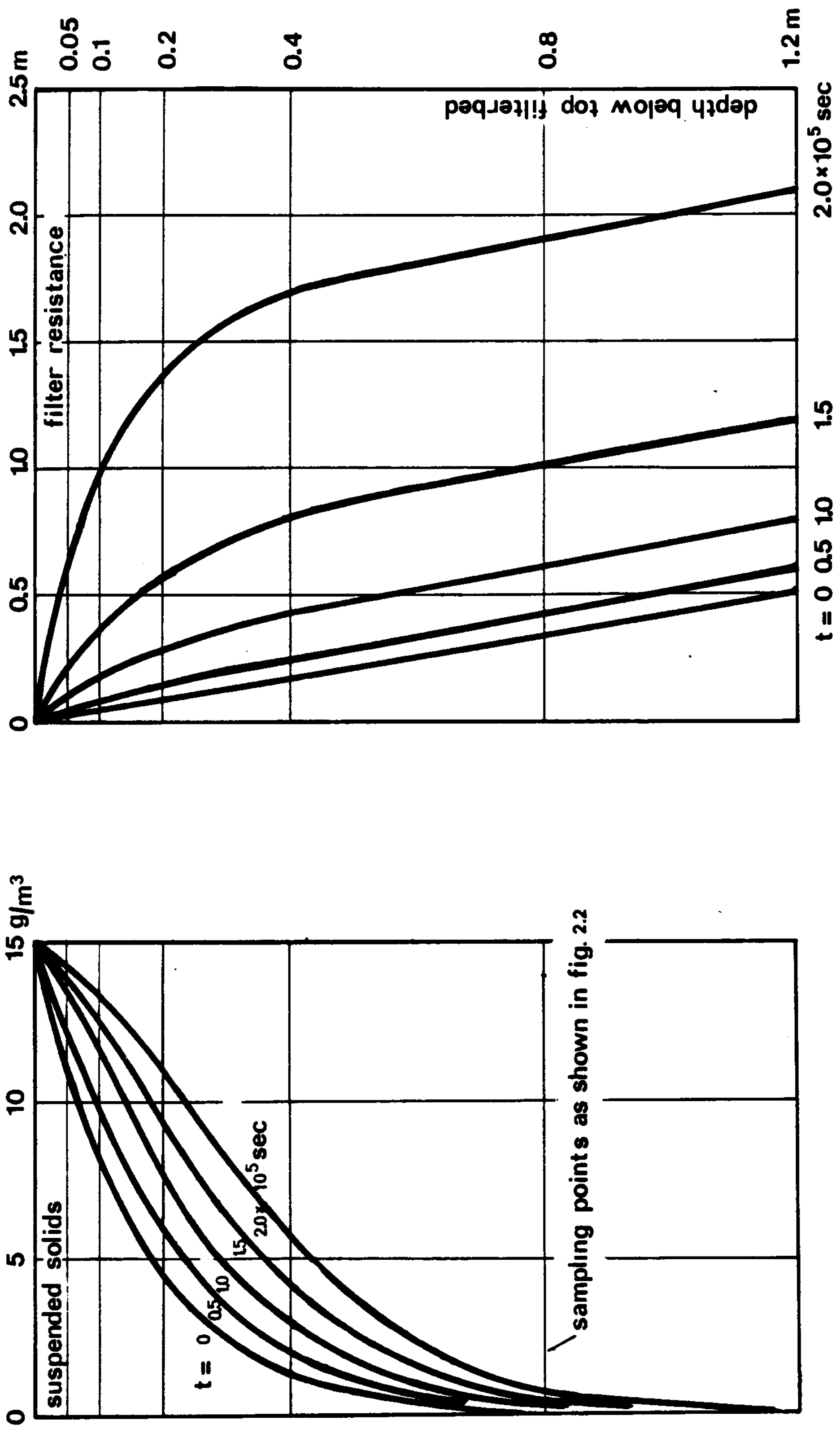



Fig. 2.4  Results of an experimental filterrun

fluence at all on the cost of construction. An increase in the depth of supernatant water or the thickness of the filterbed raise these costs only to a limited extent, but a decrease in filtration rate has a large influence, as the total filterbed area to be applied is inverse proportional to this value. Indeed with fine grained filtering materials, say 0.6 mm, and a small penetration of impurities into the filterbed (surface filtration) an excellent effluent quality can be obtained, but to prevent rapid clogging low filtration rates must be used, greatly increasing the cost of filtration. To allow higher rates of filtration, coarse grains with a greater depth of penetration of impurities into the filterbed (deep bed filtration) must be applied. Notwithstanding the use of greater filterbed thicknesses,however,a deterioration of effluent quality with time must now be expected. This measn that the results of rapid filtration can be expressed in two parameters

a. the length of filterrun $T_q$ during which the effluent quality satisfies the set standard;
b. the length of filterrun $T_r$ during which the filter resistance is less than the maximum allowable value.

Both lengths of filterrun depend on two sets of variables

c. the physical, chemical and bacteriological composition of the raw water to be treated;
d. the filtration rate and the composition of the filterbed, the latter factor tc be subdivided into the bed thickness on the one hand and on the other hand the grainsize, the grainsize distribution and the composition of the filtering material.

The quality of the raw water may show seasonal fluctuations and may be altered by pre-treatment, but is otherwise a fact, meaning that the desired results in terms of $T_q$ and $T_r$ can only be obtained by a judicious combination of the factors mentioned above under d. In practice a continuous monitoring of the quality of the effluent emerging from the various filtering units is impossible and $T_q$ must therefore be larger than $T_r$ under all operating conditions. For a low cost solution moreover, $T_q$ and $T_r$ should not differ too much and for normal operating conditions be equal to 1 or 1.5 days, say $(1)10^5$s.

To get some idea how to start the experiments, plants treating roughly the same type of water should be visited and the operators asked about their experience and their views on changes in design to obtain better results and lower construction costs. Many people hate to admit mistakes and great tact and perseverance is therefore required to get behind the truth. With a completely new source of water about which no experience whatsoever is available, the experimental filters of fig. 2.2 with inside diameters of 0.1-0.15 m should be used. As already indicated the results obtained are not very reliable, but still adequate to start the final experiments. As an example of the way to run a pilot plant along the lines described above, a case will be studied where rapid filtration is used for clarification only. The raw water is assumed to have a constant suspended matter content of 15 $g/m^3$, while the effluent standard is set at 0.5 $g/m^3$. As filtering material various grades of sand are available, each composed of spherical grains with one and the same diameter d. The maximum allowable filter resistance H is provisionally set at 1.5 m water column. With regard to the required improvement in water quality, a reduction in suspended matter content by a factor no less than 30, the operator decides for a modest filtration rate v of $(2)10^{-3}$ m/s and not to coarse filtering materials. The pilot plant is equipped with 3 (sets) of experimental filters and the investigations are therefore started with grain sizes of 0.7, 0.8 and 0.9 mm, at equal depths of 0.8 m. After a breaking-in period of a few weeks, the results are fairly constant. They are shown graphically in fig. 2.5, from which the following table can be composed

| d = 0.7 | 0.8 | 0.9 | mm |
|---|---|---|---|
| $T_q$ = 1.96 | 1.26 | 0.36 | x $10^5$ sec |
| $T_r$ = 1.34 | 2.00 | 2.84 | x $10^5$ sec |

From these data the following conclusions can be drawn

a. the finest filtering material, d = 0.7 mm, satisfies the requirements $T_q > T_r$ and $T_q$, $T_r$ > (1) $10^5$s. Both lengths of filterrun, however, are rather great, meaning that a higher rate of filtration could be considered;

b. with the middle grainsize of 0.8 mm both lengths of filterrun are again fairly large, but this set-up has the serious disadvantage that effluent quality deteriorates below the set standard long before the filter resis-

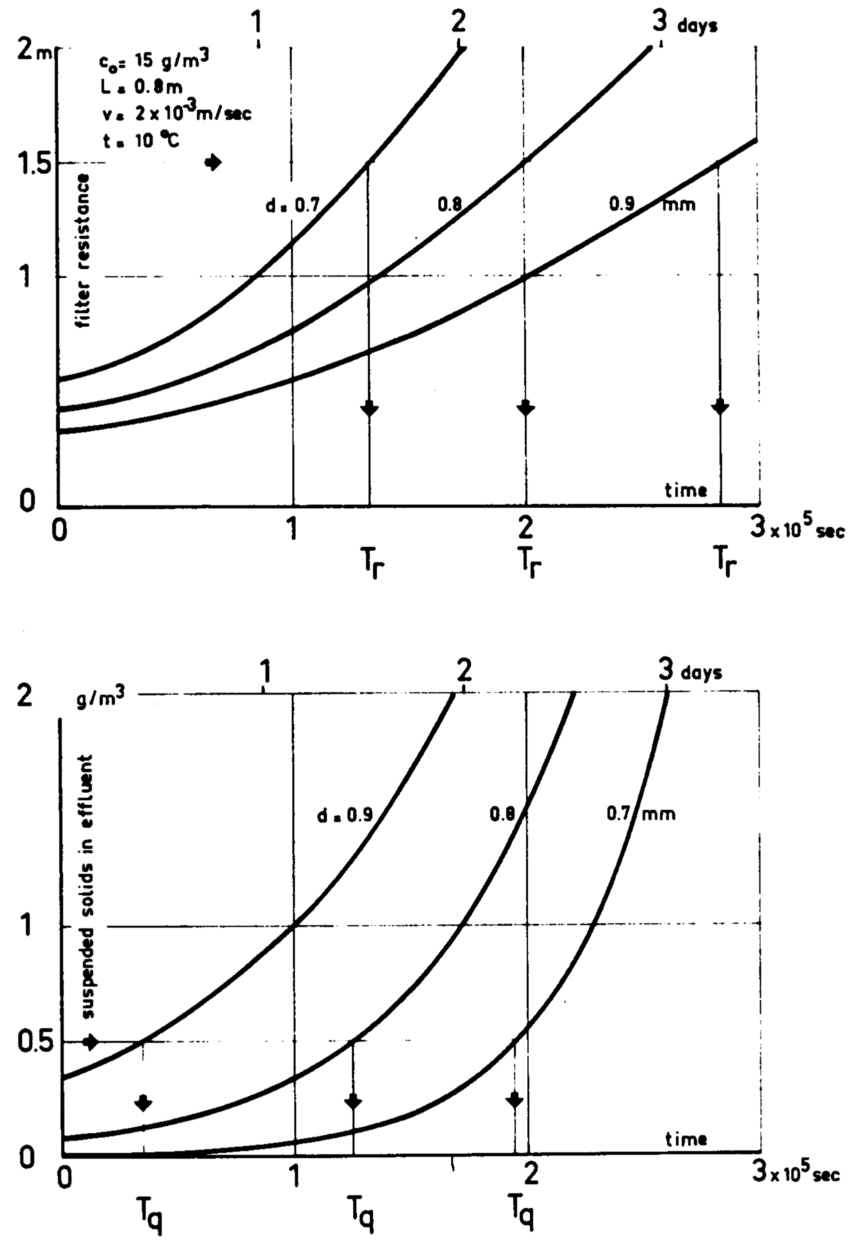



Fig. 2.5 Results of experimental filterruns using different grain sizes

tance reaches its maximum allowable value. Effluent quality is much more difficult to measure than filter resistance and for the two lengths of filterrun, a reversed sequence is therefore highly preferable. This could easily be obtained by increasing the filterbed thickness by which $T_q$ will be greatly enlarged and $T_r$ slightly reduced. Both lengths of filterrun will now be so long that again a higher rate of filtration could be contemplated;

c. with a grainsize of 0.9 mm the length of filterrun $T_q$ is much too short and the length of filterrun $T_r$ much too large. To obtain acceptable results, a large increase in filterbed thickness is now required.

Going out from the conclusions mentioned above, the operator decides to continue the experiments with a filtration rate of $(3)10^{-3}$ m/sec, a grain size of 0.8 mm and bed thicknesses L of 1.0, 1.2 and 1.5 m respectively. The results obtained are shown graphically in fig. 2.6, from which the table given below can be composed

| | | | |
|---|---|---|---|
| L = 1.0 | 1.2 | 1.5 | m |
| $T_q$ = 0.70 | 1.27 | 2.12 | x $10^5$ sec |
| $T_r$ = 0.95 | 0.81 | 0.55 | x $10^5$ sec |

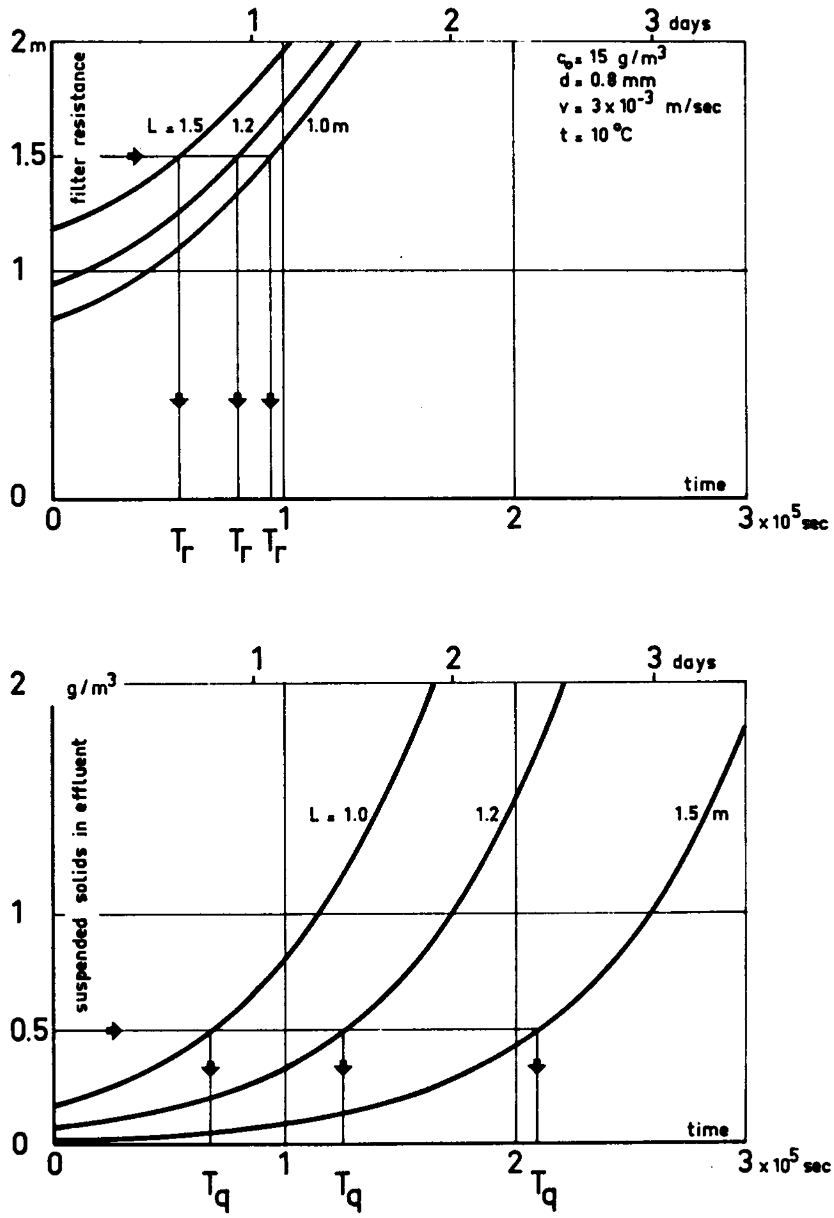



Fig. 2.6 Results of experimental filterruns using various bed thicknesses

According to these data, a filterbed thickness of 1.2 m nearly satisfies all requirements. Some economy in the cost of construction and operation, however, may still be obtained by considering that in practice a length of filterrun $T_r$ of about 1 day or $(0.9)10^5$ sec is usually sufficient. The filter resistances occurring at this moment are therefore taken from fig. 2.6 and plotted in fig. 2.7 as function of the filterbed thickness. As a factor of safety, the length of filterrun $T_q$ as determined by effluent quality must be longer, for instance $(1.0)10^5$ sec. Effluent turbidities at this moment are again read from fig. 2.6 and also plotted in fig. 2.7. With the effluent quality set at a suspended load less than 0.5 $g/m^3$, figure 2.7 finally gives a required filterbed thickness of 1.10 m and a filter resistance not surpassing a value of 1.52 m. Both values in the meanwhile are still fairly low, indicating that also higher filtration rates of say (3.5) $10^{-3}$ m/sec are possible. This certainly has the advantage of a smaller filterbed area, but it requires a greater filterbed thickness as well as a greater depth of supernatant water to allow a larger filter resistance, resulting perhaps in a less economical solution. Optimization of filter design, however,asks for such a multitude of data, that experiments alone are seldom sufficient. This is only possible with the help of a filtration theory, allowing interpolation and extrapolation of the experimental results obtained, as will be explained in the next sections.

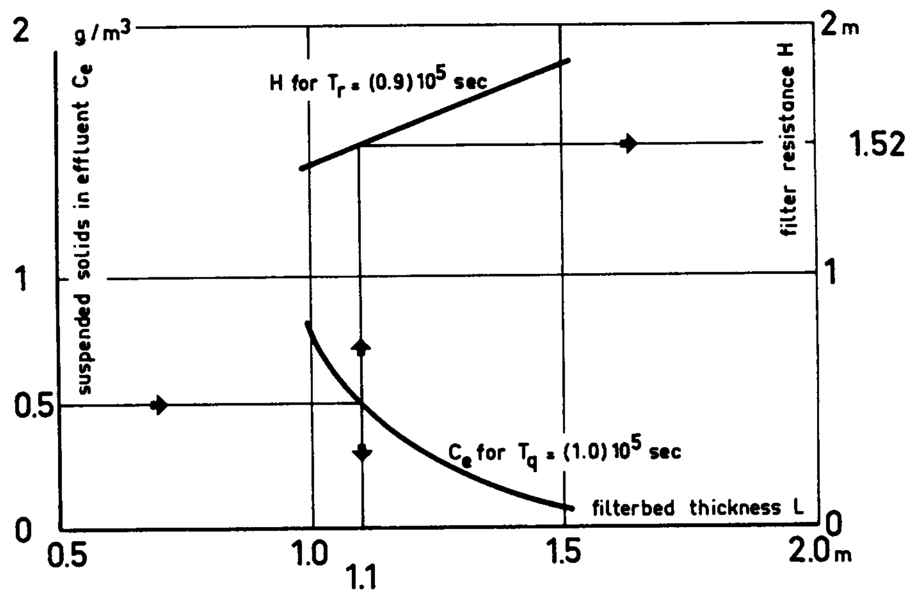



Fig. 2.7 Selection of bed thickness and filterresistance from the filtration results of fig. 2.6

## 2.3. Mathematical theory of filtration; effluent quality

To simplify calculations, the mathematical theory of filtration assumes the clean filterbed with thickness L to be composed of spherical grains with a uniform diameter $d_o$ and a porosity $p_o$. During filtration impurities from the raw water are transferred to the filtergrain surfaces. This means on one hand that the amount $c_o$ of impurities carried by the raw water is decreased to c at a depth y below the filterbed surface (fig. 2.8) and to $c_e$ in the effluent, while on the other hand the grain size in the filterbed with unchanged depth L increases from $d_o$ to d and the pore space decreases from $p_o$ to p. With v as constant rate of filtration,the real velocities of flow inside the pores of the filterbed thus increase from $v/p_o$ to $v/p$.

For every problem of mechanics, two equations are available, the equation of motion and the equation of continuity. For the concentration c of impurities carried by the flowing interstitual water it may be assumed that the decrease is proportional to the concentration still present (Fick's law). This gives as equation of motion.

$$-\frac{\partial c}{\partial y} = \lambda c$$

with $\lambda$ as proportionality coefficient , the so-called coëfficient of filtration. With the notations of fig. 2.8 the equation of continuity may be written as

in = out + storage + deposition

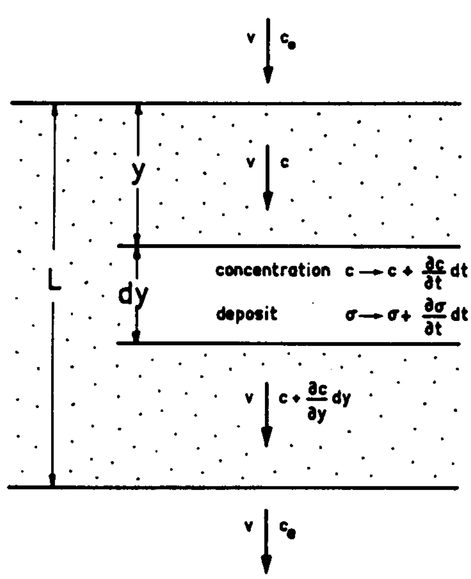



Fig. 2.8 Improvement in water quality

$$v\ c\ dt = v(c + \frac{\partial c}{\partial y}\,dy)dt + p\,\frac{\partial c}{\partial t}\,dt\ dy + \frac{\partial \sigma}{\partial t}\,dt\ dy \text{ simplified}$$

$$\frac{\partial \sigma}{\partial t} = -\,v\,\frac{\partial c}{\partial y} - p\,\frac{\partial c}{\partial t}$$

The concentration changes strongly with depth but little with time, allowing a further simplification to

$$\frac{\partial \sigma}{\partial t} = -\,v\,\frac{\partial c}{\partial y}$$

with $\sigma$ as gravimetric concentration of impurities. The reduction in pore space, however, is determined by the volumetric concentration.

$$\sigma_v = \frac{\sigma}{\rho_d}$$

with $\rho_d$ as mass density of these deposits.

To solve the set of equations derived above, the value of $\lambda$ must be known. Theoretical considerations are only of little help and $\lambda$ has to be determined from filtration experiments, using the set-up of fig. 2.2 with which $\frac{\partial c}{\partial y}$ at various combinations of y and t can be determined

$$\lambda = -\,\frac{1}{c}\,\frac{\partial c}{\partial y}$$

When plotted against $\sigma_v$

$$\sigma_v = -\frac{v}{\rho_d}\int_{o}^{t}\frac{\partial c}{\partial y}\,dt$$

a graph as shown in fig. 2.9 is found. The initial rise in the value of $\lambda$ is due to an increase in straining efficiency accompanying a narrowing of pores by clogging and due to a better adherence of suspended particles to

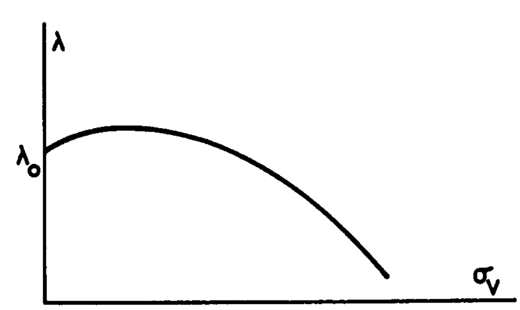



Fig. 2.9 Change of the filtration coefficient $\lambda$ during operation

the sticky gelatinous coating formed on the filter grain surfaces by previously deposited bacteria and colloïdal matter. As clogging proceeds, however, the interstitial velocities rise from $v/p_o$ to $v/(p_o - \sigma_v)$, hindering deposition and lowering the value of $\lambda$. Ultimately scour will prevent a further transfer of impurities from the water to the filter grain surfaces. At this depth $\lambda$ drops to zero, shifting the burden of removal to greater depths and augmenting the concentration of impurities in the effluent. When the effluent quality no longer satisfies the chosen standard, the filter must be taken out of operation for back-washing.

More than 50 research workers have calculated $\lambda$ as function of $\sigma_v$, but due to variations in the composition of the water treated, many of them have found different results. Some are expressed in simple formulas, for instance

Ives and Diaper $\quad \lambda = \lambda_o(1-k_1\sigma_v^2)$

Iwasaki, O'Melia and Ali $\quad \lambda = \lambda_o(1+k_2\sigma_v)$

Lerk, Shektman $\quad \lambda = \lambda_o(1-\frac{\sigma_v}{p_o})$

Mackrle(1), Deb $\quad \lambda = \lambda_o(1-\frac{\sigma_v}{p_o})^{n_1}$

Maroudas $\quad \lambda = \lambda_o(1-k_3\frac{\sigma_v}{p_o})$

while other formulas are more complicated

Adin and Rebhun $\quad \lambda = \lambda_o(1-\frac{\sigma_v}{p_o}) - k_4\frac{\sigma_v}{c_v}$

Ives(1) $\quad \lambda = \lambda_o(1 + k_5\sigma_v - k_6\frac{\sigma_v^2}{p_o-\sigma_v}$

Ives(2), Mohanka $\quad \lambda = \lambda_o(1 + k_7\frac{\sigma_v}{p_o})^{n_2}(1-\frac{\sigma_v}{p_o})^{n_3}(1-k_8\frac{\sigma_v}{p_o})^{n_4}$

Mackrle(2) $\quad \lambda = \lambda_o(1 + k_9\frac{\sigma_v}{p_o})^{n_5}(1-\frac{\sigma_v}{p_o})^{n_6}$

with $k_i$ as coefficients and $n_j$ as exponents to be determined experimentally.

The simple formulas mentioned above are applicable for the case under consideration or not, but when they are, equations can be derived for c and $\sigma_v$ as functions of y and t. The more complicated formulas can always be made to suit experimental results by adjusting the values of the various coefficients and exponents (in the formulas of Ives (2) and Mohanka together with $\lambda_o$ no less then 7), but to find the course of c and $\sigma_v$ with time and depth computer calculations are necessary, limiting the number of possible combinations of filtration rate, filterbed thickness, grain size and lengths of filter-run to be investigated.

For the subsequent calculations in this section, the theory of Maroudas will be followed, replacing $k_3$ by 1/n.

$$\lambda = \lambda_o\left(1 - \frac{\sigma_v}{n\, p_o}\right) \text{ with } n < 1,$$

meaning that purification stops when the pores are filled for a fraction n with deposits. This gives as basic differential equations.

removal $$-\frac{\partial c}{\partial y} = \lambda c = \lambda_o\left(1 - \frac{\sigma_v}{n\, p_o}\right) c$$

clogging $$-\frac{\partial c}{\partial y} = \frac{1}{v}\frac{\partial \sigma}{\partial t} = \frac{\rho_d}{v}\frac{\partial \sigma_v}{\partial t}$$

With the boundary conditions

$y = 0$, $c = c_o$ and $t = 0$. $\sigma_v = 0$, substituting moreover

$$\alpha = \frac{v\, c_o\, \lambda_o}{n\, \rho_d p_o}$$, these equations have as solution

$$c = c_o \frac{e^{\alpha t}}{e^{\lambda_o y} + e^{\alpha t} - 1}, \quad c_e = c_o \frac{e^{\alpha t}}{e^{\lambda_o L} + e^{\alpha t} - 1} \quad g/m^3$$

$$\sigma_v = n\, p_o \frac{e^{\alpha t} - 1}{e^{\lambda_o y} + e^{\alpha t} - 1}$$, with as average value

$$\bar{\sigma}_v = \frac{1}{L}\int_0^L \sigma_v\, dy = n\, p_o\left(1 - \frac{1}{\lambda_o L} \ln \frac{e^{\lambda_o L} + e^{\alpha t} - 1}{e^{\alpha t}}\right) \quad m^3/m^3$$

For actual results, comparing the effects of various rates of filtration and different grain sizes, the value of $\lambda_o$ (and $\alpha$) still needs to be known. The various research workers mentioned above have also determined this interrelationship. All results may be written as

$$\lambda_o \sim \frac{1}{\nu^a v^b d_o^c} \qquad \text{with } \nu \text{ als kinematic viscosity}$$

The values of a, b and c, however, differ greatly, depending on the composition of the raw water, on the most important purification processes described in section 2.1

| | a | b | c |
|---|---|---|---|
| Fair | 1 | 0 | 1.67 |
| Hall (1) | 0 | 0 | 2.5 |
| Hall (2) | 1 | 1 | 1 |
| Ison | -1.4 | 4 | 1.4 |
| Ives and Sholji | 2 | 1 | 1 |
| Iwasaki | 0 | 1 | 1 |
| Lerk | 1 | 1 | 3 |
| Ling | 0 | 0 | 1.5 |
| Mackrle | -0.5 | 1 | 2 |
| Maroudas | 0 | 1 | 0 |
| Mintz and Krishtul | 0 | 0.7 | 1.7 |
| Mohanka | 1 | 0.25 | 1.35 |
| Stanley | 0 | 1.56 | 2.46 |
| Stein | 0 | 0 | 3 |

Following Lerk's theory and fixing the proportionality constant at $(9)10^{-18}$ gives

$$\lambda_o = \frac{(9)10^{-18}}{\nu\, v\, d_o^3} \quad , \alpha = (9)10^{-18} \frac{c_o}{n\, \rho_d\, p_o \nu\, d_o^3}$$

and with the assumptions $n = 0.67$, $\rho_d = 50\ kg/m^3$ ( silt with 98% water), $p_o = 0.38$ and at $t = 10^oC$, $\nu = (1.31)\ 10^{-6}\ m^2/s$

$$\lambda_o = \frac{(6.87)10^{-12}}{v\, d_o^3} \; , \alpha = (5.40)10^{-13} \frac{c_o}{d_o^3}$$

From the formula for $c_e$, the required filterbed thickness L for a length of filterrun $T_q$ may now be calculated

$$c_e = c_o \frac{e^{\alpha T_q}}{e^{\lambda_o L} + e^{\alpha T_q} - 1} \quad \text{or}$$

$$L = \frac{1}{\lambda_o} \ln \left(1 + (\frac{c_o}{c_e} - 1)\, e^{\alpha T_q}\right)$$

Assuming as before $c_o$ = 15 g/m$^3$ (0.015 kg/m$^3$), $c_e$ = 0.5 g/m$^3$ and $T_q$ = $10^5$ s gives with the values of $\lambda_o$ and $\alpha$ calculated above

$$L = \frac{v\, d_o^{\,3}}{6.87}\, 10^{12} \ln \left( 1 + 29\, e^{(8.1)10^{-10}/d_o^{\,3}} \right)$$

For various values of v and $d_o$, the required depth L of the filterbed in the example under consideration is tabulated below

| | v = 2 | 3 | 4 | mm/s |
|---|---|---|---|---|
| $d_o$ = 0.7 mm | L = 0.57 | 0.86 | 1.14 | m |
| 0.8 | 0.74 | 1.11 | 1.48 | |
| 0.9 | 0.95 | 1.43 | 1.91 | |
| 1.0 | 1.22 | 1.83 | 2.44 | |

The combination of high filtration rates and small grain sizes look most attractive, but it may be that now filter resistances increase too fast with time. This will be investigated in next section.

## 2.4 Mathematical theory of filtration ; filter resistance

The friction losses accompanying the flow of water through the pipeline of fig. 2.10 with a hydraulic diameter D (equal to 4 times the ratio between the wetted cross-sectional area and the wetted perimeter)can be calculated with

$$I_o = \frac{H}{L} = f \frac{1}{D} \frac{v_p^2}{2g}$$

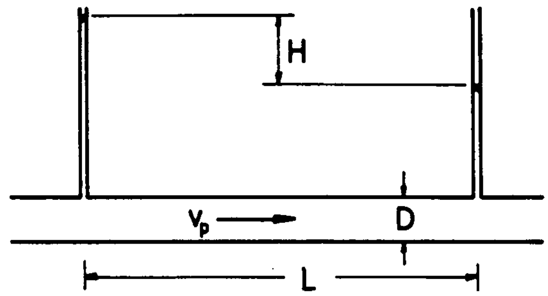



Fig. 2.10 Pipe flow

with f as friction factor and g as gravity constant. When the Reynolds number

$$Re = \frac{v_p D}{\nu} \text{ is less than 2000, the flow is laminar}$$

and f is given by

$$f = \frac{a}{Re} \text{ with the constant a varying between}$$

64 and 96, depending on the shape of the cross-sectional area. With the average value a = 80, substitution yields

$$I_o = 40 \frac{\nu}{g} \frac{v_p}{D^2}$$

To apply this formula for the flow of water through the clean filterbed of fig. 2.11, it will be assumed that the pores between the sand grains form capillaries, m per $m^2$ with length l ( > L), diameter $e_o$ and capacities v/m with v as rate of filtration. The pore velocity becomes

$$v_p = \frac{\frac{v}{m}}{\frac{\pi}{4} e_o^2} \quad \text{substituted together with}$$

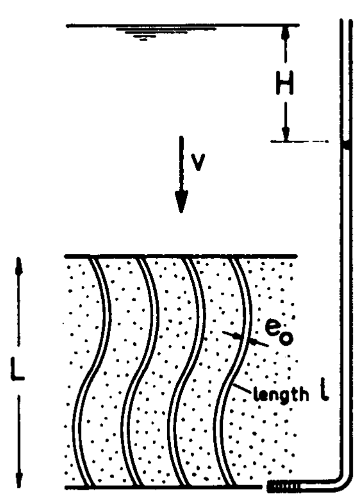



Fig. 2.11 Capillary filter model

$D = e_o$ gives

$$I'_o = \frac{H}{l} = \frac{160}{\pi} \frac{\nu}{g} \frac{v}{m e_o{}^4} \quad \text{and}$$

$$I_o = \frac{H}{L} = \frac{160}{\pi} \frac{\nu}{g} \frac{v}{m e_o{}^4} \frac{l}{L}$$

The volume of the pores equals the pore space $p_o L$

$$p_o L = m\, l\, \frac{\pi}{4} e_o{}^2$$

while their circumferential area in the same as the combined surface area of the grains. With $S_o$ as the area per $m^3$ of the clean filterbed

$$S_o L = m\, l\, \pi\, e_o \qquad \text{Together this gives}$$

$$e_o = \frac{4 p_o}{S_o} \quad , \quad m = \frac{S_o{}^2}{4\pi\, p_o} \frac{L}{l} \quad , \text{ substituted}$$

$$I_o = 2.5 \frac{\nu}{g} \frac{S_o{}^2}{p_o{}^3} \left(\frac{l}{L}\right)^2 v \quad \text{and with}$$

the tortuosity factor l/L estimated at $\sqrt{2}$

$$I_o = 5 \frac{\nu}{g} \frac{S_o{}^2}{p_o{}^3} v$$

which formula is fully confirmed by experiments all over the world.

When during filtration clogging occurs, the pore space will decrease from $p_o$ to p and the specific surface area from $S_o$ to S. This gives

$$I = 5 \frac{\nu}{g} \frac{S^2}{p^3} \quad \text{or}$$

$$I = I_o \left(\frac{p_o}{p}\right)^3 \left(\frac{S}{S_o}\right)^2$$

In the capillary model the pore diameter will decrease from $e_o$ to e, but the number and length of the pores remains the same, giving at any combination of time and depth

$$m = \frac{S_o^2}{4\pi\, p_o} \frac{L}{l} = \frac{S^2}{4\pi\, p} \frac{L}{l} \quad \text{or} \quad \left(\frac{S}{S_o}\right)^2 = \frac{p}{p_o}$$

Substituted

$$I = I_o \left(\frac{p_o}{p}\right)^2 \quad \text{and with } \sigma_v = p_o - p$$

$$I = I_o \left(\frac{p_o}{p_o - \sigma_v}\right)^2$$

According to last section

$$\sigma_v = n\, p_o \frac{e^{\alpha t} - 1}{e^{\lambda_o y} + e^{\alpha t} - 1}$$

meaning that clogging increases with time and is less at greater depth. The slope I is therefore not a constant and the total head loss H must be calculated from

$$H = \int_o^L I dy \quad \text{from which follows}$$

$$H = \frac{I_o}{\lambda_o} \left\{ \frac{\lambda_o L}{(1-n)^2} - \frac{n^2 \;(e^{\lambda_o L} - 1)(e^{\alpha t} - 1)}{(1-n)\left(e^{\lambda_o L} + (1-n)(e^{\alpha t} - 1)\right)\left((1-n)e^{\alpha t} + n\right)} - \right.$$

$$\left. - \frac{n(2-n)}{(1-n)^2} \ln \frac{e^{\lambda_o L} + (1-n)(e^{\alpha t} - 1)}{(1-n)e^{\alpha t} + n} \right\}$$

This formula looks horrible, but offers no difficulties whatsoever for a programmable pocket calculater. With the same assumptions as made before and the filterbed thicknesses as calculated at the end of last section, this gives with $T_r = (0.9)10^5$s

| | v = 2 | 3 | 4 mm/s |
|---|---|---|---|
| $d_o$ = 0.7 mm | H = 0.88 | 1.99 | 3.53 m |
| 0.8 | 0.68 | 1.53 | 2.71 |
| 0.9 | 0.59 | 1.32 | 2.35 |
| 1.0 | 0.55 | 1.23 | 2.19 |

In fig. 2.12 the values of L and H are plotted as function of v and $d_o$. Any combination satisfies the chosen requirements, examples of which are given in fig. 2.13 for a medium rate of filtration and a shallow filterbox and in fig. 2.14 for a high rate of filtration and a much greater depth of supernatant water and filtering material. In the upper part of these graphs

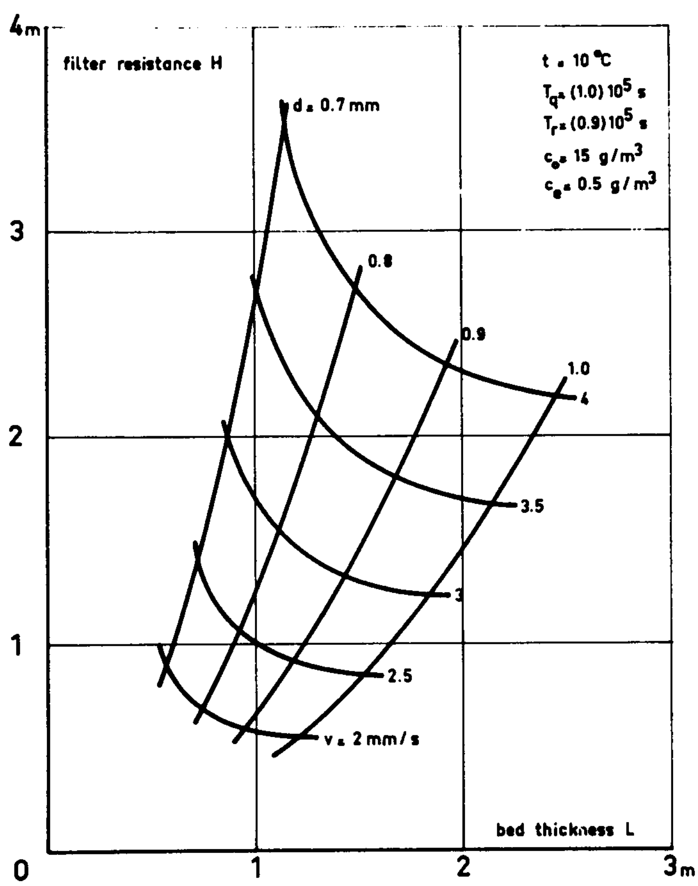



Fig. 2.12 Possibilities for rapid filtration as calculated in sections 2.3 and 2.4

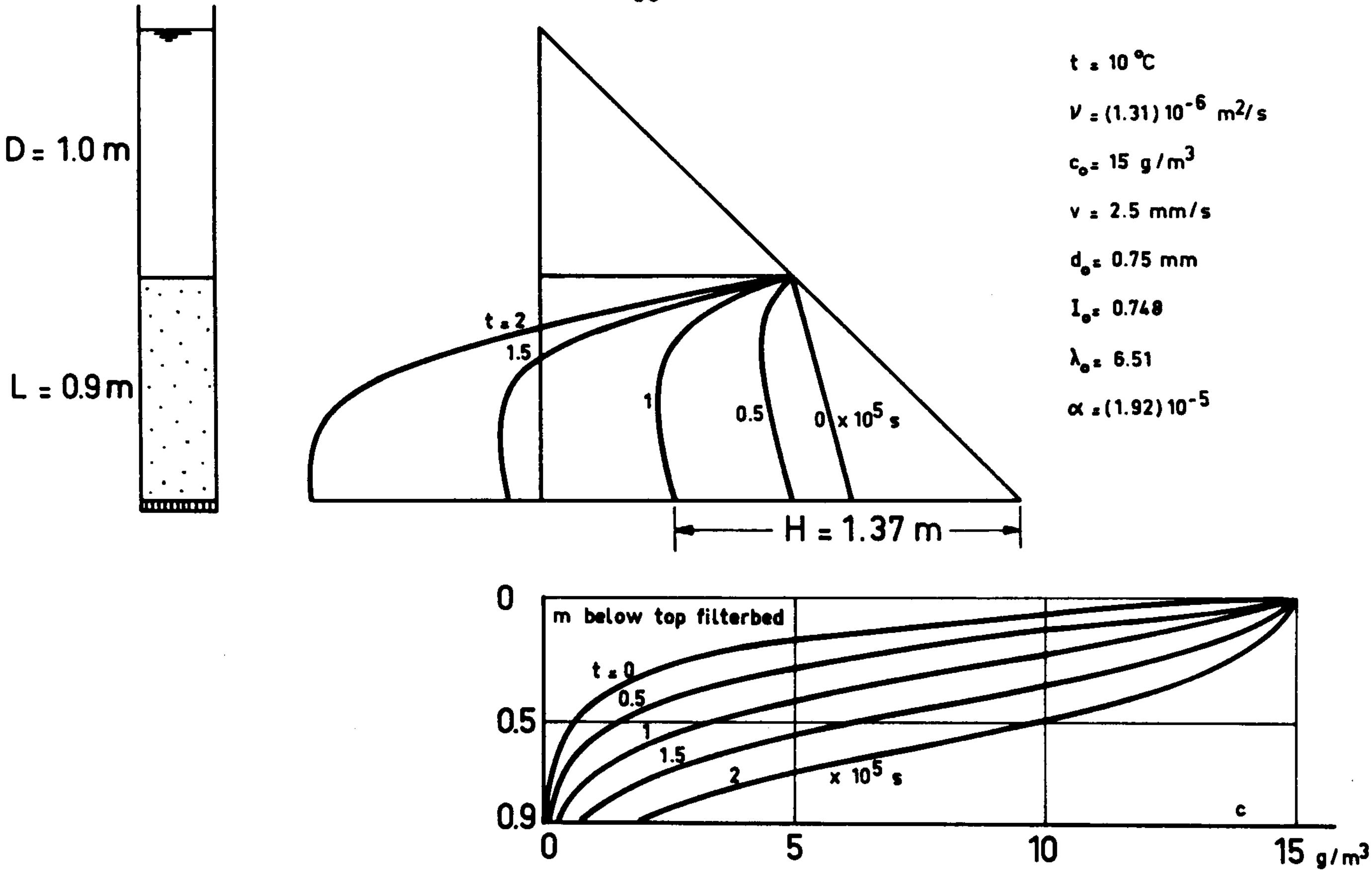



Fig. 2.13 Distribution of pressure and suspended matter in the bed of a rapid filter.

the straight line under 45° indicates the hydrostatic pressure distribution (v=0) and the straight line with a steeper slope the pressure distribution in the clean bed (t=0). Clogging will result in a greater pressure loss, resulting in curved lines more to the left.

Although not so much important for design purposes, the deposition of suspended matter in the filterbed can also be calculated with the formulas given above. At the time t and over the depth L of the filterbed

$$\bar{\sigma} = n\, p_o L\, \rho_d \left(1 - \frac{1}{\lambda_o L} \ln \frac{e^{\lambda_o L} + e^{\alpha t} - 1}{e^{\alpha t}}\right) \text{ kg/m}^2$$

For the example of fig. 2.13 this gives

| T = | 0 | 0.5 | 1 | 1.5 | 2.0 | $\times 10^5$ s |
|---|---|---|---|---|---|---|
| supply | 0 | 1.88 | 3.75 | 5.63 | 7.50 | kg/m$^2$ |
| storage | 0 | 1.87 | 3.72 | 5.53 | 7.26 | kg/m$^2$ |
| discharge | 0 | 0.01 | 0.03 | 0.10 | 0.24 | kg/m$^2$ |

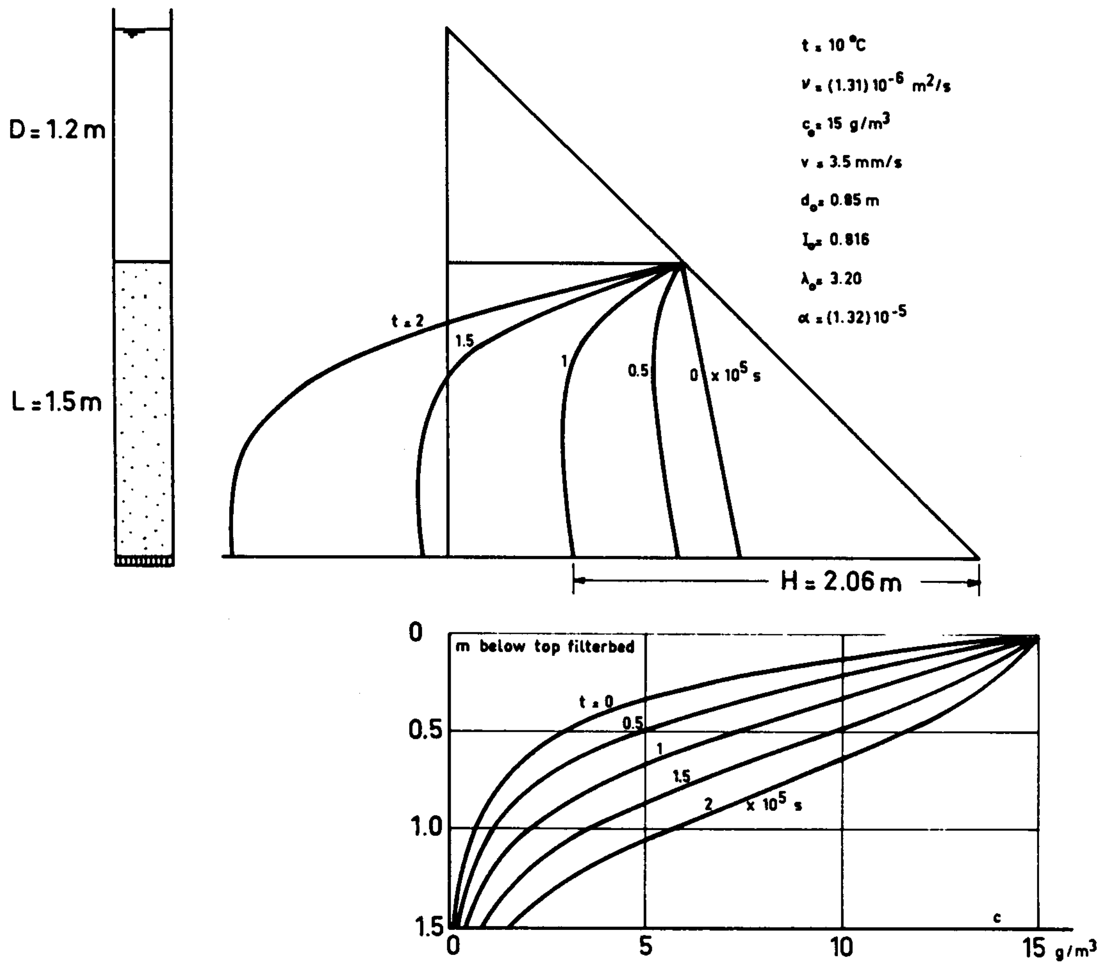



Fig. 2.14 Distribution of pressure and suspended matter in the bed of a rapid filter.

showing clearly the break-through of suspended matter after prolonged periods of filter run.

Before proceeding with the subject proper, the reader may be informed that the formula for the friction losses accompanying the flow of water through a clean bed of granular material

$$I_o = 5 \frac{\nu}{g} \frac{S_o^2}{p_o^3} v$$

is often written in a different form. When 1 $m^3$ of filtering material contains m' spherical grains of diameter $d_o$

$$p_o = 1 - m' \frac{\pi}{6} d_o^{\,3},\ S_o = m'\pi\, d_o^{\,2}, \text{ combined}$$

$$S_o = \frac{6}{d_o}(1 - p_o), \text{ substituted}$$

$$I_o = 180 \frac{\nu}{g} \frac{(1-p_o)^2}{p_o^{\,3}} \frac{v}{d_o^{\,2}}$$

the well-known Carman-Kozeney equation. This lineair relation between resistance and filter rate holds strictly true when the eddying resistance is negligeable compared to the viscous forces. According to experiments this is the case when

$$Re = \frac{1}{p_o} \frac{v\, d_o}{\nu} < 5$$

or with $p_o = 0.38$, $d_o = 1$ mm and at $t = 10^oC$, $\nu = (1.31)\ 10^{-6}\ m^2/s$, for $v < 2.5$ mm/s. For rates up to 5 mm/s, however, the differences are negligeable.

The kinematic viscosity $\nu$ finally is a function of the temperature

| | | | | | | |
|---|---|---|---|---|---|---|
| $t$ = 0 | 5 | 10 | 15 | 20 | 25 | 30 $^0C$ |
| $\nu$ = 1.792 | 1.519 | 1.310 | 1.146 | 1.011 | 0.898 | 0.804 x $10^{-6} m^2/s$ |

which relation may be approximated by

$$\nu = \frac{(497)\ 10^{-6}}{(t + 42.5)^{1.5}}$$

## 2.5 Negative heads and air binding

In the experimental plant of fig. 2.2, the pressure distributions of fig. 2.13 and 2.14 can also be measured directly. This was first advocated by the Swedisch waterworks engineer Lindquist, after whom these curves are named. The actual distribution, however, also depends on the depth of supernatant water. Two extreme cases can be distinguished

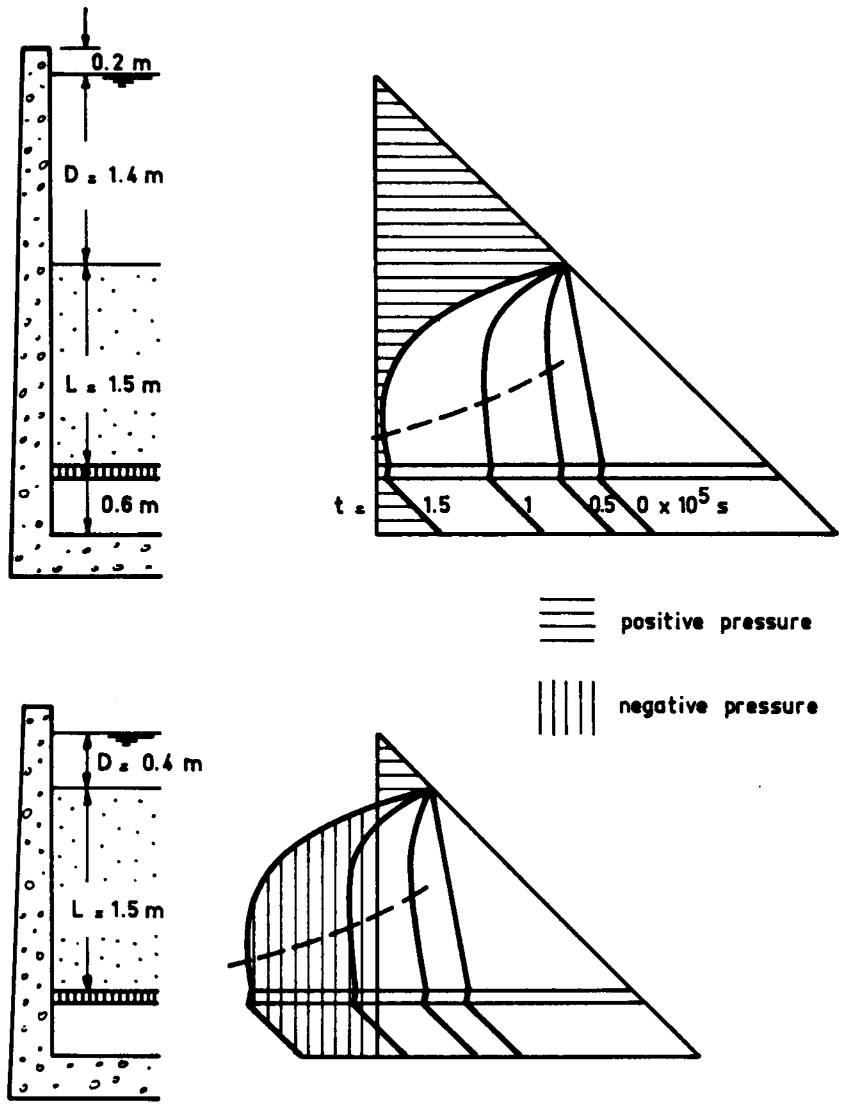



Fig. 2.15 Pressure distribution in the filterbed dependent on the depth of supernatant water

a. a large depth of supernatant water, say 1.4m, operating the filter by overpressure (fig. 2.15, top);
b. a small depth of supernatant water, say 0.4 m, operating the filter by suction (fig. 2.15, bottom).

The last solution has the advantage of a shallower filter box, reducing the cost of construction, but it brings with it the danger of air-binding. To understand this phenomenon, it is assumed that the water on top of the filterbed is saturated with atmospheric air, meaning in reverse that the sum of the gas pressures, including that of water vapour, will equal atmospheric pressure. As the water moves downward through the depth of supernatant water, the waterpressure increases but the total gas pressure remains the same. The water pressure decreases as soon as the water enters the filterbed. When no chemical reactions occur in this filterbed, for instance during clarification of surface water for removal of inorganic turbidity, the total gaspressure will again stay constant and surpass the waterpressure the moment negative heads arise. Gases carried by the water will now come out of solution. The released gasbubbles will accumulate in the pores between the sandgrains, hindering downward watermovement, increasing filterresistance and prematurily ending filterruns. When airbinding occurs over part of the filterbed only, other portions will be overloaded. The more rapid rise of filterresistance is perhaps hardly noticeable in this case, but the overloading may result in a deterioration of effluent quality. The accumulated gases may also break through the filter, leaving openings through which the water is able to move downward with insufficient purification, again lowering effluent quality. Gasbubbles adhering to the filtergrains finally will increase their buoyancy, thus promoting loss of filtering material during backwash.

When during groundwater treatment oxygen is consumed, the total gas pressure goes down, while the reaction products formed are so highly soluble that they do not contribute to this pressure in any extend. Airbinding with all its consequences will now only occur when the negative head has assumed certain values, larger as the oxygen consumption is greater and the watertemperature is higher. At 20°C the volumetric composition of atmospheric air in rural areas equals

$N_2 = 76.2\%$, $O_2 = 20.6\%$, $H_2O = 2.3\%$, argon and other gases = 0.9%

while the solubility of oxygen amounts to 44.3 g/m$^3$ per atmosphere partial pressure. This gives as saturation concentration of oxygen in water

$$c_s = (0.206)(44.3) = 9.13 \text{ g/m}^3$$

A negative head of 1.5 m water column = 0.1452 atmosphere will give no problems of air binding when the oxygen concentration has dropped by

$$\Delta c = (0.145)(44.3) = 6.43 \text{ g/m}^3 \quad \text{to}$$

$$c = 9.13 - 6.43 = 2.70 \text{ g/m}^3$$

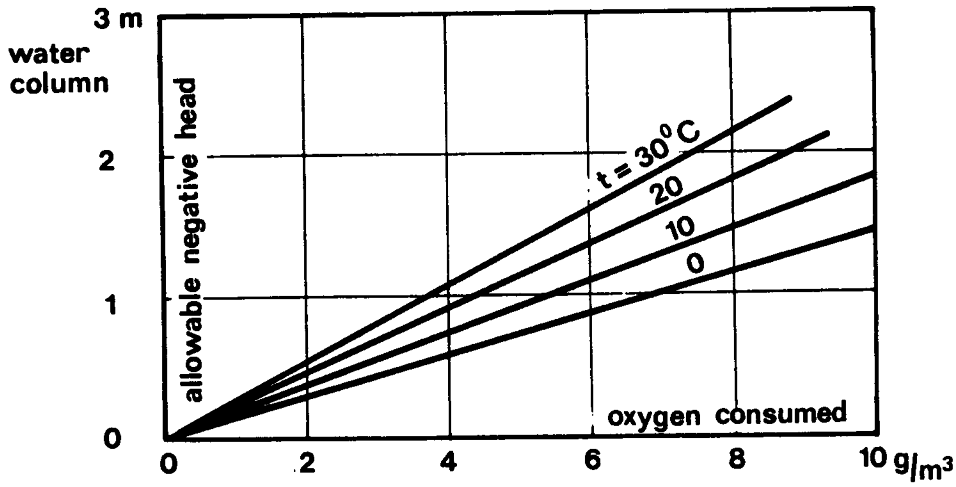



Fig. 2.16 Relation between oxygen consumed and allowable negative head

For various temperatures, the relation between the amount of oxygen consumed and the allowable negative head is shown in fig. 2.16. As the creation of air bubbles takes time, an additional negative head of about 0.5 m water column = 0.0484 atmosphere may be allowed during short periods at the end of the filterrun. For the example given above, this means in reverse a required drop in oxygen concentration by

$$\Delta c = (0.1452 - 0.0484)(44.3) = 4.29 \text{ g/m}^3 \text{ to}$$

$$c = 9.13 - 4.29 = 4.84 \text{ g/m}^3$$

making the problem of air binding even less serious.

For surface water with a high oxygen demand, the problem is in principle the same as described above for groundwater. The composition of this water, however, will not change as years go by, while by treatment of domestic and industrial wastes before discharge surface water quality may improve greatly in future. When the filter has been built according to the set-up of fig. 2.15 at the bottom, serious difficulties will now be encountered. In practice, troubles with airbinding have been experienced all over the world, deteriorating effluent quality, shortening filterruns and promoting loss of filtering material during back-wash. In some cases these troubles were so severe, that part of the filterbed had to be removed so as to increase the depth of supernatant water. Provision of a larger depth from the very beginning is then certainly a better proposition.

For the treatment of surface water, negative pressures should better be avoided. In case $I_o < 1$, this is the case when according to fig. 2.17

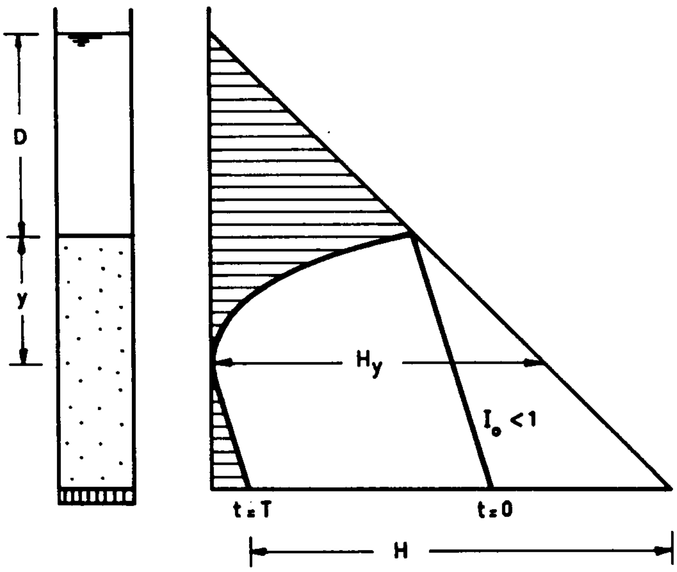



Fig. 2.17 Avoiding negative heads from occurring

$$D \geqslant H_y - y$$

with the depth y determined from

$$\frac{dH}{dy} = I_o \left( \frac{p_o}{p_o - \sigma_v} \right)^2 = 1 \qquad \text{with}$$

$$\sigma_v = n\, p_o \, \frac{e^{\alpha t} - 1}{e^{\lambda_o y} + e^{\alpha t} - 1} \qquad \text{this gives}$$

$$y = \frac{1}{\lambda_o} \ln \left\{ \left( \frac{n}{1 - \sqrt{I_o}} - 1 \right) (e^{\alpha t} - 1) \right\}$$

for which depth $H_y$ can be calculated from the resistance formula. For $I_o > 1$, fig. 2.18, the solution is more simple

$$D > H - L$$

For the cases shown in fig. 2.12, but assuming for safety $T = (1.5)\ 10^5$ s, these equations give

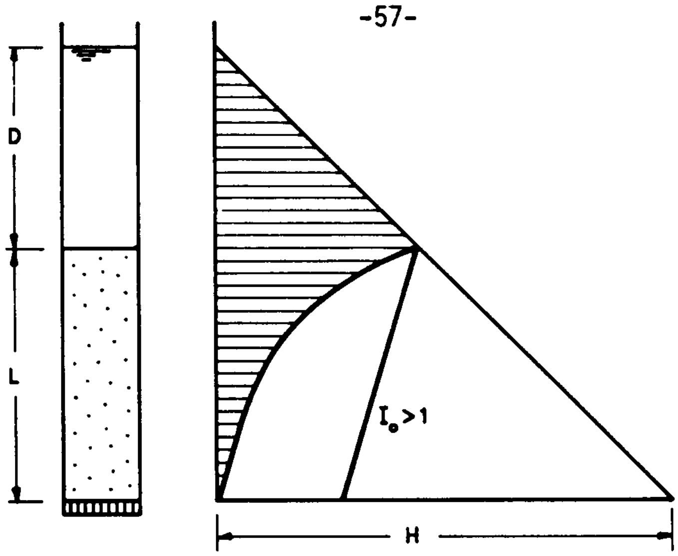



Fig. 2.18 Avoiding negative heads from occurring

| v = | | 2 | 2.5 | 3 | 3.5 | 4 | mm/s |
|---|---|---|---|---|---|---|---|
| d = 0.7 mm | L = | 0.57 | 0.72 | 0.86 | 1.00 | 1.14 | m |
| | D = | 0.96 | 1.66 | 1.13 | 1.70 | 2.39 | |
| 0.8 mm | L = | 0.74 | 0.92 | 1.11 | 1.30 | 1.48 | |
| | D = | 0.42 | 0.79 | 1.29 | 1.94 | 1.23 | |
| 0.9 mm | L = | 0.95 | 1.19 | 1.43 | 1.67 | 1.91 | |
| | D = | 0.13 | 0.30 | 0.56 | 0.92 | 1.38 | |
| 1.0 mm | L = | 1.22 | 1.53 | 1.83 | 2.14 | 2.44 | |
| | D = | 0.01 | 0.06 | 0.17 | 0.35 | 0.60 | |

With a free-board of 0.2 m, a minimum water depth D = 0.4 m and a thickness of the underdrainage system equal to 0.6 m, the total depth $L_t$ of the filterbox equals

| v = | 2 | 2.5 | 3 | 3.5 | 4 | mm/s |
|---|---|---|---|---|---|---|
| d=0.7 mm | $L_t$=2.33 | 3.18 | 2.79 | 3.50 | 4.33 | m |
| 0.8 | 1.96 | 2.51 | 3.20 | 4.04 | 3.51 | m |
| 0.9 | 2.15 | 2.39 | 2.79 | 3.39 | 4.09 | m |
| 1.0 | 2.42 | 2.73 | 3.03 | 3.34 | 3.84 | m |

The dimensions of the filterbox are now fully known, allowing a fair estimate of construction costs to be made .
For a capacity of 1 $m^3/s$ and assuming the vertical walls at f $400/m^2$, filtersand at f $300/m^3$ and foundation, floor and underdrainage systems together at f $2000/m^2$, the total costs of the filterboxes in million guilders amounts to

| v = | 2 | 2.5 | 3 | 3.5 | 4 | mm/s |
|---|---|---|---|---|---|---|
| d=0.7 mm | 1.33 | 1.16 | 0.98 | 0.92 | 0.86 | |
| 0.8 | 1.32 | 1.13 | 1.04 | 0.99 | 0.83 | |
| 0.9 | 1.37 | 1.15 | 1.03 | 0.97 | 0.90 | |
| 1.0 | 1.44 | 1.22 | 1.09 | 1.01 | 0.93 | |

From this table two conclusions can be drawn

a. the cost of construction goes down as filtration rates go up, but the decrease is less as filtration rates are higher;
b. for the same rate of filtration, the selection of a finer or coarser grain size has little influence on the costs of construction.

Money is important, but the final selection should be made according to the preference of an experienced designer.

The dotted line in fig. 2.15 is the locus of the points below which the pressure line is parallel to the one in the clean filterbed. These points consequently indicate the deepest penetration of impurities into the filterbed. According to fig. 2.19, however, chemical reactions not contributing to clogging still occurs at greater depths.

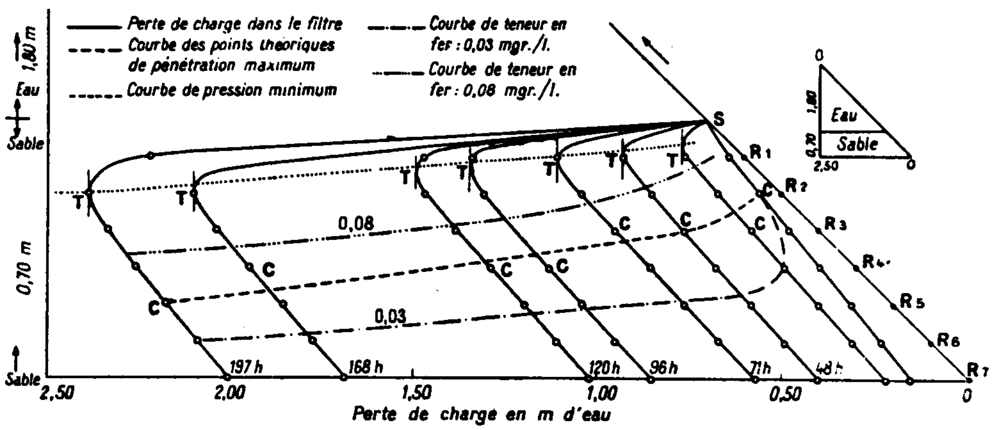



Fig. 2.19 Observed pressure distribution in rapid filters used for the deferrisation of groundwater

## 2.6 Changes in operating conditions

When a filter has been designed in the cheapest way, so that it just satisfies effluent requirements, any change for the worst in operating conditions will result in an unpleasant shortening of the filter runs. With for instance $c_o = 15\ g/m^3$, $c_e < 0.5\ g/m^3$, $t = 10°C$, $v = 3$ mm/s, $d = 0.8$ mm, $L = 1.1$ m and $H = 1.5$ m and the values of n, λ and α as mentioned before

$$T_r = (0.88)10^5\ s \qquad T_q = (0.98)10^5\ s$$

Increasing $c_o$ from 15 to 30 $g/m^3$ gives

$$T_r = (0.44)10^5\ s \qquad T_q = (0.27)10^5\ s$$

A drop in water temperature from 10 to 0°C results in

$$T_r = (0.47)10^5 s \qquad T_q = (0.17)10^5\ s$$

while an increase in filtration rate from 3 to 4 mm/s produces

$$T_r = (0.39)10^5 s \qquad T_q = (0.19)10^5\ s$$

The lowering of $T_r$ from 1 to about 0.5 days is till acceptable, when the adverse conditions do not last too long. A length of filterrun $T_q$ equal to 0.2 days or 5 hours, however, is too short, asking for a larger filterbed thickness. Increasing this thickness to 1.2 m gives values of $T_q$ of $(0.41)10^5$, $(0.46)10^5$ and $(0.41)10^5$s respectively, quite acceptable for short periods.

In case the filtration plant possesses a larger number of filtering units,the effluent of which is put together in the clear well, then the decisive factor is the quality of the mixed effluent. When moreover the filters are backwashed at equal intervals (fig. 2.20), this quality equals the average quality over the full length of the filterrun. With

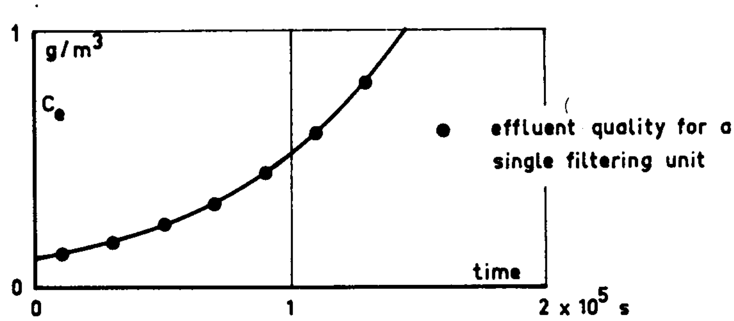



Fig. 2.20 Effluent quality for a number of filtering units

$$c_e = c_o \frac{e^{\alpha t}}{e^{\lambda_o L} + e^{\alpha t} - 1}$$

this average quality can be calculated as

$$c_a = \frac{1}{T} \int_o^T c_e \, dt = \frac{c_o}{\alpha T} \ln \frac{e^{\lambda_o L} + e^{\alpha T} - 1}{e^{\lambda_o L}}$$

from which T can be found by trial and error.
With $c_a = 0.5$ g/m$^3$, this gives for the situation mentioned above

| | |
|---|---|
| normal conditions | $T_q = (1.65)10^5$ s |
| $c_o$ from 15 to 30 g/m$^3$, | $T_q = (0.48)10^5$ s |
| t from 10 to 0 $^o$C, | $T_q = (0.34)10^5$ s |
| v from 3 to 4 mm/s, | $T_q = (0.37)10^5$ s |

which values are more or less acceptable for limited periods. Increasing the bed thickness from 1.1 to 1.2 m gives

| | |
|---|---|
| normal conditions | $T_q = (2.05)10^5$ s |
| $c_o$ from 15 to 30 g/m$^3$, | $T_q = (0.70)10^5$ s |
| t from 10 to 0$^o$C, | $T_q = (0.86)10^5$ s |
| v from 3 to 4 mm/s | $T_q = (0.75)10^5$ s |

from which no problems whatsoever will result.

Whether filter design should be based on normal or on (a combination of) adverse conditions also depends on the treatment system as a whole. In case rapid filters are used as a preparatory treatment to be followed by slow sand filtration, an occasional deterioration of effluent quality is not objectionable. Slow filters do have an enormous reserve capacity and a lower quality of the water brought upon them may quicken filter clogging, shortening the filterrun, but will not affect the quality of the water going into supply. Also when rapid filters are used for industrial water supplies, a limited deterioration in effluent quality is often acceptable and again here the designer may proceed with more daring in his search for the most economical solution. When rapid filters constitute the final treatment for drinking water purposes, however, effluent quality may not be tampered with and should satisfy strict standards under all conditions. It must be realised moreover that such standards change during the years, becoming more strict as time goes on. When the new standards cannot be obtained by a better pre-treatment of the water to be filtered, lower filtration rates allowing finer filtering materials must be used. Anticipating such developments, lower rates or larger bed thicknesses should be applied from the very beginning.

## 2.7 Length of filterrun

In the preceding sections, the length of filterrun $T_r$ has been set at $(0.9)10^5$ s = 1 day for conditions of average demand and average water quality. The reasoning behind this assumption is that now all filtering units must be cleaned daily, but when their number is not too large this can be accomplished by one filter attendant during an 8-hour shift. Back-washing proper only takes a few minutes, but to this must be added the time necessary for draining and back-filling the filtering unit and for walking to and from it, increasing the total time to 20 or 30 minutes. About 20 units can thus be cleaned during one shift. When in large plants more units are present, 3 options are open

a. doubling or tripling the number of filter attendants, back-washing the filters during 2 or 3 shifts. This is a simple expedient, but it increases the cost of operation;
b. extending the back-washing facilities so that 2 units can be cleaned simultaneously increasing the cost of construction;
c. actuating the back-washing facilities automatically when a pre-set resistance is surpassed, allowing the cleaning to be spread over 24 hours per day.

In western-type countries automatic back-washing using small computers is most attractive, while in developing countries increasing the number of operating personnel has the advantage of providing jobs for people out of work.

When demand increases or water quality drops, the length of filterrun goes down. A reduction to 0.5 days means that all units must be cleaned twice daily, asking for automatic back-washing or two shifts of filter attendants. When adverse conditions are of short duration only and do not occur too frequently, the second shift can be manned by other employees, temporarily relieved from their normal duties. Another solution is to increase the length of filterrun to 2 days for normal conditions. Operation is now more flexible, less hectic, but it asks for a larger bed thickness and a greater depth of supernatant water, increasing the cost of construction.

When a filtering unit is taken out of operation for back-washing, its load is taken over by the remaining units, augmenting their rate from v to $\frac{n}{n-1}$ v with n as number of filtering units. The resistance of these units rises accordingly, meaning that cleaning must be accomplished before the maximum allowable resistance $H_r$ is reached. In fig. 2.21 it is assumed that the filter resistance growth linearly with time and that the filters are back-washed one after the other. Taking filter A out of operation for back-washing, increases the resistance of filter B instantaneously to the value indicated by the point B' and during the back-washing of filter A gradually to point B''. The latter value may not be larger than the maximum allowable one $H_r$, giving for the resistance $H_B$ at back-washing the relation.

$\frac{n}{n-1} H_b = H_r$ , shortening the length of filterrun from $T_r$ to $T_b$.

With $H_i$ as resistance of the clean bed

$$T_b = T_r \frac{H_b - H_i}{H_r - H_i}$$

Assuming $H_i = 0.5\ H_r$ gives

$$T_b = \frac{n-2}{n}\ T_r$$

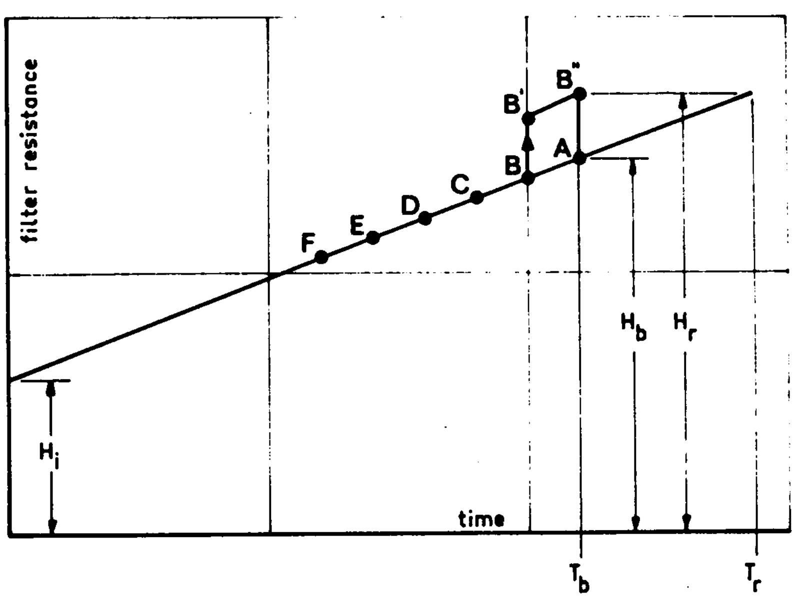



Fig. 2.21 Change in filter resistance during back-washing

With a large number of filtering units, the reduction in the length of filterrun can be undone by a slight increase in the maximum allowable resistance $H_r$. With the minimum number n = 4 and $T_b$ = 0.5 $T_r$, however, a much larger design length of filterrun should be chosen.

When after cleaning a filter is returned to normal duty, the quality of the effluent will first equal that of the backwash water applied. After this water has been displaced by the downward percolating raw water, a sudden deterioration of effluent quality sets in, raising the concentration of impurities carried by the filtered water to a multiple of the normal one. After reaching a maximum value, this concentration declines only very gradually, taking one to two hours to reach steady state conditions. Fig.2.22 shows this phenomenon with regard to the quality of the effluent from filters treating an aerated ground-water, containing about 5 g/$m^3$ iron (Cleasby, 1963). At a filtration rate of $(1.3)10^{-3}$ m/sec, the iron content of the effluent first rises to over 1 g/$m^3$, dropping in the course of 2 hours to a value of 0.1 g/$m^3$, which just satisfies normal drinking water standards. Due to the higher iron content during the first 2 hours, however, the average values goes up, more as the length of filterrun is shorter. With $T_r$ = 1 day, the increase amounts to 20%, dropping to a more acceptable 8% when $T_r$ is increased to 2 days. In principle, the lowering of effluent quality

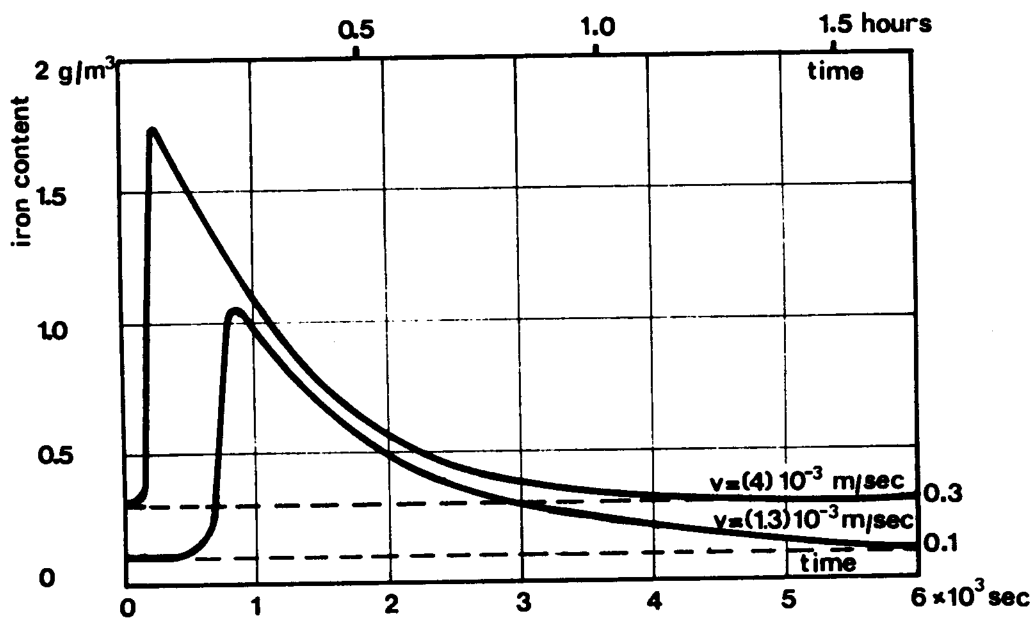



Fig. 2.22 Effluent quality of rapid filters treating aerated groundwater with iron content of 5 g/$m^3$.

thus obtained can be prevented by carrying the effluent to waste during the first 0.5 to 1 hour after backwashing or by gradually increasing the filtration rate from zero to the full value over the same period. The necessary equipment, filter to waste connections (and recycling installation) or slow start controllers add to the cost of construction, while the reduction in capacity per unit filterbed area requires a larger filterplant with again a higher building cost, more as the length of filterrun is shorter.

Next to adverse operating conditions and a reduced length of filterrun, also better than average circumstances occur, increasing the length of filterrun to many days. To prevent a deep penetration of impurities into the filterbed, however, it is good practice to back-wash the filter at least every 3 days.

## 2.8 Application of filtration theory

By its very nature, a mathematical treatment tends to make an exact impression. It must never be forgotten, however, that this only concerns the calculation process itself, while the results fully depend on the assumptions made when setting up the basic equations. The selection of the filtration theory to be applied is therefore of paramount importance. In the preceding section Lerk's theory has been used

$$\lambda = \lambda_o \left(1 - \frac{\sigma_v}{n\, p_o}\right) \text{ with } \sigma_v = \frac{\sigma}{\rho_d} \text{ and}$$

$$\lambda_o = \frac{\beta}{\nu^a\, v^b\, d_o^{\,c}} \text{ , assuming } a = 1,\ b = 1,\ c = 3$$

and $\beta = (9)10^{-18}$. In these equations $\nu$, $v$ and $d_o$ are fully known, $p_o$ and $\rho_d$ can be estimated with fair accuracy, but the values of $n$, $\beta$, $a$, $b$ and $c$ are mere guesswork. Their influence on the results of the calculations are enormous, meaning in reverse that they have to be determined experimentally. For the design of a rapid filtration plant using the mathematical theory of filtration, experiments are indispensable, for instance those of which the results are shown in fig. 2.5 and 2.6. With regard to the changes in purification efficiency during the breaking-in process, these experiments must be carried out for at least 3 months and when large seasonal fluctiations in raw water quality occur, for more then one year. Depending on the outcome of these tests, two choices must be made

a. which filtration theory to be applied;
b. which values to be given to the parameters occurring in the chosen theory,

so that the calculated results in terms of changes in effluent quality and filter resultance with time cover those measured as well as possible. The most attractive filter construction can now be determined, but due to ambiguities in the mathematical analysis, it is essential that this solution is again investigated experimentally. With this second set of experimental results, the filtration theory and the values of its parameters can be adjusted and the most attractive construction of the filtration plant determined anew. Commonly the deviation between this and the former optimization is so small , that extrapolation by the mathematical theory is warranted. If not, a third set of experiments must be carried out. The total time involved varies from a minimum of 0.5 or 1 year to a maximum of 2 or 3 years. This time is only available when the decision to built the plant is taken well in advance!

## 2.9 Filtering material

The actual work of a rapid filtration plant is done by the filterbed for which consequently the best materials should be chosen. As first requirements must be mentioned that the filtering material should be clean and durable, free of clay, loam, dust, dirt or organic matter and able to resist mechanical, chemical or biological attack. Clean sand satisfies these standards, but as found in nature, the variation in grain size is too large. During back-washing a hydraulic grading occurs, bringing the fine particles to the top and the coarse ones to the bottom of the filterbed. The fine material in the upper part of the filterbed has a high filtration efficiency, resulting in a rapid clogging and shortened filter runs, while the coarse material in the lower part does not add to effluent quality. To prevent this as much as possible,the natural sand must be sieved to remove the coarse and fine fractions, in this way reducing the coefficient of uniformity U as defined in fig. 2.23 to values below 1.5 under all circumstances and preferably below 1.3. Values less than 1.2 have little advantage and add considerably to the cost. To avoid interfaces in the filterbed with a sharp increase in grain size, the grain size distribution of the

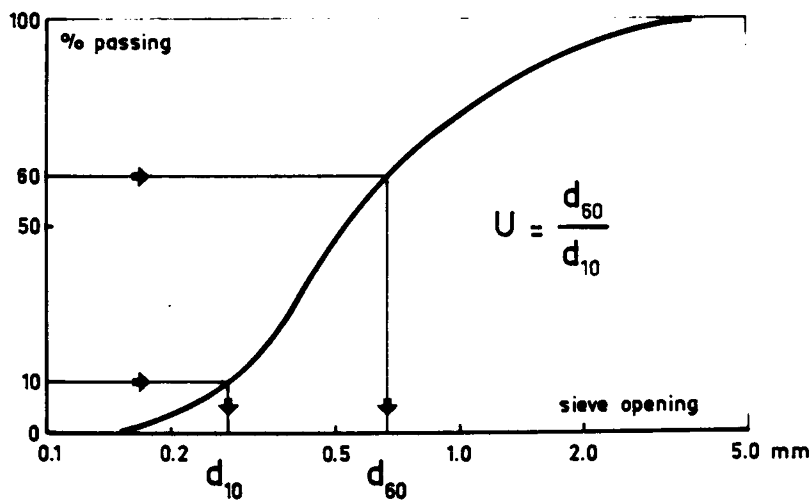



Fig. 2.23 Grain size analysis

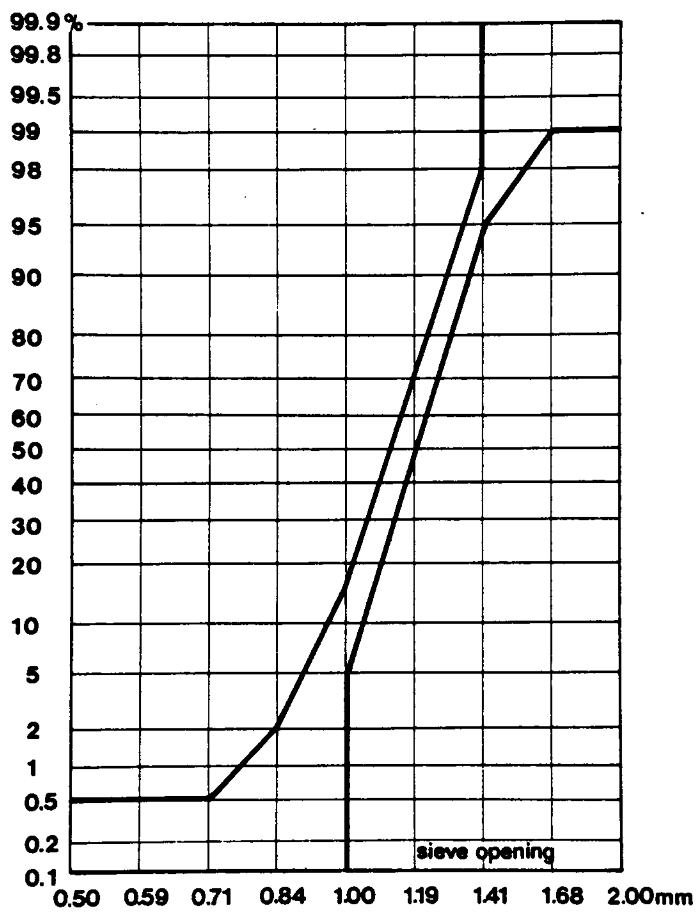



Fig. 2.24 Specifications of filtersand for pre-treatment of river water

graded material should show a smooth curve, which can be obtained by setting maximum and minimum limits for the percentages of filtering material passing various standard sieves. For better visualization these may be plotted on a graph of which fig. 2.24 gives an example.

In the mathematical theory of filtration, filter grains are supposed to be spherical with a uniform diameter $d_o$ when clean. Both assumptions, however, do not hold true in practice where on one hand the grain shape deviates more or less from the spherical one, while on the other hand a variation in grain sizes occur. Today this variation is determined by separating the filtering material into fractions, using square woven wire sieves (fig. 2.25) of which in The Netherlands the clear opening s increases by a factor $2^{\frac{1}{4}} = 1.189$. The most practical approach to the two-sided problem mentioned above can now be had by considering that the combined surface area of the grains is the deciding factor, both with regard to filtration efficiency as with respect to filter resistance. For uniform spherical grains of diameter $d_o$ the total surface $S_o$ area per unit volume of clean filtering material equals (section 2.4)

$$S_o = \frac{6}{d_o} (1 - p_o) \text{ with } p_o \text{ as pore space.}$$

For non-uniform spherical grains with diameters varying evenly from $d_i$ to $d_j$ this area is given with good approximation (error less than 0.5%) by

$$S_o = \frac{6}{\sqrt{d_i d_j}} (1 - p_o), \text{ valid for } \frac{d_j}{d_i} < \sqrt{2}$$

Square wove wire sieves pass spherical grains of diamter d = s, giving for the surface area of a fraction between the consecutive sieve $s_i$ and $s_j$

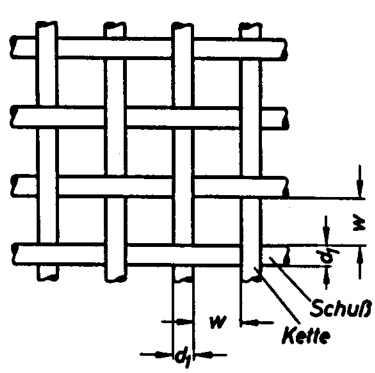



Fig. 2.25 Square woven wire sieves

$$S_o = \frac{6}{\sqrt{s_i s_j}} (1 - p_o)$$

Non-spherical grains have on one hand a larger surface are to volume ratio, while square woven wire sieves may now also pass elongated grains with a volume in excess of $\frac{\pi}{6} s^3$. Both factors are taken into account simultaneously by a shape factor $\phi$, giving for a fraction between the sieve sizes $s_i$ and $s_j$

$$S_o = \frac{6}{\phi_i \sqrt{s_i s_j}} (1 - p_o)$$

For filter grains of constant shapes, prisms for instance, the value of $\phi$ can easily be calculated. In practice, however, many shapes occur simultaneously and here the value of $\phi$ is determined by equating the measured resistance of a clean filterbed composed of grains between the consecutive sieve sizes $s_i$ and $s_j$ to the value following from the Carman-Kozeny equation (section 2.4)

$$H_o = \frac{180\ \nu}{g} \frac{(1 - p_o)^2}{p_o^3} \frac{v}{(\phi_i \sqrt{s_i s_j})^2} L$$

When a sample of filtering material of weight W contains n fractions with weights $W_1$ to $W_n$ between the consecutive sieve sizes $s_1$ to $s_{n+1}$, the surface area can be found by addition

$$S_o = \frac{W_1}{W} S_{01} + \frac{W_2}{W} S_{02} + \ldots\ldots\ldots\ldots\ldots \frac{W_n}{W} S_{0n} \qquad \text{or}$$

$$S_o = \frac{6(1-p_o)}{W} \left( \frac{W_1}{\phi_1 \sqrt{s_1 s_2}} + \frac{W_2}{\phi_2 \sqrt{s_2 s_3}} + \ldots\ldots + \frac{W_n}{\phi_n \sqrt{s_n s_{n+1}}} \right)$$

For this area may also be written

$$S_o = \frac{6}{d_s} (1 - p_o)$$

with $d_s$ as specific grain size to be used for $d_o$ in the mathematical theory of filtration

$$\frac{W}{d_s} = \frac{W_1}{\phi_1 \sqrt{s_1 s_2}} + \frac{W_2}{\phi_2 \sqrt{s_2 s_3}} + \ldots\ldots + \frac{W_3}{\phi_n \sqrt{s_n s_{n+1}}}$$

In the laboratory for Sanitary Engineering of the Department for Civil Engineering at the University of Technology in Delft, the value of the shape factor $\phi$ has been determined for various filtering materials and sieve fractions (G.H. Corstjens, Journal $H_2O$, 1972), using square woven wire sieves. The results of these measurements are summarized below.

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| lower sieve opening $s_i$ | 0.5 | 0.56 | 0.63 | 0.71 | 0.8 | 0.9 | 1.0 | 1.12 | 1.25 | 1.4 | 1.6 | 1.8 | 2.0 | mm |
| upper sieve opening $s_j$ | 0.56 | 0.63 | 0.71 | 0.8 | 0.9 | 1.0 | 1.12 | 1.25 | 1.4 | 1.6 | 1.8 | 2.0 | 2.24 | mm |
| $\sqrt{s_i s_j}$ | 0.529 | 0.594 | 0.669 | 0.754 | 0.848 | 0.949 | 1.058 | 1.184 | 1.323 | 1.497 | 1.697 | 1.898 | 2.118 | mm |
| Meuse sand $\phi$ = | 0.92 | 0.92 | 0.91 | 0.90 | 0.89 | 0.88 | 0.87 | 0.86 | 0.84 | 0.81 | 0.78 | 0.75 | 0.72 | |
| broken gravel | | | | constant at 0.665 | | | | | | | | | | |
| magnetite | | | | constant at 0.75 | | | | | | | | | | |
| Wales anthracite | | | | constant at 0.70 | | | | | | | | | | |
| Hydro-anthracite | 0.65 | 0.65 | 0.64 | 0.64 | 0.63 | 0.63 | 0.62 | 0.61 | 0.60 | 0.57 | 0.55 | 0.52 | 0.49 | |

In case no data are available, $\phi$ may be estimated from

| shape: | spherical | nearly spherical | rounded | worn | angular | broken |
|---|---|---|---|---|---|---|
| $\phi \simeq$ | 1.00 | 0.95 | 0.9 | 0.85 | 0.75 | 0.65 |

Assuming for instance sand from the river Meuse and as grainsize distribution

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| s = | 0.71 | 0.8 | 0.9 | 1.0 | 1.12 | 1.25 | 1.4 | mm |
| W = | | 1.5 | 6.5 | 34 | 45 | 10 | 3 | % |

the diameter $d_s$ to be used for $d_o$ in the calculations of the preceding sections follows from

$$\frac{100}{d_s} = \frac{1.5}{(0.90)(0.754)} + \frac{6.5}{(0.89)(0.848)} + \frac{34}{(0.88)(0.949)} + \frac{45}{(0.87)(1.058)} +$$

$$+ \frac{10}{(0.86)(1.184)} + \frac{3}{(0.84)(1.323)} \text{ or}$$

$$d_s = 0.885 \text{ mm}$$

Next to sand other inert filtering materials may be used, having a different mass density

| | |
|---|---|
| pumice | 1100 $kg/m^3$ |
| crushed coconut shell | 1350-1450 |
| anthracite | 1400-1700 |
| sand | 2600 |
| garnet | 3500-4300 |
| barite | 4500 |
| magnetite | 5200 |

In single bed filters they may be applied to avoid troubles in back-washing very fine or very coarse filtergrains. During back-washing the grains float in the rising wash-water stream, which exerts a shearing stress $\tau$ that keeps the grains in suspension. The shearing force equals the submerged weight

$$\tau \pi d^2 = \frac{\pi}{6} d^3 (\rho_f - \rho_w) \text{ , simplified}$$

$$\tau = \frac{d}{6} (\rho_f - \rho_w)$$

The shear stress $\tau$ determines the cleaning action, but its value decreases as the grain size goes down. According to experiments sand grains with a diameter less than 0.8 mm are difficult to keep clean by back-washing with water alone. According to the formula given above, however, $\tau$ may now be augmented by using a heavier filtering material such as garnet.

During back-washing an expansion of the filterbed should take place, allowing the liberated cloggings to escape more freely with the wash-water. This asks for large back-wash rates, higher as the grains are coarser and the mass density of the filtering material differs more from that of the surrounding water. To avoid excessive back-wash rates necessary for expanding coarse sand grains, a lighter filtering material such as antracite may now be applied.

In some cases finally reactive filtering materials may have advantages such as crushed marble or burned dolomite for the deferrisation of iron containing groundwater having a low pH.

# 3. CLEANING

## 3.1. Introduction

When during filtration the hydraulic resistance attains its maximum allowable value or the quality of the effluent drops below the set standards, cleaning of the filter is necessary to restore its capacity and / or to improve the quality of the filtered water. Today without exception mechanical cleaning is used, effected by reversing the direction of flow, admitting wash-water to the underside of the filterbed (fig. 3.1).At a rate many times larger than the filtration rate, this washwater flows upward, taking the impurities accumulated in the pores of the filterbed with it to above, where wash-water troughs and gutters are present to convey it to be a drain leading outside the filter (fig. 1.1). This backwashing process has two purposes

a. to dislodge impurities adhering to the filter grain surfaces by the shearing action of the rising wash-water stream, flowing at high rates past the stationary grains;
b. to expand the filterbed, to increase the pore space allowing the liberated cloggings to escape more easily with the wash-water.

As explained in last section, the shearing action is less with smaller grainsizes and in many cases will prove insufficient when the diameter drops below 0.8 mm with sand as filtering material. Either heavier (and rather expensive) filtering material must now be used or the shearing action improved by an auxilliary scour, commonly by back-washing with air.

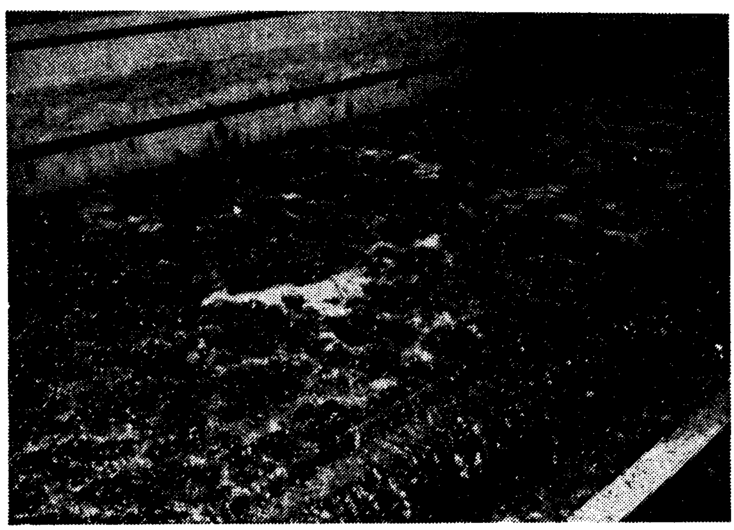


Fig. 3.1 Backwashing of a rapid filter

To expand a filterbed composed of coarse sand grains asks for enormous back-wash rates, which can be reduced by applying filtering material of a lower mass density. Commonly, however, such filters are back-washed without expansion, but for long periods, 10-20 minutes, to obtain a more complete removal of the loosened impurities.

Back-washing is commonly characterised by the amount of sandbed expansion obtained. With the notations of fig. 3.2

$$E = \frac{L_e - L}{L}$$

which factor is chosen larger as the grain sizes are smaller. In the past high amounts of sandbed expansion were applied, 50% for finishing filters with fine grains and 30% for pre-filters with coarser material. Today much smaller values are used, 15-20% for grains of 0.8 mm and 10% for grains of 1.2 mm size. With the total amount of filtering material unchanged

$(1 - p) L = (1 - p_e) L_e$, substituted

$$E = \frac{p_e - p}{1 - p_e} \quad \text{or} \quad p_e = \frac{p + E}{1 + E}$$

for which the required back-wash rate will be calculated in next section.

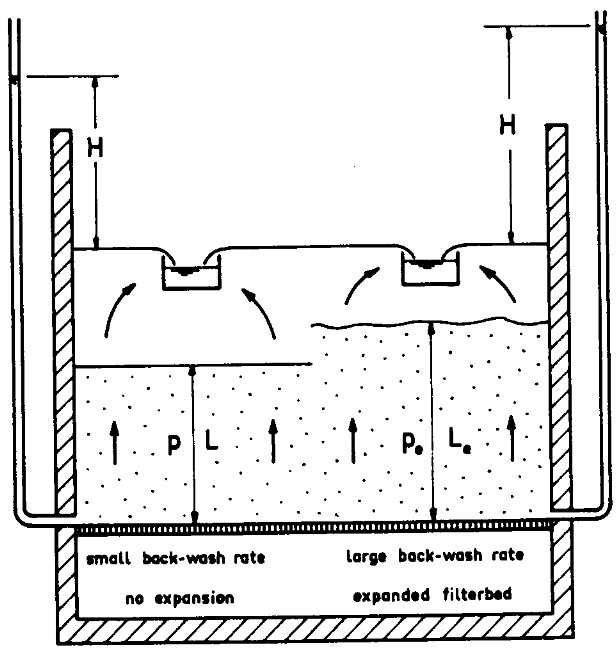



Fig. 3.2 Expansion during back-wash

### 3.2. Hydraulics of backwashing

As already mentioned in section 2.4, the head loss H accompanying the laminar flow with approach velocity v through a granular bed of thickness L, porosity p and composed of spherical grains of uniform diameter d, is given by the Carman-Kozeny equation as

$$H = 180 \frac{\nu}{g} \frac{(1-p)^2}{p^3} \frac{v}{d^2} L$$

with g as gravity constant and $\nu$ as kinematic viscosity of the fluid concerned. With the Reynolds number

$$Re = \frac{1}{1 - p} \frac{vd}{\nu}$$

this equation may also be written as

$$H = \frac{360}{Re} \frac{L}{d} \frac{1 - p}{p^3} \frac{v^2}{2g}$$

and shows indeed the inverse proportionality between H and Re, typical for laminar flow. This laminar flow in the meanwhile is only present when the Reynolds number is small, less than about 5. With flow of water at a temperature of $10^0C$, $\nu = (1.31)10^{-6}$ $m^2/sec$, through a bed with grain diameter $d = (1)10^{-3}$ m and porosity p = 0.4, this requirement limits the velocity to $v = (4)10^{-3}$ m/sec, a value which is mostly not surpassed in normal filtration practice. During backwashing, however, much higher velocities are applied, up to $(30)10^{-3}$ m/sec and sometimes even more. This means a flow in the transition region between laminar and turbulent water movement. For this region no exact equation can be drawn up, but many empirical formulae have been developed by various investigators. In the first part of the transition region, 5 < Re < 100, one of the best approximations reads

$$H = \frac{260}{Re^{0.8}} \frac{L}{d} \frac{1 - p}{p^3} \frac{v^2}{2g}$$

and after substitution of the value of Re

$$H = 130 \frac{\nu^{0.8}}{g} \frac{(1-p)^{1.8}}{p^3} \frac{v^{1.2}}{d^{1.8}} L$$ and for the expanded filterbed

$$H = 130 \frac{\nu^{0.8}}{g} \frac{(1-p_e)^{1.8}}{p_e^{\,3}} \frac{v^{1.2}}{d^{1.8}} L_e$$

The use of this formula is demonstrated in fig. 3.3, indicating for a sandbed $L = 1.2$ m thick with porosity $p = 0.4$ and various grain sizes d, the relation between the head loss H and the backwash rate v at a temperature of $10^{o}$C. When this head loss is not calculated, but actually measured, the values for small backwash rates will show good correspondence, but above a certain rate the resistance will remain constant. This happens when the head loss H equals the submerged weight of the filterbed and fluidization occurs

$$\rho g H = (1 - p)\, L\, (\rho_f - \rho_w) g \qquad \text{or}$$

$$H = (1 - p)\, L \frac{\rho_f - \rho_w}{\rho_w}$$

with $\rho_w$ and $\rho_f$ as mass densities of water and filtering material respectively. With spherical sand grains of one size,

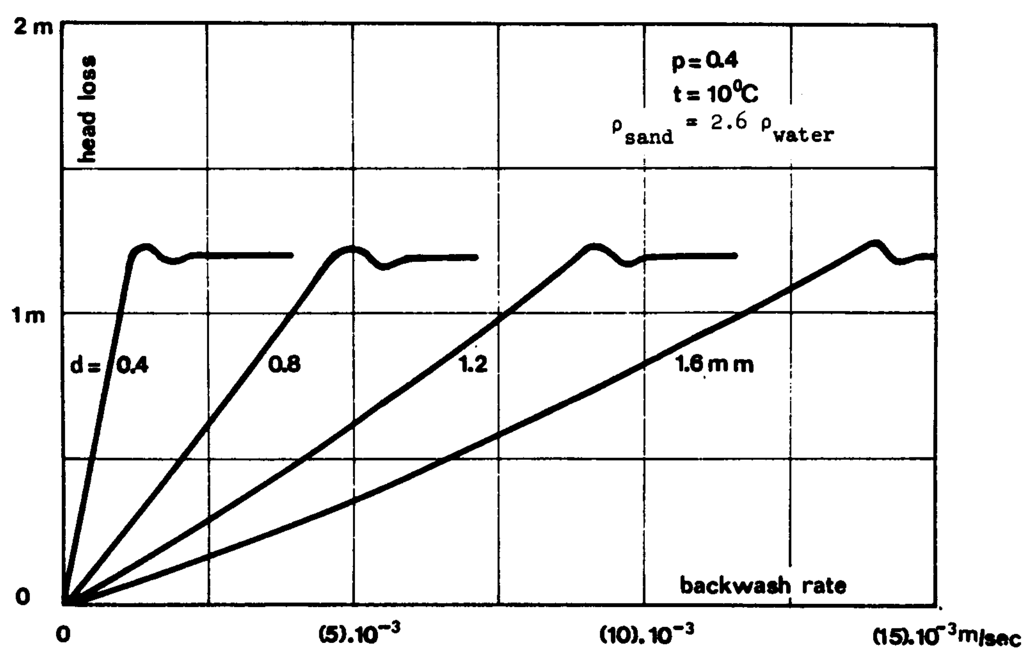



Fig. 3.3 Head loss of a sandbed 1.2 m thick, grainsize d, during backwashing

$$p \simeq 0.4, \quad \frac{\rho_f - \rho_w}{\rho_w} \simeq 1.6, \text{ giving } H \simeq L$$

that is to say a maximum head loss equal to about the thickness of the sandbed. This maximum head loss in the meanwhile will be higher when the original porosity p is smaller. Especially with fine filtergrains, adhering water and coatings of calciumcarbonate, ferric and aluminium hydroxyde, etc., may appreciably reduce the average mass density $\rho_f$, resulting on the other hand in a lower value of the maximum head loss.

The backwash rate v necessary to obtain an expansion E and porosity $p_e$ may be found by equating the two values of H calculated above

$$(1 - p)\, L\, \frac{\rho_f - \rho_w}{\rho_w} = 130\, \frac{\nu^{0.8}}{g}\, \frac{(1 - p_e)}{p_e^{\,3}}\, \frac{v^{1.2}}{d^{1.8}}\, L_e \quad \text{and with}$$

$$(1 - p)\, L = (1 - p_e)\, L_e \qquad \text{after rearranging terms}$$

$$v^{1.2} = \frac{g}{130\, \nu^{0.8}}\, \frac{\rho_f - \rho_w}{\rho_w}\, \frac{p_e^{\,3}}{(1 - p_e)^{0.8}}\, d^{\,1.8} \qquad \text{with}$$

$$\nu = \frac{(497)10^{-6}}{(\tau + 42.5)^{1.5}}$$ and $p_e$ calculated from the required amount of

sand bed expansion

$$p_e = \frac{p + E}{1 + E}$$

With these formula and a programmable pocket calculator, the required back-wash rate is not difficult to calculate. For sand as filtering material ($\rho_f/\rho_w = 2.6$), a grain size of 1.0 mm and a porosity p before backwashing equal to 0.38 they are tabulated below

| | E = 0 | 10 | 20 | 30 % | |
|---|---|---|---|---|---|
| t = 0°C | v = 4.5 | 6.8 | 9.3 | 11.9 | mm/s |
| 10 | 5.6 | 8.4 | 11.5 | 14.7 | |
| 20 | 6.6 | 10.0 | 13.6 | 17.5 | |
| 30 | 7.7 | 11.6 | 15.8 | 20.3 | |

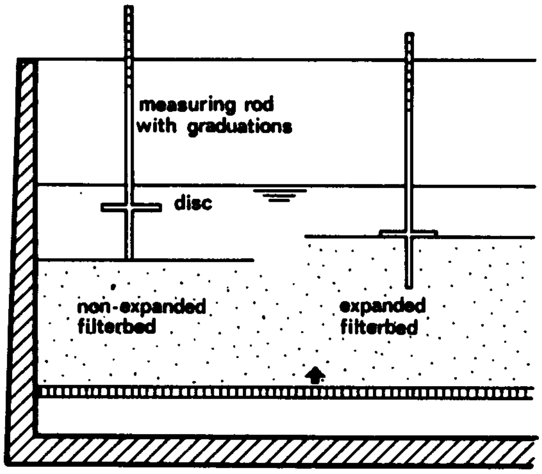



Fig. 3.4 Measurement of filterbed expansion during backwashing

As could be expected, the required back-wash rate increases strongly with the amount of filterbed expansion, but also with temperature. With river water in temperate climates summer conditions are therefore decisive. The amount of filterbed expansion can be measured with the device of fig. 3.4 which is self-explanatory.

The calculations given above in the meanwhile only hold true for uniform spherical grains. When the filtergrains are all of the same size, but with a non-spherical shape, the sandbed expansion at the same rate of backwash will usually be smaller, the free movement of the grains in the expanded sandbed enabling them to take a position which offers the least resistance to the upward flowing washwater. This influence cannot be calculated, but it can be measured with an experimental filter and subsequently taken into account by replacing in the formulae above the diameter d by the factor $\phi$'s with s as the clear opening of square woven wire sieves which just passes the grains and with $\phi'$ as correction factor smaller than unity. The results obtained in the Laboratory for Sanitary Engineering of the Department for Civil Engineering at the University of Technology in Delft (G.H. Corstjens, Journal $H_2O$, 1972) are shown in the table below

| lower sieve opening $s_i$ | 0.5 | 0.56 | 0.63 | 0.71 | 0.8 | 0.9 | 1.0 | 1.12 | 1.25 | 1.4 | 1.6 | 1.8 | 2.0 | mm |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| upper sieve opening $s_j$ | 0.56 | 0.63 | 0.71 | 0.8 | 0.9 | 1.0 | 1.12 | 1.25 | 1.4 | 1.6 | 1.8 | 2.0 | 2.24 | mm |
| $s = \sqrt{s_i s_j}$ | 0.529 | 0.594 | 0.669 | 0.754 | 0.848 | 0.949 | 1.058 | 1.184 | 1.323 | 1.497 | 1.697 | 1.898 | 2.118 | mm |
| Meuse sand $\phi^1 =$ | 1.02 | 1.02 | 1.01 | 1.00 | 0.99 | 0.97 | 0.95 | 0.92 | 0.89 | 0.85 | 0.80 | 0.76 | 0.70 | |
| broken gravel | 0.84 | 0.83 | 0.82 | 0.80 | 0.79 | 0.77 | 0.75 | 0.73 | 0.71 | 0.68 | 0.64 | 0.61 | 0.57 | |
| magnetite | 0.89 | 0.88 | 0.86 | 0.85 | 0.83 | 0.81 | 0.79 | 0.77 | 0.74 | 0.71 | 0.67 | 0.63 | 0.59 | |
| Wales anthracite | 0.97 | 0.96 | 0.95 | 0.94 | 0.92 | 0.91 | 0.90 | 0.88 | 0.86 | 0.84 | 0.81 | 0.78 | 0.76 | |
| Hydro-anthracite | 0.84 | 0.83 | 0.81 | 0.80 | 0.78 | 0.76 | 0.74 | 0.71 | 0.68 | 0.65 | 0.60 | 0.56 | 0.52 | |

With non-uniform filtering materials, backwashing will result in a stratification, with the fine grains in the upper and the coarse grains in the lower part of the filterbed. Backwashing such beds at low rates will only expand the upper part, while in the lower part the grains remain stationary, thus hampering the removal of impurities accumulated there during the previous filterrun. When for this reason the backwash rate is augmented to provide an adequate expansion of the lower part of the bed, the expansion of the upper part will be so high that a serious loss of filtering material might occur. This phenomenon can best be demonstrated with an example, assuming on one hand a uniform spherical material of 0.9 mm size and on the other hand a non-uniform spherical material of the same average size, but consisting of 5 equal portions with diameters of 0.7, 0.8, 0.9, 1.0 and 1.1 mm respectively. According to the formula given above, a 15% expansion of the uniform spherical material with 0.9 mm diameter, requires at $20^0$C and 38% original porosity a backwash rate of $(10.1)10^{-3}$ m/sec. For the upper portion of the non-uniform bed with 0.7 mm diameter, this rate provides an expansion of 29%, while for the lower portions of 1.1 mm size this expansion is only 6%. To raise the latter value to 10%, an increase of the backwash rate to $(11.5)10^{-3}$ m/sec is necessary. Judged by itself this has little disadvantages, but the expansion of the upper layer now rises to 35%. With sandbeds of 1.2 m thickness, the expansions at the latter backwash rate increase the thickness of the bed with non-uniform material by 0.25 m, while for the bed of uniform material this increase amounts to 0.18 m. To prevent, a loss of fine filtering materials during backwashing, the washwater troughs should be built with adequate freeboard, with their overflow edges according to fig. 3.17 about 0.6 m above the top of the unexpended sandbed. Any increase in this distance, however, hampers the removal of accumulated cloggings that have been floated to above, ultimately resulting in many filter troubles. Also with regard to back-washing, as uniform filtering materials as can be obtained should be used with the coefficient of uniformity at least below 1.5 and preferable below 1.3.

## 3.3. Equality of washwater distribution

In the preceding section the head losses accompanying the upward flow of washwater has been calculated, in the meanwhile neglecting the resistance of the filter bottom against water passage. At first sight this seems logical as any resistance of this bottom would increase energy consumption and therewith the price of water treatment. Absence of resistance, however, might impair the equal distribution of washwater over the full area of the filterbed, the point being that notwithstanding all precautions some irregularities will always occur.
In this way it is possible that locally, over area A of fig. 3.5 for instance, the backwashrate v + dv is slightly higher than the value v over the remaining part of the filterbed. This higher velocity will result in a higher porosity of the expanded bed, but not in a larger bed thickness, as the excess material flows away laterally. This means that over area A less filtering material is present, offering less resistance to the upward flow of washwater with a further increase of the backwash rate as unavoidable result. In its turn this higher rate will cause another increase of sandbed expansion, augmenting the porosity and lowering the resistance, from which again a rise in flow rate will follow, and so on, and so on. Finally nearly all the filtering material is removed from area A, forming a so-called sand boil as indicated in fig. 3.6. When the sandbed is supported by graded lay-

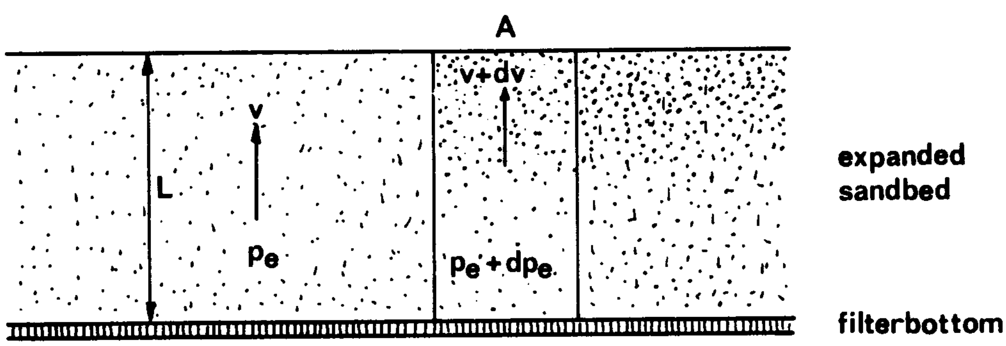



Fig. 3.5 Unequal washwater distribution

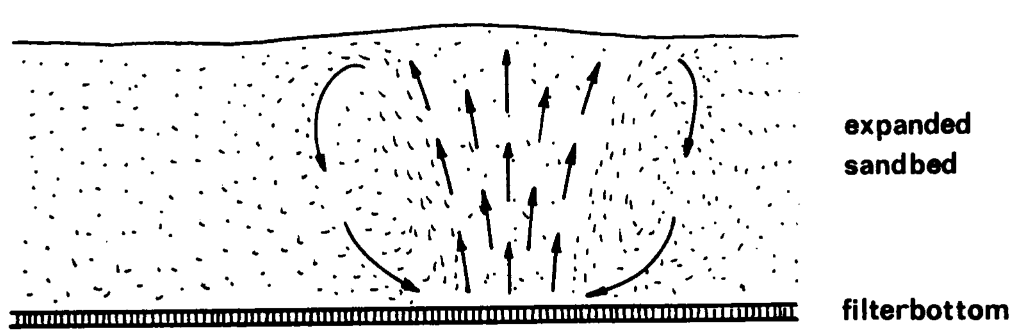



Fig. 3.6 Sandboil

ers of gravel, also the fine gravel grains at the top may become suspended, allowing the filtersand to penetrate and clog the underlaying coarser gravel (fig. 3.17). This increases the flow velocities in the pores between the gravel grains, bringing successively coarser layers in suspension. Ultimately the sand reaches and blocks the underdrains, after which expensive repairs are unavoidable. Initial disturbances will occur more easily and will have more serious effects, when the washwater is supplied and discharged under variable heads. As shown in fig. 3.18 , this is nearly unavoidable in practice. Remedial actions are now clearly indicated. As in similar cases, damping promises best result and in its turn this can be obtained by providing the filter bottom with a large resistance against the passage of washwater.

According to fig. 3.18 , the difference between the heads at which the washwater is supplied and discharged, varies between H and H + dH,

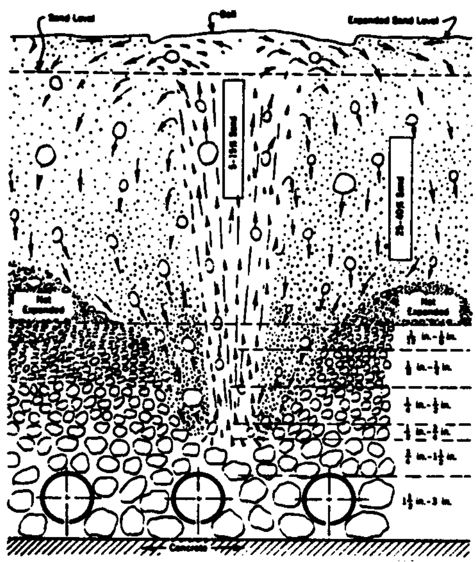


Fig. 3.7 Sandboil disturbing underlying gravel layers (after Baylis)

with

$dH = d_1H$ + or $-d_2H$, depending on local circumstances. This

head is used to overcome the resistance of the filterbottom and of the expanded sandbed

$$H = H_{bottom} + H_{bed}$$

The flowvelocities when passing the filterbottom are very high, resulting in turbulent watermovement and a quadratic resistance law

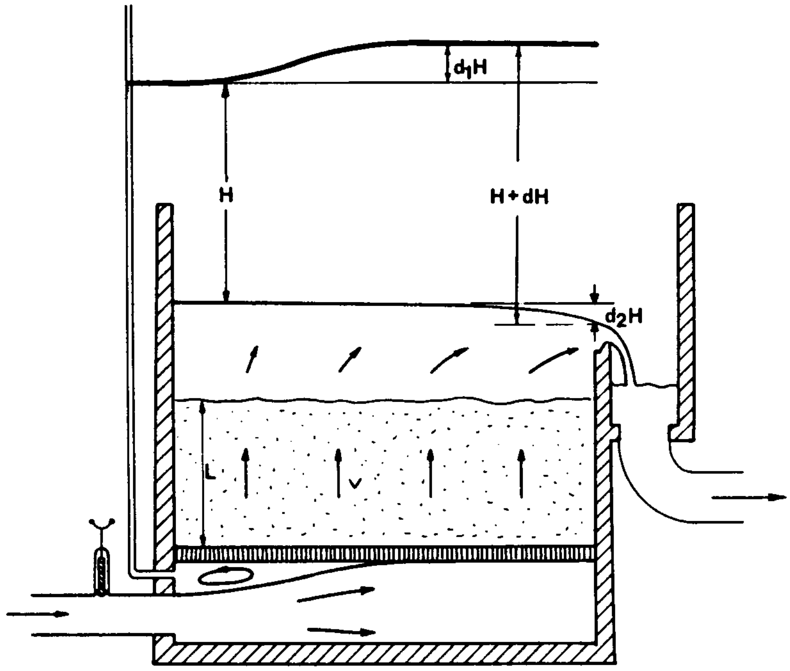



Fig. 3.8 Pressure distribution during backwashing

$$H_{bottom} = \alpha v^2$$

with v as backwashrate and $\alpha$ as proportionallity constant, the value of which depends on the construction of the underdrainage system. The resistance of the expanded filterbed equals its submerged weight

$$H_{bed} = (1 - p_e)\, L_e \; \frac{\rho_f - \rho_w}{\rho_w}$$

When locally the available head loss H increases by dH, an increase in backwashrate by dv and in pore space by $dp_e$ will occur, while the thickness of the filterbed $L_e$ remains unchanged. This changes the resistances of filterbottom and filterbed by

$$dH_{bottom} = 2\ \alpha v\ dv$$

$$dH_{bed} = -\ L_e \frac{\rho_f - \rho_w}{\rho_w} dp_e \quad , \qquad \text{together}$$

$$dH = 2\alpha v\ dv - L_e \frac{\rho_f - \rho_w}{\rho_w}\ dp_e$$

According to the preceding section the resistance of the expanded filterbed may also be written as

$$H_{bed} = 130 \frac{\nu^{0.8}}{g} \frac{(1-p_e)^{1.8}}{p_e^{3}} \frac{v^{1.2}}{d^{1.8}} L_e$$

As mentioned before, an increase in H only changes the values of v and $p_e$. The formula above may therefore be simplified to

$$H_{bed} = \beta \frac{(1-p_e)^{1.8}}{p_e^{3}} v^{1.2} \quad \text{with } \beta \text{ as a constant. With}$$

$$dH_{bed} = \frac{\partial H_{bed}}{\partial v} dv + \frac{\partial H_{bed}}{\partial p_e} d\ p_e$$

$$dH_{bed} = \beta \frac{(1-p_e)^{1.8}}{p_e^{3}}\ 1.2\ v^{0.2}\ dv + \beta\{\frac{-1.8(1-p_e)^{0.8}\ dp_e}{p_e^{3}} - \frac{3(1-p_e)^{1.8} dp_e}{p_e^{4}}\} v^{1.2}$$

or

$$dH_{bed} = H_{bed}\ \{\frac{1.2\ dv}{v} - \frac{1.8\ dp_e}{1-p_e} - \frac{3\ dp_e}{p_e}\}$$

Substitution of the values for $dH_{bed}$ and $H_{bed}$ mentioned above gives

$$-L_e \frac{\rho_f - \rho_w}{\rho_w} dp_e = (1-p_e)\ L_e \frac{\rho_f - \rho_w}{\rho_w} \{\frac{1.2\ dv}{v} - \frac{1.8\ dp_e}{1-p_e} - \frac{3\ dp_e}{p_e}\}$$

from which follows

$$dp_e = \frac{1.2(1-p_e)p_e}{3-2.2\ p_e} \frac{dv}{v}$$

The total increase in resistance thus becomes

$$dH = 2\ \alpha v\ dv - L_e \frac{\rho_f - \rho_w}{\rho_w} \frac{1.2\ (1-p_e)p_e}{3-2.2\ p_e} \frac{dv}{v}$$

which may be simplified to .

$$dH = 2\ H_{bottom} \frac{dv}{v} - 1.2\ H_{bed} \frac{p_e}{3-2.2\ p_e} \frac{dv}{v}$$

From this equation the required resistance of the filterbottom may be calculated

$$H_{bottom} = 0.6\ H_{bed} \frac{p_e}{3 - 2.2\ p_e} + \frac{1}{2} \frac{v}{dv}\ dH$$

which for p = 0.38, 20% filterbed expansion and $p_e$ = 0.48 may further be simplified to

$$H_{bottom} = 0.15\ H_{bed} + \frac{1}{2} \frac{v}{dv}\ dH$$

With sand as filtering material, the resistance of the bed against back-washing is about equal tot the bed thickness say 1.2 m. Allowing moreover a 3 % variation in backwashrate, dv = 0.03 v and assuming dH = 0.05 m gives finally

$$H_{bottom} = (0.15)(1.2) + \tfrac{1}{2} \frac{100}{3} (0.05) = 0.18 + 0.83 = 1.01\ m$$

a large value indeed. In practice moreover, a value of 0.05 m for dH is rather small and this is the reason that filterbottoms often have resistances against backwashing as high as 2 or 3 m water column. This resistance is proportional to the square of the flowrate, meaning that during filtration values of only a few centimeters occur.

### 3.4. Supply of washwater

Water needed for backwashing a filter may be supplied in different ways, by the distribution system, by special washwater pumps connected to the clear well or by an elevated washwater reservoir. Which solution is most attractive in a particular case depends primarily on the required backwash capacity compared to the production of the plant as a whole and on the minimum time interval between two successive cleanings in relation to the actual washing period.

Taking back-wash water from the distribution system is bad practice. Unless the number of filtering units is large and the back-wash rate low, this will result in strong variations in system pressure.

With this pressure usually much higher than the head required for backwashing an appreciable loss of energy will moreover occur and finally there is always the danger that a failure of the pressure reducing device results in a backwash rate many times larger than the intended one. This will overturn the filterbed, even flushing filtering material over the walls of the filterbox.

As indicated above, the capacity of washwater pumps discharging directly into the washing system must be rather large. This means big and expensive pumps and when driven by electricity from the public grid, a high charge for connected power. As with all moving machinery, these pumps and motors are subject to wear and tear and to sudden failures, asking for reserve units, installing for instance four pumps of which only two are used simultaneously, keeping one in reserve when the fourth one is being repaired. In case the water temperature and/or the clogging properties of the raw water vary during the year, the required backwash rate will show great seasonal fluctuations. Even when for greater flexibility the number of pumps is increased, throttling down will still be necessary to obtain the exact rate wanted, augmenting energy losses. Washwater pumps moreover work intermittently, requiring storage space for which the capacity of the clear well must be augmented accordingly.

Elevated washwater tanks (fig. 3.9) have the enormous advantage of unlimited flexibility, permitting backwashing at any rate, even at rates higher than anticipated. They are filled between washings by relatively small pumps,

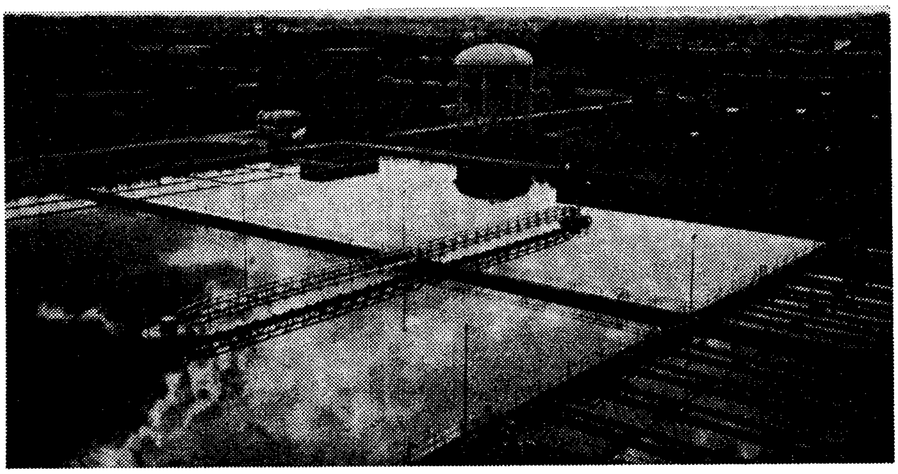


Fig. 3.9 Elevated wash-water tank

combined capacity only 10 or 20% of the backwash rate, working more or less continuously at the highest efficiency, while additional storage space in the clear well can be omitted. These advantages must be balanced, however, against the cost of construction, which will be higher as a larger volume is required and as the tanks must be set at a greater elevation. Even when nearly empty, the head supplied by the tank must be sufficient to deliver the desired flow of washwater to the most remote filtering unit, asking for an adequate distance between the bottom of the tank and the top of the washwater trough (fig. 3.10). In practice this distance varies between 5 and 10 m, smaller when the washwater tank is installed in the centre of the filtration plant and larger when for economic reasons the washwater pipelines are designed for high velocities, 3 or 4 m/sec for instance. Washwater tanks should have sufficient capacity to take care of maximum requirements, in larger plants allowing two consecutive washings at rates and during periods somewhat larger than usual. When for instance the filters are normally washed during not more than 180 sec at a rate not exceeding $(15)10^{-3}$ m/sec, corresponding with a washwater consumption of 2.7 $m^3$ per $m^2$ of filterbed area, the tank volume should be chosen at 120% of (2)(2.7) or 6.5 $m^3$ per $m^2$. With a large number of filtering units and occasional short filter runs due to a deterioration of raw water quality,

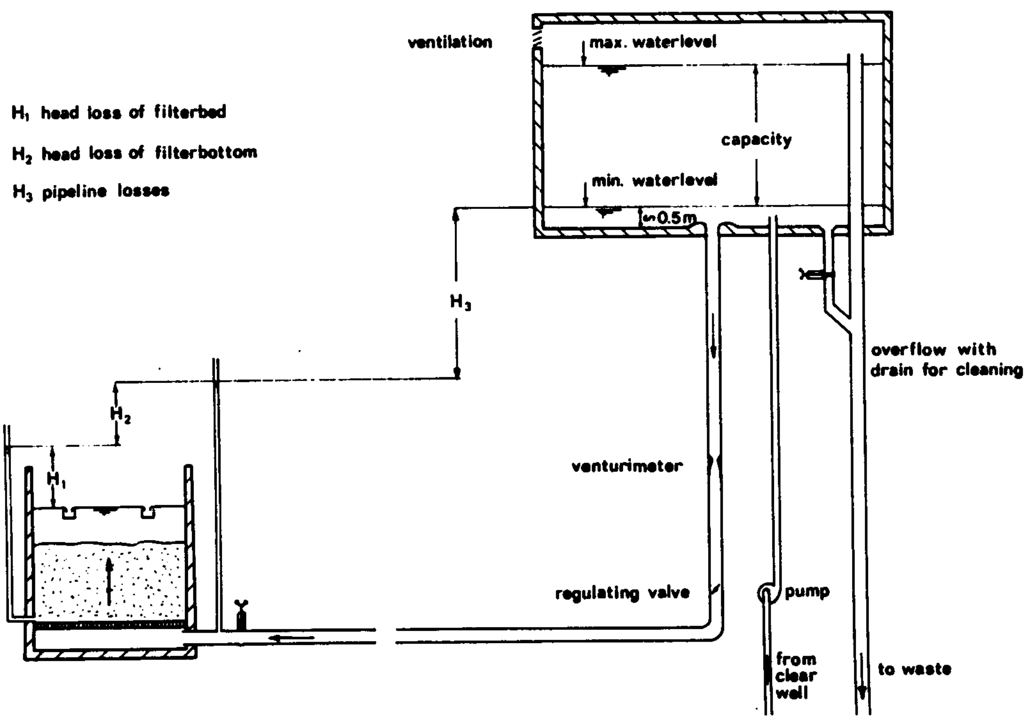



Fig. 3.10 Arrangement of washwater tank

the possibility of backwashing two filters simultaneously should be anticipated. Either double washwater tanks and washwater supply lines should be installed or the size of the tank and piping increased accordingly. In the latter case the tank volume should be sufficient for at least 3 backwashings under unfavorable conditions.

Summing up, taking backwash water from the distribution system should be avoided as much as possible. When small filtering units can be backwashed the year round at the same rate, washwater pumps will give satisfactory results at the lowest price. With large filtering units washwater tanks may be more economical, while they are **certainly** more attractive when backwash rates vary strongly from one period to another. Whatever system of backwashing is applied, the amount of washwater consumed should be carefully recorded. Study of these records provides excellent information about the efficiency of the backwash process. Normally this consumption varies between 1 and 3% of filtered water production. This amount is so small that there is little sense in trying to achieve further economies, which also might endanger the filtration process by unsufficient cleaning of the filterbed.

During backwashing, many valves must be opened and closed. The washwater supply valve in particular should be operated with care, opened slowly and just far enough to obtain the desired backwash rate and closed slowly to allow the expanded filterbed to settle down evenly. Manual operation of these valves is nowadays an exception and mostly they are operated hydraulically, pneumatically or electrically from a control table on the operating **floor.** When this table is located near the filter to be backwashed, the filter **attendant** is able to observe any defects as soon as they appear, allowing timely repairs before much damage is done. In Western-type countries, however, this work has nowadays little appeal, in particular on a 24 hours per day basis. This has lead to the development of fully automated backwash installations needing only periodic adjustments of backwash rate (with surface water sources for every 5 $^{0}$C variation in water temperature) and duration. Small computers are nowadays cheap and the expense of such automatically operated installations is therefore small. They have the great advantage that each time the backwash is carried out exactly in the way prescribed, without any human errors or inaccuracies. They lack, however, the eye of the master that **fatters** the horse!

When the raw water is taken from a spring or a mountain stream some distance above the area of distribution, the filtered water can be transported by gravity. No energy is required for this purpose and a great simplification could now be obtained when also back-washing could be accomplished without pumping. This possibility is shown in fig. 3.11, where the filter is back-washed with the effluent of the other ones. To obtain a high enough back-wash rate, the number of filters should be at least 4 to 6, while to overcome the resistance against back-washing, the depth of supernatant water must be large, 3 to 4 m, increasing the cost of construction.

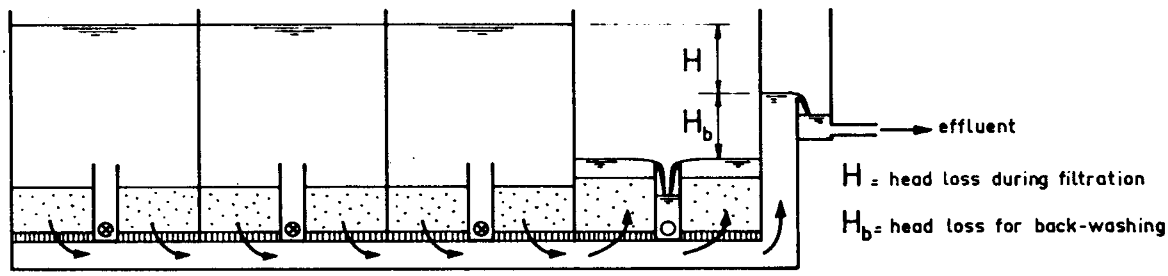



Fig. 3.11 Backwashing a filter with the effluent of the others

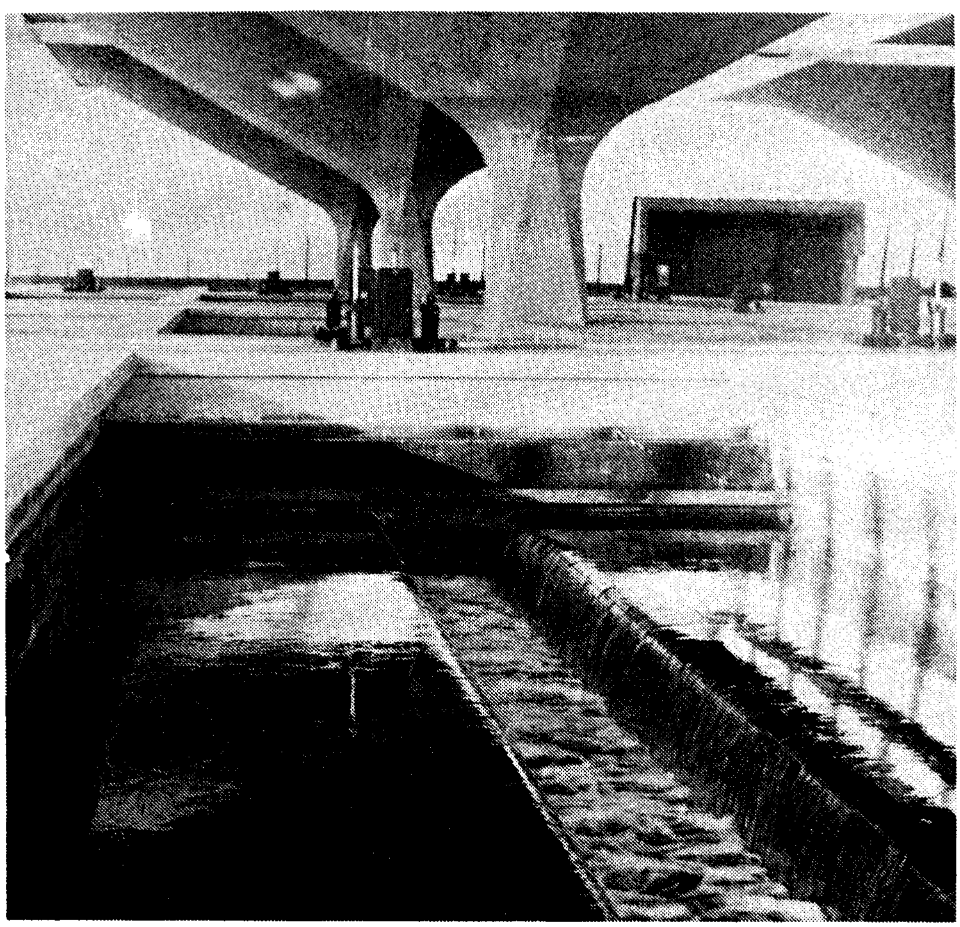


Fig. 3.12 Wash-water discharge

## 3.5. Discharge of washwater

After passing the filterbed, the washwater together with the impurities removed from the openings between the sand grains must be discharged to waste,(fig. 3.12)for which a system of troughs and gutters is commonly provided. This system must be arranged such as to limit the horizontal travel of the dirty water, the point being that the upward velocity of the washwater decreases by a factor 1.5 to 2.5 the moment it leaves the filterbed. Particulate suspended matter with a settling velocity of for instance 1.2 times the backwash rate is easily floated to above, but will as easily settle in the depth of water above the expanded filterbed. In practice the maximum permissible length of horizontal travel varies from 0.75 to about 2.5 m, larger as backwashing occurs at higher rates and the washed-out impurities are more finely divided and of lower specific gravity. Various arrangements of washwater troughs are shown in fig. 3.13. In larger filters, the troughs discharge their water into a central gutter, over which edge no water is taken. The distance between troughs may be increased and a saving in cost of construction obtained, by flushing the depth of water above the filterbed with an additional supply of water. Mostly raw water is used for this purpose as shown in fig. 3.14. With this so-called water sweep, the length

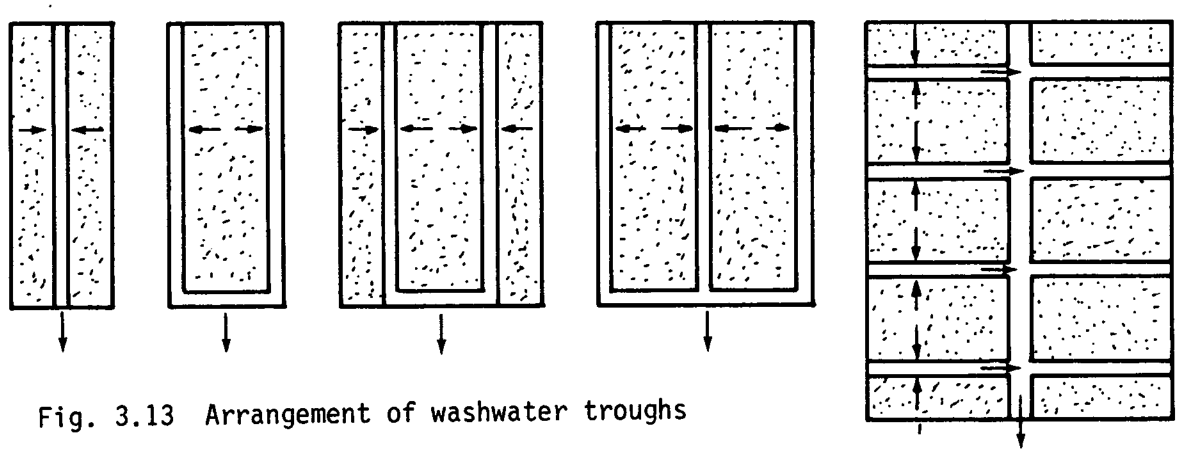


Fig. 3.13 Arrangement of washwater troughs

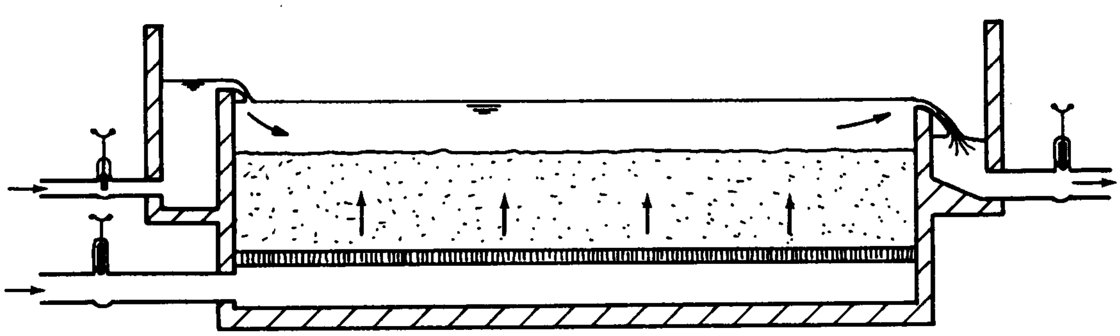


Fig. 3.14 Water sweep.

of horizontal travel may be increased to 10 m and sometimes even more. It has the disadvantage, however, of changing the head available for backwashing ($\Delta_2H$ in fig. 3.8 ), thus resulting in a less equal distribution of the washwater supplied. For a limited period only, the horizontal velocity may also be increased by replacing the fixed discharge weir of fig. 3.14 by a collapsible one (fig. 3.15 ) or by a syphon (fig. 3.16), with as added advantages that the depth of perhaps still dirty water left on top of the sandbed after washing is reduced and a regular inspection of the filterbed is possible.

The upper, overflow edge of the washwater troughs should be placed sufficiently near to the surface of the sand so that the washed-out impurities are removed easily and in short time and no large quantity of washwater is left in the filter after completion of washing. On the other hand, however, this upper edge should be set a minimum distance of about 0.25 m above the top of the expanded sandbed to prevent loss of sand during washing as much as possible. For the same reason the bottom of the trough must be kept at least 0.05 m above the expanded sandbed (fig. 3.17). With a

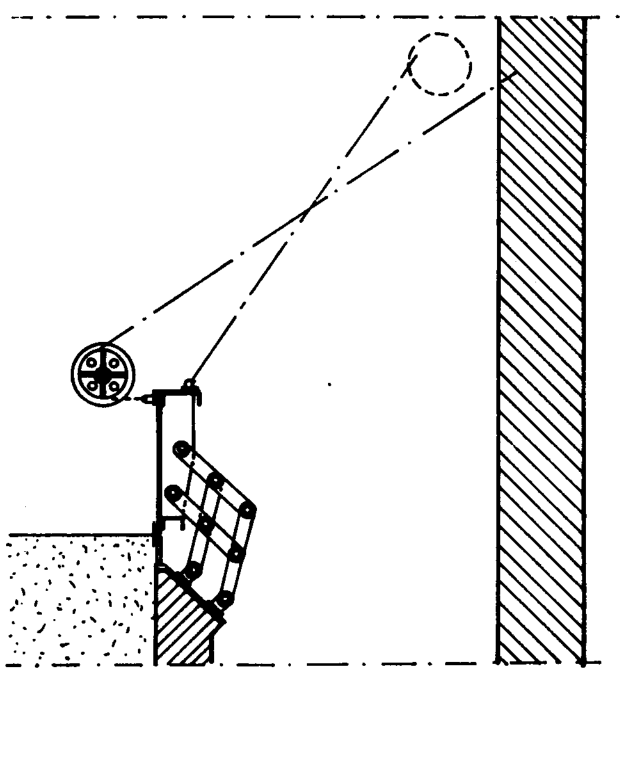


Fig. 3.15 Collapsible weir for washwater removal

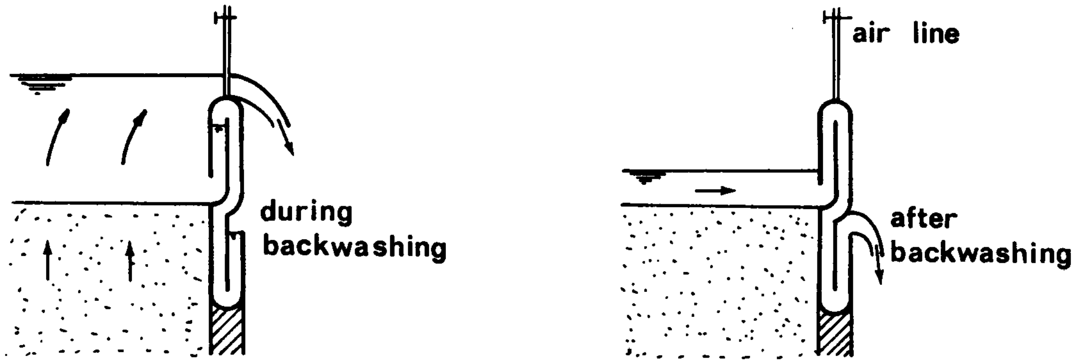



Fig. 3.16 Syphon for washwater removal

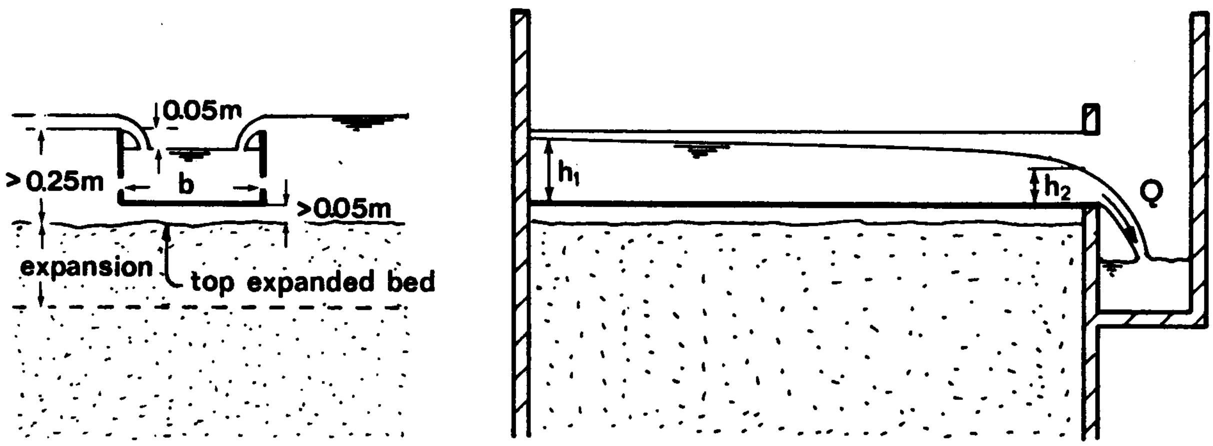



Fig. 3.17 Washwater gutter

sandbed 1.2 m thick and 20% expansion during backwashing, this means a vertical distance between the upper edge of the washwater trough and the unexpanded sandbed of 0.5 to 0.6 m, depending on the size of the trough itself.

The cross sectional area of the troughs should be large enough to carry the maximum amount of washwater with at least 0.05 m freeboard, so preventing submergence and unequal abstraction. For the hydraulic design of these troughs it may be assumed that the kinetic energy of the water falling into it does not contribute to the lateral velocity, that friction is negligeable and that the flow is substantially horizontal in direction. The depth $h_2$ at the outlet end of the trough depends on the conditions prevailing in the central gutter. The depth $h_1$ at the other end can be calculated with the momentum theory. With a horizontal gutter of rectangular cross-section, constant width and discharging an amount of Q $m^3$/sec

$$h_1 = \sqrt{h_2^{\,2} + \frac{2Q^2}{gb^2h_2}}$$

With free discharging troughs, $h_2$ closely approximates critical depth

$$h_2 = \sqrt[3]{\frac{Q^2}{gb^2}}\ , \qquad\qquad h_1 = \sqrt{3}\,h_2$$

With gutters of varying cross-section and/or sloping bottoms, the drop in level will be slightly larger. Calculation of the water movement in the central gutter follows the same pattern, with the only difference that the flow increases stepwise instead of uniformely. In case the gutters are of great length, the friction loses may not be neglected and should properly be taken into account.

## 3.6. Washwater disposal

There are still cases indeed where the washwater after performing its duty in cleaning the rapid filterbed, can be discharged to waste, into a sewage system or back to the river from which the raw water has been taken. With the growing concern for environmental pollution, however, this is nowadays an exception and mostly some treatment before discharge is needed. Looking only at the cost of construction, plain sedimentation using simple dug basins without a lining (fig. 3.18) certainly gives the cheapest solution. Taking into account the cost of operation, they only remain economical when the raw water to be treated has a low silt content so that cleaning of the settling basins by draining and digging is only necessary once in a while. With somewhat higher silt contents and the necessity to remove sludge deposits at intervals of one to a few years, suction dredging can be used to advantage. With heavily silt laden waters, however, mechanical sludge removal in sedimentation tanks constructed from reinforced or prestressed concrete becomes a necessity (fig. 3.19). If the effluent of such basins has to satisfy high standards, coagulants or flocculants may be added to the incoming water to increase settling efficiency, while in extreme cases sedimentation must be followed or replaced by filtration. Effluent quality may now be better, or only slightly less than the quality of the raw water going to the rapid filtration plant, allowing recirculation of the washwater and doing away with the necessity of disposal alto-

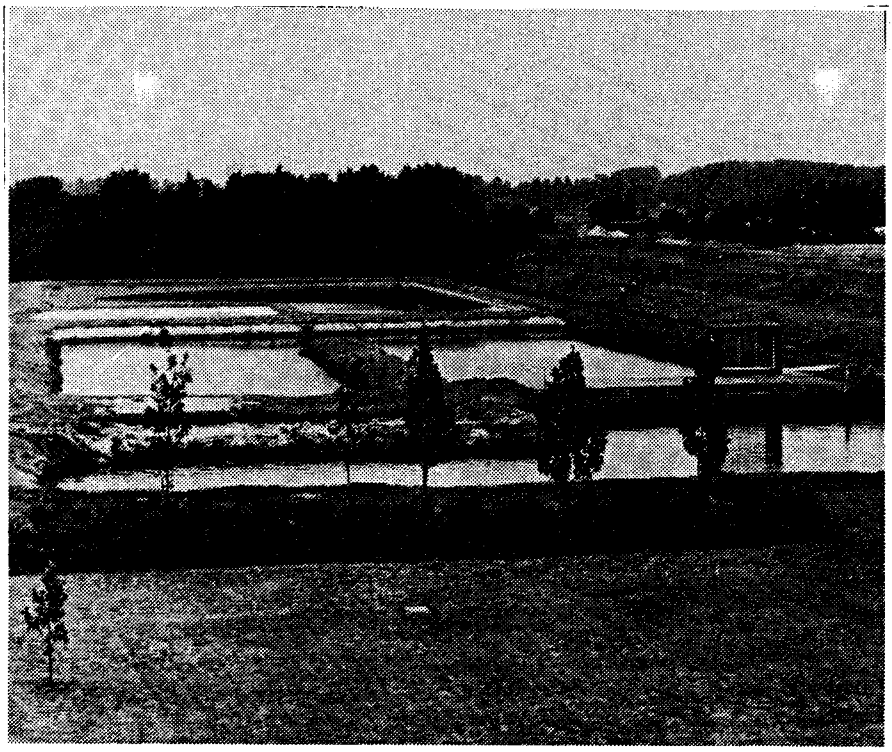


Fig. 3.18  Simple dug basins for washwater purification by settling

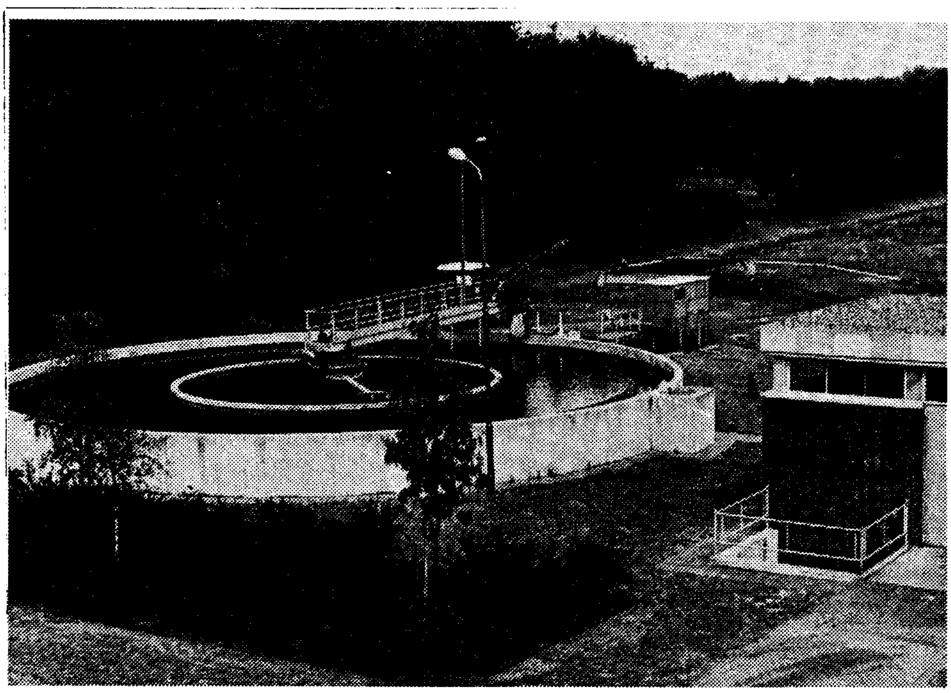


Fig. 3.19  Coagulation supported upward flow sedimentation of washwater at the dune-water treatment plant of Amsterdam Municipal Water Works

gether. Washwater re-use is certainly attractive when the raw water source is located at a great distance from the treatment plant, so that it already carries a high cost of transportation. The same holds true when this water has been submitted to an extensive and expensive system of pre-treatment such as softening, artificial recharge and so on.

Sedimentation and filtration in the meanwhile only separate the suspended matter from the dirty washwater, but do not destroy it, leaving a difficult sludge disposal problem. Mostly the water content of this sludge is very high, 99% or more and direct transportation is only possible by pipelines or tanks. Ordinary lorries can be applied for this purpose after the water content has been lowered to about 60 or 70%, using sludge thickeners (fig. 3.20) and one of the many systems for natural or artificial sludge drying (fig. 3.21 and 3.22). In case the sludge is of mineral, inorganic origin only, it may subsequently be dumped, using it for land-fills

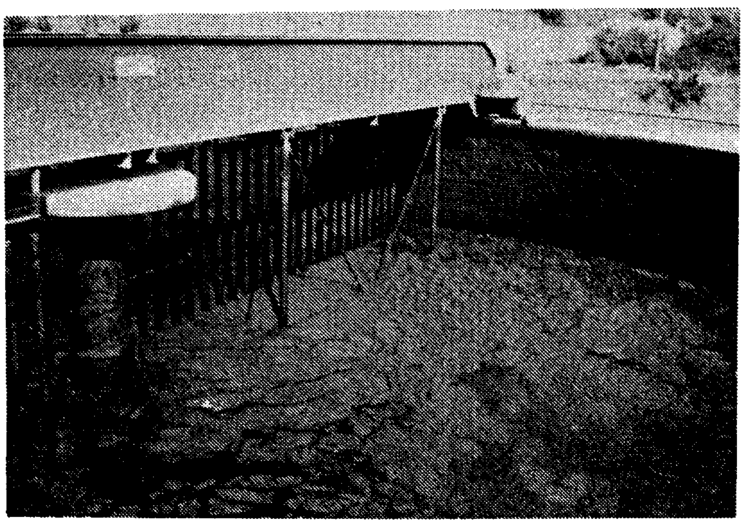


Fig. 3.20 Sludge thickening

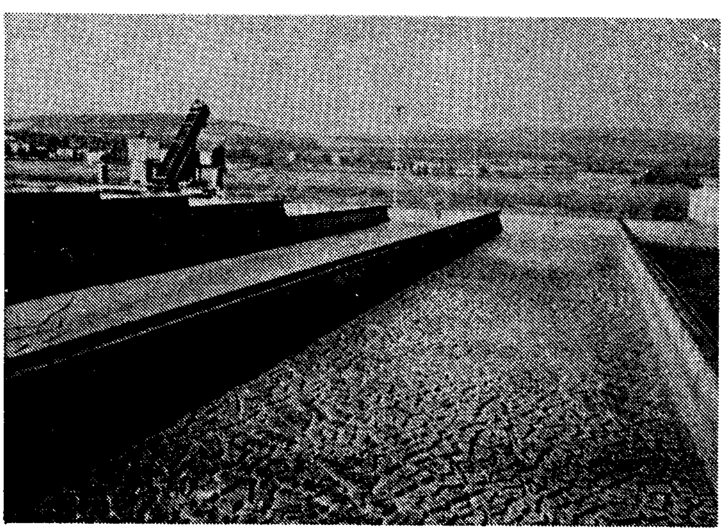


Fig. 3.21 Sludge drying beds (Wiesbaden)

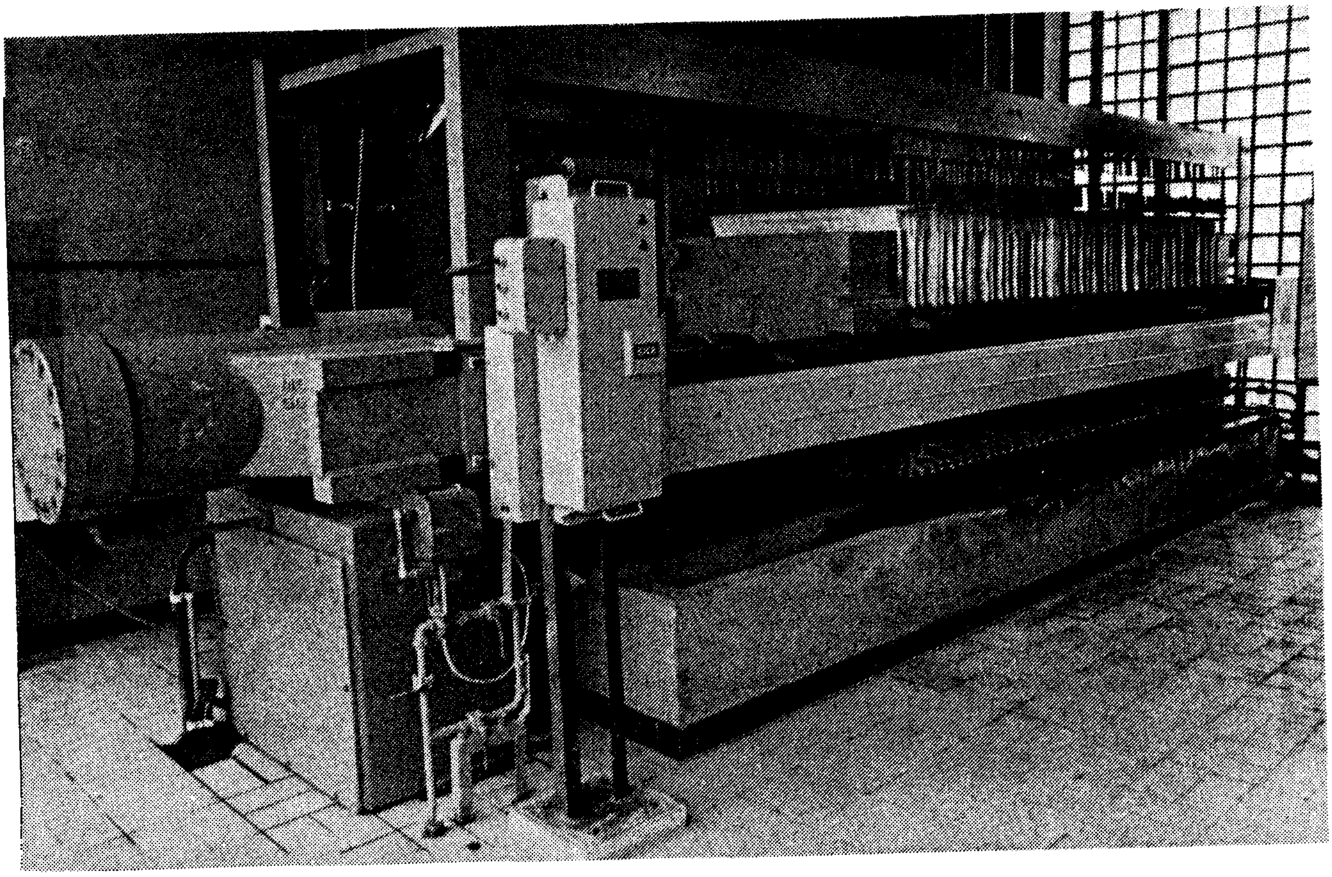


Fig. 3.22 Mechanical sludge drying

of abandoned pits, quarries, etc. With a high organic content, putrifaction of the sludge will ultimately set in, causing bad odors, attracting flies etc. Sanitary landfills, if possible together with the solid refuse of a neighbouring community, must now be practised or the sludge stabilized before disposal, using one of the many methods for anaerobic or aerobic sludge digestion, including wet or dry oxidation, again preferable in combination with other sludges, of sewage treatment plants for instance.

To promote flocculation of suspended matter, in this way improving filtration efficiency, the raw water to be treated is sometimes dosed with iron or aluminium salts .

Recovery of these salts from the sludge mentioned above and reuse after processing has certainly the advantage of reducing environmental pollution. In some instances it is also an economic proposition.

## 3.7. Filterbed troubles

With fine-grained filtering material, suspended matter from the raw water is mostly deposited on top and in the very upper part of the filterbed. This phenomenon is called surface or cake filtration and has as result that only over a small depth l of the filterbed, the resistance against downward water movement increases with time. At the end of the filterrun, this thin layer of filtering material is loaded by a large water pressure (A-B in fig. 3.23), which must be taken up by the grain pressures below, with a compression of this layer as unavoidable result.

In many cases these grains carry a sticky gelatinous coating by which the compression forms a tough crust, which during backwashing is not desintegrated but only broken up in smaller and larger bits. Some of these bits are so large, that the upward flow of washwater is unable to carry them to waste. They remain in the filterbed indefinitely, grow together again and form with adhering sand grains so-called mud balls of higher specific gravity (fig. 3.24) . After some time these mud balls have collected so much of the original filtering material that their specific gravity is larger than that of the sand-water mixture present in a well expanded sandbed during backwashing. In this way they are able to sink to the bottom of the filterbed, where they grow together into mud banks, clogging part of the filterbottom (fig. 3.25). As well during backwash as during filtration,

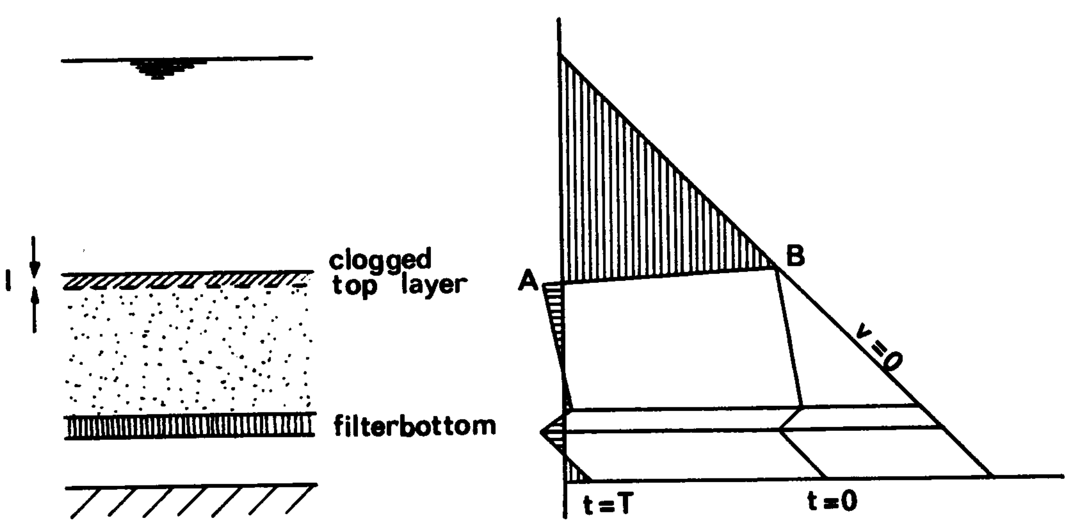
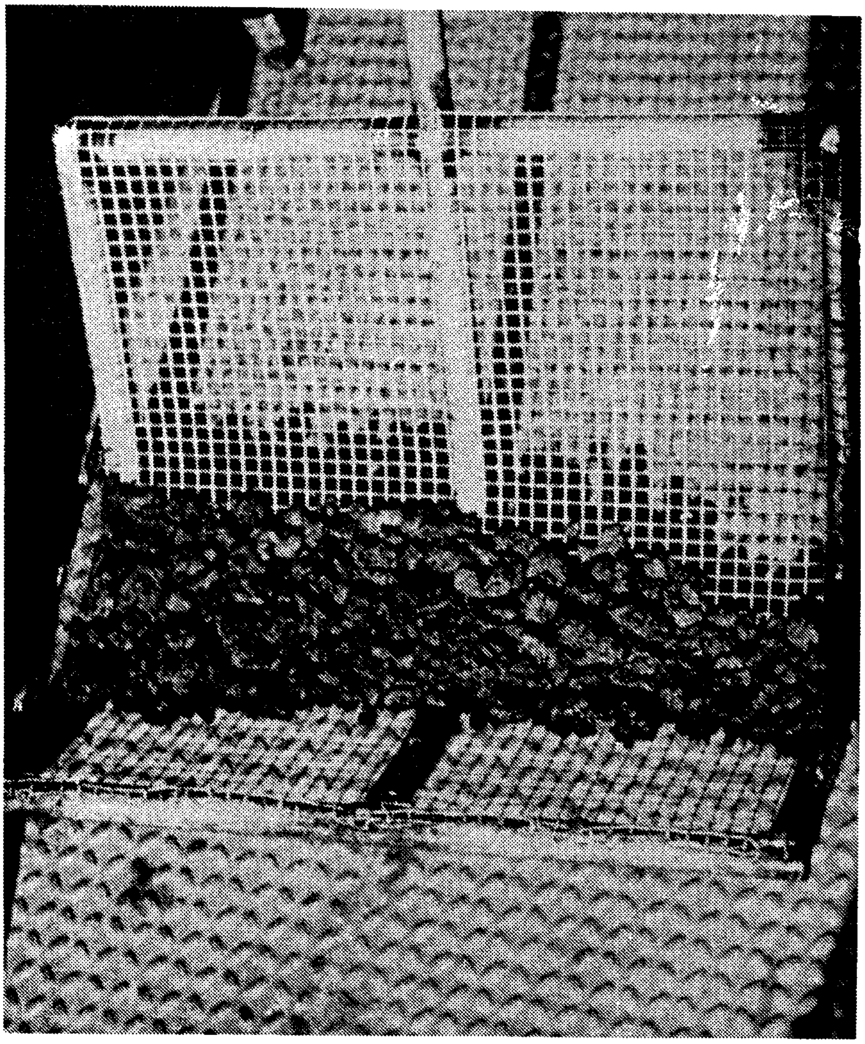



Fig. 3.23 Pressure distribution with surface filtration



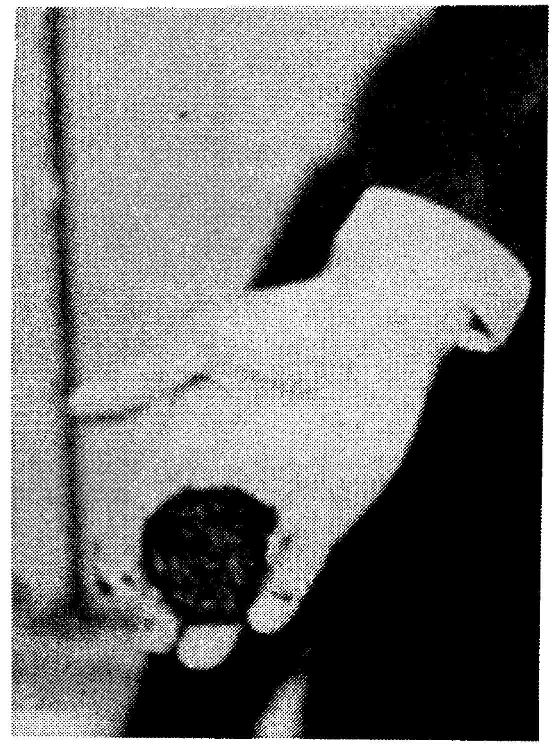


Fig. 3.24 Mud balls on top of a rapid filter

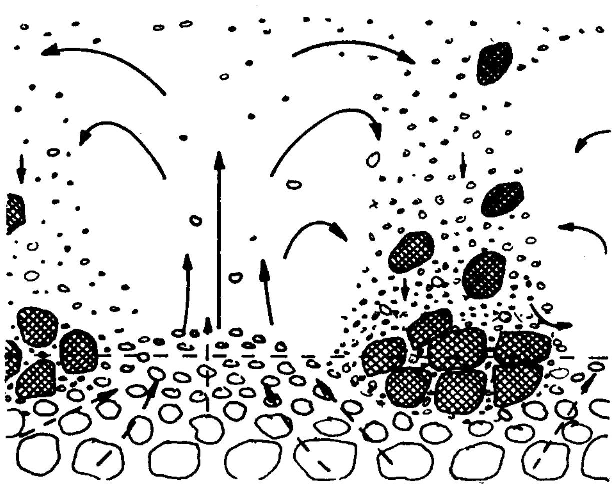


Fig. 3.25 Mud balls clogging a filterbottom

only the remaining portion of the filterbed is now effective, increasing actual filtration rates, in this way deteriorating effluent quality and shortening filterruns, while the increase in actual backwash rate will result in an appreciable loss of filtering material. When the filterbottom is provided with graded layers of gravel, the lateral deflection of the washwater under the clogged area may carry some of the fine gravel with it. In the course of time the whole top layer of fine gravel under such spots is displaced, after which mud balls and filtering material alike have access to the coarser grained gravel below, thoroughly clogging the filterbottom. The only remedy is rebuilding the filter, an unpleasant, expensive and time consuming job.

Along the more or less smooth walls of a rapid filterbox, the resistance against downward water movement will always be smaller than in the filterbed proper. Head losses along these walls will consequently be less than in the body of the filterbed (fig. 3.26), giving rise to an excess water pressure which tries to move the filtering material away from the walls. With clean coarse grained sand this will have no adverse effects, but with fine grained material filter cracks may develop (fig.3.27) when by surface filtration the pressure differences are larger and the grains are coated with soft and compressible material. Through these cracks raw water may penetrate the filterbed to great depth, reducing filtration efficiency and deteriorating effluent quality. The deposition of suspended matter from the raw water in these cracks, will also result in mud banks,

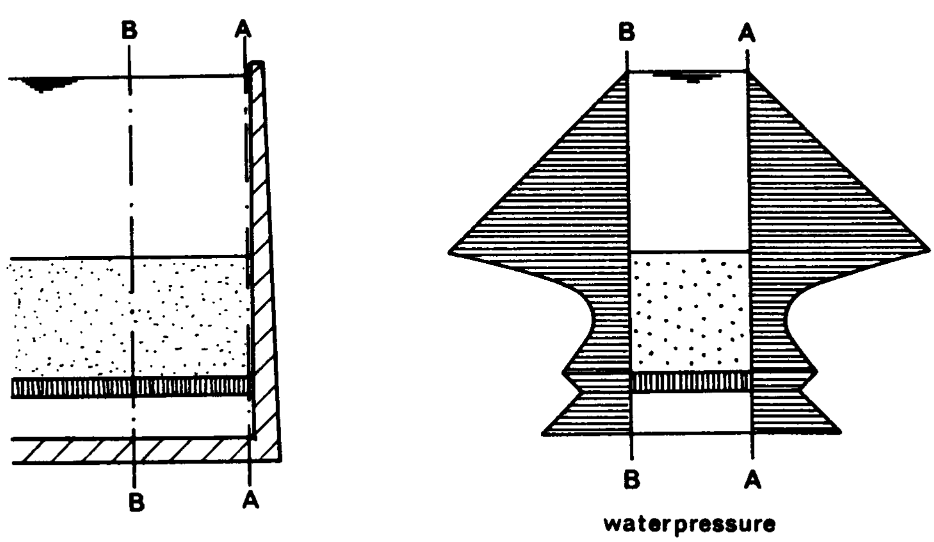



Fig. 3.26 Wall effect

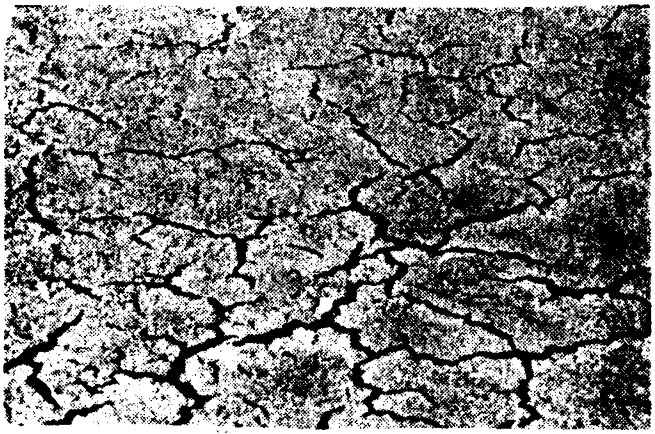


Fig. 3.27 Filter cracks

extending now from the walls into the filterbed and again disturbing both the process of filtration as backwashing. Incidentally, the occurrence of filter cracks cannot be prevented by giving the walls of the filterbox a rough surface. On contrary, along such rough walls the filtering material will settle more slowly after expansion during backwash, resulting in a higher porosity and a much increased permeability.

The filterbed troubles of mud balls and filtercracks are primarily due to the use of fine grained filtering material. Already with a low rate of backwash and a correspondingly small amount of hydraulic scour, a large filterbed expansion occurs. separating the grains and reducing the effect of scouring against each other. Coatings of organic material are thus not fully removed from the grains, resulting in vertical as well as horizontal compression when loaded by hydraulic forces. From this description it will be clear that the best way to avoid these filterbed troubles is the use of coarser filtering material which on one hand can be kept cleaner by backwashing with water alone, allowing on the other hand a deeper penetration of suspended and colloïdal matter from the raw water (deep-bed filtration), thus reducing pressure differences. When with regard to effluent quality a coarser grained filtering material cannot be applied, filterbeds can be kept cleaner by the use of a filtering material with a higher mass density such as magnetite and garnet or by the use of an auxiliary scour. With regard to the cost involved, heavier filtering materials are seldom used for this purpose, but an increase of the mechanical scour by an additional stirring of the filterbed during backwash is quite popular. Different systems are available for this purpose as will be explained in next section.

## 3.8. Auxiliary scour

To keep filterbeds clean on the long run, the scour produced by backwashing with water alone is insufficient with light-weight filtergrains and an additional stirring of the expanded sandbed is necessary for this purpose. This auxiliary scour can be obtained in different ways, in chronological order of application, mechanically by rotating rakes, pneumatically by compressed air and hydraulically by a surface wash, as shown schematically in fig. 3.28.

Mechanical agitation of the filtering material during backwash by means of revolving rakes was applied universally in the dawn of rapid filtration at the end of last century and in the beginning of the present one (fig. 3.29). Excellent results were obtained in this way, but the technology of that time required drive mechanisms consisting of long spindles and gears, pulleys and belts which were heavy, cumbersome and vulnerable, while the necessity to give such filters a circular plan added further to the

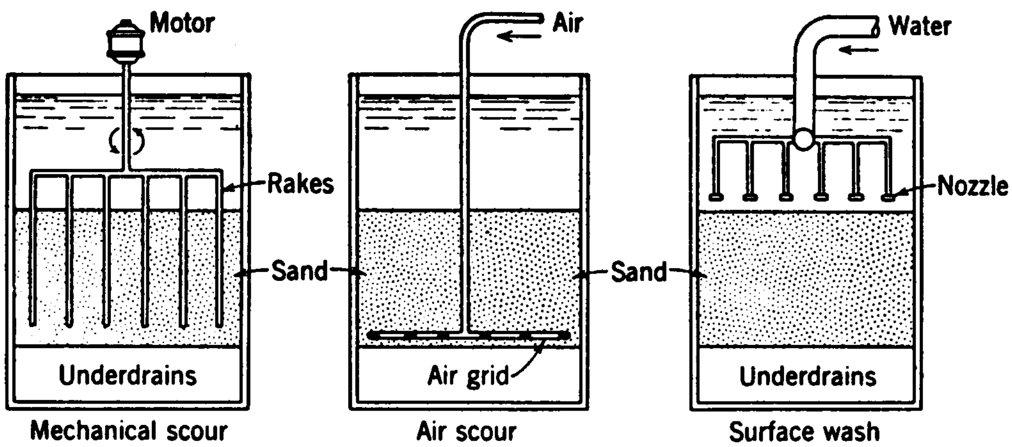



Fig. 3.28 Auxiliary scour

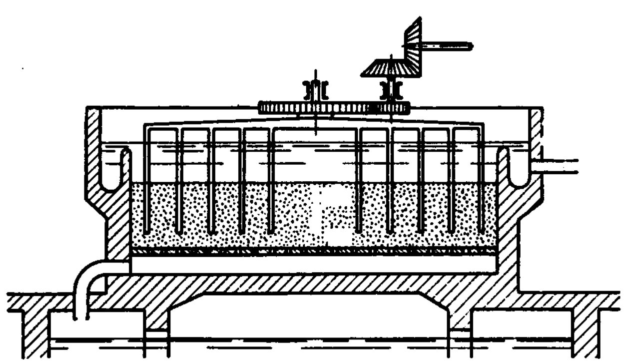


Fig. 3.29 Mechanical rakes for added agitation during backwashing

cost of construction (fig. 3.30). These disadvantages have lead to a complete abandonment of this method and today it is difficult to find such a filter in operation. This may seem strange as nowadays excellent electric motors, cheap, rugged and reliable are available for individual drive, while the use of pre-stressed concrete or steel as building material for the filterbox makes a circular plan attractive anyway. In future, after hesitation to leave the beaten tracks has been overcome, a new application of this mechanical agitation with rotating rakes may be expected.

Air-wash as a mean for additional agitation during backwash has gained enormous popularity in Europe and here practically all rapid filters built during the last decades have been equipped with it, even in those cases where by the presence of coarse filtering material an added scour was not strictly necessary. In some cases air-wash is even used prior to backwashing with water, the air serving to scour the grains, to remove the accumulated impurities from the filtergrain surfaces and the subsequent waterwash to flush the loosened material upward and out of the filter (fig. 3.31). This system, however, is not recommendable. Whatever construction of filterbottom is used, the air is administred by a limited number of openings only, 30 to 100 per $m^2$ of filterbed area. Due to the large difference in specific gravity compared with the surrounding water, this air rises more or less vertically to above, entraining the neighbouring water in the same way as an air-lift pump does. With no supply from

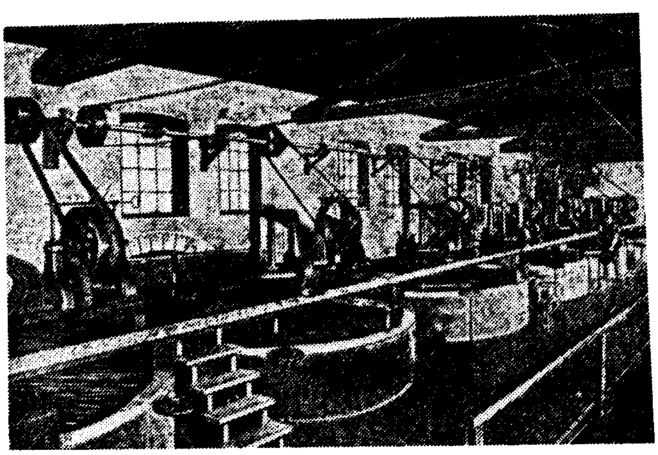


Fig. 3.30 Battery of circular filters with mechanical rakes

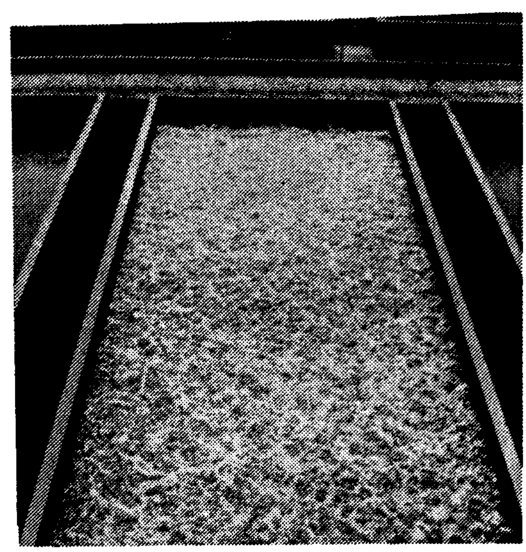


Fig. 3.31 Air wash prior to water wash

below, the water thus displaced has to flow back in the space between the jets of air, taking pollutions from the surface of the filter to below. To prevent these return flows, filterbed agitation with air must be accompanied by a limited upward flow of wash-water, of such a magnitude that no filterbed expansion occurs (fig. 3.32). Measured as atmospheric air, this air-wash usually proceeds at rates of about $(10)10^{-3}$ to $(20)10^{-3}$ m/sec (that is $m^3$ of atmospheric air per $m^2$ of filterbed area), while the vertical rise of the washwater is in the neighbourhood of $(4)10^{-3}$ m/sec. This combined air-water wash with a duration of 2 to 3 minutes produces a vigorous scrubbing of the sand grains, thoroughly loosening even strongly adhering coatings from the filtergrains. The loosened material is removed by subsequent backwashing with water alone, for 3 to 5 minutes, at rates sufficient to produce a sandbed expansion of 10 to 30%, depending on local circumstances and personal preferences (fig. 3.33). Air-wash in the mean-

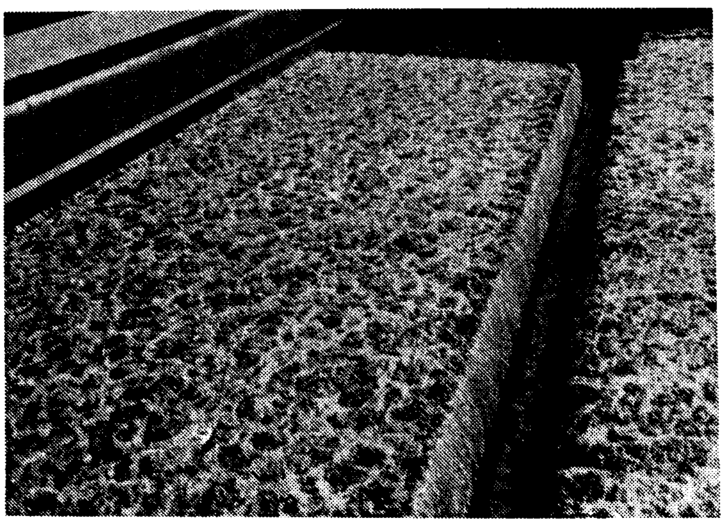


Fig. 3.32 Air wash combined with water wash

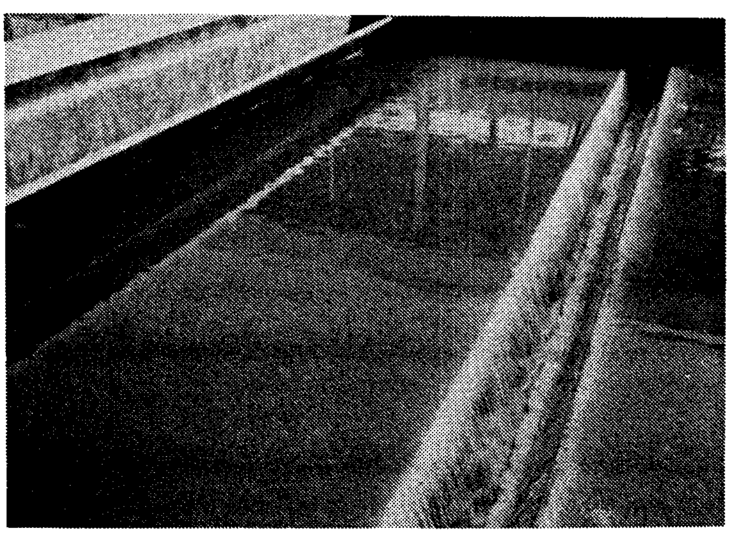


Fig. 3.33 Water wash following air wash

while is also used for the cleaning of coarse filtering material, composed of such heavy grains that enormous rates of washwater would be necessary to obtain expansion. Air is now applied in great amounts, $(40)10^{-3}$ m/sec for instance, together with water at rates of $(4)10^{-3}$ to $(6)10^{-3}$ m/sec for long periods, sometimes over 20 minutes.

Air for air-washing is commonly supplied directly by ventilators (up to 0.5 atmosphere) or compressors, while with small filtering units air vessels could be used to reduce power requirements. With regard to the limited amount of air now available and the loss of energy during decompression, this system is not recommendable. To subdivide the air equally over the full underside of the filterbed, an artificial loss of head must again be introduced at the point where the air emerges from the supply system. With regard to the lower resistance of the filterbed to flow of air, this controlling loss of head may be smaller than when backwashing with water, a value of 0.2 to 0.5 m water column being most common. With the low mass density of air, however, even this small loss of head asks for extremely fine openings of one to a few millimeters diameter only, which are easily clogged by suspended or dissolved impurities still carried by the water at this depth or by particulate matter transported by the air.

Surface wash to intensify the cleaning for the top layer of the filterbed originates in the U.S.A., where rapid filters are used primarily as finishing filters to remove the last traces of impurities carried over from the preceding coagulation/sedimentation process. With fine grained filtering material, impurities from the water to be treated are now retained for the greater part on top and in first millimeters depth of the filterbed, forming a tough crust which even air scour will find difficult to disintegrate completely. From a surface wash attacking this crust directly from above, better results may now be expected.

The stationary type of surface wash consists of a pipe distribution grid, suspended about 1 m above the filterbed and provided with vertical branches having nozzles at their lower ends. The nozzles are set about 0.1 m above the top of the unexpanded sandbed, at intervals of 0.5 to 1 m (fig. 3.34). During backwashing with water at a rate giving about 10% sandbed expansion, the nozzles are submerged by the sand-water mixture and when now water under high pressure is admitted to this system, the jets of water emerging from the openings will create an extreme turbulence that breaks up mudballs and thoroughly scours the sand in the upper part of the filterbed where clogging is heaviest. This surface wash is applied at rates of about $(3)10^{-3}$ to $(5)10^{-3}$ m/sec under pressure of 1 to 2 atmosphere (jet

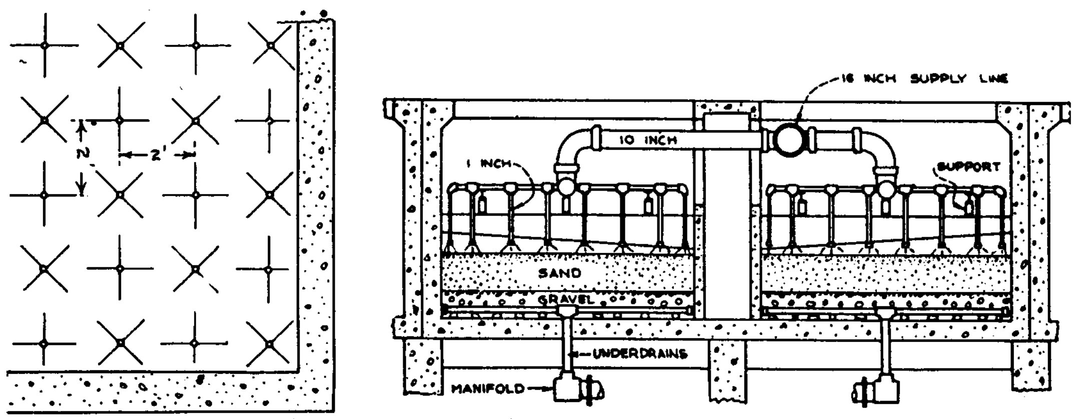



Fig. 3.34 Surface wash with stationary nozzles according to Baylis

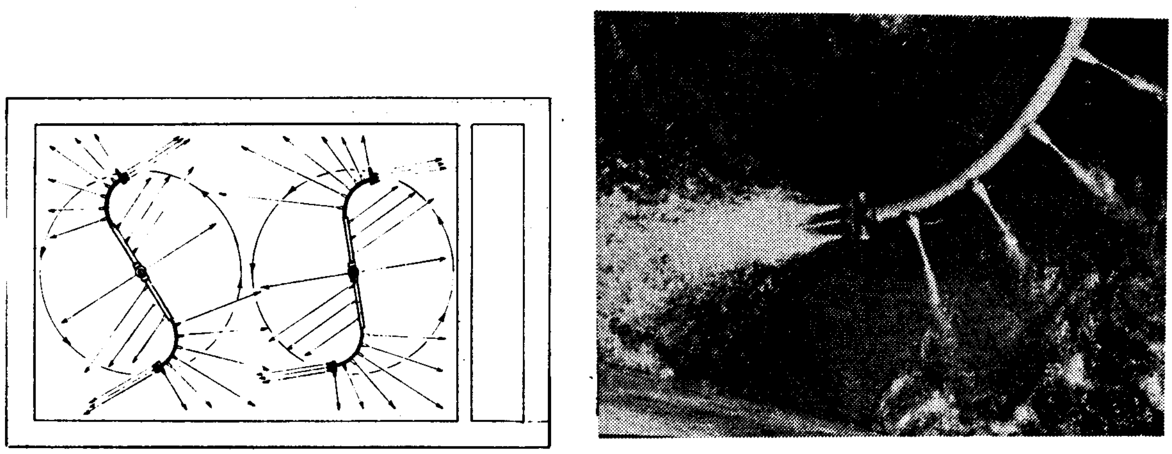


Fig. 3.35 Palmer sweep

velocity 14 to 20 m/sec) for a period of 1 to 3 minutes, after which backwashing from below is continued for another 2 to 3 minutes with sandbed expansions of 20 to 40%, allowing the loosened and disintegrated material to escape with the washwater.

It goes without saying that the pipe distribution grid mentioned above hinders any repair job to be carried out inside the filterbox, while it also adds considerably to the expense of construction. These are the reasons that the fixed type of surface wash has never become popular and that next to it revolving and removable types have been developed. The best known revolving type is the Palmer sweep, shown in fig. 3.35 and requiring a rate of $(0.3)10^{-3}$ to $(0.5)10^{-3}$ m/sec only. The pressure applied is fairly high, 3-5 atmosphere, giving nozzle velocities of 25 - 30 m/sec, able to keep the filterbeds in good condition except in cases of severe clogging. The removable type of surface wash is mounted on a traveling bridge (fig. 3.36), by which it can be moved from one filtering unit to another. Specifications are the same as for the stationary type, but with regard to the trouble of transferring, it can only be used when it is expected that an occasional surface wash suffices to keep the filterbeds clean.

Surface wash has been developed to obviate one of the disadvantages of surface filtration. As explained in chapter 2, this method of filtration has other and even more important drawbacks, which have lead to the application of deepbed filtration. When here an additional scour is required, surface wash has no sense and either air wash or revolving rakes must be applied.

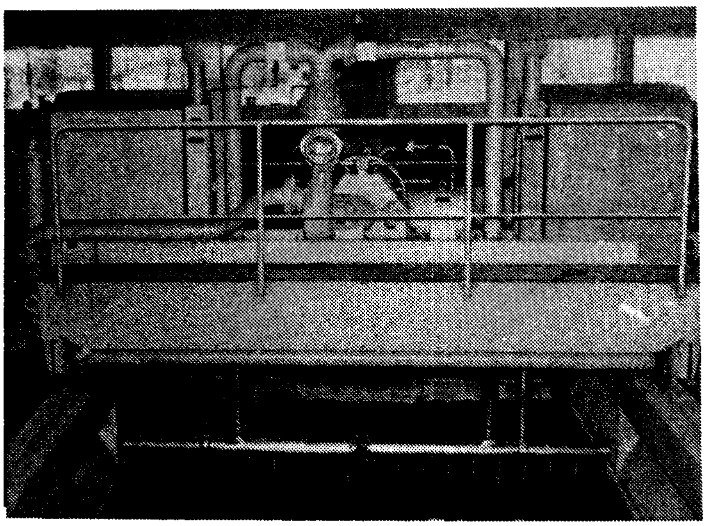


Fig. 3.36 Surface wash mounted on a moving bridge

# 4. DESIGN AND CONSTRUCTION OF A RAPID GRAVITY FILTER PLANT

## 4.1. Plant size

Primarily the size of a filtration plant depends on the total filterbed area A, being the quotient between the amount $Q_f$ of water to be treated and the filtration rate v to be applied

$$A = Q_f/v$$

In its turn, filtered water demand $Q_f$ equals water consumption $Q_c$ minus the amount of water taken from storage $Q_s$(fig. 4.1). Generally speaking water consumption is higher in summer than in winter, higher on working days then in the week-end and higher in day-time than during the small hours of the night. This allows various rates of consumption to be distinguished, in particular.

a) the average demand $Q_o$ being the yearly consumption divided by $(31.5)10^6$ s per year;
b) the average demand on the maximum day, say 1.5 $Q_o$
c) the maximum demand on the maximum day, for instance (1.5)(1.8) = 2.7 $Q_o$.
d) the average demand on the maximum week, with the figures given above about 1.2 $Q_o$

The ratios quoted above are not constant, but change from one community to the other, being less as the climate is more uniform, industrial consumption is relatively larger and more water is lost by leakages.

When no storage at all is present, the capacity of the filtration plant must equal the maximum demand on the maximum day, $Q_f$ = 2.7 $Q_o$ when the figures mentioned above are followed.With a small amount of storage, equal to about 25% of maximum daily demand(or $1^o$/oo of yearly consumption) the variations in hourly consumption can be satisfied with a constant supply,

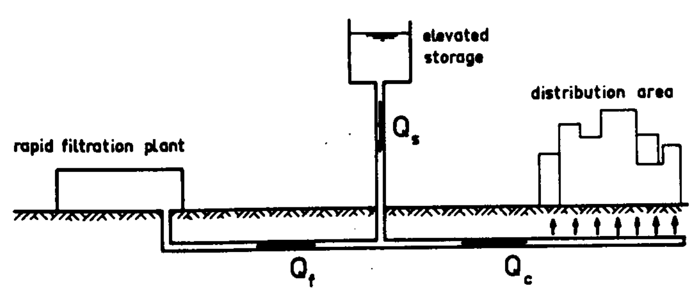



Fig. 4.1 Water consumption and water production

$Q_f = 1.5\ Q_o$ as shown in fig. 4.2. A larger amount of storage, about 75% of maximum daily demand even allows to balance the demand over the maximum week (fig. 4.3), reducing supply to $Q_f = 1.2\ Q_o$.

With a chosen composition of the filterbed, the maximum allowable filtration rate depends on raw water quality. When using groundwater, this quality is constant during the year, but with surface water, in particular from rivers, strong seasonal fluctuations occur. In temperate climates the temperature is high in summer and low in winter, while suspended matter content in riverwater is low in summer and high in wintertime. Both factors allow a much larger filtration rate in summer than in winter. Assuming

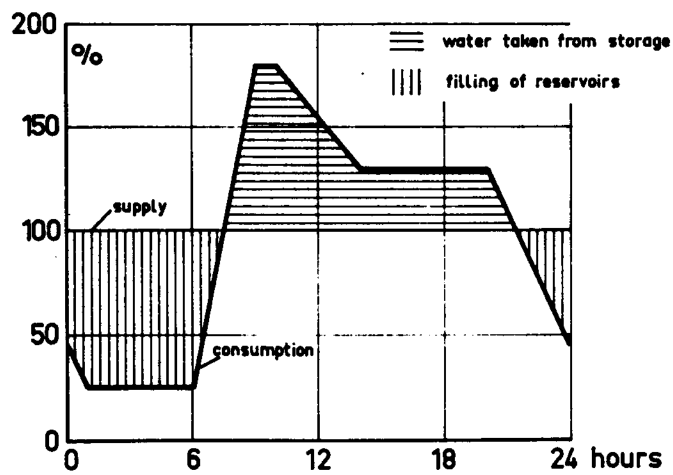



Fig. 4.2 Schematized hourly water consumption in Amsterdam in % of daily consumption

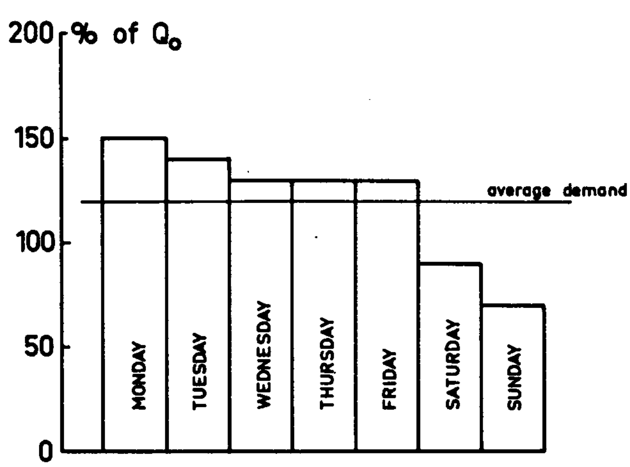



Fig. 4.3 Water consumption during maximum week

values of 4 and 2 mm/s respectively and maximum daily demands in summer and winter equal to 150 and 120 % of the yearly average gives

$$A_{summer} \approx \frac{1.5\ Q_o}{4} \approx 0.375\ Q_o$$

$$A_{winter} \approx \frac{1.2\ Q_o}{2} \approx 0.6\ Q_o$$

meaning that in winter the required surface area is 0.6/0.375 = 1.6 times larger than in summer.

It is perhaps needless to say that the capacity $Q_o$ is not the present demand, but the one expected to occur over 10 to 15 years.

## 4.2. Unit capacity and filter arrangement

The filterbed area A as calculated in the preceding section is always spread over a number of filtering units, each with a filterbed area a. Taking into account the loss in capacity when backwashing one or two filters simultaneously, this area should equal

$$a = \frac{A}{n-1} \text{ respectively } \frac{A}{n-2}$$

with n as number of filters. In practice this number varies between 4 and about 40, larger as the size of the plant increases. With the minimum number of 4 filters, of which 3 can satisfy maximum requirements, taking out of service another unit for maintenance or repairs increases the filtration rate by no less then 50%, which is only possible in case periods of good filtrability combine with low demand. In this respect 8 or 12 filters give more flexibility as now the increase in filtration rate is not more then 15 to 10%, which will be allowable during major parts of the year.

With big plants a further increase in the number of filters is necessary to keep the unit filterbed area down, mostly below 100 to 150 $m^2$, so as to reduce the size of filter piping and appurtenances which otherwise would be heavy and cumbersome, difficult to install and to replace. With regard to the required capacity of back-washing facilities, large filters are sometimes built in 2 halves, operated as one whole during filtration, but cleaned one after the other. An increase of unit size to 2 x 100 = 200 $m^2$ is now possible. Large filtering units also have the adventage of economy, reducing the cost of construction by a smaller number of filters with accessories and the cost of operation when manually actuated cleaning can be effected during one 8-hour shift per day. With regard to the lower filtration efficiency along the more or less smooth walls of the filterbox finally, the filterbed area should never drop below 10 $m^2$, and preferably stay above 20 $m^2$. Summing up these considerations gives

| | minimum | maximum |
|---|---|---|
| number of units n | 4 | indefinite |
| filterbed area a | 10 - 20 | 100-200 $m^2$ |

The final choice of the number of filters and the unit size of filterbed area should be based on comparative designs. A first estimate, however, may be had with the empirical formulae

$$n \approx 12 \sqrt{Q} \qquad \text{or} \qquad a \approx 3.5\ n$$

with Q as average capacity in $m^3$/sec and a as unit filterbed area in $m^2$.

For economy in construction and operation the filtering units of a rapid filtration plant should be set in a compact group, with influent and effluent lines as short as possible to reduce head losses. Common facilities such as wash water pumps and tanks, compressors for air wash, pressure vessels and pumps for hydraulic or pneumatic operation, etc, are located in a service building, which may also contain offices, laboratory, store rooms, central heating and ventilation equipment, chemical handling, storage and feed devices when necessary, sanitary facilities and so on. Many designs place this service building in the centre, while in wings extending in one or two directions the various filtering units are arranged on one or both sides of a two-level corridor (fig. 4.4).

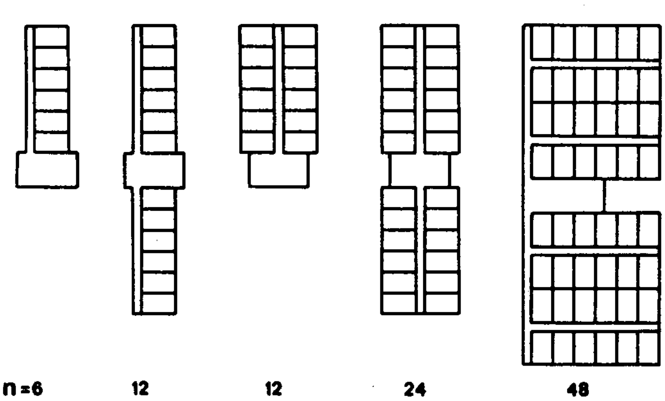



Fig. 4.4 Filter arrangements for n units

In this set-up, the filters are placed with their longitudinal axis perpendicular to the corridor, while their length is a little to many times larger as their width. The corridors are always housed, but in hot climates the filters are built in the open air (fig. 4.5). In cold climates the filters must be covered to prevent freezing in winter time (fig. 4.6), while this is preferable for hygienic reasons when dealing with groundwater. This

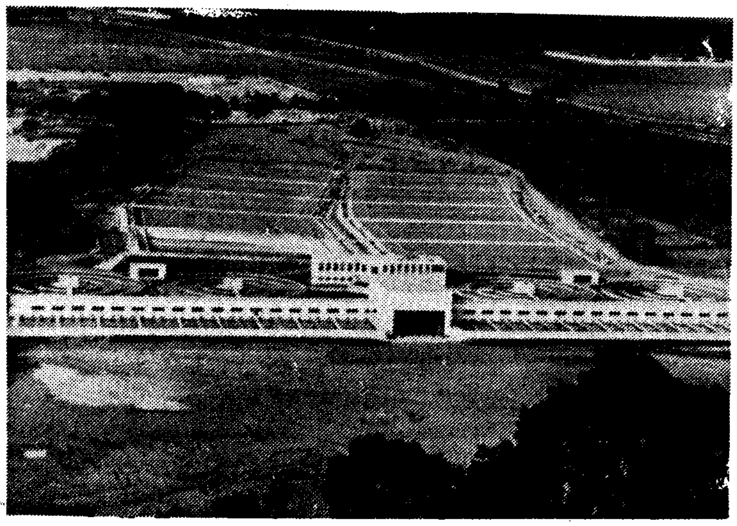


Fig. 4.5 Rapid filtration plant of Lima in Peru

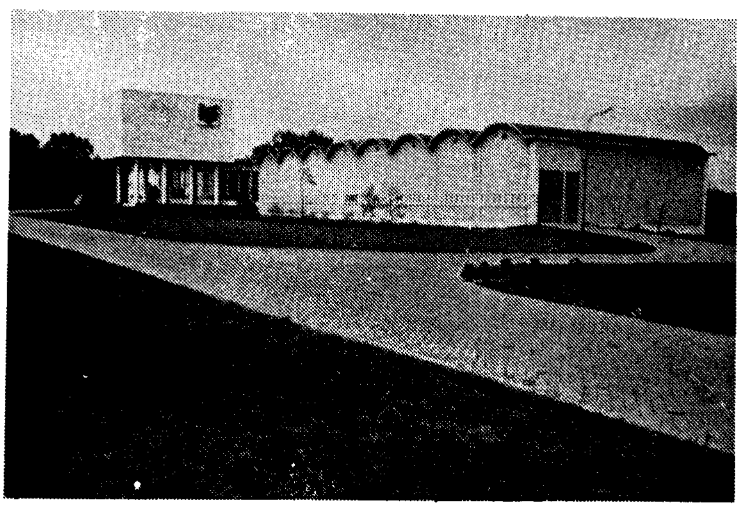


Fig. 4.6 Rapid filtration plant of Amsterdam in Leiduin

appreciably increases the cost of construction with as consequence that in moderate climates as much as possible building in the open air is practiced although in severe winters some protection may now be necessary (fig.1.8).. Whatever design is used, convenience to the operator, economy in operation and provisions for future extensions must be provided. The grouping of the

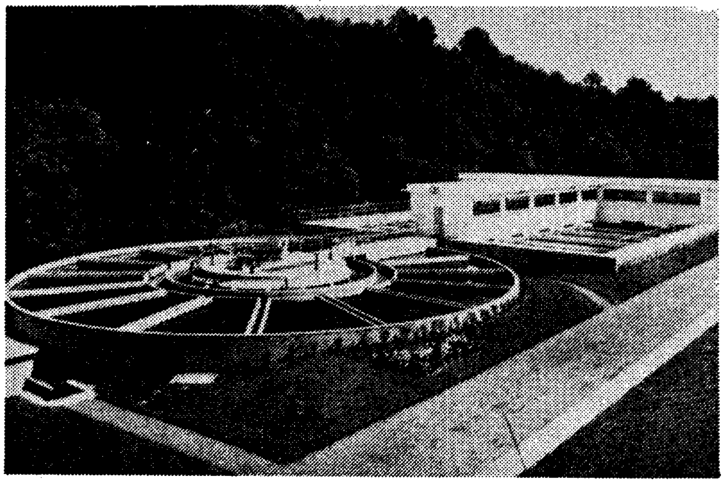


Fig. 4.7 Rapid filtration of the Syndicat Intercommunal de la Forêt de Mervant

filters should also permit a good architectonical concept. The public not acquainted with the technicalities of water purification, is likely to judge the quality of the water as much from the appearance of the plant, both inside and out, as from the appearance and the taste of the water. In this respect also gardening carries a heavy weight, requiring in particular exemplary maintenance(fig.4.7).

## 4.3. Filter control

During operation of a rapid gravity filter, impurities brought up by the raw water are deposited in the pores of the filterbed, increasing the resistance against downward water movement. With the other factors unchanged, a drop in filtration rate would thus occur. A similar drop in filtration rate would take place when the raw water level above the filterbed goes down or the filtered water level downstreams of the bed goes up, while the reverse movements would result in an increase of the rate of filtration. With regard to effluent quality, however, the filtration rate should be kept as constant as possible, while in particular sudden fluctuations should be avoided. An abrupt increase in filtration rate might cause impurities from the raw water to break through the filterbed, impairing effluent quality, while with negative heads a sudden reduction in the rate of filtration might release gas bubbles that have accumulated in the filterbed.

When these gas bubbles travel upward, holes might be produced in the filterbed, through which the raw water can pass without proper treatment. A positive control of the rate of filtration finally is necessary to adapt the production of the filtration plant to the supply of raw water or the abstraction of filtered water.

According to the mathematical theory of filtration, the hydraulic resistance of a filterbed is proportional to the filtration rate v, or in reverse

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

with $H_1$ and $H_2$ as piezometric levels of raw and filtered water respectively (fig. 4.8). With a clean bed, at the beginning of a filter run, the value

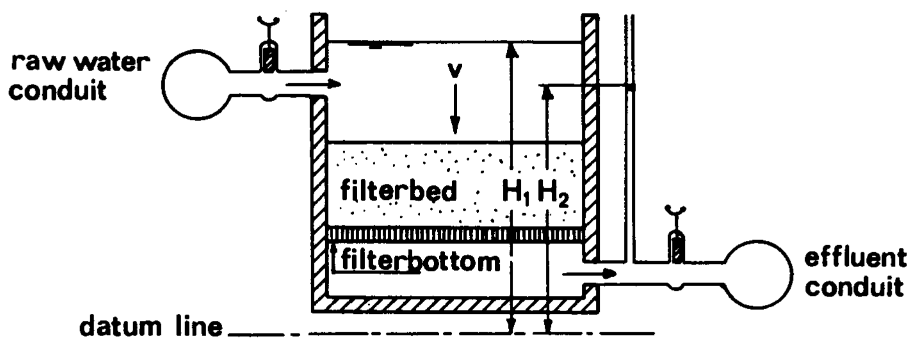
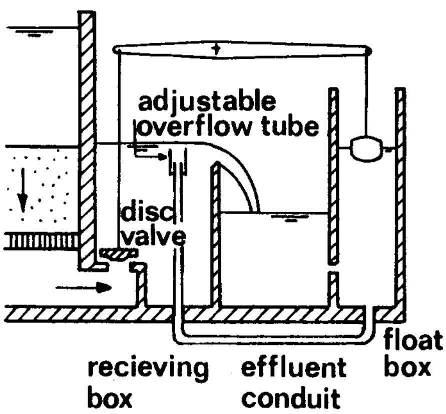



Fig. 4.8 Basic factors in filter control

of the proportionality constant $\alpha$ depends on water temperature and on the properties of the filterbed, that is on filterbed thickness, grain shape, grain size, grain size distribution and porosity. As filtration goes on, the porosity will decrease by the amount of deposited impurities, lowering the value of $\alpha$ and requiring a larger difference in head ($H_1$ - $H_2$) to keep the filtration rate at the intended value. At any time, however, $\alpha$ has a definite value and the formula above gives a relation between the 3 variables v, $H_1$ and $H_2$ of which now only 2 may be chosen at will. Nearly without exception one of these controlled variables is the rate of filtration v, while the other is eiter the raw water level $H_1$ above the filterbed or the piezometric level $H_2$ of the filtered water below this bed.

Rate control is obtained by inserting an additional loss of head in influent line (upstream control) or effluent line (downstream control) and adjusting this loss of head in such a way as to keep the supply of raw water or the abstraction of filtered water constant at the desired value. In principle these adjustments can be made by hand, but with the short lengths of filterrun commonly applied, a rapid change in operating conditions occurs, requiring constant supervision and making automatic control by mechanical, hydraulical, pneumatical or electrical means more attractive. In the past each filter was equipped with its own rate controller for which in the course of time an enormous variety of often very ingenious constructions have been developed. As examples only, fig. 4.9 and 4.10 show two constructions of downstream rate control. With the open type of fig. 4.9, the water flows from the filter through a disc valve into a receiving box. The greater part of this water is discharged over a fixed weir into the effluent conduit, serving the respective battery of filters. A very



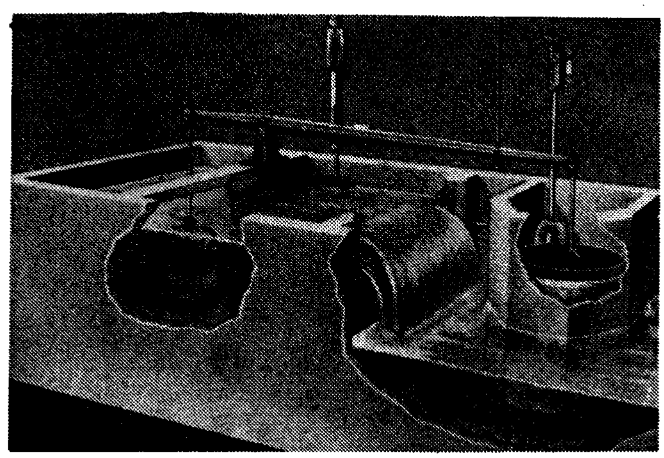



Fig. 4.9 Open filter rate controller (Paterson engineering co LTD,London)

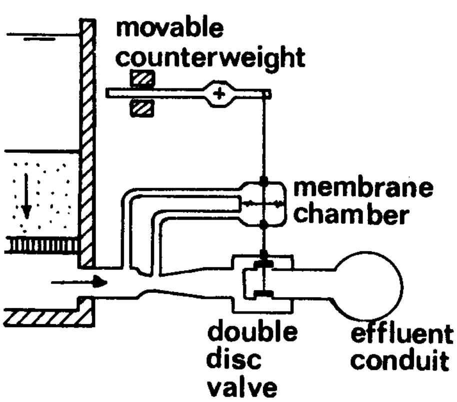



Fig. 4.10 Venturi rate controller with mechanical operation

minor part of the filtered water, however, enters the adjustable overflow-tube and flows to the float box from which it is discharged into the effluent conduit by a small hole. When now in the course of time the resistance of the filterbed increases and the amount of filtered water tends to drop, the flow of water into the slightly submerged overflow tube will be greatly reduced. As a consequence, the waterlevel in the float box drops, the float goes down, opening the disc valve, decreasing the resistance against the flow of filtered water and restoring the original rate of flow. In case a larger capacity is required, the overflow tube must be raised after which a new equilibrium will establish itself with a higher water level in the receiving box and a correspondingly higher discharge over the fixed weir into the effluent conduit. The closed rate controller of fig. 4.10 is based on the circumstance, that according to Bernouilli's law the water pressure in the throat of the venturi is smaller than the upstream one. When now these pressures are conveyed to both sides of a membrane chamber, the difference between them tries to move the membrane down, requiring a counter force to obtain equilibrium. An increase in filter resistance will again lower the filtration rate, decreasing the difference in pressure. With the same counter force, the membrane will move upward, opening the double disc valve, reducing the resistance against the flow of filtered water and increasing the filtration rate till again the original value is obtained. In case a large rate of flow is required, a greater counter force is necessary. In the construction of fig. 4.10 this is obtained by moving the counterweight to the left. When upstream rate control must be practised, the closed construction of fig. 4.10 may be used without change, while fig. 4.11. shows two of the many possibilities of an open construction. Here the raw water flows from the supply conduit into a float box and thence over a fixed weir or down through a calibrated orifice into the filterbox. The disturbing factor is now the variation in pressure under

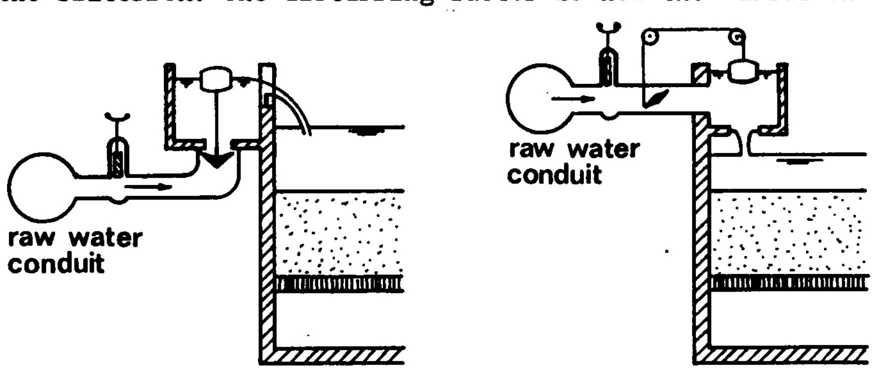



Fig. 4.11 Open filter controllers in raw water influent lines

which the raw water is supplied. This variation is nullified and the water level in the floatbox is kept constant by a float controlled throttling valve in the raw water connection. For another rate of flow, the length of rod or cable between the float and the valve should be changed.

Individual rate controllers have many disadvantages. They are expensive to install and cause large losses of head even when fully opened, increasing the cost of operation. When the total demand of filtered water changes or the total supply of raw water varies, the controllers for all the various filtering units moreover need resetting, a rather laborious procedure. With a master control this can be effected from a central point, but this will again increase the cost of installation. These are the reasons that today individual rate controllers are seldom applied with new installations and that mostly another principle, that of flow splitting is used. This method incorporates devices by means of which the total flow of raw water is supplied equally to all filtering units or the total demand of filtered water is abstracted equally from all units, while the filtration rate itself equals the ratio between the capacity and the available filterbed area. In fig. 4.12 the raw water conduit serving the battery of filters under consideration, has a large cross-sectional area by which the drop in water level due to losses of friction and turbulence as well as the rise in water level due to recovery of velocity head are negligeable. In front of all raw water inlets, the conduit has consequently the same water level and when now the free overflowing weirs at the left are set at one and the same elevation, each filter will receive the same amount of water. In fig. 4.12 on the right the calibrated orifices pass equal amounts of water when the water level in the raw water conduit is again uniform and next to this the upper water level in the filters is at one and the same

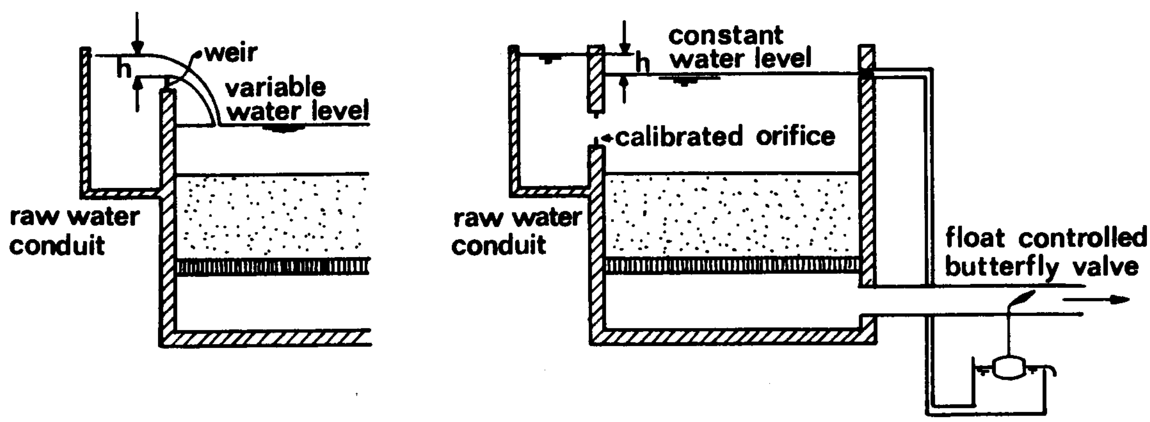



Fig. 4.12 Equal supply of raw water to all filtering units

elevation for all units. This asks for an additional regulating device, but has the advantage that a deviation in the resistance h has less influence on the rate of filtration v

$$\text{weirs} \qquad v \sim h^{1.5}, \quad dv \sim 1.5h^{0.5}\,dh, \quad \frac{dv}{v} = 1.5\,\frac{dh}{h}$$

$$\text{orifices} \qquad v \sim h^{0.5}, \quad dv \sim 0.5\,\frac{dh}{h^{0.5}}, \quad \frac{dv}{v} = 0.5\,\frac{dh}{h}$$

In both cases the filtration rate is larger when the supply of raw water goes up, raising the water level in the raw water conduit and increasing the value of h. For an equal abstraction of filtered water from all units, the effluent conduits of fig. 4.13 have again a large cross-sectional area, assuring a uniform level. With the water level in the float boxes kept

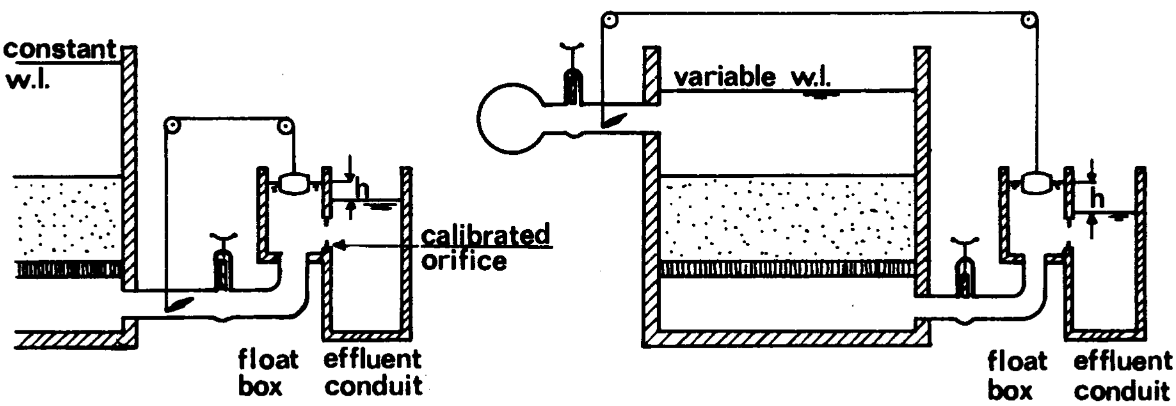



Fig. 4.13 Equal abstraction of filtered water from all filtering units.

constant at one and the same elevation for all units, all calibrated orifices will discharge the same amount of water, more as filtered water demand goes up and the water level in the effluent conduit drops, increasing the resistance h. In case a filter is taken out of service for back-washing, flow splitting will divide its load automatically over the remaining units and again here no additional controls are necessary.

Neither with individual rate controllers nor with flow splitting is it possible to obtain filtration rates that are exactly the same for all units. Some deviations must always be allowed with as limiting factor that the adverse effects on lengths of filterrun and/or effluent quality are not too serious. According to the mathematical theory of filtration (section 2.3)

$$c_e = c_o \frac{e^{\alpha T_q}}{e^{\lambda_o L} + e^{\alpha T_q} - 1} \text{ with } \lambda \sim \frac{1}{v}$$

Assuming $e^{\alpha T_q} = 4$ and $\lambda_o L = 5$ gives with $c_o = 20$ mg/l a value of $c_e$ equal to 0.550 mg/l. Varying the filtration rate with + or - 10%, changes this concentration to 0.877 and 0.313 mg/l respectively, in average 0.595 mg/l or 8% higher than with a constant rate. This is not yet objectionable, but a further variation would ask for shorter filterruns, hindering the proper operation of the plant. When in fig. 4.12 on the left, the water level in the raw water conduit varies from one unit to the other by 0.01 m, the requirement $\frac{dv}{v} < 0.1$ requires an overflow height h greater than

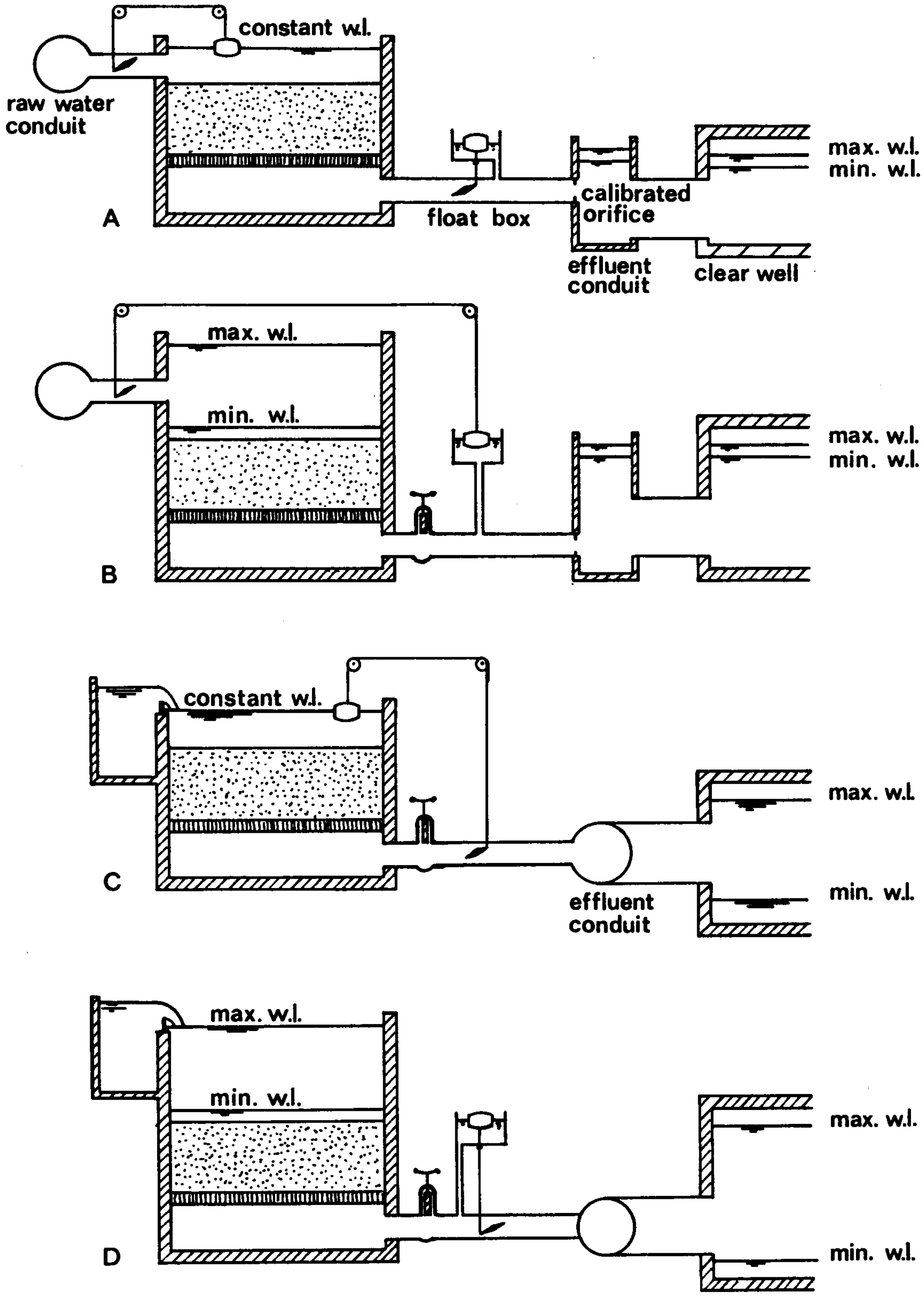
$$h = 1.5 \, dh \frac{v}{dv} = (1.5)(0.01)(10) = 0.15 \text{ m}$$

a rather large value, asking for every effort to reduce dh assumed above at 10 mm.

For fully regulating the operation of a rapid filter, rate control must be supplemented by water level control, governing either the level of the raw water above the filterbed or the level of the filtered water below this bed. On one hand this control serves to make filter operation independent from variations in the pressure under which the raw water is supplied or the filtered water is abstracted, while on the other hand it must compensate the increase in filterresistance accompanying clogging of the filterbed during the filterrun. This water level control is again obtained by inserting an additional loss of head in influent or effluent line and adjusting this loss of head in such a way as to keep the relevant water level constant. It goes without saying, however, that rate control and water level control can never be set behind each other in the same line. This means that downstream rate control must be combined with upstream waterlevel control and conversely, giving altogether 4 possibilities

downstream rate control and upstream control of raw water level;
downstream rate control and upstream control of filtered water level;
upstream rate control and downstream control of raw water level;
upstream rate control and downstream control of filtered water level.

Going out from the principle of flow splitting, examples of these 4 combinations are shown in fig. 4.14.




A en B - upstream water level control,downstream rate control.
C en D - upstream rate control,downstream water level control.
A en C - constant raw water level,variable filtered water level.
B en D - variable raw water level,constant filtered water level.

Fig. 4.14 Systems of filter control

A. the raw water level is kept constant by a float controlled butterfly valve in the raw water inlet pipe, while a similar device keeps the water level in the effluent pipe constant, at one and the same elevation for all filtering units. When moreover the effluent conduit has a large cross-sectional area and consequently a uniform water level over its entire length, the calibrated orifices at the end of the various effluent pipes will discharge equal amounts of water, more as the water-level in clear well and effluent conduit is lower;

B. with a float controlled butterfly valve in raw water inlet pipe, the filtered water level is kept constant at one and the same elevation for all filtering units. With a uniform water level in the effluent conduit, the calibrated orifices at the end of the various effluent pipes will again discharge equal amounts of water, more as the water level in clear well and effluent conduit is lower;

C. the raw water is kept constant by means of a float controlled butterfly valve in the effluent pipe. With the raw water conduit of ample cross-section, its water level will be uniform over the entire length by which the weirs set in the various raw water inlets at equal elevations will supply equal amounts of water, more as the water level in the raw water conduit is higher;

D. the filtered water level is kept constant by float controlled butterfly valves in the effluent pipe, while an equal distribution of raw water over all filtering units is again assured by raw water inlet weirs set at one and the same elevation together with a raw water conduit of ample cross-section and uniform water level.

With regard to the relative merits of the constructions described above, it may be noted that in the solutions B. and D. the filtered water level is kept constant at a short distance above the top of the filterbed, while the increase in filter resistance during filtration is taken up by a rise of the raw water level. Negative heads and air-binding are thus impossible, but the depth of the filterbox must be rather large, increasing the cost of construction. In the solutions A. and C., negative heads can be prevented when the constant raw water levels are chosen at a large distance above the top of the filterbed and filterruns are broken off the moment that the filtered water level drops below the surface of this bed. With these constructions, however, it is also possible to provide only a shallow depth of raw water, 0.25 or 0.4 m for instance, and operating the filter by suction. Large negative heads will now develop during filtration but the depth of the filterbox is much smaller, reducing the cost of construction and making these solutions rather popular with firms specialising in the building of rapid filters. With the upstream rate control in solutions C. and D. finally, the water level in the clear well may vary between wide limits, giving an appreciable storage capacity. With the downstream rate control of solutions A. and B. on the other hand, the water level variations in the clear well are small, asking for additional provisions to balance raw water supply and filtered water demand over short periods, a half to one hour for instance.

Which solution must be chosen in a specified case, depends on local circumstances and above all on the preferences of the designer. As general rules it may be mentioned, however, that with rapid filtration serving to remove the carry-over of coagulant flocs from the preceding settling tanks, all care must be taken to prevent a desintegration of these flocs as this would only render the work of the filters more difficult. This rules out the use of upstream rate control with its accompanying resistance and high flow velocities, as well as the application of a variable raw water level, leaving the construction of fig. 4.14 A as only possibility. Still better results may now be obtained with the construction of fig. 4.15 A, where the depth of raw water above the filterbed is larger and its level is regulated by changing the capacity of the raw water supply pumps. A variation of this level, 0.5 m for instance, gives now sufficient storage capacity to operate the filtered water pumps independently, while the larger depth of raw water is able to prevent the occurence of negative heads. The simplest solution can be obtained by replacing the filtered water level control of fig. 4.14 D by a fixed weir in the effluent line, as shown in fig. 4.15 B. With no moving parts, nothing can go out of order, making this construction very popular for rapid filters preceding slow sand filtration. The variable raw water level brings with it, however, that the walls of the filterbox are periodically submerged or visible. By deposits of silt, iron, manganese, etc, these walls may become very unsightly, adversely affecting the sanitary aspects of a drinking water treatment plant. In this respect the constructions of fig. 4.14 C or fig. 4.15 C are better suited. In the latter figure, the butterfly valve in the effluent line is replaced by a syphon with an air inlet at the top,decreasing the discharge capacity.These syphons are cheap and reliable, but to make their operation independent from water

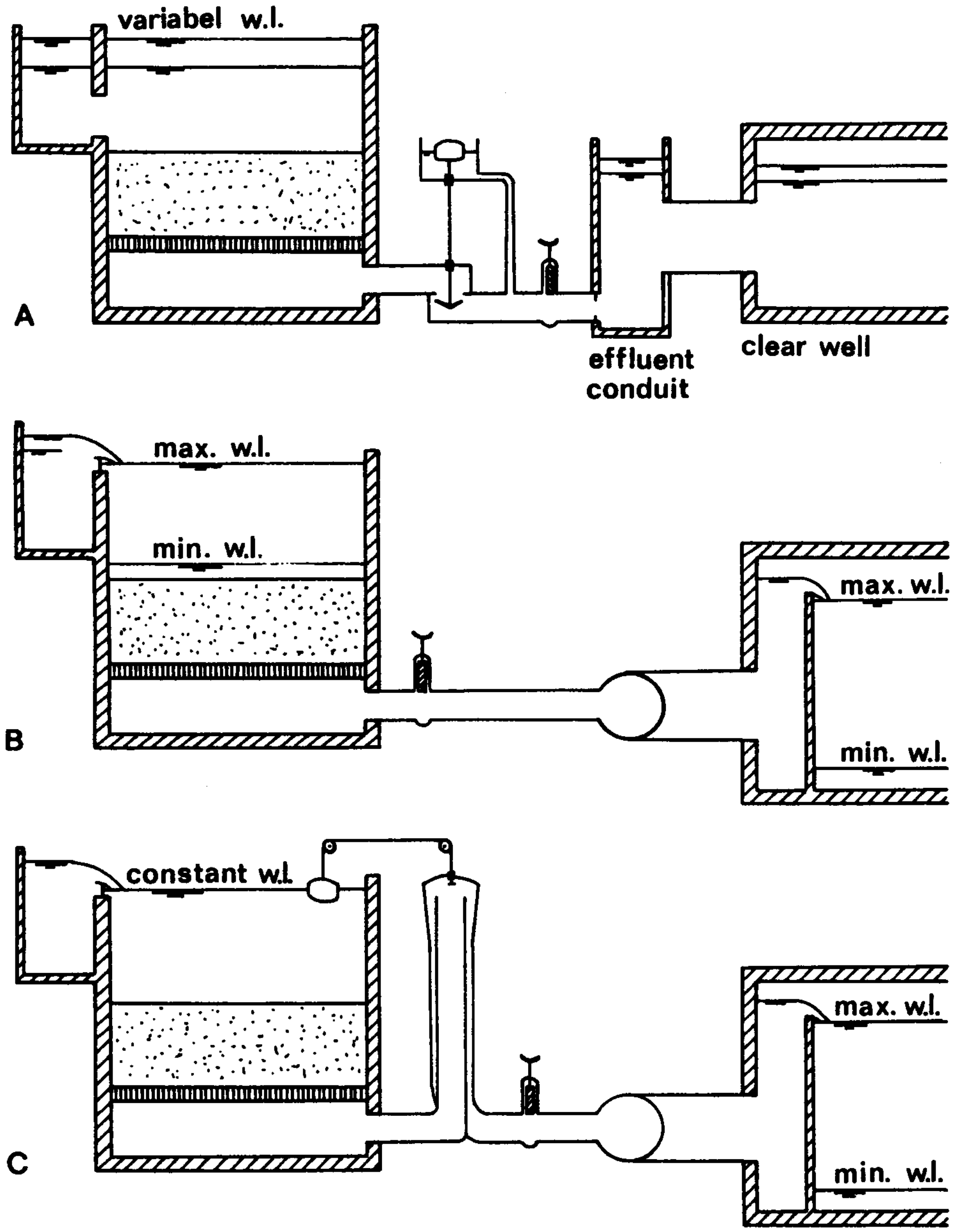



Fig. 4.15 Systems of filter control

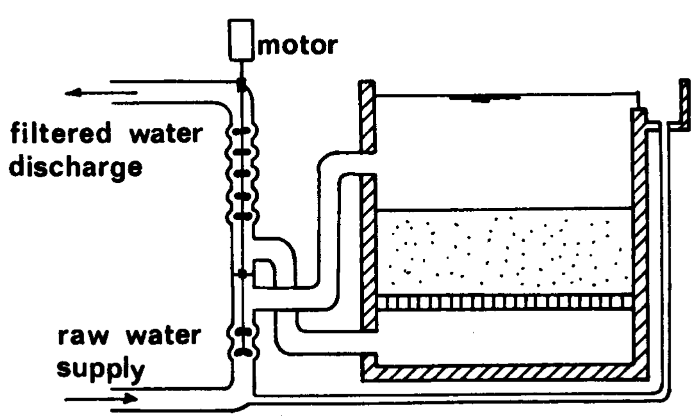



Fig. 4.16 Pump control of a rapid filter

level variations in the clear well, an additional weir is required. This weir, however, also prevents negative heads and provides an often very welcome amount of aeration. A constant raw water level must be applied when in the depth of water above the filterbed flocculation occurs, asking for a constant detention time before entering the filterbed. When with high-rate filtration this detention time is small, a great depth of raw water on top of the filterbed is necessary. The construction of fig. 4.14 B is seldom used. It seems to have some advantages when the raw water is supplied under a high and variable pressure.

It is needless to say that fig. 4.14 and fig. 4.15 only show examples, indicating the basic principles of filter control. As regards details, however, an enormous variety of constructions is commercially available and still today new solutions are emerging continuously. Most of these do not show the simplicity of the constructions sketched in fig. 4.14, but an exception may be made for the pump control of fig. 4.16. Here raw water pump and filtered water pump are driven by the same motor, while under all operating conditions the capacity of the raw water pump is slightly higher than that of the filtered water pump. The excess amount of raw water is removed by an overflow and discharged to waste or returned to the raw inlet. This certainly means a loss of energy, but still the increase in operating cost is next to negligeable.

In the set-up of fig. 4.15 A, the filtration rate depends on filtered water demand, going up as the water level in clear well and effluent conduits drops, creating a larger head loss across the orifices at the end of the effluent pipes from the various filters. The amount of raw water supplied has no influence on this rate and only changes the depth of raw water above the filterbed. In case the same set-up is desired, but with a filtration rate dependent on raw water supply, additional regulating devices are necessary. With the control system of fig. 4.17 A, an increase in raw water supply raises the raw water level in the respective battery of filters, opening the butterfly valve in the discharge pipe between the effluent conduit of this battery and the clear well. The water level in the effluent conduit will thus go down, increasing the rate of filtration

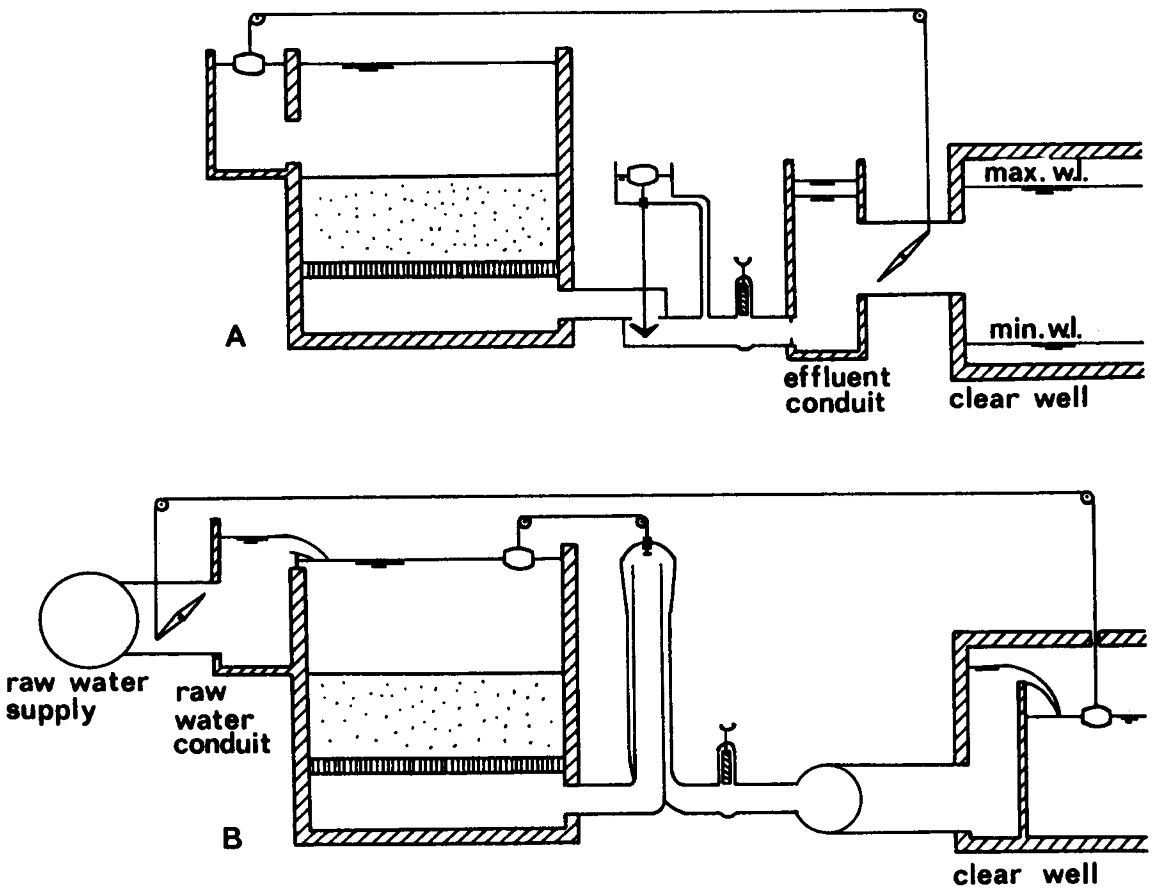



Fig. 4.17 Systems of filter control

in the same way as described above, till equilibrium with the capacity of the raw water supply is obtained. The reverse situation is found in fig. 4.15 C, where filtered water demand changes the water level in the clear well, but does not affect the filtration rate, which only depends on the amount of raw water supplied. With the additional regulating device of fig. 4.17 B, an increase in filtered water demand lowers the water level in the clear well, opening a butterfly valve in the pipe line connecting the raw water supply to the raw water conduit of the respective battery of filters. The water level in this conduit will consequently go up and all raw water influent weirs will start to discharge more water, raising the water level in the clear well till equilibrium with filtered water demand is again established.

With a constant supply of raw water or a constant abstraction of filtered water, the control systems of this section are meant to keep the filtration rate constant. Unfortunately this is not completely obtained, resulting in a (slight) deterioration of effluent quality.

When for instance the system of 4.15 A is applied, it is tacitly understood that the weight of float,disc valve and connecting bar together with the resultant of the hydraulic forces acting upon the disc valve are always counterbalanced by the buoyancy of the float. In reality, however, discrepancies may occur, the difference being taken up by the friction between the bar and the bushings in float box and valve box. When now filtered water demand surpasses raw water supply, a gradual lowering of the raw water level on top of the filter will occur. Due to the frictional forces mentioned above, the disc valve will first maintain its original position, by which the water level in the float box drops and the filtration rate goes down. The buoyancy of the float thus gradually decreases till the difference surpasses the frictional resistance and the float suddenly moves down, moving friction being appreciably less than friction at rest. The downward movement of the disc valve augments its discharge opening with a sudden increase of filtration rate as unavoidable result. This phenomenon is known as hunting and has many other causes as the frictional resistance mentioned above. When for instance in the same figure 4.15 A the disc valve is not properly designed, the Kármán vortex trail emanating from it may again give rise to serious oscillations with a periodic decrease and increase in filtration rate. Many an unsatisfactory operation of a rapid filtration plant must be attributed to such minor ailments, which long may go unnoticed.

The filtration rate has to change when raw water supply or filtered water demand increases or decreases. According to experience this does not deteriorate effluent quality when the increase in rate remains below 3 % per minute. With the situation of again fig. 4.15A this is automatically obtained as even a sudden increase in the capacity of the filtered water pumps only gradually lowers the water level in the clear well.

## 4.4. Declining rate filtration

The flow splitting devices of the preceding section are quite simple and inexpensive (fig. 4.15B), but still they do entail some costs (fig. 4.15C). The cheapest system is no control at all, shown schematically in fig. 4.18. Here the raw water level is kept constant at an equal elevation for all filtering units by manipulating the raw water supply pumps, while the piezometric level of the filtered water upstreams of the clear well is kept constant by a fixed weir. The difference between both levels given the head loss H, which is the same for all filtering units and constant during the whole length of filterrun. This means, however, a high rate of filtration for the clean filterbed directly after back-washing, gradually declining as filtration goes on and clogging occurs. When the capacity of the plant no longer satisfies requirements, the filter longest in operation and having the lowest rate of filtration is cleaned by back-washing. According to Lerk's filtration theory, the value of $\lambda_o$ is inverse proportioned to v. At the beginning of the filterrun $t = o$ with a high rate of filtration and a low value of $\lambda_o$ the suspended matter content

$$c_e = c_o \, e^{-\lambda_o L}$$

will consequently go up. As filtration continues clogging occurs. On one hand this lowers the rate of filtration, increasing $\lambda_o$ and improving water quality, while on the other hand a break-through of suspended matter takes place. Altogether, the change in effluent quality will be more gradual than with constant rate filtration, but the average one will be somewhat less. This can easily be compensated, however, by a slight increase in bed thickness.

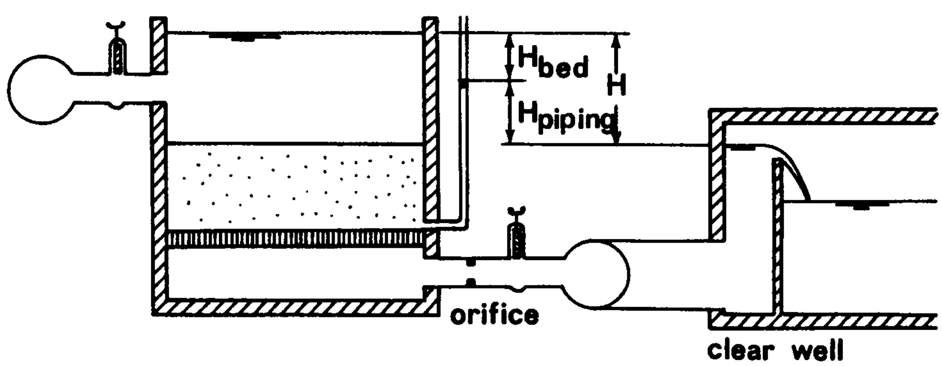



Fig. 4.18 Declining rate filtration

The head loss H is necessary to overcome the linear resistance of the filterbed and the quadratic resistance of filterbottom and effluent piping

$$H = H_{bed} + H_{piping} = \beta_1 v + \beta_2 v^2$$

According to the graph of fig. 2.12 a possibility for constant rate filtration is given by the combination $v = (2.5)10^{-3}$ m/s, $d_o = 0.8$ mm, $L = 0.9$ m and $H = 1.1$ m. This gives at $10^0$C and $p_o = 0.38$ as initial head loss of the filterbed a value of 0.59 m. Assuming for this rate a quadratic resistance of 0.1 m gives for the initial rate of filtration

$$\frac{v}{(2.5)10^{-3}}\ 0.59 + \left(\frac{v}{(2.5)10^{-3}}\right)^2 0.1 = 1.1 + 0.1 = 1.2 \text{ and}$$

$$v = (4.00)10^{-3} \text{ m/s}$$

According to section 2.3

$$\lambda_o = 3.355 \text{ m}^{-1} \text{ and } c_o = 0.73 \text{ g/m}^3,$$

which value is much higher than the maximum allowable one of 0.5 g/m$^3$. Improvement may be had by installing an orifice, increasing the quadratic resistance to 0.5 m

$$\frac{v}{(2.5)10^{-3}}\ 0.59 + \left(\frac{v}{(2.5)10^{-3}}\right)^2 0.5 = 1.6$$

$$v = (3.23)10^{-3} \text{ m/s}, \lambda_o = 4.155 \text{ m}^{-1} \text{ and}$$

$$c_o = 0.36 \text{ g/m}^3$$

## 4.5. Filterbox and filterbottom

The filterbed together with the underdrainage system below and the supernatant water above are encased in a box with a depth of 2 to 4 m, a surface area of 15 to 150 $m^2$ and almost without exception constructed of reinforced or prestressed concrete. With regard to the backwashing facilities, all units have the same surface area, while to facilitate the construction of the filterbottom a rectangular plan is strongly recommendable. In view of positioning the various filtering units along pipe gallery and operating floor, the length of the filterbox is commonly many times its width. For small plants in particular, filters built of steel with circular plans may be more economic, but the difficulties of obtaining a pleasing architectonical design should not be overlooked.

A section over the filterbox again shows a rectangular shape, with walls of constant thickness vertical and walls of upward declining thickness slightly sloping backward. As already mentioned in section 3.7, short-circuiting of the raw water along the walls of the filterbox cannot be prevented by giving these walls a rough or even grooved surface, while with regard to fouling and easy cleaning an as smooth surface as possible is strongly advisable, for instance by applying steel shuttering. When short-circuiting must be prevented, this can be done by using a small number of larger units with a more favorable ratio between surface area and circumference. When this results in a very small number of units and less flexible operation, the same effect can be obtained by giving small filters a more square or even circular plan.

The underdrainage system or filter bottom of a rapid filter serves the threefold purpose of supporting the filtering material, providing an outlet for the water passing through the filter and supplying washwater to the underside of the filterbed. It goes without saying that the filterbottom must be constructed in such a way that no loss of filtering material can occur and that filtered water is collected and washwater distributed evenly over the whole area of the filterbed, so as to assure that during filtration all parts of the filterbed perform as nearly as possible the same amount of work and when washed receive nearly the same amount of cleansing. Because washwater is applied at rates many times greater than the filtration rate, the hydraulic design of the filterbottom is governed primarily by the necessity of delivering washwater evenly to the entire underside of the filterbed.

As shown in section 3.3, this can only be obtained by providing the filter-bottom with a large resistance against the passage of washwater, greater as the variations in head accompanying the flow of washwater over the length and width of the filterbottom are larger.

The number of filterbottom constructions that have been applied in practice is nearly uncountable and there is no single detail of rapid filter construction that has aroused so many controversies and has evoked such heated arguments as the selection of the underdrainage system best suited for a particular case of rapid filtration. In the subsequent pages only the major systems can be dealt with, treated in such a sequence as to show a logical development although the actual history was quite different.

One of the oldest and still most widely used filterbottom is the perforated pipe underdrain system, consisting of a manifold to which a serie of laterals are connected, the latter provided with openings in the lower portion as shown in fig. 4. 19. Through these openings the washwater is directed downward, either vertically or under an angle of 30° to 45° with the vertical (fig. 4.20). In both cases, however, the kinetic energy of the jets emerging from the openings is dissipated by collision with the bottom of the filterbox or the sides of the surrounding pebbles and there is no danger of disturbing the filterbed. The pebbles around the perforated lateral are placed by hand in such a way that no blocking of the openings

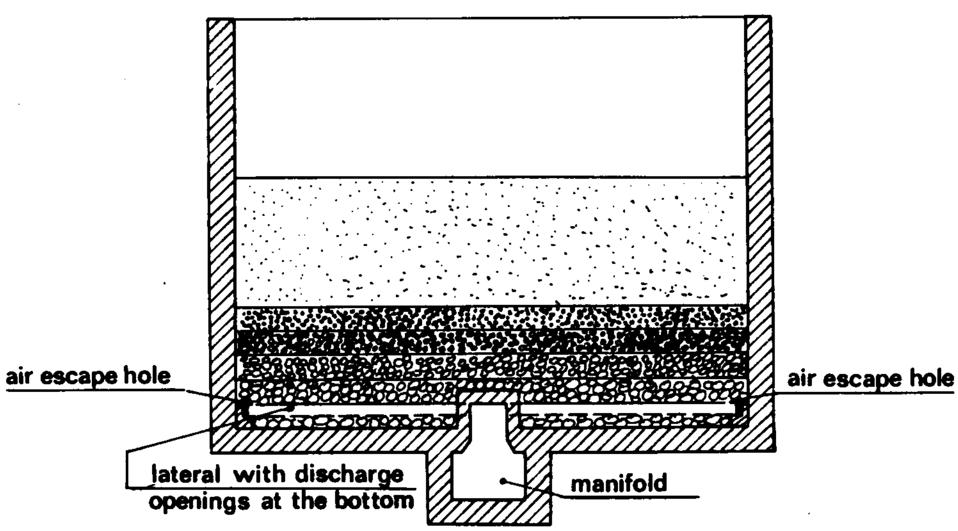



Fig. 4.19 Perforated laterial underdrain system for backwashing with water


**holes vertically down for instance 10mm ∅ at 100mm on centers**

**holes under 30° with vertical for instance 10mm ∅ at 100mm on centers staggered**

Fig. 4.20 Holes in lateral

occurs. The resistance of this filterbottom consequently equals the velocity head of the jets issuing from the openings. With n openings of diameter $D_o$ per $m^2$ of filterbed area and backwashing at a rate v $m^3/m^2/sec$, this velocity head equals

$$\frac{v_o^2}{2g} = \frac{8}{\pi^2 g \mu^2} \frac{v^2}{n^2 D_o^4}$$

or with the discharge coefficient μ of the openings assumed constant at 0.7

$$\frac{v_o^2}{2g} = \frac{1}{6} \frac{v^2}{n^2 D_o^4}$$

With a backwash rate v of $(15)10^{-3}$ m/sec for instance and 50 openings $\phi$ = 10 mm per $m^2$

$$\frac{v_o^2}{2g} = \frac{1}{6} \frac{(225)10^{-6}}{(2500)(10^{-8})} = 1.5 \text{ m}$$

In practice this resistance varies from 1 to 4 m, asking for about 25 to 75 openings per $m^2$, with diameters between 6 and 15 mm. To assure an equal distribution of washwater, the resistance of the filterbottom must be larger as the head under which the washwater emerges from the various openings differs more over the length and width of the underdrainage system. With the direction of the jets perpendicular to the flow in the lateral, the deciding head is the difference in piezometric level inside and outside the lateral. Outside the lateral the piezometric level may be considered constant, but inside the underdrainage system it will increase by recovery of velocity head and decrease by losses due to friction and turbulence. In fig. 4.21 the greatest variation will occur between the openings A and C. When the losses $A^1$-A and $B^1$-B are assumed to be equal, this difference amounts to the increase in piezometric level over the length $A^1$-$B^1$ of the manifold and over the length B-C of the lateral (fig. 4.22)

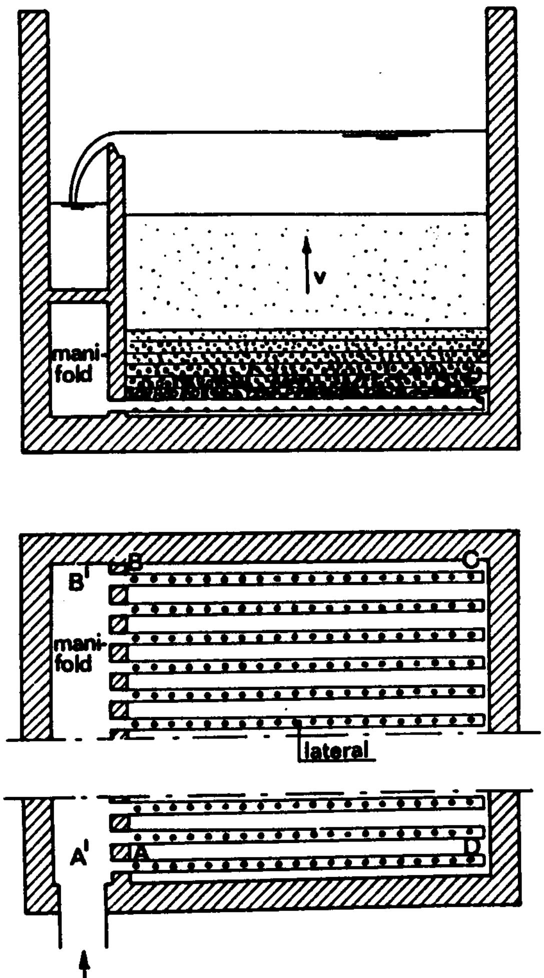



Fig. 4.21 Plan and cross-section of a rapid filter

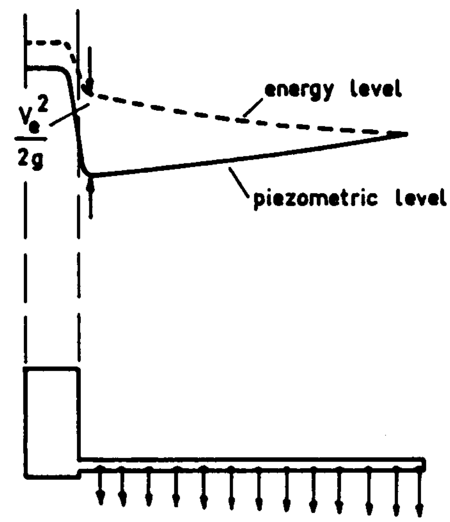



Fig. 4.22 Unequal wash-water distribution due to partial recovery of velocity head

$$\Delta_{A'-B'} = \frac{v_m^2}{2g} - \frac{1}{3}\lambda_m \frac{L_m}{D_m} \frac{v_m^2}{2g} - \frac{1}{n_m} \frac{v_m^2}{2g} = \alpha_m \frac{v_m^2}{2g}$$

$$\Delta_{B-C} = \frac{v_l^2}{2g} - \frac{1}{3}\lambda_l \frac{L_l}{D_l} \frac{v_l^2}{2g} - \frac{1}{n_l} \frac{v_l^2}{2g} = \alpha_l \frac{v_l^2}{2g}$$

in which L is the length, D the (hydraulic) diameter, $\lambda$ the friction coefficient, v the entrance velocity and n the number of outflows of manifold (index m) and lateral (index l) respectively. The total variation in piezometric level thus equals

$$\Delta_{A-C} = \alpha_m \frac{v_m^2}{2g} + \alpha_l \frac{v_l^2}{2g}$$

with $\alpha_m$ and $\alpha_l$ commonly between 0.5 and 0.8. According to the example given at the end of section 3.3. the minimum required resistance of the filterbottom is given by

$$H_{bottom} = 0.18 + 17\ dH$$

in which dH is the variation in piezometric level under which the washwater is supplied and abstracted. In the case under consideration, the variation in supply pressure equals the value of $\Delta_{A-C}$ calculated above, while the water level variations above the filterbed may be neglected. This gives finally

$$\frac{v_o^2}{2g} = 0.18 + 17\left(\alpha_m \frac{v_m^2}{2g} + \alpha_l \frac{v_l^2}{2g}\right)$$ or vor with $\alpha_m = 0.6$, $\alpha_l = 0.7$

$v_m = 0.75$ m/s and $v_l = 1.25$ m/s

$$\frac{v_o^2}{2g} = 0.18 + 17(0.073) = 0.18 + 1.24 = 1.42 \text{ m}$$

Large design values of $v_m$ and $v_l$ allow small sizes of manifold and lateral to be used, lowering the cost of construction, but they increase the head under which the washwater must be supplied, augmenting the cost of installation and operation of the washwater facilities. With regard to the resistance of the filterbottom, an ample safety factor is moreover required as in reality the discharge coefficient $\mu$ is not constant but increases when the velocity of the main-flow becomes smaller. This is another reason that lateral B-C will receive more water than lateral A-D and opening C will discharge more water than opening B. In principle these differences may be compensated by varying the intervals between laterals along the length of the manifold and the intervals between holes along the length of the lateral.Unfortunately the experimental data for a judicious selection of these increases are still insufficient.

As regards the construction of the perforated pipe underdrain system, the manifold is commonly made of cast iron, steel with a concrete jacket and asbestic cement or built from reinforced or pre-stressed concrete. In the latter case the cross-sectional area is mostly so large that access for inspection, maintenance and repair is possible. For the laterals cast iron, steel and copper are seldom applied nowadays, asbestic cement and hard plastic being most popular. The internal diameter of manifold and laterals should be large enough to satisfy the hydraulic requirements elaborated above, while their wall thickness should provide sufficient structural strength, to support the filterbed and to withstand the sudden vibration of water pressure put upon them when starting the backwash. The internal diameter of laterals varies from 0.05 to about 0.12 m, their interval from 0.15 to 0.30 m, while the perforations in the laterals are of 6 to 15 mm diameter at 0.10 to 0.25 m centers along the pipe. It should never be forgotten that once installed, the underdrains are relatively inaccessible. All care should therefore be given to their design and construction and when the filtered water is agressive(for instance by oxidation of organic matter in the raw water, forming $CO_2$ and lowering the pH), they should be made of corrosion resistant material or protected against corrosion, for instance by a coating with plastic. Erosion of softer, non-metallic materials around the holes may be prevented by lining these holes with brass or bronze bushings. Common arrangements of perforated pipe laterals are shown in fig. 4.23. The purpose of the double unit at the bottom left is to cut the washwater requirements of the filter in half by washing the two component units in succession.

The perforated pipe underdrainage system in the meanwhile is not complete with manifold and laterals alone, a system of supporting layers of gravel still being required to prevent filtering material from entering and blocking the underdrains and to aid in a more uniform distribution of washwater, emerging from a limited number of openings only. The size and the depth of these gravel layers should moreover be chosen such as to accomplish both purposes without being displaced by the rising wash-water. To satisfy these requirements, the supporting gravel system is built up from various layers, fine at the top and coarse at the bottom each layer composed of carefully graded grains with the 10 and 90% diameters passing not

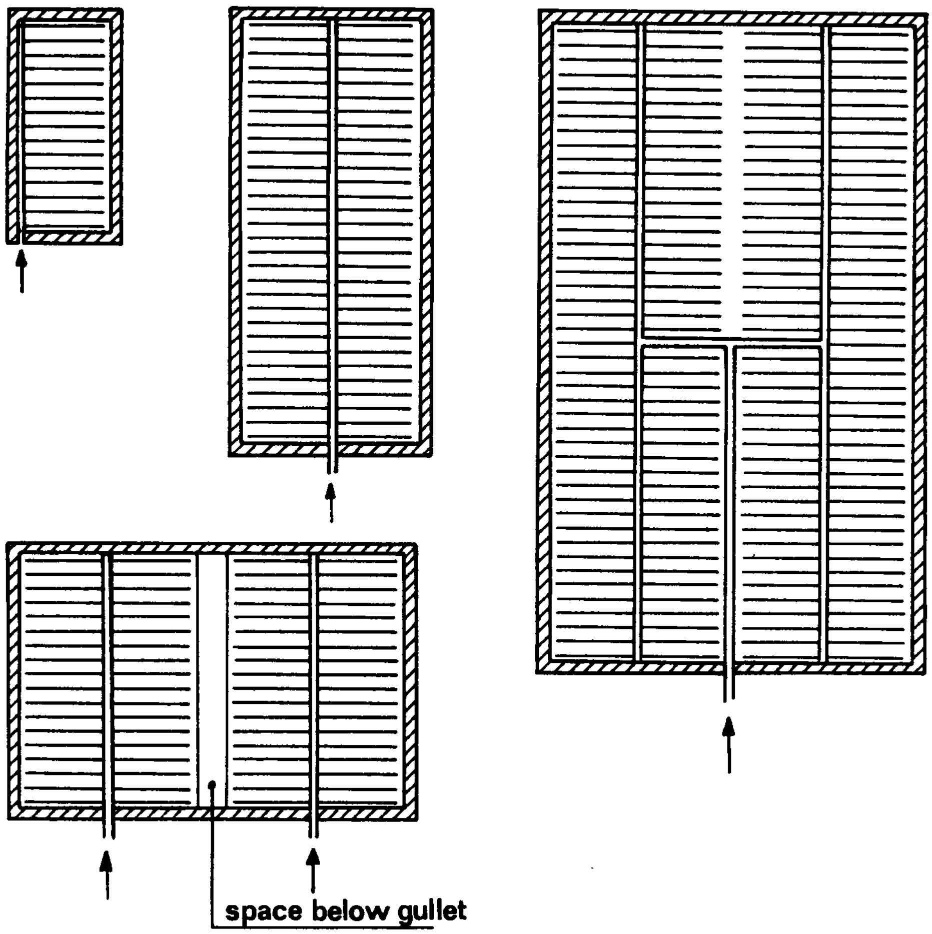



Fig. 4.23 Common arrangements of perforated pipe underdrains

more than a factor $\sqrt{2} = 1.41$ apart. The gravel in the top layer should on one hand be fine enough to prevent filtering material from entering and clogging the openings between the gravel grains, while on the other hand it should be so coarse that it is not expanded during even high-rate backwashing. When the latter danger is imminent, the gravel in this layer should be as uniform as possible with the lower grain size limit from 4 to 4.5 times the effective (10% passing) size of the filtering material, while otherwise the upper grain size limit of the gravel can better be chosen at this value. From layer to layer the gravel size should increase by a factor not exceeding 4 as ratio between the upper grain size limit of the gravel below and the lower grain size limit of the gravel above. The gravel in the bottom layer finally should be so coarse, that it cannot be dislodged by the jets emerging from the orifices in the pipe laterals and that it cannot block these openings. A size of 30-60 mm with the lower grain size limit 2 to 3 times the orifice diameter has been found to satisfy both requirements. The thickness of each layer should be at least 0.07 m and at least 3 times the upper grain size limit of the gravel under consideration, augmenting the thickness of the bottom layers to 0.1 or 0.15 m. Examples of gravel systems built to the rules given above are shown in fig. 4.24, on the left when a great number of openings are present and in the middle when the distance between holes along the lateral or the interval between laterals is larger and the gravel system must help in spreading the wash-water equally over the full underside of the filterbed.

The hydraulic resistance of the gravel system may be calculated with the Carman-Kozeny equation of section 2.4. or set at a value of 0.4 m for a backwash rate of $(15)10^{-3}$ m/sec.

Gravel for rapid filters should consist of hard rounded stones with a specific gravity not less than 2.5 and should be carefully washed to remove sand, clay, loam, dirt and organic impurities of any kind. The gravel should not contain more than 2% by weight of thin, flat or elongated pieces and not more than 5% by weight should be lost after immersion for 24 hours in warm, concentrated hydrochloric acid. The grains of the gravel layers should be carefully packed, the larger size even by hand to prevent instability during backwashing, which would result in miniature land slides, disturbing the gravel system and allowing filtersand to reach and clog the perforated laterals.

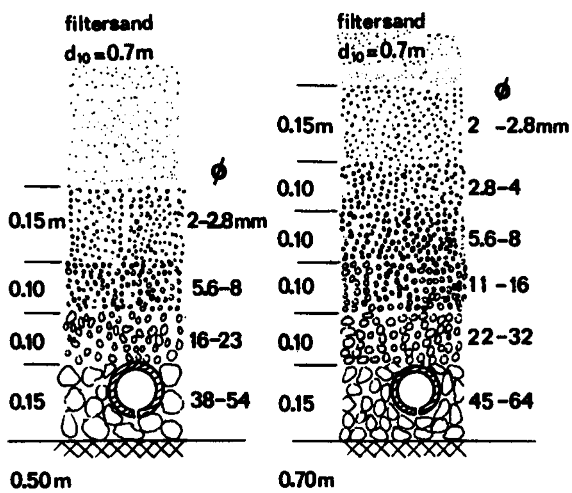



Fig. 4.24 Composition of gravel layers in rapid filters

When it is expected that backwashing the filter with water alone is insufficient to keep the filterbed clean on the long run, an air-wash system may be installed. The simplest and cheapest solution is to administer this air with the perforated pipe underdrain system already present, providing the laterals with small diameter air holes in the top, as shown in fig. 4.25. Especially when backwashing the filter with air and water simultaneously, however, more certainty of equal air and water distribution can be obtained with a separate distribution system for the wash-air, allowing also a greater number of air openings, 50 to 100 per $m^2$ for a more equal distribution. The design of this system follows the same rules as given above for the wash-water distribution system. As mentioned in section 3.8, the rate of air-wash is about equal to that of water-wash, mostly between $(10)10^{-3}$ and $(20)10^{-3}$ m/sec, measured as atmospheric air. The mass density of air in the meanwhile is much less than that of water, at a pressure of 1.3 atmosphere being a factor of 600 smaller. In the air-distribution grid much larger velocities are consequently allowed, 10 to 15 m/sec, resulting in very small pipe diameters, commonly between 15 and 25 mm. With the controlling loss of head between 0.2 and 0.5 m water column, the openings are also extremely small, not more than 1-2 mm. Asbestic-cement is now unsuited,

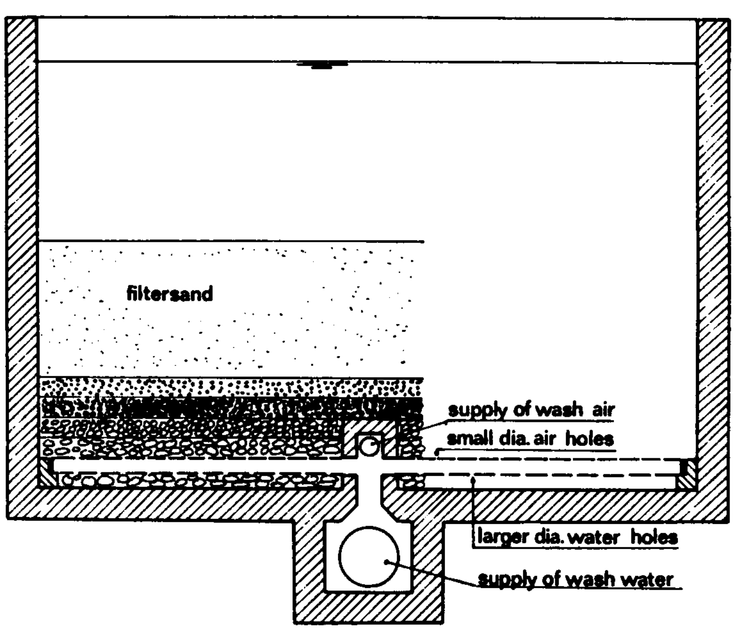



Fig. 4.25 Perforated lateral underdrains for backwashing with water and air

making copper and hard plastic the most attractive materials for construction of the air pipes. These materials, however, are quite soft and small diameter pipes made of them will consequently bend easily. With respect to the difference in mass density between the air in the pipes and the surrounding water it is on the other hand essential that all air openings are situated at one and the same level to preserve an equal air distribution. With regard to this danger of sagging, the air distribution system can best be placed directly on top of the laterals of the underdrainage system for water, as shown in fig. 4.26. This also assures that the air pipes are surrounded by coarse gravel, 20-30 mm, eliminating the danger of gravel displacement by the high-velocity jets of air. When not in use, the air pipes will fill with water by which the small discharge openings of 1-2 mm diameter tend to clog with suspended matter still present in the filtered water and in particular by bacterial growth. This clogging can be prevented by a periodic chlorination or by keeping the air lines full of air, maintaining a minimum air pressure under all circumstances, sufficient to prevent the entry of water.

Summing up it may be said that the perforated lateral system of underdrains has been very popular for many years with as result that the majority of existing filter plants are equipped with this type of filter bottom. When properly designed and executed, they give excellent results, while their usefull life is nearly unlimited. As yet no cheaper system is available and for many developing countries it has the added advantage that it can be constructed locally with a minimum of foreign exchange. As absolute

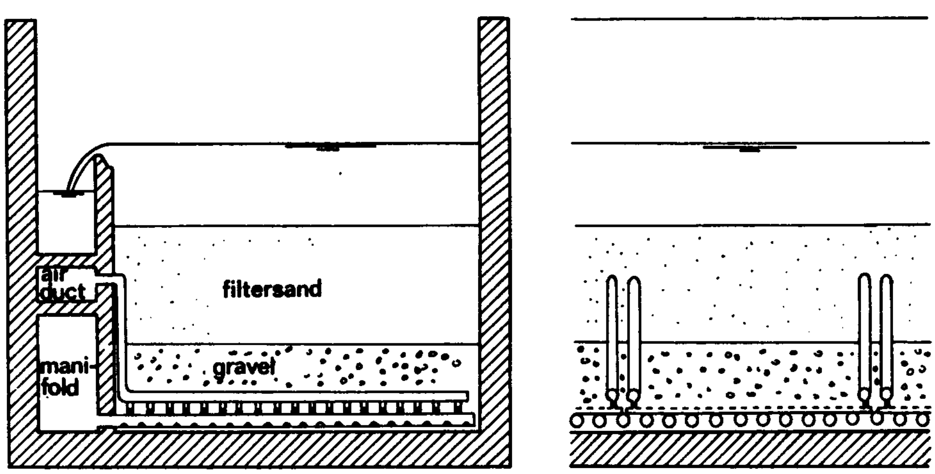



Fig. 4.26 Seperate system of perforated laterals for backwashing with water and air

pre-requisite must be mentioned, that the designing engineer is well versed in hydraulics. In the past many mistakes have been made in this respect, of which a beautiful (and all to frequent) example is shown in fig. 4.27 at the top. Here the washwater rate is adjusted to the desired value by partially closing the valve in the connection to the washwater supply main. Especially when the washwater is taken from an elevated reservoir, this valve must be able to destroy large amounts of head, with as result that the washwater enters the manifold at extremely high velocities, 7 or 10 m/sec for instance. This means a velocity head of 2.5 to 5 m of which part will be recovered as the water moves along the manifold. At the downstream end of the manifold, the water pressure will consequently be much higher, resulting in a higher backwash rate, a larger amount of sand-bed expansion and a forward movement of the filtering material. After a while the filterbed thickness will vary strongly over the length of the filterbox, appreciably reducing filtration efficiency and deteriorating effluent quality. The solution of this problem in the meanwhile is rather simple, as shown in fig. 4.27 at the bottom where the washwater rate is adjusted centrally, for instance upon leaving the elevated washwater reservoir (fig. 3.10), while the conical enlargement of the connecting pipe assures a low entrance velocity of the washwater, equally distributed over the height and width of the lateral.

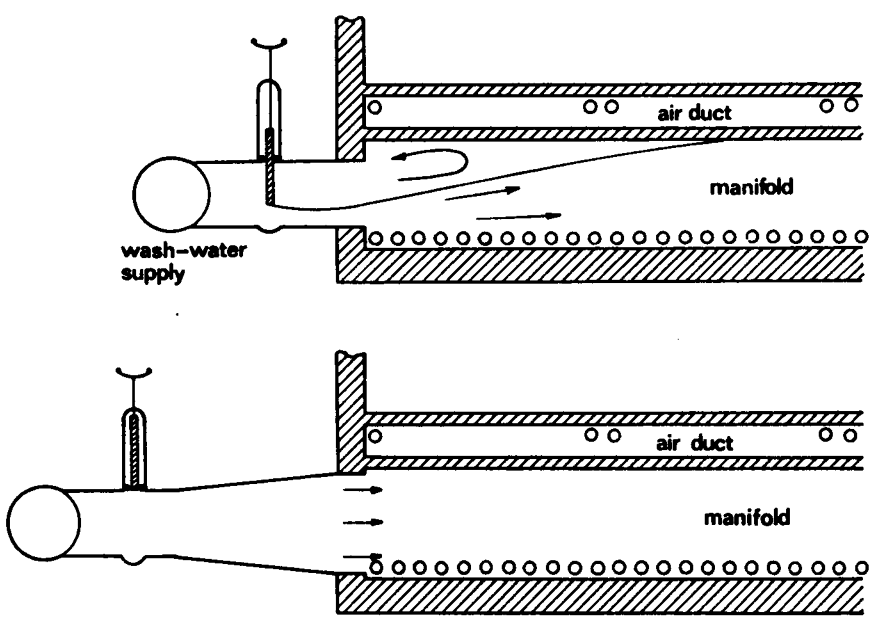



Fig. 4.27 Washwater connection to manifold

The situation of fig. 4.27 at the top can even be improved by a judicious use of baffles, the size, shape and position of which may be determined with a model test in a hydraulic laboratory. A disadvantage of the perforated pipe underdrainage system is indeed the presence of a 0.5 to 0.7 m thick bed of gravel between the filterbed and the laterals, increasing the depth of the filterbox and augmenting the cost of construction without adding to the efficiency of the filtration process. When not properly designed and executed, this bed of gravel may again lead to many failures, for instance by a dispersion of the upper gravel layers though the filterbed and a penetration of the filtering material into the underdrainage system. Whether the design failures indicated above are responsable or not, a decline in the popularity of the perforated pipe underdrain system is a fact. Without any doubt this is promoted by the human dislike of old and so-called old-fashioned constructions. Unfortunately, however, this leads to a preference of modern solutions, even if they have not yet proved their worth in practice. If this tendency exists, attention must be drawn to fig. 4.28, showing the perforated pipe underdrain system in a new shape. In case demand is large and mass production possible, it is also cheaper than the standard system composed from individual pipes.

The disadvantage of a large depth of gravel between the perforated laterals and the filterbed proper may partially be obviated by application of the pipe -and- strainer underdrain system of fig. 4.29. Here the holes are set in the top of the laterals and provided with strainers. These strainers in their turn are supplied with a large number of small openings,

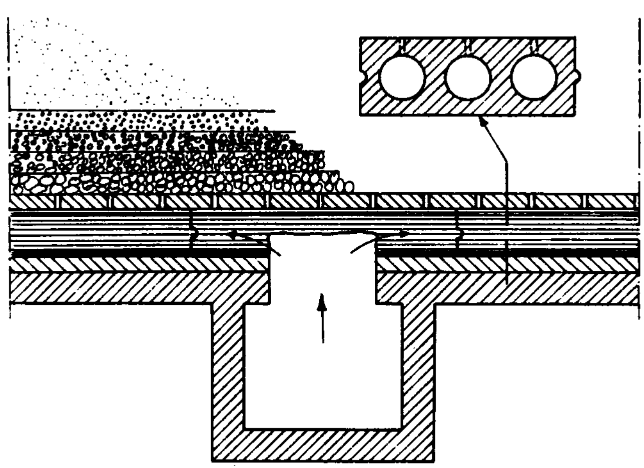


Fig. 4.28 Modern design of perforated pipe underdrain system

discharging the washwater horizontally into the surrounding gravel. Jets from small openings, however, cannot dislodge even fine gravel and the same fine gravel is already coarse enough to prevent a blocking of the small openings by the individual gravel grains. With slits of 1 mm for instance and the filtersand of fig. 4.24, one layer of gravel $\phi$ 2-2.8 mm in a thickness of 0.15 m is sufficient, while under all circumstances layers of gravel with a total thickness of 0.2-0.25 m satisfy normal requirements. The shallow depths of gravel are not able to disperse the rising washwater equally over the full underside of the filterbed. This must now be accomplished by the strainers themselves, by setting them closer together, in a number of 50 to 70 per $m^2$. As failure of a strainer will result in a large loss of filtering material into the underdrainage system, blocking this system completely and asking for costly and time consuming repair jobs, the strainers must be made with sufficient structural strength from corrosion resistant materials as for instance brass, stainless steel or bronze. For added protection and to avoid dead spaces, the laterals are commonly embedded in lean, easily removable concrete.

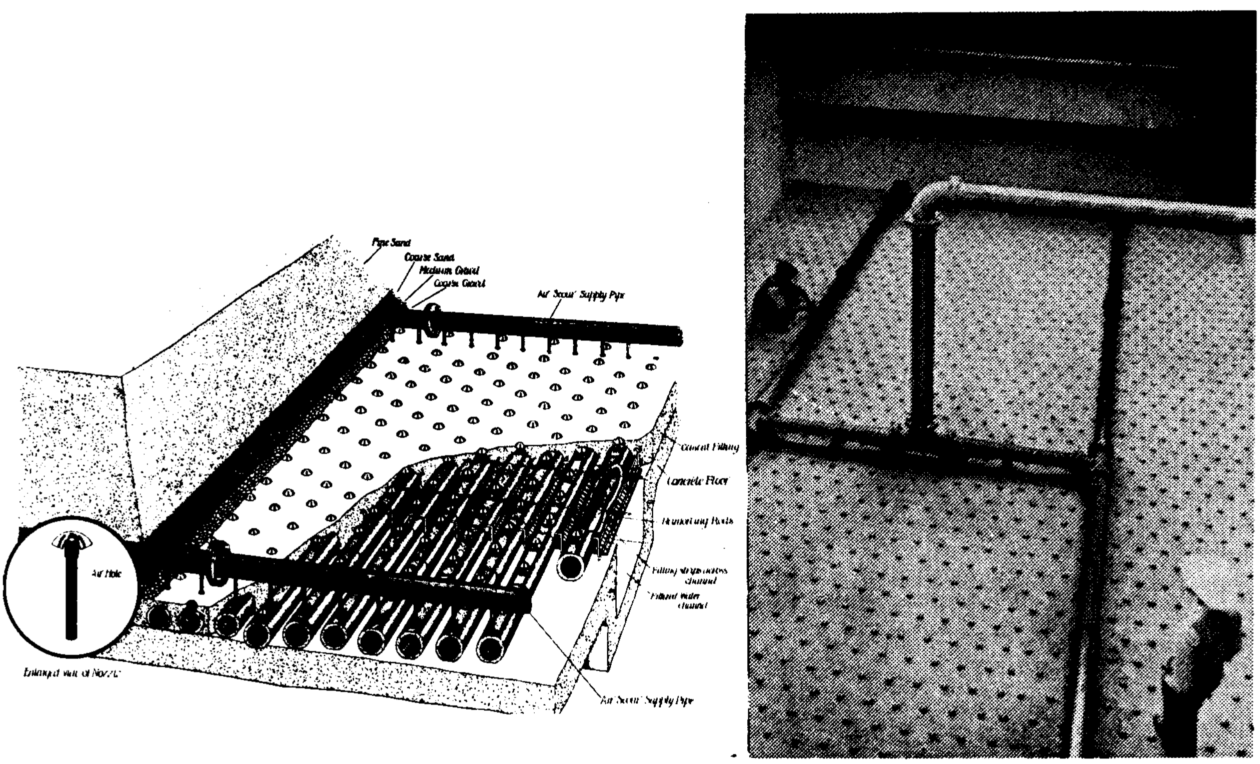


Fig. 4.29 Pipe-and strainer underdrainage system

The pipe -and- strainer underdrain system can easily be made suitable for air-wash, either separately or in combination with water-wash, by extending the strainer with a small diameter tube downward into the lateral. In the upper part of the tube a small hole is present through which air is able to enter the strainer, while the washwater is supplied at the same time through the tube, the air-water interface in the lateral being between the air hole and the bottom of the tube.

Pipe -and- strainer underdrains are no longer used, the point being that once strainers are chosen for supplying washwater to the filterbed, these strainers can better be set in a false bottom, doing away with the more complicated lateral system altogether (fig. 4.30). When below the false bottom a space of 0.2 to 0.3 m is provided, the washwater moreover has unrestricted acces to all strainers, reducing variationgs in piezometric level to nearly negligeable values, by which a small hydraulic resistance of these strainers is already sufficient to assure an equal distribution of washwater over the entire underside of the filterbed. This presupposes in the meanwhile that the entry of washwater into this space does not give rise to variations in piezometric level by partial recovery of velocity head. With this danger in mind, the filter of fig. 4.30 is provided with a concrete channel to receive the washwater from the supply main and to distribute it over the space below the false bottom with the help of a number of perforated pipes. To obtain room for these distribution pipes, the depth below the false bottom must be increased to 0.4 or 0.5 m. A further increase to 0.7 m to make this space available for inspection, maintenance and repairs is now a small step, but it completely defeats

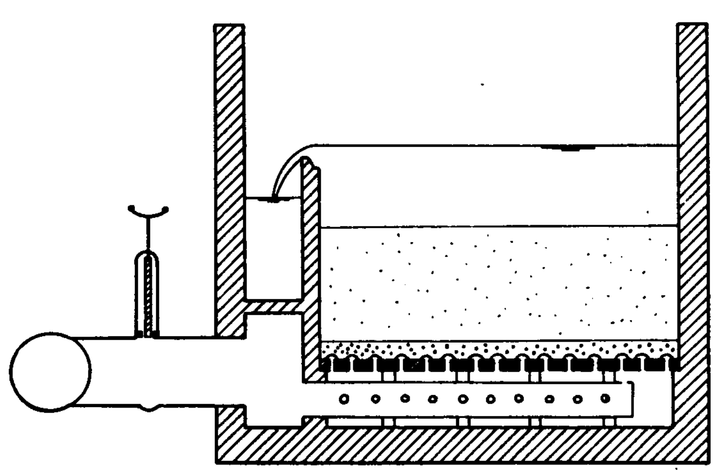


Fig. 4.30 False bottom and strainer underdrainage system for backwashing with water

the original goal of obtaining a filter bottom with a smaller depth than required for the perforated lateral underdrainage system.

As mentioned above, the more even distribution of water pressure in the space below the false bottom, allows a sizable reduction of the controlling loss of head, a value of 0.5 to 1 m being mostly sufficient. This hydraulic resistance of the strainers in the meanwhile is difficult to calculate from the constructional details as an unknown portion of the slits will be blocked by the surrounding gravel or filtering material. When these data cannot be supplied by the manufacturer, tests in a hydraulic laboratory are indispensable. With no or only a small depth of gravel around and above the strainers, a large number, 80 per $m^2$ for instance, is necessary to disperse the rising washwater equally over the full underside of the filterbed. With the modern trend of using coarser filtering materials in a greater bed thickness, this number may be reduced to about 36 per $m^2$, giving an appreciable saving in the cost of construction.A better distribution of the rising washwater and at the same time some protection against mechanieal damage may now be obtained with the countersink mounting of fig. 4.31.

False bottoms are commonly made in sections, about 0.6 m square, from steel, asbestic cement or reinforced concrete and supported by ridges, short columns or even bolts cast into the reinforced concrete bottom of the filterbox (fig. 4.32). Much care must be taken to prevent leakage between the individual sections, for which special joint constructions and filling materials are nowadays available. Strainers were formerly made of strong and corrosion resistant materials such as copper, bronze, stainless steel and porcelain, able to resist any attack but rather expensive. This is the reason that today plastic is almost used exclusively. In the past many a plastic strainer has been broken, after which a nearly unlimited loss of filtering material into the underdrainage system occured. Repairs are expensive and time consuming, while damage may already have been inflicted on valves and other appurtenances. With expert design and a proper selection of materials, the danger of breaking a plastic strainer is nowadays small, but never absent, another reason for making the space below the false bottom accessible for repairs, if only temporarily by closing the bottom of the broken strainer. To limit the number of gravel layers or with coarser filtering materials to omit these layers altogether, there is a tendency to equip the strainers with very fine slits, down to 0.5 mm. It must never be forgotten, however, that such narrow slits are easily clogged

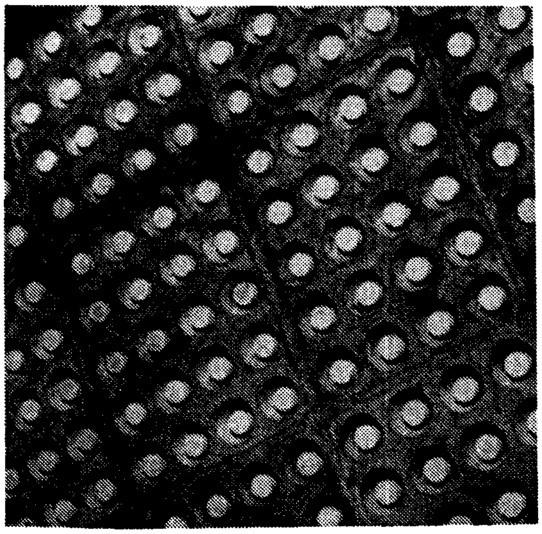
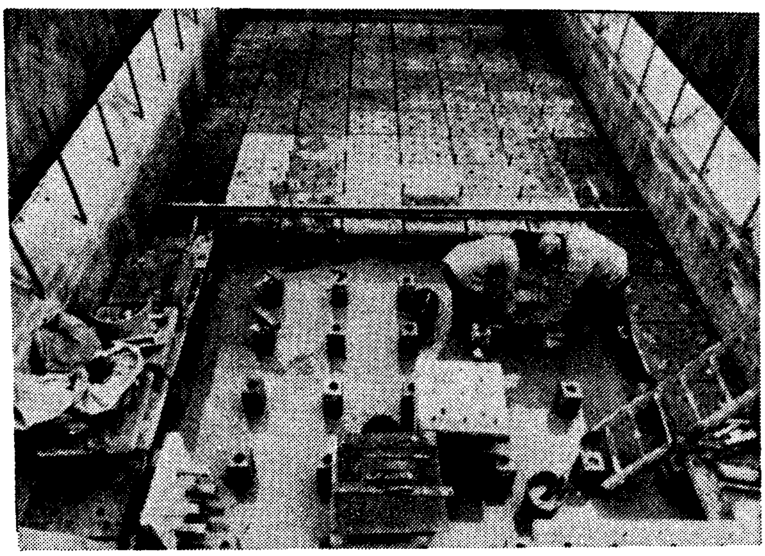


Fig. 4.31 Countersink mounting of strainers in a false bottom



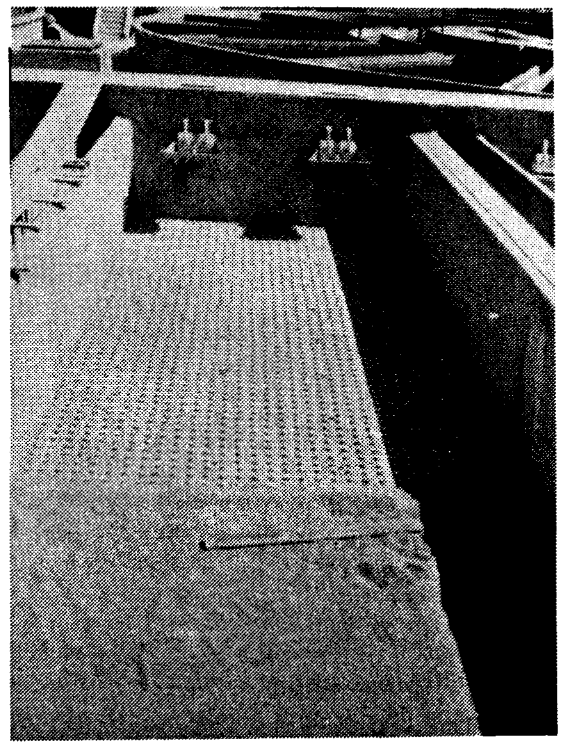


Fig. 4.32 Construction of false bottom and strainer underdrainage system

by algae or small animal life, originating from the space below the false bottom. Backwashing a filter at a rate of $(15)10^{-3}$ m/sec is an impressive sight when observing the boiling sand bed, but a vertical velocity of 15 mm/sec in the space below the false bottom is insufficient to carry small vegetable and animal matter through the strainer openings into the filterbed and thence to waste. Aquatic life will flourish in this space, producing a large amount of clogging matter (fig. 4.33). Already after blocking of a few strainers, an uneven distribution of washwater will result, while blocking of a larger number of strainers increases the hydraulic resistance of the false bottom to such an extend that it is unable to withstand the waterpressure during backwash. It will now burst upward, destroying the rapid filter completely. Aquatic growth may be prevented by chlorination of the raw water or even of the washwater only, but this inhibits any biological activity of the filterbed and even of the subsequent slow filters when present.

False bottom -and- strainer underdrains can easily made fit for a separate or simultaneous air-wash by providing the strainers with a long stem, extending downward in the space below the false bottom (fig. 4.34). During backwash, washwater enters this stem at the lower end, while for introduction of wash-air the stem has a hole in the upper part. For an equal distribution of the wash-air, these holes must be small and all of exactly the same diameter. Formerly instead of holes long narrow slits were used for this purpose. Not to disturb the equal distribution of wash-air, the top of these slits had to be set at exactly one and the same level, a rather laborious and expensive job.

With the false bottom -and- strainer type of underdrain, a better distribution of the washwater over the underside of the filterbed can be obtained by increasing the number of strainers per $m^2$. Ultimately this leads to the use of porous plate filter bottoms, supplying washwater evenly over

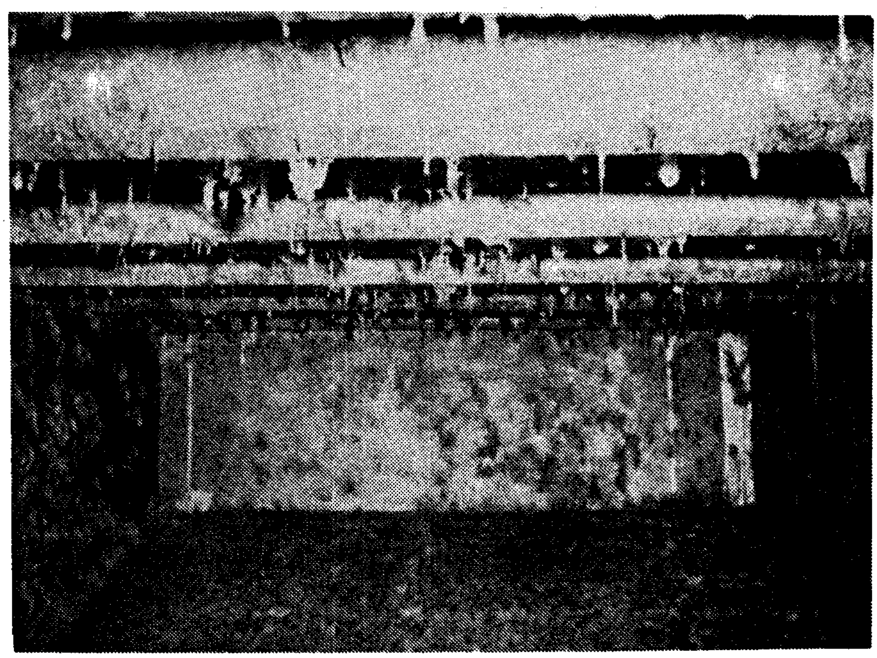


Fig. 4.33 Fouling of the space below a false bottom by aquatic growth

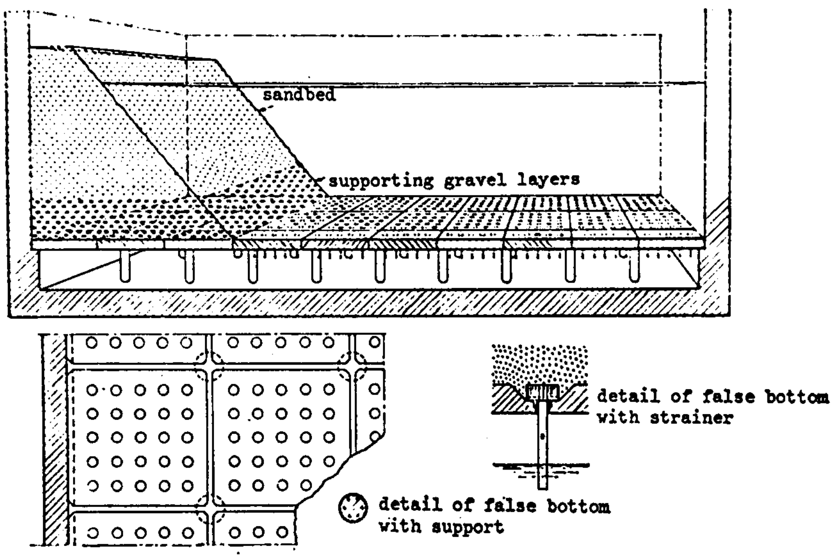
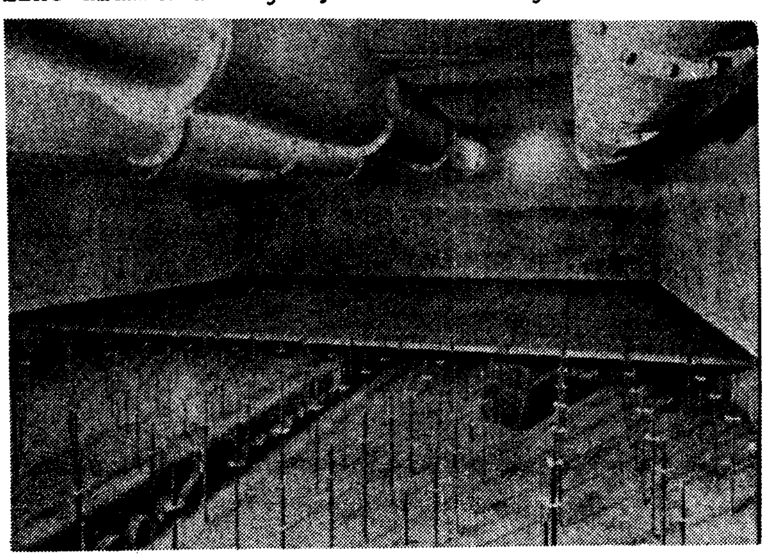



Fig. 4.34 False bottom and strainer underdrainage system for backwashing with water and air

the entire area of the filterbed. The openings in these porous plates are so small that even fine sand can be placed directly on top. Gravel layers are thus unnecessary, effecting some economy in the cost of construction and above all eliminating difficulties resulting from a dispersal of this gravel through the filterbed. Porous plate filter bottoms are again made of sections, about 0.6 m square and supported at a distance of 0.2 to 0.3 m or more above the bottom of the filterbox by beam or ridges, columns of concrete or asbestic cement or even steel bolts (fig. 4.35). Also here much care must be given to the construction of the joints between the individual sections, assuring completely watertight connexions. The porous plates themselves can be made of different materials. In the U.S.A. vitrified crystalline aluminium oxyde, more commonly known as corundum is used for this pur-



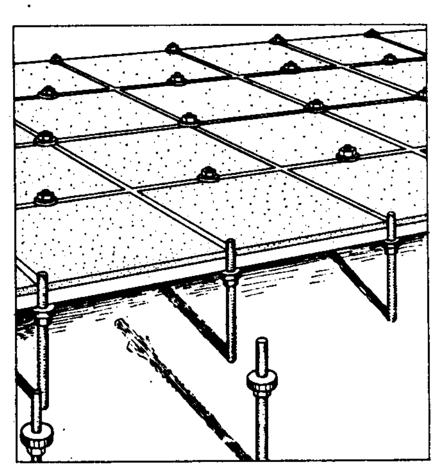


Fig. 4.35 Porous plate filter bottom

pose, while in Europe such plates have been made of no fines concrete.

Without any doubt, porous plate filter bottoms have an enormous appeal, giving the simplest solution for the problem at hand. To assure an equal distribution of washwater in the meanwhile some resistance of this bottom is still required, for instance 0.5 m at a backwash rate of $(15)10^{-3}$ m/sec. With the porous plate bottom pervious over its entire area, this asks for extremely fine openings, of the same size or only slightly larger than the pores in the filterbed above. Filtered water, however, still carries some impurities in suspension, which may be removed by the openings of the filterbottom, while even dissolved substances such as iron, manganese, calcium, magnesium, etc, may be deposited here. After some period of service clogging of the filter bottom will thus occur, increasing the resistance against the upward flow of washwater, which now must be supplied at a higher pressure till ultimately the filter bottom breaks away to above. This phenomenon cannot be prevented entirely, but it may be retarded and made less serious by a periodic cleaning of the porous filterbottom with a 2% NaOH or a 5% inhibited HCl solution, depending on the nature of the cloggings. Needless to say that this is only allowable when the filterbox with adjoining pipelines and appurtenances is able to resist the subsequent chemical attack. This will ask for additional provisions, further augmenting the already high cost of this type of filterbottom. Summing up it must be said that how attractive a porous plate filterbottom may look at first sight, a general application cannot be advised. This is even more so when air wash is necessary, for which a separate distribution grid must now be provided. To prevent blocking of the openings in the porous plates by air bubbles, this grid must be set above the filterbottom, where it will result in a serious disturbance of the filterbed during backwashing and a larger loss of filtering material into the washwater troughs and gulleys. This may be prevented by covering the air grid with one or two layers of gravel, but this eliminates many advantages of this underdrainage system. Some engineers are so fascinated by the simplicity of a porous plate filterbottom that they go to all extremes in their endeavour to improve its applicability. Above all the rapid clogging of the fine pores must be avoided, with as most direct approach an enlargement of these pores by the use of coarser grains, for instance no-fines concrete composed of pea gravel. Needless to say that the resistance of such a bottom is too small to assure an equal distribution of washwater over the entire area of the filterbed. This, however, may also be obtained separately by the application of a second false bottom, composed of ordinary concrete and provided with a limited number of small holes to create the desired resistance. For backwashing with water or with water and air, these double false bottoms are shown in fig. 4.36. A bottom pervious over its entire area, without the use of gravel layers, is certainly attractive, but it remains debatable whether the solutions of fig. 4.36 are not too complicated and too expensive.

As mentioned before, the number of underdrainage systems that have been applied in practice is a multiple of the systems dealt with in this section. Disregarding failures, the majority of these systems operate along the same general lines as elaborated above, with sometimes only slight differences in construction to make them better suited under special local conditions in terms of availability of material, cost of labor, tradition, preferences of the management, etc. It would be impossible to mention them all, but an exception may be made for the Wheeler false filterbottom, whose beauty has not yet been surpassed (fig. 4.37). The proprietary systems are mostly developed to enhance the competitive powers of the respective firm. Although claimed otherwise, they are not always better than existing systems, but mostly more expensive!

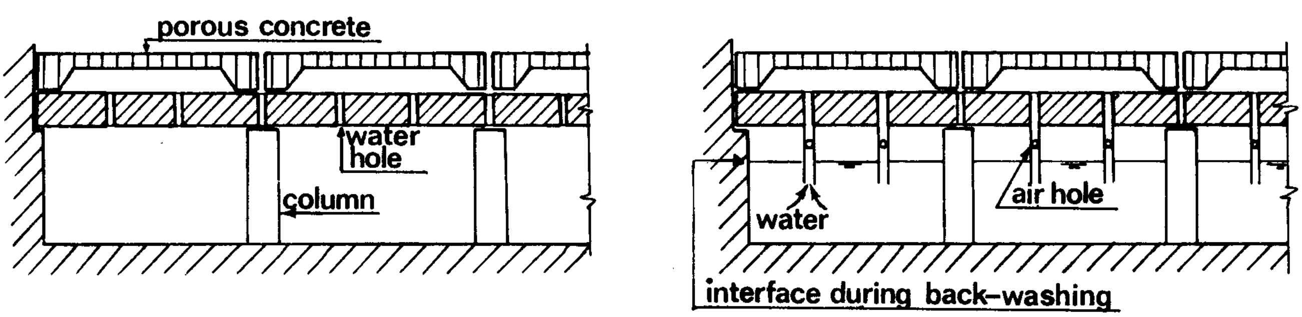



Fig. 4.36 Double false bottom underdrains

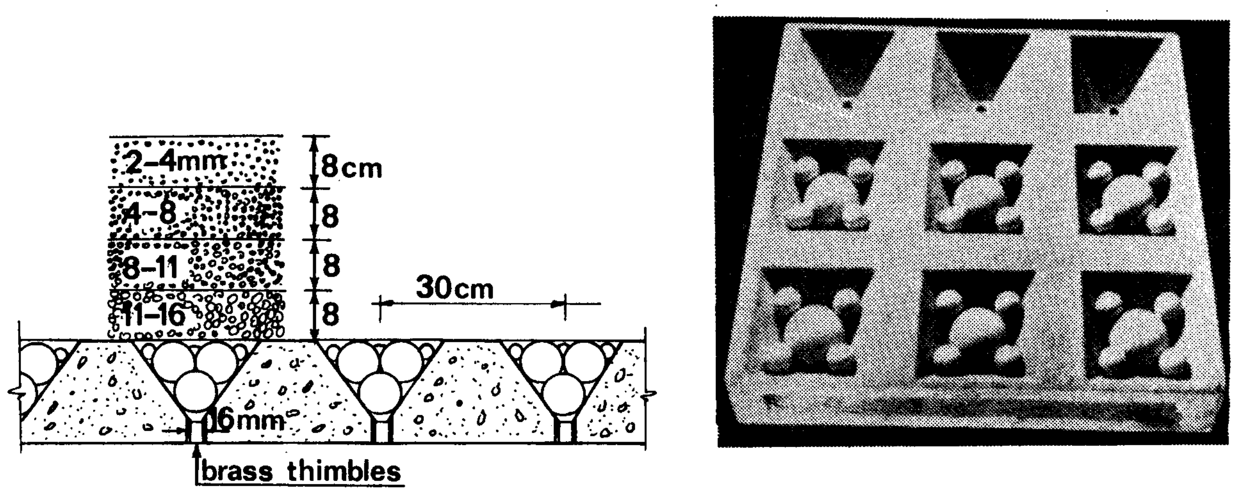


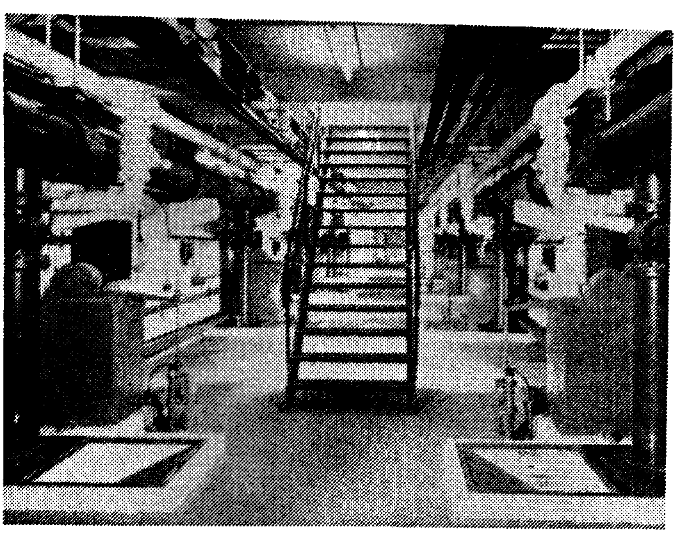

Fig. 4.37 Wheeler false filter bottom

## 4.6. Pipe gallery and operating floor

As mentioned in section 4.2, the various filtering units are commonly arranged on one or both sides of a two-level corridor, the lower part of which forms the pipe gallery and the upper part the operating floor.

The pipe gallery houses the pipes and other conduits for carrying raw and filtered water, wash and waste water, wash air, etc, together with the necessary valves, filter controls and so on. Also pressure lines for hydraulic operation, electric cables, ventilation equipment, heating pipes, etc, must be accomodated in this space. Altogether this means a large amount of equipment, complicating the design of the pipe gallery to a considerable extend. With regard to the cost of construction, the gallery should be as small as possible, any waste space in this area increasing the width of the operating floor and the volume of the filter building beyond normal requirements. Although economy is a factor, this gallery should on the other hand offer adequate space for convenience of inspection and for removal of faulty equipment. One should be able to walk the length of the pipe gallery without having to climb over piping and without walking through puddles of water and it should be possible to remove any individual valve without the disassembly of larger amounts of piping. Ample points of access should furthermore be provided for ease in handling of heavy pieces of equipment. Especially with regard to this pipe gallery, the designer should use his ingenuity to develop an arrangement of piping that satisfies all functional requirements and insures ease of maintenance and operation. Good examples are shown in fig. 4.38.

Although all care must be exercised to obtain watertight joints and connections, some leakage of water must still be expected in the pipe gallery, asking for floor drains with sump and sump pumps to discharge the collected drainage. This leakage in the meanwhile will also result in a damp atmosphere, attacking metal parts by corrosion. Formerly this danger was obviated by using cast iron for pipes and appurtenances. With regard to its heavy weight and high cost, however, cast iron is now replaced by steel and although good protective coatings are available, ventilation or when necessary even complete air conditioning should be installed to assure a dry atmosphere in the pipe gallery. By the advance of electric operation, telemetering and tele-control this air-conditioning is even essential to assure safe and reliable operation.



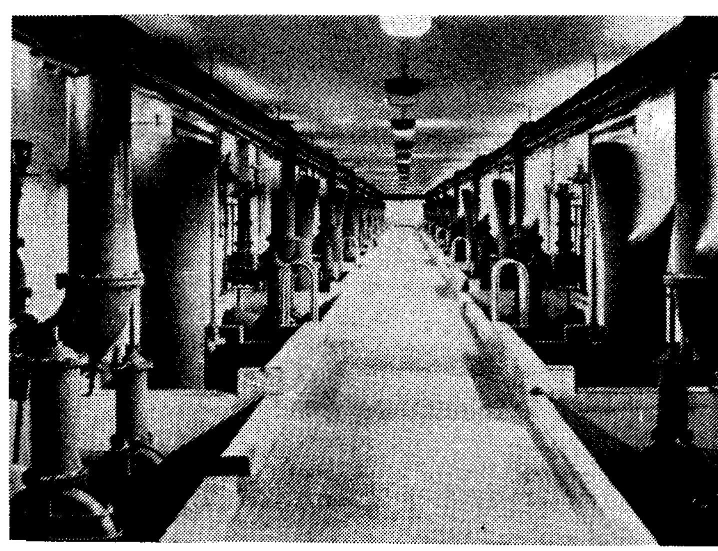


Fig. 4.38 Pipe galleries

The operating floor should be designed for maximum convenience to the operating personnel, including ease of maintenance and provision of facilities to place and replace filtering material. Under all circumstances this operating floor is housed, while in moderate to warm climates the filters themselves may be built in the open air.When treating deep ground water, safe in bacteriological respect by virtue of its origin, all possibilities of pollution should be avoided. The filters must therefore be installed in a building, separated from the operating floor by a glass partition wall. The operating floor is the focal point of visitors to the plant and is therefore commonly well decorated, finished and lighted, as shown in fig. 4.39. Some designers prefer a direct connection between operating floor and pipe gallery, of which system fig. 4.40 gives a nice example.

Nowadays manual operation of valves in a filter building is an exception and commonly they are driven by hydraulic, pneumatic or electric force. These valves are handled from an operating table near the respective filtering unit, which table also contains controls, gauges, etc. Again here, much attention is given to outward appearance as may be gathered from fig. 4.41. The demonstration panel of the bottom right of this picture should never be used. Even the best quality filtered water contains minute amount of impurities, on the long run still able to stain the glass container and making an unfavorable impression on the visiting public.

With regard to the rising cost of labour and also because the job of filter attendent on a round-the-clock basis has little appeal, the majority of future rapid filtration plants will be operated by remote control. Commands may be given from a central control-room, perhaps a large distance away or from a small process computer. With no personnel on the operating floor, other designs will emerge.

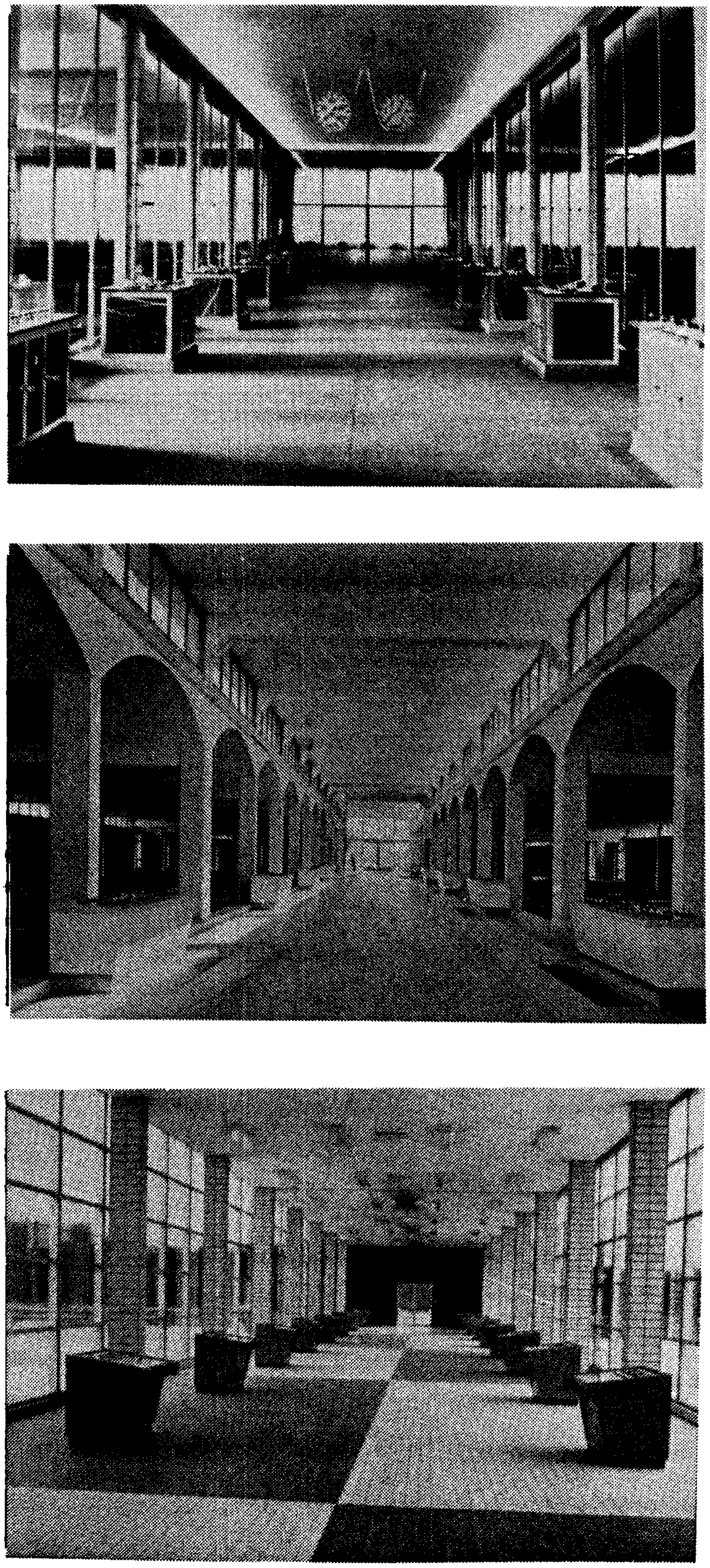


Fig. 4.39 Operating floors

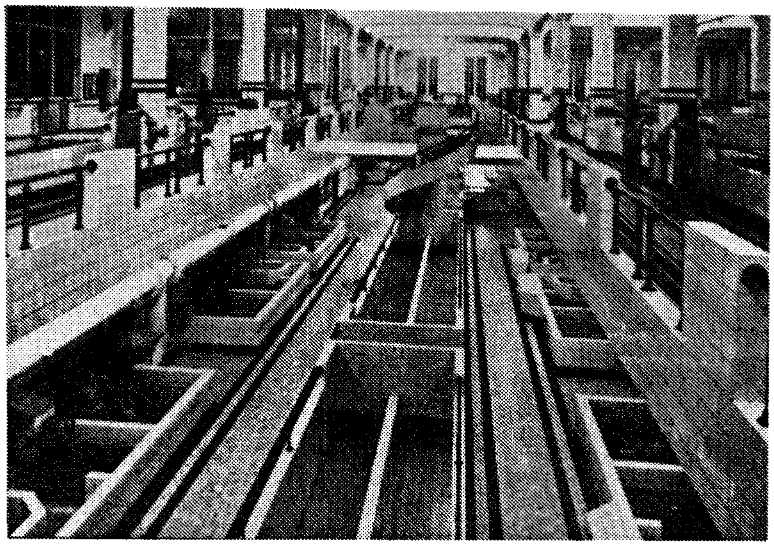
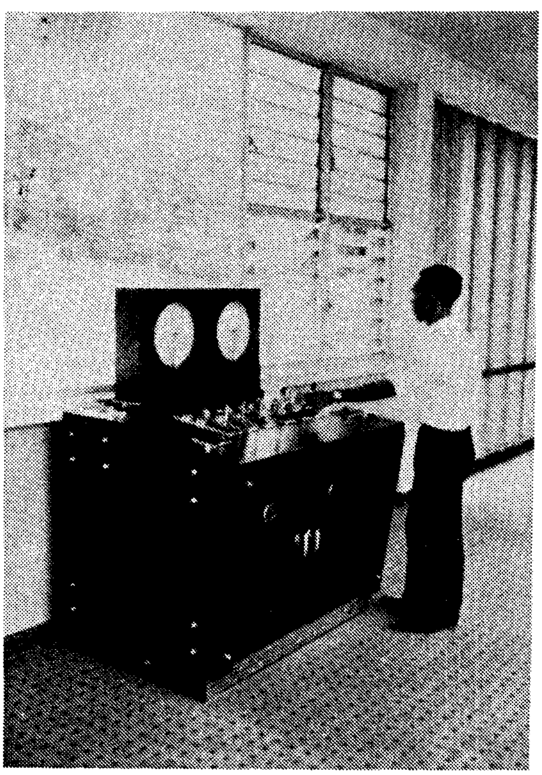


Fig. 4.40 Direct access from operating floor to pipe gallery



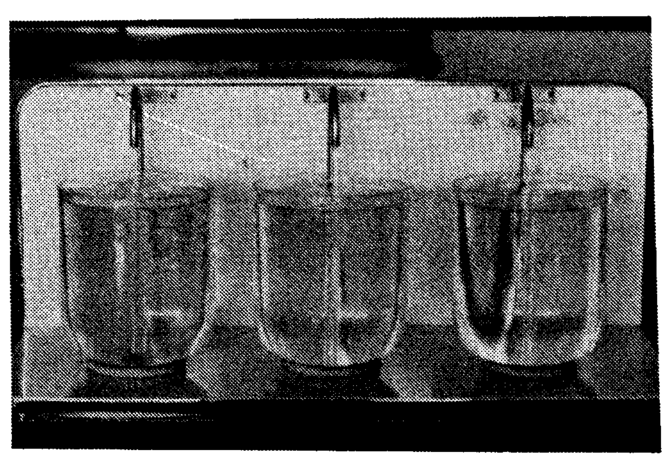


Fig. 4.41 Operating tables

## 4.7. Structural requirements

Filter buildings are commonly constructed of reinforced concrete, the design of which follows normal rules with the added difficulties, however, that the atmosphere in a filter building is usually damp and that the water retaining parts such as filterbox, reservoirs, conduits, etc, must be absolutely water-tight. In some countries special standards have been devised for these structures. As good example may be mentioned the British Standard Code of Practice CP 2007 for the design and construction of reinforced and pre-stressed concrete structures for the storage of water and other aqueous liquids.

As most important features in the design of concrete for filter buildings may be mentioned, that ample covering for protecting the reinforcing bars from corrosion should be provided and that all bars should be placed far enough apart to permit the concrete to surround them entirely. To prevent cracks with subsequent penetration of moisture, corrosion of the steel bars and spalling off of concrete, tensile stresses in the concrete as well as in the steel reinforcement must be limited and tensile stresses due to drying shrinkage, temperature changes and differences in soil subsidence prevented as much as possible by subdividing the entire building in a number of independent sections. Much attention should be paid to the design of water-tight expansion joints connecting the different sections, as well as to the contruction joints, which must be able to resist the load placed upon them without the danger of cracks and leakages. All construction joints should be planned beforehand in such a way, that concrete can be placed in any given section in a single operation.

As regard the preparation of concrete for filter buildings, imperviousness and an as small drying shrinkage as possible are the most desirable qualities. Unless concrete is impervious, the devastating effect of frost action and leaching of calcium and aluminium components out of the cement will soon ruin the construction. The materials of which the concrete is composed should conform to rigid standards, while mixing, placing and vibrating the concrete should be done with the utmost care. The aggregate should be small enough to pass between the reinforcing bars, thus preventing its piling up on the steel, causing voids below. Much attention should also be given to the design and construction of shuttering, assuring a rigid construction, able to withstand without deformation or leakage the heavy loads of concrete acting as a fluid when being vibrated. Before pouring the concrete, shuttering and especially construction joints should be rigorously cleaned and inspected to assure that the reinforcement is properly spaced and fixed in the forms and that the required number of spacers made of impervious concrete, is present. In damp buildings a plaster finish will generally not give satisfactory results. Here a better solution is to leave the concrete without any covering and to pour it into forms made of steel, laminated wood, etc, to assure a smooth finish.

In ordinary buildings where everything is dry, many of the factors mentioned above may be disregarded without any evidence of the true conditions. With concrete exposed to moisture, however, any failure to observe the necessary precautions will all to soon become apparent.

# 5. PRESSURE FILTERS.

## 5.1. Type and application

Rapid pressure filters are based on the same principles as gravity type rapid filters, with as sole difference that the filterbed with the supporting filterbottom and the supernatant raw water are encased in a water-tight steel cylinder (fig. 1.2). This gives a closed system in which the water to be treated can be forced through the filterbed under a pressure much greater than atmospheric. On one hand this high pressure allows a large filterresistance without the danger of negative heads, while on the other hand filtered water pumps are no longer required and the filter can be set at any random level. In its turn, the application of a large filter resistance permits the use of high filtration rates, through filterbeds of great thickness with still adequate lengths of filterrun. With pressure filter, filtration rates normally vary from $(2)10^{-3}$ to $(5)10^{-3}$ m/sec, while values of $(10)10^{-3}$ or $(15)10^{-3}$ m/sec are no exception. Especially in the latter case, the time of contact between the water to be treated and the filtering material becomes a limiting factor, asking for greater bed thicknesses of 2 or 3 m for instance. With raw water pumps of adequate head, the pressure of the filtered water finally is sufficient for subsequent use by which broken pumping can be avoided (fig. 5.1) and the filters may also be set in an odd corner at a higher elevation or even vertically above each other to reduce the amount of floor space required (fig. 5.2), very important in industrial installations.

The high piezometric level at which the effluent emerges from a rapid pressure filter is of no value when these filters are used as pre-filters, to lighten the load on subsequent slow sand filters which by reason of their enormous area are always built at ground level. As final treatment after chemical coagulation pressure filters can neither be applied because the pumping necessary to force the water through might damage the coagulant flocs carried over from the preceding settling tank, reducing filtration efficiency. This means that pressure filtration is limited to those instances where it constitutes the sole clarification process to which the water is subjected. For public water supplies such a sole treatment is only acceptable when a good quality raw water is available under all circumstances. With surface water sources this is an exception, but it is quite normal for

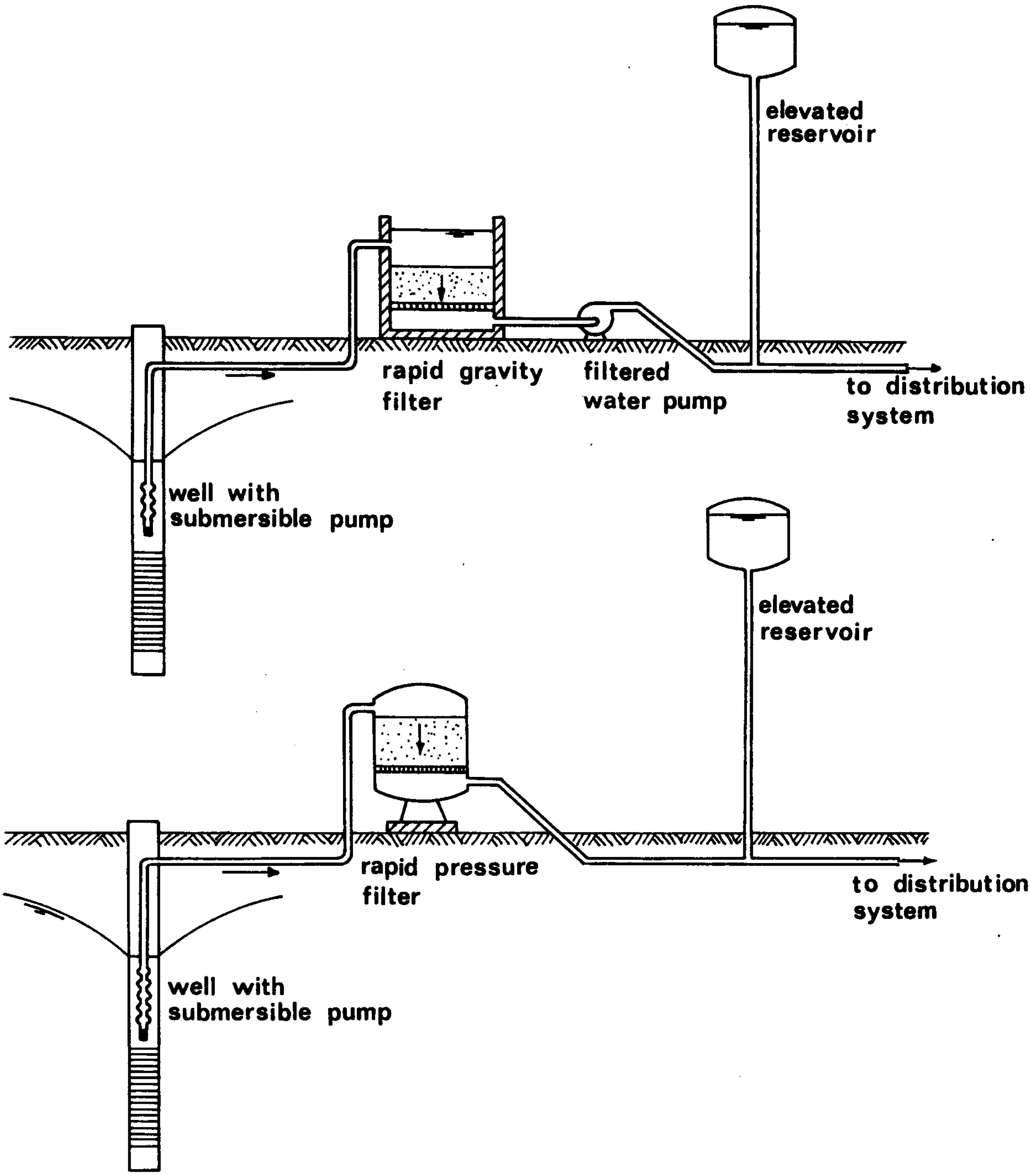
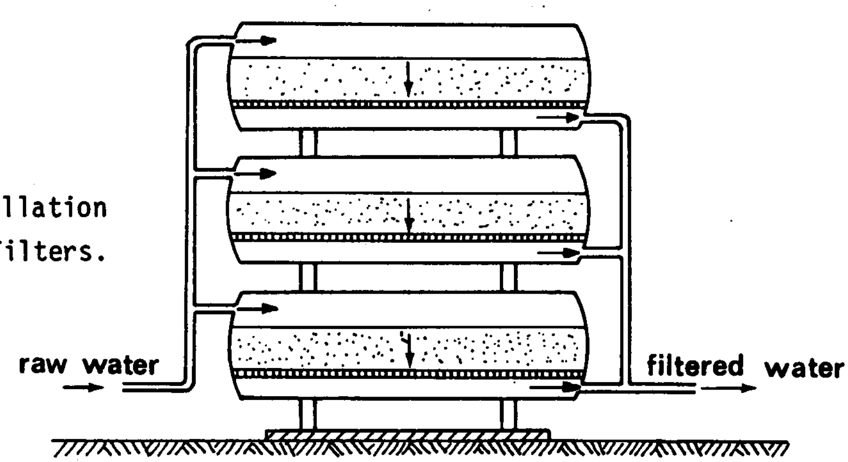



Fig. 5.1 Use of pressure filters to avoid double pumping.

Fig. 5.2 Compact installation of pressure filters.



groundwater, which by virtue of its origin is safe in bacteriological respect. Contamination of groundwater during recovery can easily be prevented, while with pressure filtration the water is not in contact with the outside air, also avoiding bacteriological pollution during treatment. When using groundwater, pressure filters mostly serve to remove dissolved impurities such as iron or manganese. The presence of these impurities, however, indicate the absence of oxygen and in many cases also the presence of agressive carbon dioxide. The oxygen content of the raw water can easily be increased by pressure aeration (using filtered air to avoid contamination), but for simultaneous removal of excessive carbon dioxide atmospheric or even vacuum de-aeration should be used, making a combination with pressure filtration less attractive.

With public water supplies, pressure filtration always has the disadvantage that regular inspection of the filterbed is impossible. This filterbed, however, is easily disturbed by inexpert backwashing or even completely overturned by the pressure of the filtered water when the raw water pumps stop, for instance by a failure of the electricity supply. In theory the latter phenomenon can be avoided by the use of no-return valves, but in waterworks practice these are rather notorious for their unreliability. Many cases are known where already after a few months of service the major part of the filterbed has been washed away, with a corresponding decrease in filtration efficiency. This is the reason that im some States of the U.S.A. pressure filters may not be used for public supplies, while in other countries their application is restricted to small supplies, less than 0.1 or 0.2 $m^3$/sec for instance, serving only a limited number of people.

Pressure filters are used on a large scale for industrial water supplies. When effluent requirements are not very strict, also more turbid surface waters can be dealt with, widening their field of application. For industrial supplies in particular pressure filters offer the advantage that they can be bought as complete units from various manufacturers, that they are cheaper than gravity filters and moreover can be shifted from one place to another and that they can be set in an odd corner at any level, reducing space requirements. By the absence of a water surface in contact with the outside air, the humidity in the building will not increase, making air-conditioning for this reason **superfluous**. In swimming pools, pressure filters are almost used exlusively.

In the future when labour costs continue their upward trend, the price difference between pressure and gravity filters will assume large proportions. On the other hand mistakes in operation can be avoided by additional controls (measuring water pressure and quality at various depths) and in particular by automation while a continuous monitoring of effluent quality will show any deficiency still occurring without delay. Notwithstanding their inherent disadvantages, a large increase in the use of pressure filters may be expected when the second industrial revolution takes effect in water industry. Pressure filtration offers great advantages when the raw water is received under a high pressure, for instance from an impounding reservoir at a much higher elevation.

## 5.2. Construction and operation

Pressure filters can be built with their tank axis vertical or horizontal (fig. 1.2). Vertical pressure filters make the best use of the space available in the steel cylinder (fig. 5.3), but with regard to the installation necessary for forging the dished end plates, their diameter is limited to 4 or 5 m, varying from one country to another. This means unit filterbed areas not exceeding 10 - 20 $m^2$, which can only be applied in small installations with a capacity below 0.2 to 0.5 $m^3$/sec. When larger

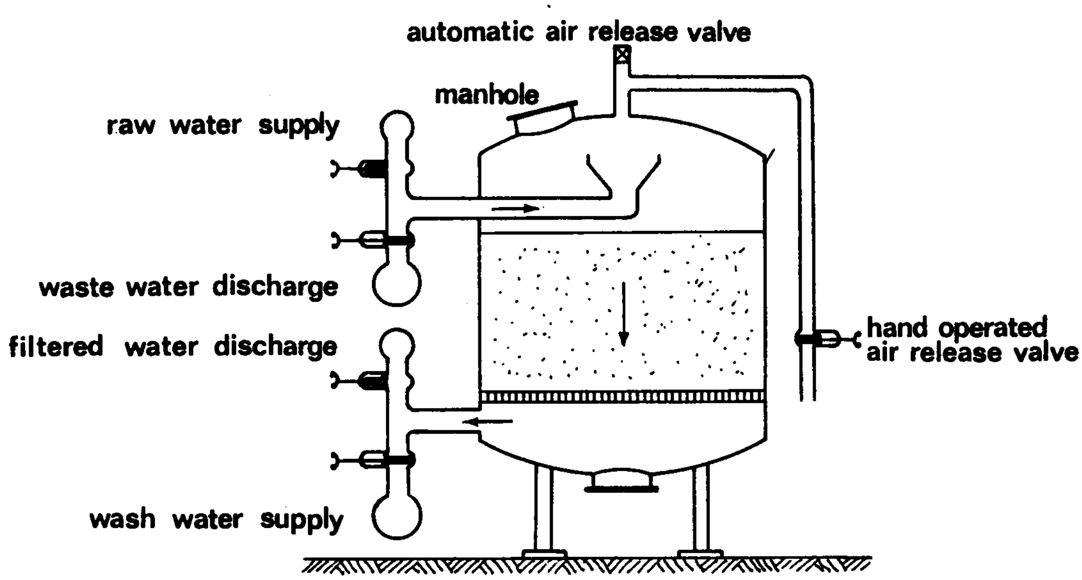



Fig. 5.3 Vertical pressure filter(during filtration).

filterbed areas are wanted, horizontal pressure filters must be applied (fig. 5.4). Here the width of the filterbed is limited to 4 or 5 m, with values of 3.5 to 4 m being most common, but the length of the tank can be increased at will. In practice, however, the length is commonly limited to 10 to 15 m, giving in the meanwhile unit filterbed areas of 35 to 70 $m^2$, in principle fit for medium sized installations with capacities somewhere between 1 and 3 $m^3$/sec.

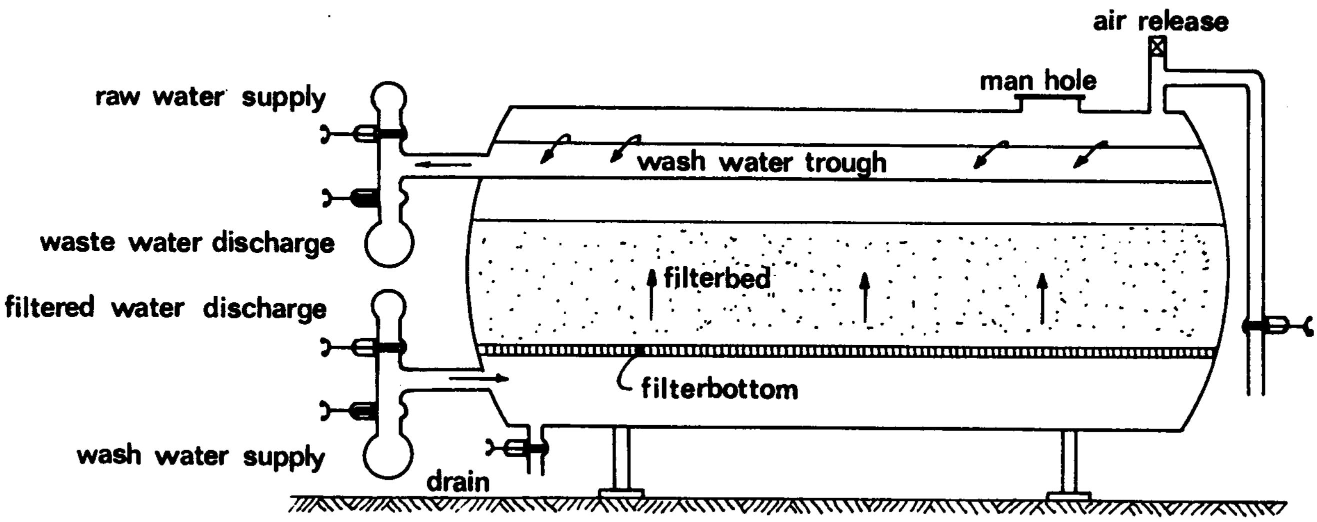



Fig. 5.4 Horizontal pressure filter(during backwashing).

With vertical filters some reduction in the cost of construction can be obtained by fitting 2 filters in a single shell, as shown in fig. 5.5 on the left. Especially with groundwaters containing large amounts of iron, better results at a lower price can be obtained by double filtration, the primary filters equipped with a rather shallow bed of coarse filtering material and the secondary filters provided with a deep filterbed composed

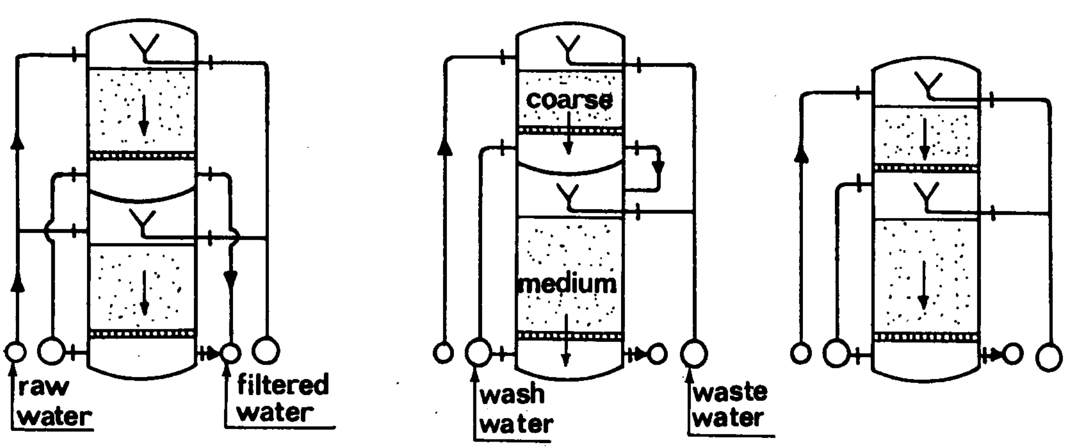



Fig. 5.5 Double pressure filters,in parallel or in series.

of fine grains. With vertical pressure filters, both stages may again be accomodated in the same shell as shown in fig. 5.5 to the right, effecting some economy in construction and above all limiting floor space requirements.

Construction and operation of rapid pressure filters follows the same general rules as elaborated in the preceding chapters with regard to rapid gravity filters. In the subsequent paragraphs therefore, attention will only be given to those elements where differences may be noted.

With pressure filters, the piezometric level of the raw water rises to a great distance above the top of the filterbed. To prevent negative heads and air binding, a large raw water depth is consequently not required and this depth is governed solely by the discharge of washwater with troughs or funnels. To prevent undue loss of filtering material, the overflow edge of these outlets should be at a distance of 0.4 to 0.6 m above the top of the filterbed, depending on the amount of sandbed expansion during backwash.

For the same raw water quality, filtration rates in pressure filters are 30 to 50% higher than with gravity filters, asking for slightly coarser filtering materials. With regard to both factors, much greater filterbed thicknesses must be applied, with values commonly between 1.5 and 2 m and values of 2.5 to 3 m being no exception. For deferrisation of groundwater, high filtration rates of $(10)10^{-3}$ or $(15)10^{-3}$ m/sec offers the advantage of increasing the electro-kinetical potential (section 2.1), thus promoting filtration efficiency. Sharp, broken filtering material in large bed thicknesses is now very attractive.

In principle, the construction of the filter bottom in pressure filters may be exactly the same as described in section 4.5 for gravity filters. As an example, fig. 5.6 shows the use of the perforated lateral system, topped by a number of gravel layers with successively finer grains. With regard to

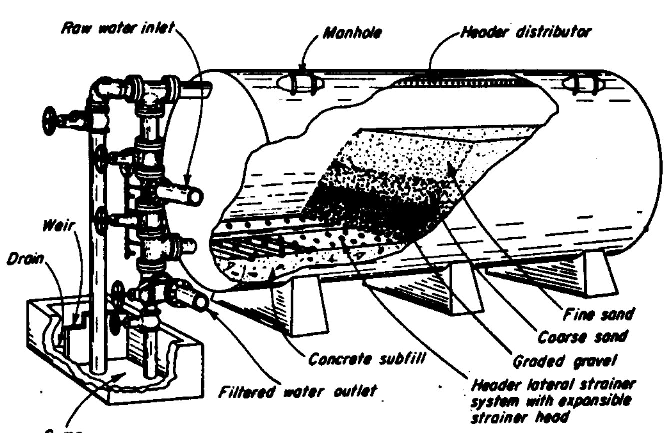
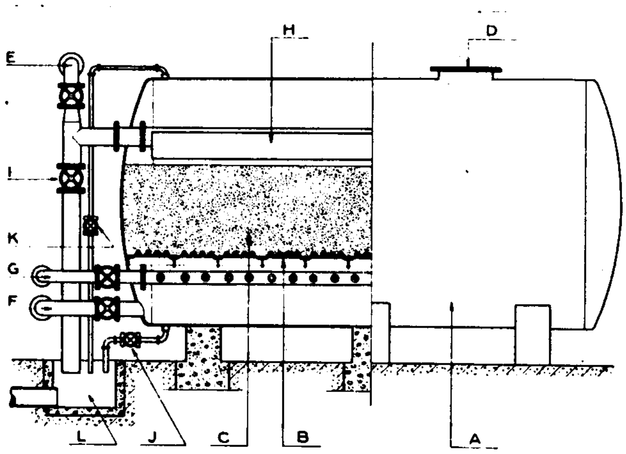


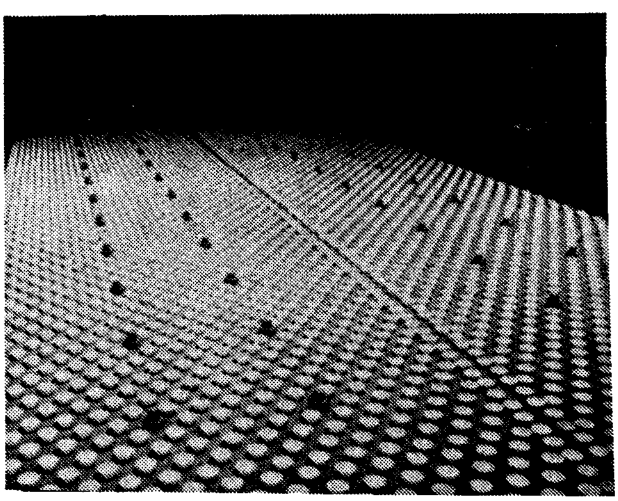

Fig. 5.6 Pressure filter with perforated pipe lateral underdrainage system. (Permutit Co)

the limited space available, however, filter bottoms having a small depth of construction are now preferable. Porous filterbottoms are seldom used, but for pressure filters the false bottom-andstrainer underdrains are very popular (fig. 5.7 and 5.8). When backwashing with water alone is insuffi-




A - steel cylinder
B - false bottom with long stem nozzles
C - filterbed
D - manhole
E - supply of raw water
F - discharge of filtered water
G - supply of wash air
H - wash water gutter
I - discharge of wash water
J - drain
K - air release
L - discharge of wash water

Fig. 5.7 Pressure filter with false bottom-and strainer underdrains. (Degrémont)



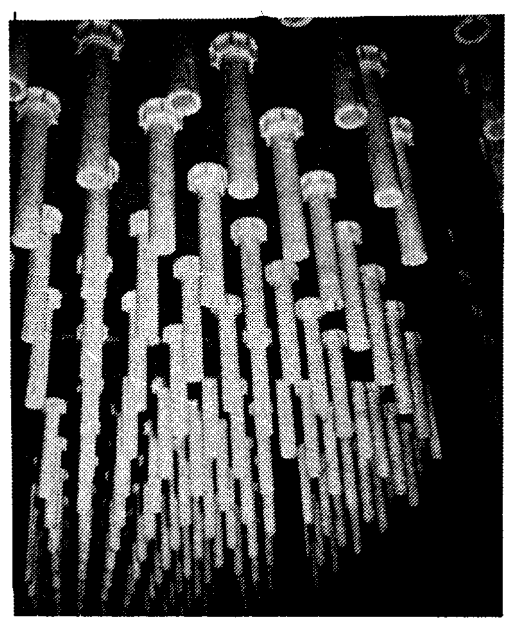


Fig. 5.8 False bottom-and strainer underdrains.

cient to keep the filterbeds clean on the long run, additional agitation is required. Fig. 5.9 shows the use of mechanical rakes, fig. 5.10 the application of air-wash and fig. 5.11 the use of surface wash.

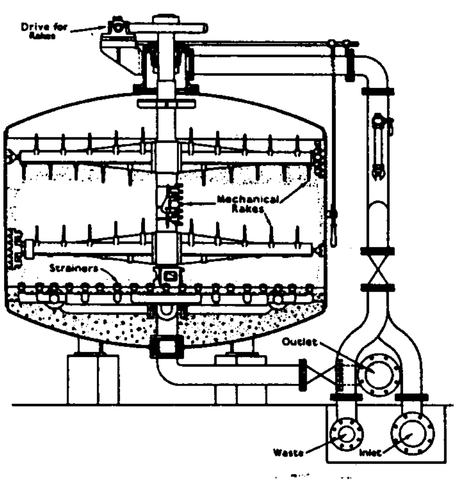



Fig. 5.9 Rake-cleaned pressure filters.
(Bell Brothers)

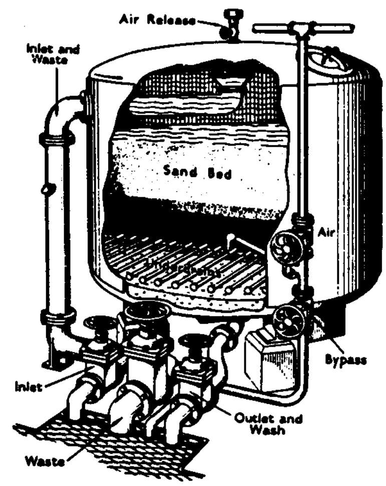



Fig. 5.10 Air cleaned pressure filters.
(Paterson Engineering Co)

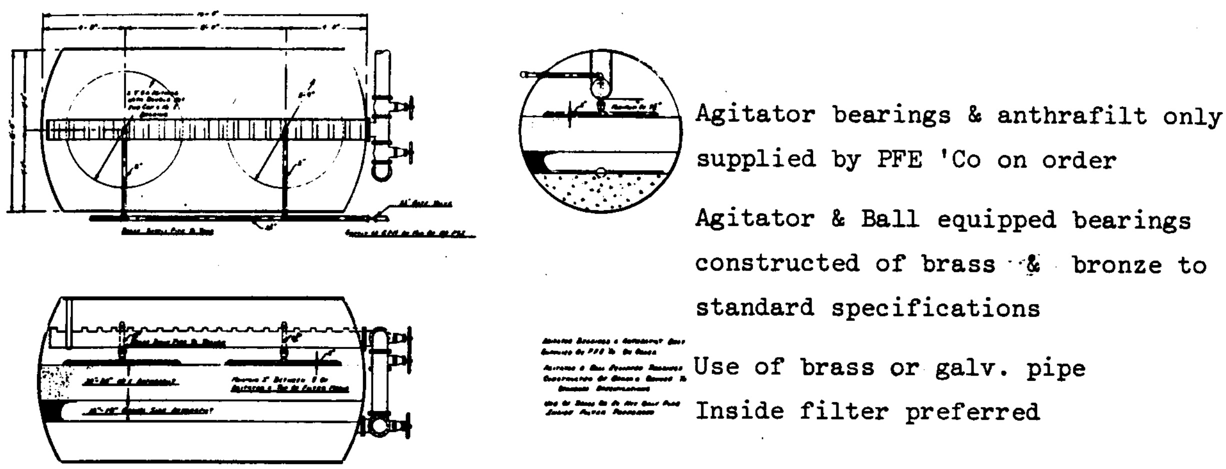



Fig. 5.11 Pressure filters with rotating surface wash.
(Palmer filter equipment Co)

**As regards filter control**, the pressure at which the raw water is supplied and **the filtered water** is discharged, is the same for all units, while these pressures rise far above the filterbed. Additional water level control is therefore unnecessary. Rate control can be obtained by providing each unit with a closed filter rate controller in influent or effluent line. Mostly, however, no control is provided, the filter operates at declining rate with only an (adjustable) orifice to limit the filtration rate through the clean filterbed directly after backwashing. In some installations a more or less constant rate of filtration is obtained by subdividing the total number of filtering units in groups. Each group is served by separate pumps, while all filters of the same group are backwashed one directly after the other, assuring the same amount of clogging and filter resistance.

In cold climates filters must be housed to prevent freezing in winter time (fig. 5.12 and 5.13). In hot climates filters can be built in open air,

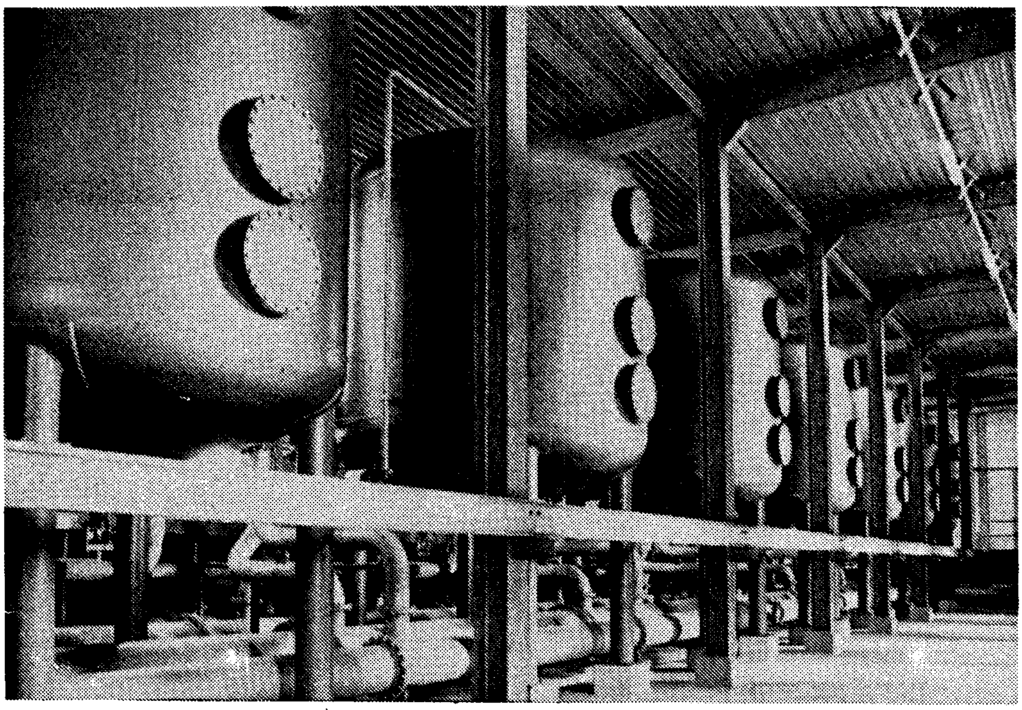


Fig. 5.12 Pumping station,Braakman.
(Public Water Supply of Zeeland)

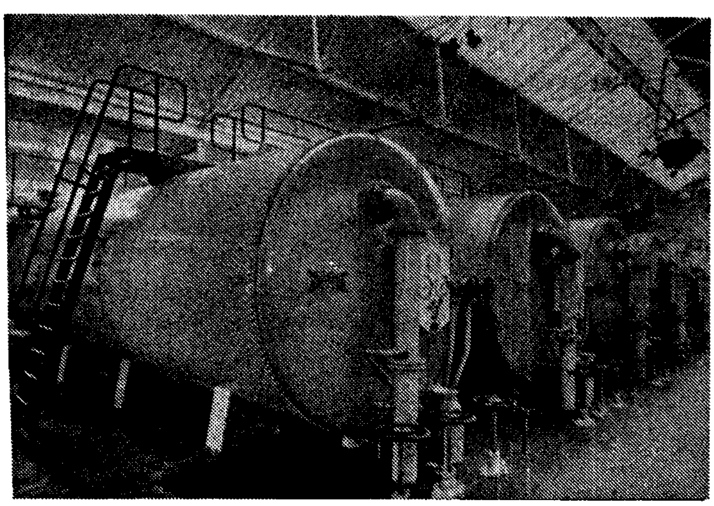


Fig. 5.13 Housing of horizontal pressure filters.
(Candy filter Co)

while in moderate and tropical climates alike the filters may partly be housed, to protect influent and effluent lines, valves, controlers, meters, etc, against adverse climatic influences (fig. 5.14 and 5.15).

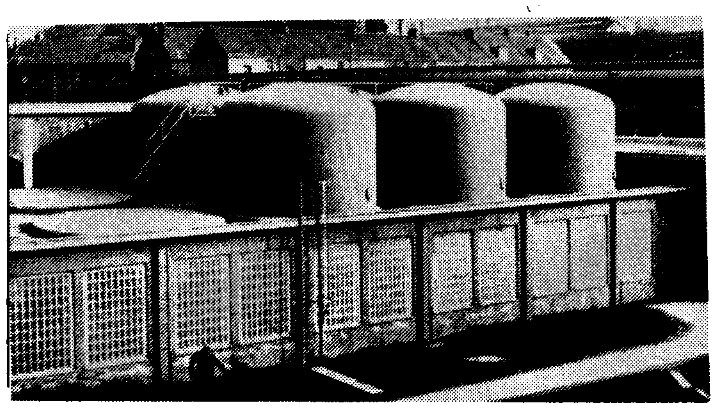


Fig. 5.14 Vertical pressure filters in the open air.
(Pintsch Bamag)

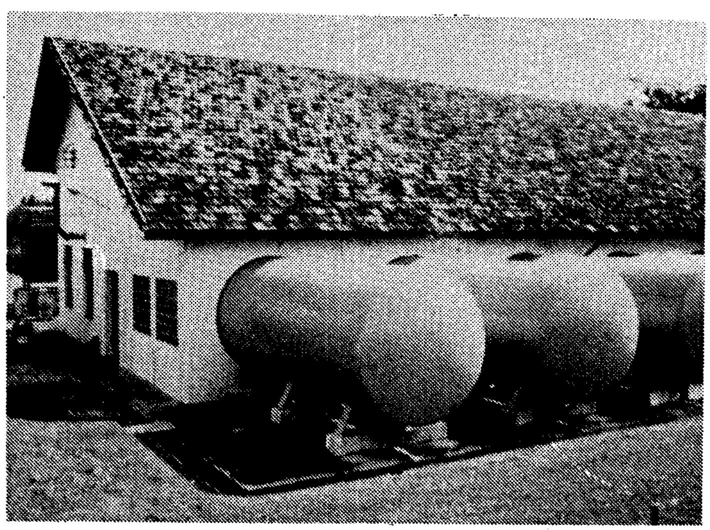


Fig. 5.15 Horizontal pressure filters in the open air.
(Alor Star, Malaysia)

# 6. UPFLOW FILTRATION

## 6.1 Coarse to fine filtration

As already mentioned at the end of section 2.6, back-washing of a rapid filterbed results in a hydraulic classification, bringing the fine parts of the grains to the top and the coarse grains to the bottom. With the filtration coefficient $\lambda_0$ being inverse proportional to the grainsize to a power between 1 and 3, the filtration efficiency will thus drop significantly in the direction of flow. This means that the upper part of the filterbed will retain the major portion of the impurities carried by the raw water, resulting in a rapid increase in filterresistance, while the remaining impurities are difficult to remove at greater depths, resulting in a rapid deterioration of effluent quality. The adverse effects of hydraulic classification may be taken from a comparison between the filter-runs of fig. 2.14 with uniform sand and of fig. 2.21 for a filtersand of the same hydraulic diameter but with a coefficient of uniformity of 1.24.

Hydraulic classification can be prevented by the use of completely uniform filtering materials. In practice, however, these are unobtainable while even better results might be expected from counter-current treatment, bringing the raw water first into contact with coarse grains and a low filtration efficiency and after that with fine grains and a large cleaning power. Without the occurrence of rapid clogging, the coarse grains retain a large part of the impurities contained in the water to be treated, leaving for the fine grained portion of the filterbed only little work to do. Notwithstanding the high filtration efficiency of this portion and the excellent effluent quality that can thus be obtained, the clogging rate will again be small, resulting in high values both for the length of filterrun $T_q$ with regard to effluent quality and for the length of filter-run $T_r$ with respect to filterresistance. This coarse to fine filtration can be realised in different ways. The simplest solution is the use of a number of filters in series of which fig. 6.1 shows an old concept by the French firm of Puech-Chabal and fig. 6.2 a modern version by the Swiss firm of Sulzer. To investigate the results that can be obtained in this way, the

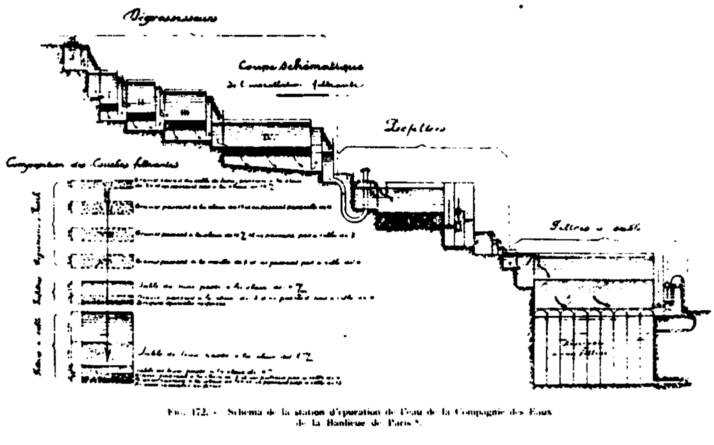


Fig. 6.1 Multi-stage rapid filtration as used by the Compagnie des Eaux de la Banlieuê de Paris.

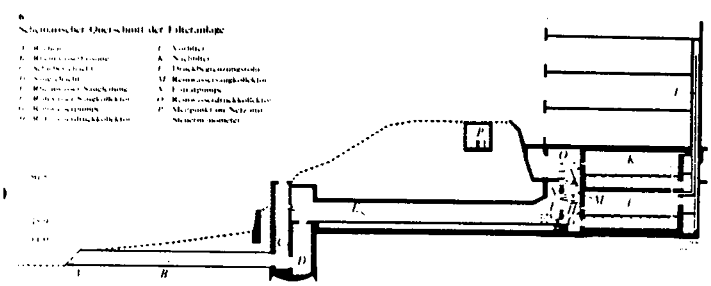


Fig. 6.2 Two-stage rapid filtration.

filterrun of fig. 2.14 with

$$L = 0.75 \text{ m}, \; d = 0.7 \text{ mm and } H = 1.5 \text{ m}$$

has been recalculated for two filters in series. Each filterbed has a thickness of 0.75/2 = 0.375 m, while the hydraulic diameters $d_1$ and $d_2$ are chosen such that for the length of filterrun $T_r = (1.62)10^5$ sec the same average effluent quality $c_a = 0.11$ g/m$^3$ is obtained. The head loss is now much smaller as indicated by the table below

| | | | | | |
|---|---|---|---|---|---|
| $d_1/d_2$ = | 0.7 / 0.7 | 0.8 / 0.66 | 0.9 / 0.63 | 1.0 / 0.61 | 1.1 / 0.59 mm |
| $\Sigma FH$ = | 1.50 | 0.95 | 0.75 | 0.95 | 1.15 m |

meaning in reverse that for the same head loss a higher filtration rate could be allowed or with finer grain sizes a better effluent quality could be obtained. Multi-stage rapid filtration has many advantages in terms of a better effluent quality and a greater length of filterrun. Without pre- or post- treatment it might even be able to convert a river-derived water with a high load of discrete particles directly into a clear drinking water. As serious drawback, however, must be mentioned that the cost of construction is rather high.

The building cost of multi-stage rapid filtration in the meanwhile can be reduced by incorporating the various filterbeds into one and the same filterbox as shown in fig 1.4. Not to disturb the composition of this multi-layered filterbed during backwashing, the coarse grains on top must now be made of a material with a mass density lower than that of sand ($\rho_f$ = 2600 kg/m$^3$) and the finer grains at the bottom with a higher mass density. Such materials are available, for instance anthracite with $\rho_f$ = 1400-1700 kg/m$^3$ for the upper layer and baryta ($BaSO_4$) with $\rho_f$ = 4900-5200 kg/m$^3$ for the lower layer, but their cost is a multiple of that of sand. Coarse to fine filtration with only sand as filtering material can be achieved by reversing the direction of the flow as shown in fig 1.3. With this upflow filtration, the hydraulic classification mentioned above is used to advantage. This may be gathered from fig 6.3 where the filtration

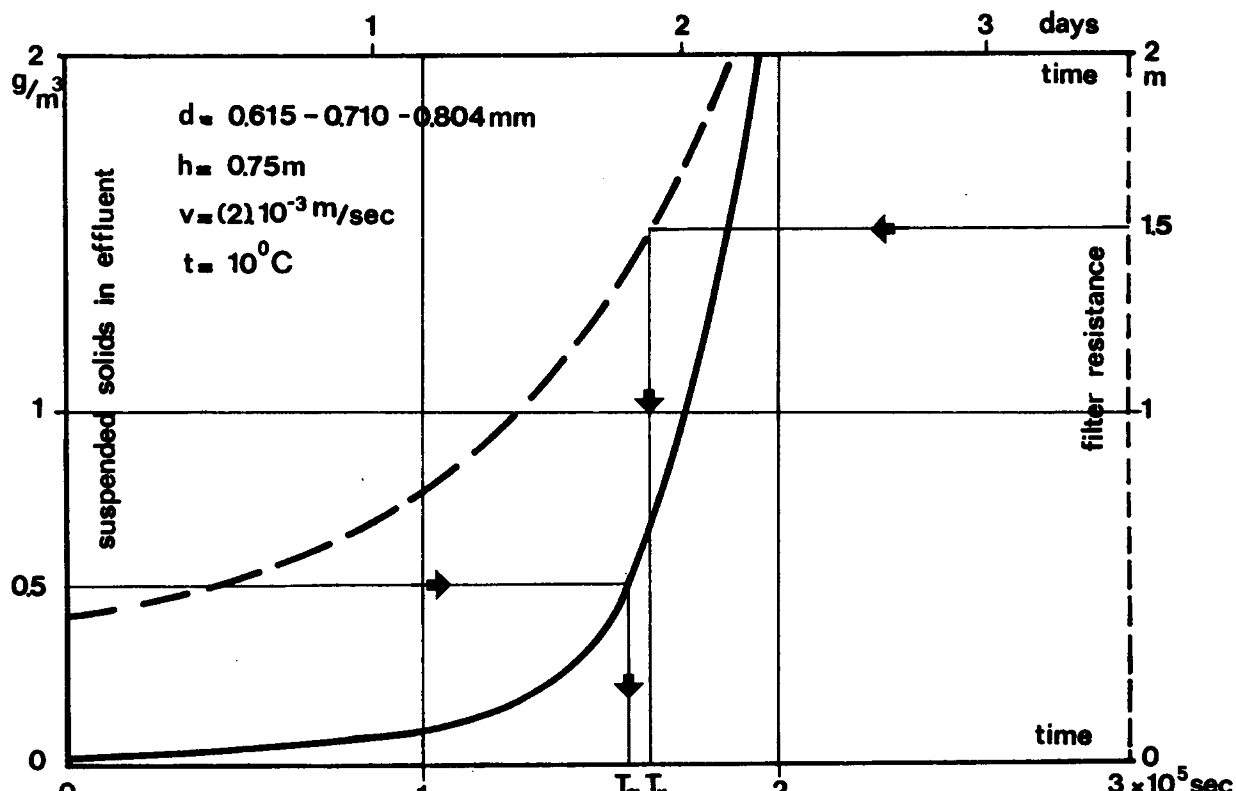



Fig. 6.3 Filtration results of fig. 2.21 recalculated for the reverse direction of flow.

results of fig 2.21, using the material of fig 2.20, have been recalculated for the reverse direction of flow. The lengths of filterrun are now 40% greater and only slightly smaller than those shown in fig 2.14 for the non-existing completely uniform filtering material. With upflow filtration a less uniform filtering material gives even slightly better results in terms of effluent quality, as may be gathered from a comparison between fig 6.3 and 6.4 where the hydraulic diameters of the mixed bed are the same, equal to 0.7 mm, but the uniformity coefficient is increased from 1.24 to 1.99.

Just as ordinary rapid filters, multi-stage or multi-layered filters will pass the majority of colloidal matter present in the raw water. As will be shown in chapter 7, this material could be retained when by the addition of coagulants to the incoming raw water it is brought to combine into larger flocs. With normal rapid filtration this procedure will result in such a rapid increase in filter resistance as to make it unpracticable. With coarse to fine filtration and a deep penetration of the impurities from the raw water into the filterbed, however, the silt storage capacity is so much higher that in many cases this floculation supported filtration can be used without adverse effects.

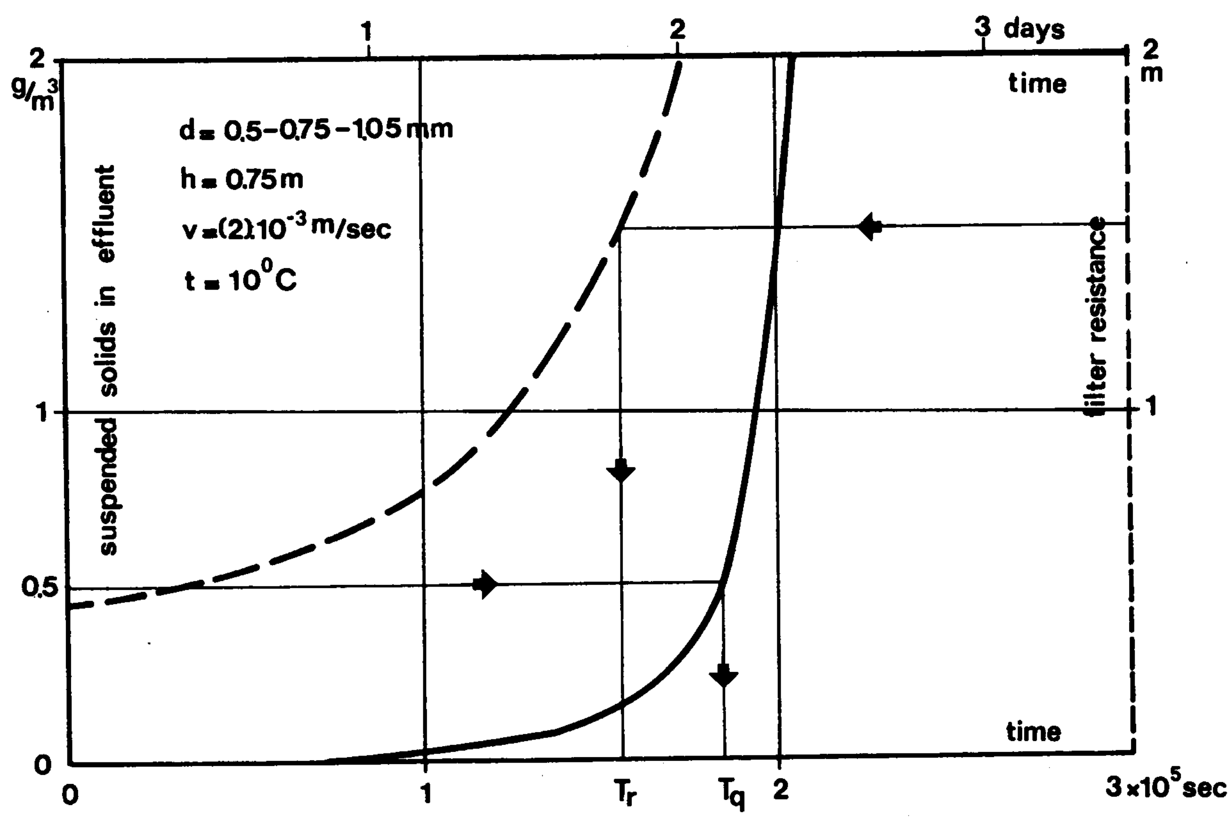



Fig. 6.4 Filtration results of fig. 6.3 recalculated for a less-uniform filtering material of the same hydraulic diameter.

## 6.2 Hydraulics of upflow filtration

With full lines fig 6.5 shows the distribution of the water pressure in the bed of an upflow filter. Due to the use of non-uniform filtering material together with hydraulic classification, the line for t = o is not straight but convex, while by a clogging of the filterbed from the bottom upward the line for t = t will be S-shaped. In the same figure the soil pressure is indicated by a dotted line. This soil pressure equals the combined weight per unit area of the filtering material, the pore water and the supernatant water above. In formula at a depth y below the top of the filterbed

$$\sigma_s = \rho_f g\ (1-p)y + \rho_w\ g\ p\ y + \rho_w\ g\ h$$

with p as pore space of the filterbed, $\rho_f$ and $\rho_w$ as mass densities of filtering material and water respectively and the other factors as indicated in fig 6.5. According to soil mechanics the grain pressure equals the difference between the soil pressure and the water pressure

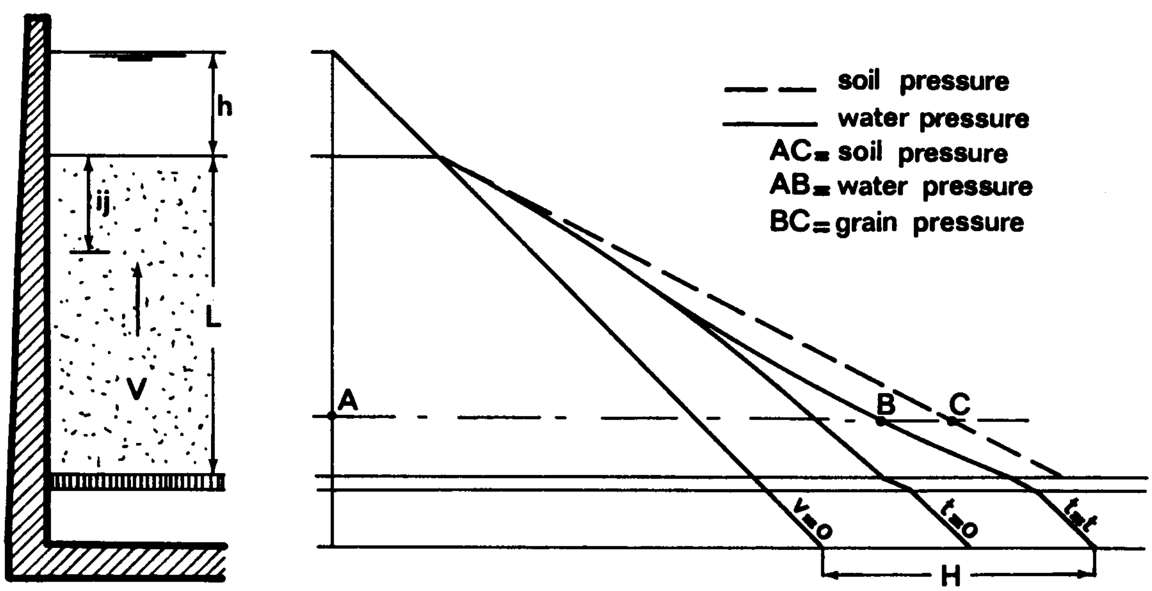



Fig. 6.5 Pressure distribution in the bed of an upflow filter.

$$\sigma_g = \sigma_s - \sigma_w$$

As shown in fig 6.5, the grain pressure at t = o increases with the depth below the top of the filterbed. During filtration the soil pressure remains constant, but the water pressure increases by clogging, fastest at the bottom of the bed. At t = t the grain pressure at the bottom of the filterbed thus becomes

$$\sigma_g = \{ \rho_f g (1-p) L + \rho_w g p L + \rho_w g h \} - \{ \rho_w g (L + h) + \rho_w g H \}$$

simplified

$$\sigma_g = (\rho_f - \rho_w) g (1-p) L - \rho_w g H$$

with H as filter resistance. When this resistance reaches such a magnitude as to make $\sigma_g$ equal to zero

$$H_m = \frac{\rho_f - \rho_w}{\rho_w} (1 - p) L$$

the grains do not longer rest upon one another. The whole filterbed will now be lifted with local breakthroughs of raw water as result. A sudden and serious deterioration of effluent quality will occur and immediately the filter must be taken out of service for backwashing. With sand as filtering material and

$$\rho_f = 2600 \text{ kg/m}^3, \; p = 40\%$$

this danger of uplifting limits the maximal allowable **head loss** to

$$H_m = \frac{2600 - 1000}{1000} 0.6 \text{ L} = 0.96 \text{ L}$$

When this head loss is too small with regard to the desired length of filterrun $T_r$, larger bed thickness could be applied, for instance. For a greater length of filterrun, large bed thicknesses are therfore re- 1.5 to 2.5 m, appreciable increasing the building costs. Better results can be obtained with heavier filtering materials. With magnetite and

$$\rho_f = 4900 \text{ kg/m}, \; P = 45\%$$

a head loss equal to 2.15 times the filterbed thickness is allowed, but this material is rather expensive, again increasing the cost of construction.

Real conditions in the meanwhile are even more complicated. Fig 6.6 shows negative grain pressures at the top and at the bottom of the filter-

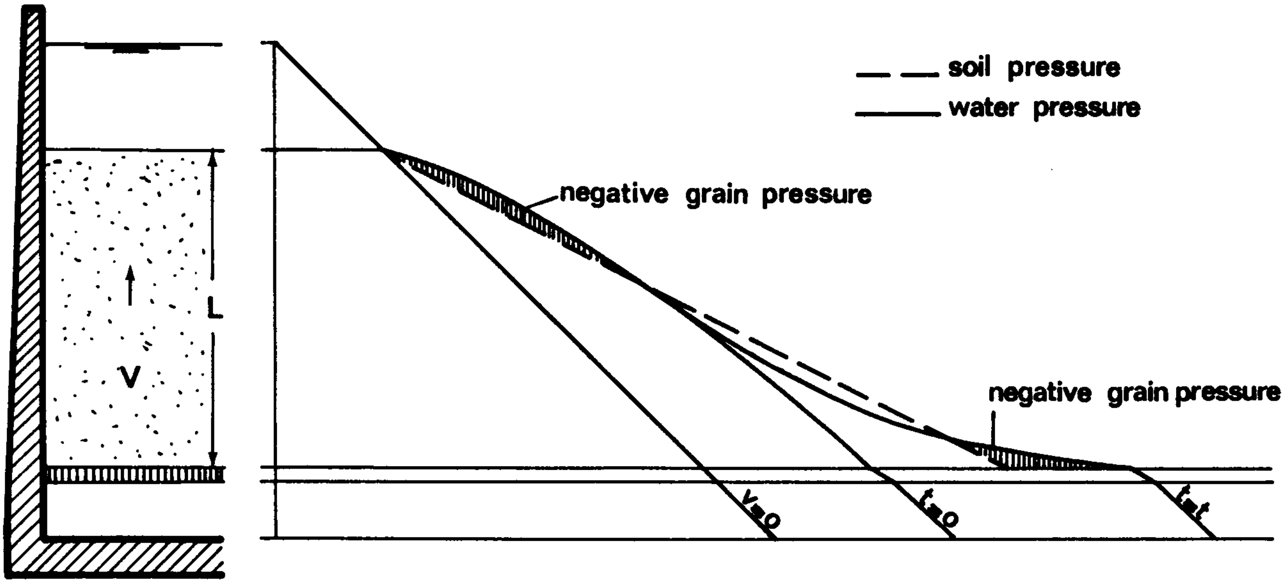



Fig. 6.6 Occurrence of negative grain pressure.

bed, the former occuring from the very beginning and the latter at the end of the filterrun. Granular non-cohesive material as filtersand in the meanwhile is unable to take up tensile stresses and negative grain pressures are therefore impossible. When they tend to occur at the top of the bed, erosion would bring the filtering material in suspension, destroying at the same time the filtering capacity of this part of the bed. To prevent an expansion of the filterbed at the top, the slope of the piezometric surface must be smaller than the value of 0.96 mentioned above for sand as filtering material.
According to Carman-Kozeny this slope equals

$$I_o = \frac{180\nu\,(1 - p_o)^2}{g \quad p_o^{\,3}} \quad \frac{v}{d_o^{\,2}}$$

giving with $p_o$ = 0.4 and $\nu = (1.792)10^{-6}$ $m^2$/sec at $0^\circ$ C
as requirment

$d_o > \sqrt{\frac{v}{5200}}$ when v is expressed in m/sec and $d_o$ in m.

To produce a water fit as a public supply, the lower limit of the grainsize distribution may not be larger than about 0.6 mm. According to the formula above, the filtration rate in this case must be limited to $(1.8)10^{-3}$ m/sec, a rather low value in modern filtration practice. High rate filtration and v equal to 4 or 6 x $10^{-3}$ m/sec is now only possible when the lower grainsize limit surpasses a value of 0.9 to 1.1 mm. This means a coarse material, which excluding the coagulation supported filtration of the next chapter is unfit in the final purification stage of a drinking water purification plant. There it may be used as preliminary treatment, to be followed by normal downflow filtration, but the widest application may be found with industrial supplies where on one hand the (occasionnal) high turbidity of the raw water asks for deep bed filtration with a large silt storage capacity, while on the other hand the high purity of drinking water is not required.

Negative pressures seem to occur at the bottom of the filterbed when at the end of the filterrun the filter resistance surpasses the weight of the filterbed below water. In reality, however, the soil pressure is now increased by friction between the stationary walls of the filterbox and the upward moving filterbed. For all practical purposes this friction may be neglected. Even with filters of small width it is not able to augment the maximum allowable filterresistance by more than a few centimeters of water column. A sizable increase in soil pressure and in the maximum allowable filter resistance may be obtained artificially, by installing a grid of steel strips in the top of the filterbed, as shown in fig 6.7, and anchoring this grid to the walls of the filterbox. When at the bottom of the filterbed negative grain pressures tend to develop and the bed

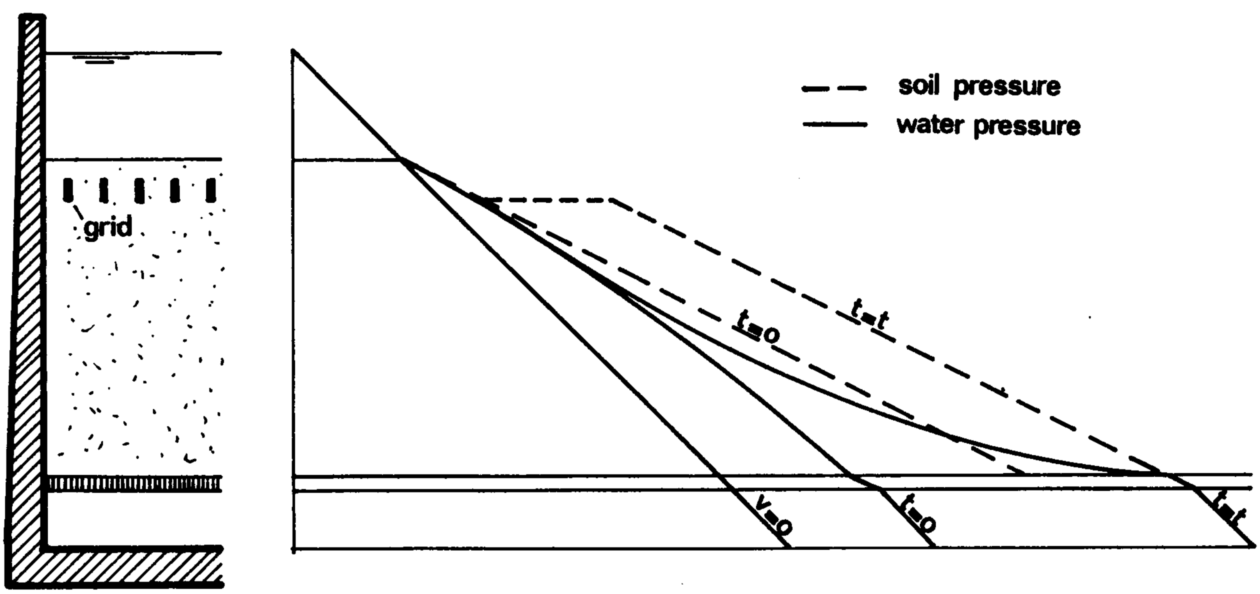



Fig. 6.7 Increase in soil pressure by the presence of a grid.

moves upward, bridges of sand grains will be formed between the steel strips, preventing a further uplifting (fig 6.8). With strips of say 15 by 80 mm, at 150 mm intervals in a filterbox 2 m wide, it is thus possible to augment the maximum allowable filter resistance by 2 m water column, a sizable increase indeed. It should not be forgotten, however, that to develop the additional soil pressure an upward movement of the filterbed is necessary, decreasing the filtration efficiency of the lower part of the filterbed.

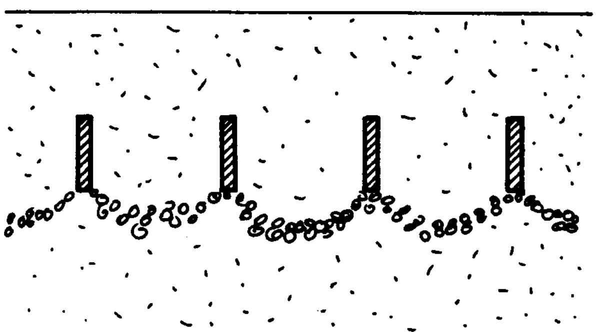


Fig. 6.8 Bridges of sandgrains preventing an upward movement of the filterbed.

## 6.3 Construction and operation

From the preceding section it will be clear, that upflow filtration has two important characteristics

a. by the sequence of coarse to fine filtering material it provides true deep bed filtration, enabling the storage of large amounts of impurities removed from the raw water;

b. fine filtergrains as desired for polishing purposes cannot be used.

Together these characteristics means that the main application of upflow filtration must be sought in the treatment of water of a high suspended load, either naturally when the water is taken from a turbid river or artificially when iron or aluminiumsalts are added as coagulants (compare chapter 7). In public water supplies, upflow filtration can only be used as a preliminary treatment, but for industrial supplies it may be the sole treatment to which the water is subjected.

As shown in fig.6.9, the water to be treated enters the filter at the lower end and passes the filterbottom before it reaches the filterbed. To prevent a clogging of this filterbottom by the impurities carried by the raw water, large openings are required and only a few of the filterbottom constructions described in section 4.5 are now applicable. The most

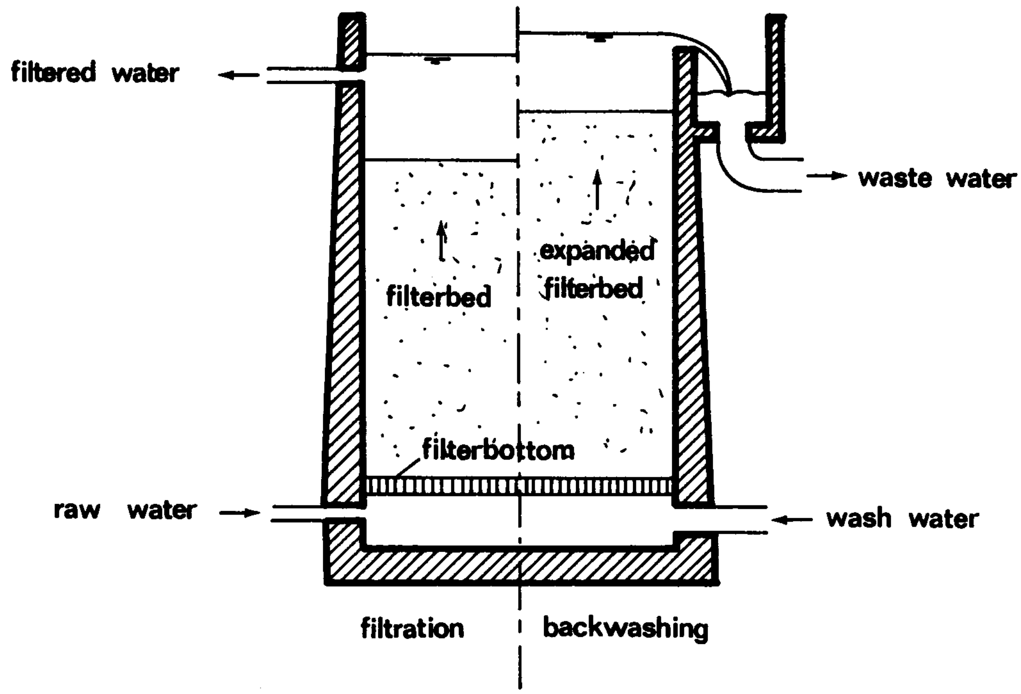



Fig. 6.9 Operation of an upflow filter.

important ones are shown in fig 6.10, to the left the perforated lateral system and to the right the false bottom and strainer underdrainage construction. The openings in the laterals must be chosen as large as possible, preferably 10 mm or more, asking at the same time for a smaller number, down to 30 or 40 per $m^2$. The greatest danger of clogging, however, occurs in the openings between the grains of the supporting gravel layers. To reduce this danger as much as possible, fine grained gravel layers must be avoided, which can best be achieved by augmenting the ratio between the upper grain size limit of the upper gravel layer and the lower grain size limit of the lower sand layer above to a factor of 5 or even 6. In fig 6.10 two gravel layers are thus enough, the upper one of a much greater thickness than normally applied to help in an even distribution of the washwater emerging from a small number of openings. With regard to the same danger of clogging, the strainers may not be equiped with fine slits. The best solution is a piece of pipe with an internal diameter of say 10 mm, covered at the top by a cup to prevent blocking by the grains of the gravel above. Their number is again small, for instance 36 per $m^2$, requiring the same gravel layers as with perforated laterals. This means a much greater depth of the filterbox, greatly increasing the cost of construction.

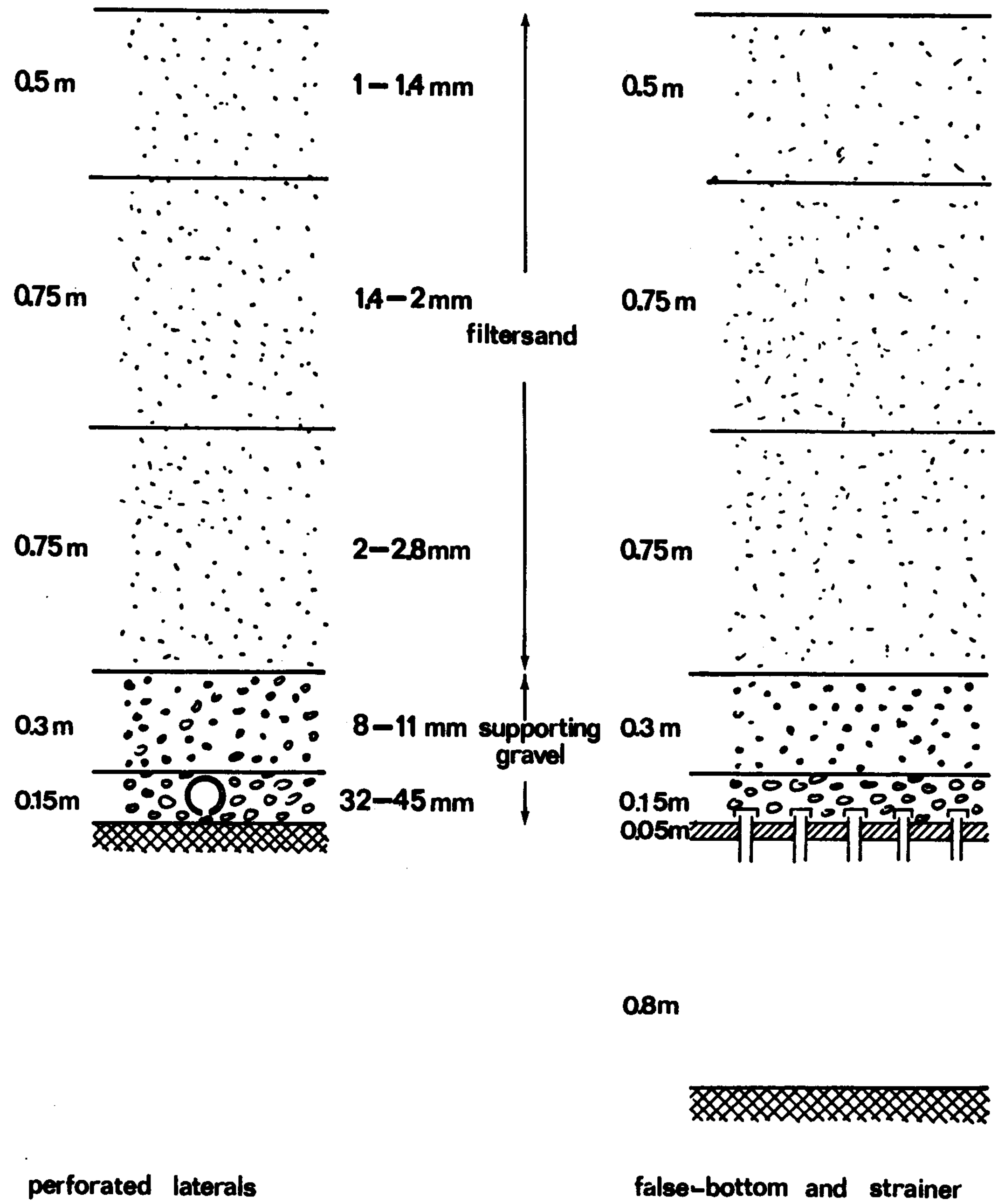


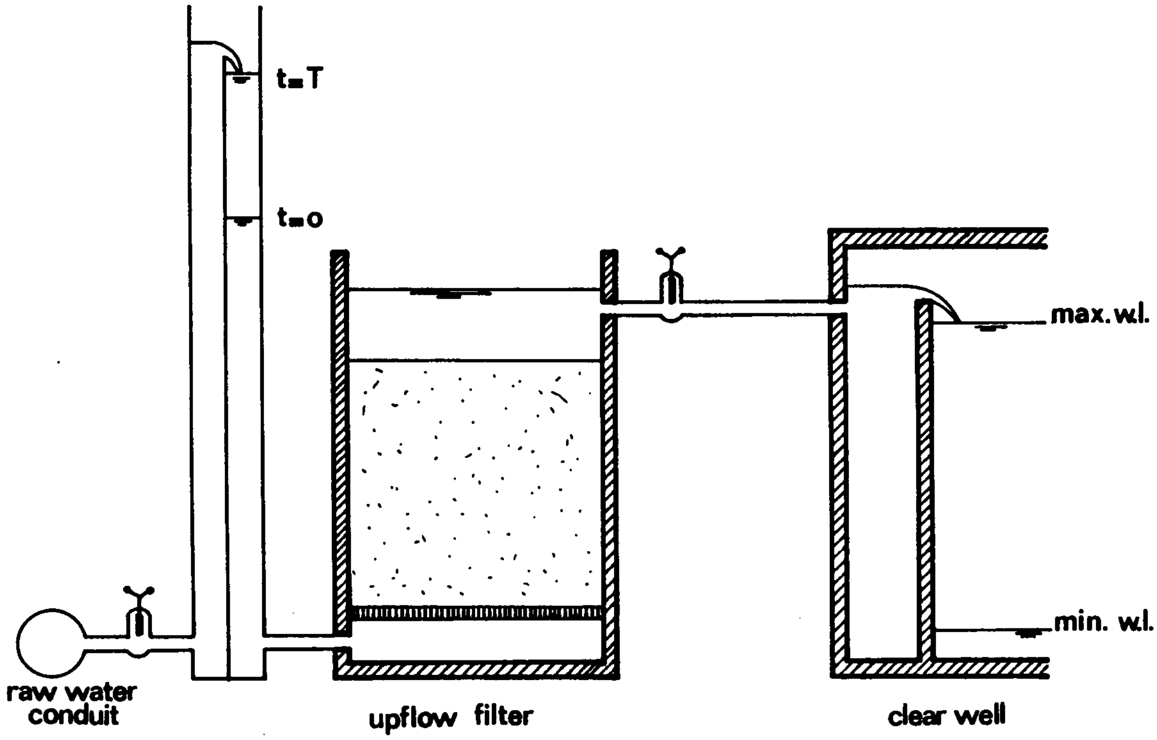

Fig. 6.10 Filterbottom construction for upflow filtration.

On the other hand the space below the false bottom allows an easier access and the possibility of cleaning a strainer blocked by grosser suspended solids or by animal and vegetable life. With the perforated lateral system such a blocking must be prevented by passing the water first through a traveling screen or strainer with openings not larger than 2 mm and preferably less. With coagulation supported filtration, this perforated lateral system has the added adventage that the time the raw water needs to reach the filterbed is small, reducing differences in floc size, density and electrical charge which otherwise might impair the effects of filtration. With the false bottom and strainer type of underdrains on the other hand the average detention time is much larger, allowing greater variations which moreover might be augmented by the nearly unavoidable presence of dead spaces.

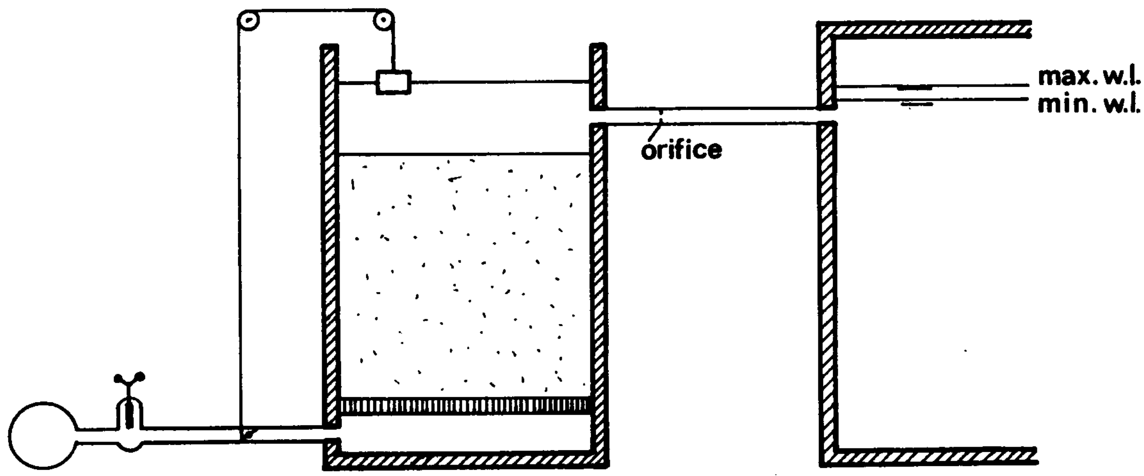
To aid in obtaining a stratified filterbed, it is usually built op of layers with upward decreasing grainsize, for instance as shown in fig 6.10. It goes without saying that the choice of the various grain sizes and the bed thicknesses requires careful thought and that for larger installations this choice should be based on extensive laboratory tests. The depth of supernatant water on top of the filterbed is governed solely by the sandbed expansion during backwashing. To obtain some expansion of the coarser grains in the lower portion of the filterbed, the expansion of the fine grains at the top will be quite large, asking for a greater depth of water, for instance 0.8 m to prevent an undue loss of filtering material. As regards the construction of the filterbox finally, a small width is indicated when grids of steel strips are used to keep the filterbed down.

With normal downflow filters and $T_q > T_r$, a delay in backwashing the filter reduces the capacity of the plant, but it does not affect effluent quality. With upflow filters on the other hand, such a delay might result in raw water breaking through the filterbed, materially reducing effluent quality. To prevent such mishaps to occur, close supervision is required, preferably automated, shutting down the filter when the head loss reaches a predetermined value well below the maximum possible one or when effluent turbidity surpasses a preset level. With respect to filter control, the small depth of supernatant water makes a constant filtered water level very attractive. This may be effected by upstream or downstream control, as shown in fig 6.11. The construction at the top has no moving parts whatsoever, but the filtration rate depends on raw water supply. When filtered water demand must be the governing factor, the construction at the bottom of fig 6.10 should be chosen or an additional control should be installed as shown in fig 4.18.

Backwashing the filter may be done with water alone or an auxiliary air scour may be used in advance. When a grid is present, the bridges of sand grains must be destroyed before the water wash starts. This can best be accomplished by a preceding air wash, first without and later on with a limited quantity of water. Expansion of the coarsest grains at the very bottom of the filterbed will never occur, neither of the supporting gravel




upstream rate control, downstream control of filtered water level




upstream control of filtered water level, downstream rate control

Fig. 6.11 Filter control.

layers. To keep these as clean as possible backwashing must be done for extended periods of time, 10 minutes for instance, appreciably increasing washwater consumption. With industrial supplies raw water may be used for backwashing, but for drinking water supplies filtered water should be used to prevent a contamination of the effluent conduit.

# 7. DUAL AND MULTI-MEDIA FILTRATION

## 7.1. Introduction

The advantage of coarse to fine filtration as described in section 6.1., can also be obtained by composing the filterbed of various layers with in downward direction a smaller grainsize and a higher mass density. Both factors must be interrelated in such a way that in downward direction the settling velocity of the filtergrains increases so that during back-washing no overturning of and neither mixing between the different layers occurs. With regard to its durability and low cost, sand is always one of the filtering materials employed, next to which can be used

a. a layer of coarser material and lower mass density on top of the sandbed;
b. a layer of finer material and higher mass density below the bed of sand.

For multi-media filtration the following materials have been used

| | |
|---|---|
| polystyrene | $\rho$ = 1040 kg/m$^3$ |
| pumice | 1100-1200 |
| PVC | 1230-1300 |
| crushed coconut shell | 1350-1450 |
| expanded slate | 1500 |
| scoria (vulcanic cinder) | 1400-2400 |
| hydro-anthracite | 1650-2150 |
| quartz sand | 2650 |
| garnet | 3830 |
| corundum | 3880-3950 |
| barite | 4500 |
| ilminite | 4680-4760 |
| magnetite | 4900-5200 |

Garnet and corundum have even a greater hardness than sand, but are extremely expensive. All the other materials are fairly soft, resulting in an appreciable amount of attrition during back-washing. Every one to a few years these materials must be replenished, increasing the cost of operation, especially when they are heavy and situated below the sandbed. This is the reason that mostly double-bed filtration is used, with natural or synthetic anthracite at the top and sand at the bottom of the filterbed.

Anthracite is 3 to 4 times as expensive as sand and is lost at a rate of 5 to 10% per year. Hydro-anthracite is more expensive, 5 to 8 times the price of sand, but the wear is less, 3 to 5 % per year.

Anthracite as filtering material not only differs from sand by its mass density, but also by its shape. Sand from rivers has a rounded shape, but due to its manufacturing process of crushing and sieving, anthracite has a more angular one, resulting in a higher porosity and a lower value of the shape factor $\phi$. Depending on back-washing procedures, sand in a filterbed at rest has a porosity of about 40% and anthracite of 45 to 60% and sometimes even more. The shape factor $\phi$ is the ratio between the effective grain size d to be used in the mathematical theory of filtration and the clear opening s of square woven wire sieves

$$d = \phi\, s$$

According to the tables on page 69 and 76

| | |
|---|---|
| filtration | |
| sand, s = 0.5-1 mm | $\phi$ = 0.9 |
| anthracite, 1-1.5 mm | 0.7 |
| hydro-anthracite, 1-1.5 mm | 0.6 |
| back-washing | |
| sand, s = 0.5-1 mm | 1.0 |
| anthracite, 1-1.5 mm | 0.85 |
| hydro-anthracite, 1-1.5 mm | 0.7 |

## 7.2. Grain size ratio's

In section 3.2, the back-wash rate v necessary to maintain a porosity $p_e$ of the expanded filterbed has been calculated at

$$v^{1.2} = \frac{g}{130\,\nu^{0.8}} \quad \frac{\rho_f - \rho_w}{\rho_w} \quad \frac{p_e^{3}}{(1-p_e)^{0.8}} \quad d^{1.8}$$

in which $p_e$ is determined by the desired amount of filterbed expansion E.

$$E = \frac{L_e - L}{L} = \frac{p_e - p}{1 - p_e} \quad \text{or} \quad p_e = \frac{p + E}{1 + E}$$

with p as the porosity of the filterbed at rest. In a double filterbed, both layers are back-washed at the same rate. For both layers the values of g, $\nu$ and $\rho_w$ are also the same, giving with "a" as index for anthracite and "s" as index for sand

$$(\rho_a - \rho_w) \frac{p_{ea}^3}{(1 - p_{ea})^{0.8}} d_a^{1.8} = (\rho_s - \rho_w) \frac{p_{es}^3}{(1 - p_{es})^{0.8}} d_s^{1.8} \qquad \text{or}$$

$$\left(\frac{d_a}{d_s}\right)^{1.8} = \frac{\rho_s - \rho_w}{\rho_a - \rho_w} \left(\frac{1 - p_{ea}}{1 - p_{es}}\right)^{0.8} \left(\frac{p_{es}}{p_{ea}}\right)^3$$

With the assumptions

$\rho_s = 2650$, $\rho_a = 1500$, $\rho_w = 1000$ kg/m$^3$

and for the filterbed at rest

$p_s = 0.38$, $p_a = 0.50$

this relation is shown graphically in fig. 7.1. With regard to the amounts of filterbed expansion shown there, various assumptions can be made.

a. the sand being fine must be back-washed at a large amount of filterbed expansion, while for the coarse anthracite a much lower value suffices, for instance

$E_s$=30%, $E_a$=10% and $d_a/d_s$=1.77

b. both layers must be back-washed at the same rate of expansion, e.g.

$E_s = E_a$=20% $\qquad$ $d_a/d_s$=1.29

c. when for appropriate filtration results a still lower grain size ratio is required, the expansion of the anthracite bed must be larger than for the sand bed, e.g.

$E_s$=15%, $E_a$= 20%     $d_a/d_s$=1.17

When considering the amounts of filterbed expansion mentioned above, it should be realized however, that for each bed the grain sizes vary between certain limits. When these limits are a factor $\sqrt{2}$ apart, the coefficient of uniformity has a low value of about 1.20, but due to hydraulic classification the amounts of filterbed expansion still vary greatly between top and bottom.

anthracite = porosity at rest 50%

| | | | | | | |
|---|---|---|---|---|---|---|
| $E_{average}$ = | 5 | 10 | 15 | 20 | 30 | % |
| $E_{min}$ = | 0 | 1 | 5 | 9 | 17 | |
| $E_{max}$ = | 15 | 22 | 28 | 34 | 47 | % |

sand, porosity at rest 38%

| | | | | | | |
|---|---|---|---|---|---|---|
| $E_{average}$ = | 5 | 10 | 15 | 20 | 30 | % |
| $E_{min}$ = | 1 | 3 | 7 | 12 | 20 | % |
| $E_{max}$ = | 12 | 18 | 24 | 30 | 43 | % |

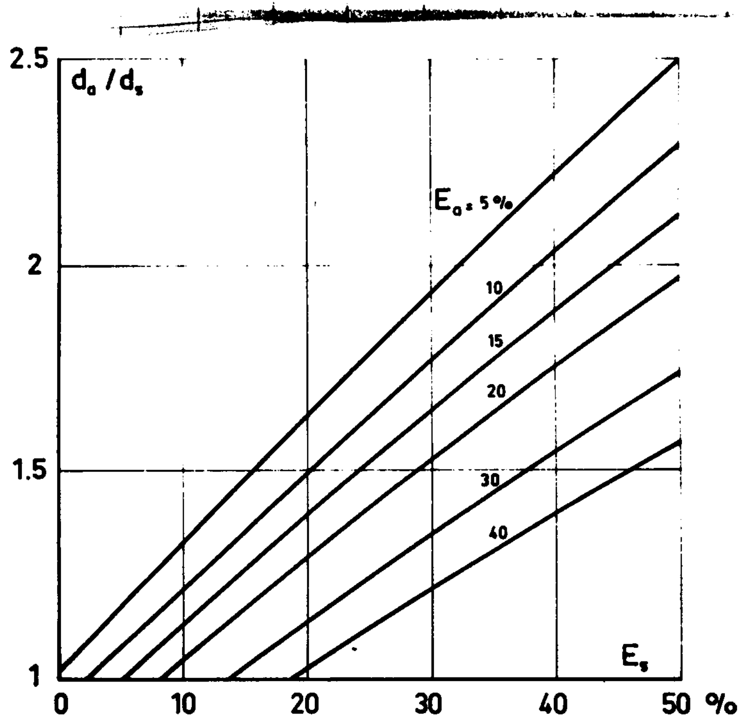



Fig. 7.1 Grain size ratio's for the conditions mentioned in the text

Not to hinder the removal of impurities more than strictly necessary, the minimum sandbed expansion during back-washing should be 10%, according to the table above conforming with an average one of 18%. Not to loose too much anthracite during back-washing the maximum expansion should be limited to 30%, conforming with an average one of 17%. For both average amounts of expansion, fig. 7.1 gives a grain size ratio larger than 1.3.
The highest allowable grain size ratio follows from the consideration that the maximum amount of sandbed expansion be less than 40%, equal to an average one of 28% and the minimum amount of anthracitebed expansion larger than 5%, equal to an average one of 15%. For both average amounts of expansion fig. 7.1 gives a grain size ratio less than 1.6.

When for appropriate filtration results, a grain size ratio smaller than 1.3 has to be used, a lower amount of sand bed expansion must be applied, increasing wash-water consumption, together with a higher amount of expansion for the anthracite bed, augmenting loss of this material.

For a grain size ratio larger than 1.6 on the other hand, the opposite measures should be taken, a higher amount of sand bed expansion and a smaller expansion of the anthracite bed. When the latter expansion drops below 9%, however, the minimum expansion at the bottom of the anthracite bed is zero. This means that the back-wash rate applied is less than the settling velocity of the largest anthracite grains. As a consequence these grains will sink into the expanded sand bed, resulting in a mixing of both materials, reducing the pore space at the interface. According to practical experience, however, this mixing is negligeable up to a grain size ratio of 1.8 and small for grain size ratio's between 2 and 2.4. Some water engineers even prefer a little amount of mixing as this prevents a sharp interface between anthracite and sand and cake filtration in the upper part of the sand bed.

When ordering filtering materials, the sieve sizes s must be specified for which the differing values of the shape factor $\phi$ have to be taken into account. With a grain size ratio of for instance 1.4

$$d_a = 1.4\ d_s \quad \phi_a\ s_a = 1.4\ \phi_s\ s_s$$

and with the values of $\phi$ mentioned for back-washing at the end of last section

$$(0.85)s_a = (1.4)(1.0)\ s_s \text{ or } s_a = 1.65\ s_s$$

for instance sand 0.6-0.85 mm and anthracite 1-1.4 mm. The shape factors for filtration are different, giving

$$d_a = 0.7\sqrt{(1)(1.4)} = 0.83 \text{ mm}$$

$$d_a = 0.9\sqrt{(0.6)(0.85)} = 0.64 \text{ mm}$$

and a grain size ratio of 1.29. The recommended ratio's of 1.3 and 1.6 mentioned above for back-washing must similarly be reduced to 1.2 and 1.5 for application in the mathematical theory of filtration to be dealt with in next section.

## 7.3. Double-bed filtration

In section 2.3 and 2.4 Lerk's mathematical theory of filtration has been used to calculate effluent quality (page 45, top) and filter resistance (page 48, bottom). The same formulas may be applied to determine the results of anthracite-sand filtration, assuming the effluent $c_e$ of the anthracitebed to be the influent $c_o$ for the sand bed and the total filter resistance H to be the sum of the resistance of the individual beds. For both layers the filtration coefficients have different values, depending on grain size $d_o$ and original porosity $p_o$. According to page 44 at the bottom

$$\lambda_o = \frac{(6.87)10^{-12}}{v\ d_o^{\ 3}} \qquad \alpha = (2.05)10^{-13}\ \frac{c_o}{p_o\ d_o^{\ 3}}$$

and for sand $p_o$=0.38, for anthracite $p_o$=0.50. With a programmable pocket calculator the results of double-bed filtration are now easy to determine.

The composition of single bed filters to reduce suspended matter content from 15 to 0.5 g/m$^3$ at 10$^o$C is shown in fig. 2.12, from which for instance may be chosen

| | |
|---|---|
| filterbed thickness | $L = 1.1$ m |
| size of sand grains | $d_s = 0.8$ mm |
| filtration rate | $v = 3$ mm/s |

The increase in filter resistance and of effluent suspended matter content with time is shown graphically in fig. 7.2 from which follows

$T_q = (0.98)10^5$ s and H= 1.62 m

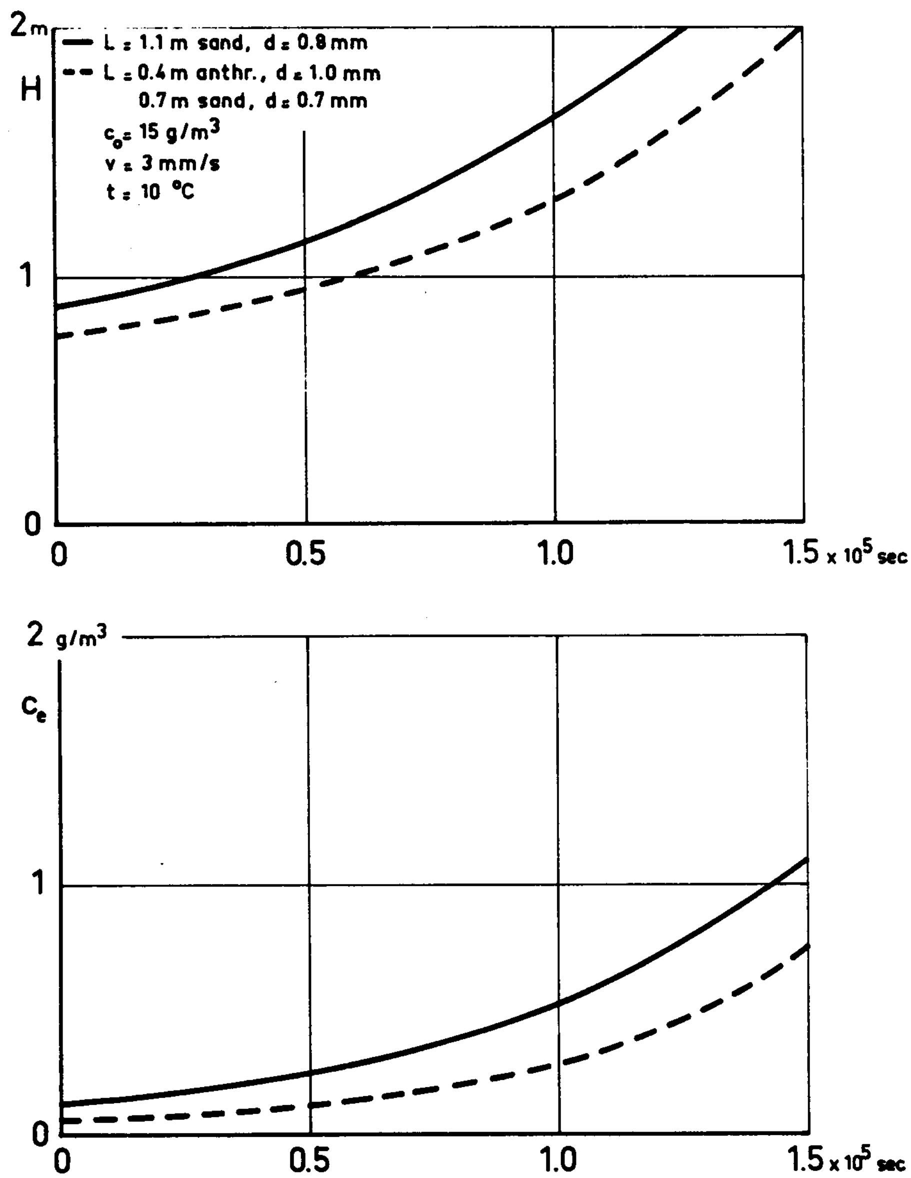



Fig. 7.2 Comparing single and double-bed filtration

To obtain better results with double bed filters of the same total thickness, the grain size of the sandbed must be decreased, say to 0.7 mm. According to the calculations of the preceding section, this gives as recommended sizes for the anthracite bed

$$d_a = (1.2 \text{ to } 1.5)(0.7) = 0.84 \text{ to } 1.05 \text{ mm}$$

Choosing a grain size of 1.0 mm in a bed thickness of 0.4 m, gives the results as shown in fig. 7.2 with a dotted line. The length of filter run $T_q = (1.30)10^5$s is now 30% larger, while the head loss at this moment, H=1.70m is only a little higher than before.

A better proposition in the meanwhile would be to maintain the length of filterrun $T_q$ at $10^5$s, allowing higher rates of filtration and a corresponding reduction in the cost of construction. For different thicknesses and grain sizes of the antracite bed, a total bed thickness of 1.1 m and a size of the sand grains equal to 0.7 mm, the various possibilities are shown in fig. 7.3. For the case under consideration, $L_a$=0.4m, $d_a$=1.0mm, the allowable filtration rate rises to 3.33 mm/s, while the head loss drops a little to 1.60 m. Even better results could be obtained by lowering the

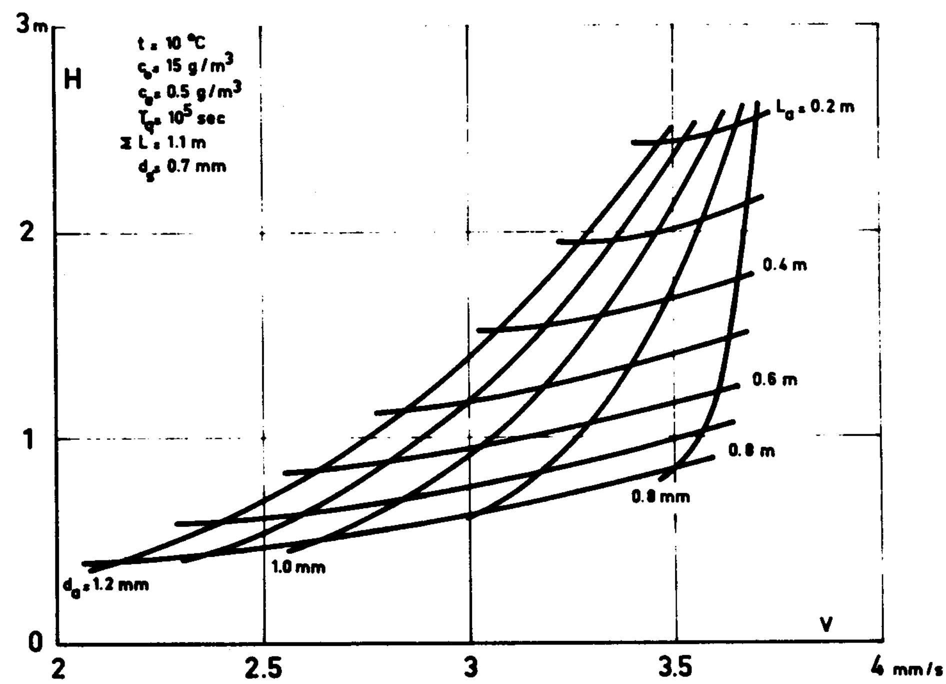



Fig. 7.3 Possibilities for double-bed filtration

size of the anthracite grains to the minimum value of 0.84 mm mentioned above. With a thickness of the anthracite bed of 0.35 m, the filtration rate rises to 3.61 mm/s and the head loss to 1.89 m. According to the pressure diagram of fig. 7.4, the minimum depth of supernatant water necessary to present negative heads and airbinding from occurring, now equals this head loss minus the filterbed thickness or D = 1.89 - 1.10 $\simeq$ 0.8 m, quite a low value. With anthracite sand filtration, air-binding also has the disadvantage that air bubbles adhere to the light-weight anthracite grains, increasing their buoyancy and promoting loss of this material during backwashing.

Compared with the single bed filter of fig. 7.2 indicated there with fully drawn lines, the double bed filter of fig. 7.4 allows a 20% increase in filtration rate. This is nice, but not impressive. A much larger increase in filtration rates can be obtained by augmenting the total thickness of the filterbed, for instance to 1.5 m for which the results are shown in fig. 7.5. The most economic solution is again obtained for the smallest allowable grain size of the anthracite bed, d = 0.84 mm. With a depth of supernatant water of 1.2 m, the maximum allowable head loss equals 2.7 m, asking for an anthracite bed of 0.67 m thickness. The allowable filtration rate rises to 4.83 mm/s, equal to 17.4 m/hour! For this situation the distribution of pressure and clogging is shown in fig. 7.6.

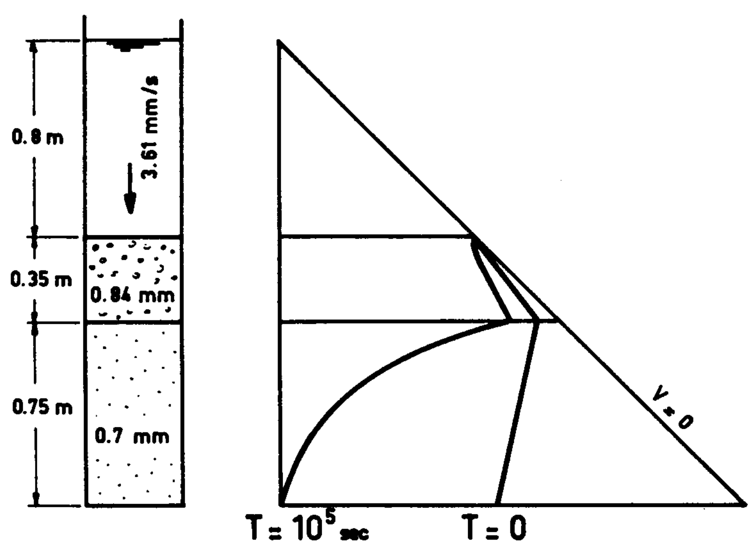



Fig. 7.4 Pressure distribution in double-bed filter

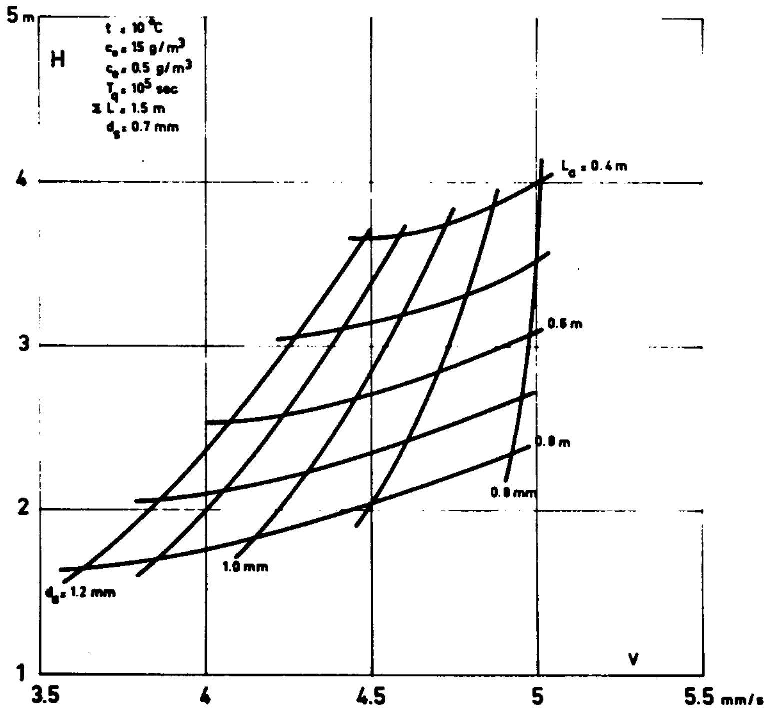



Fig. 7.5 Possibilities for double-bed filtration

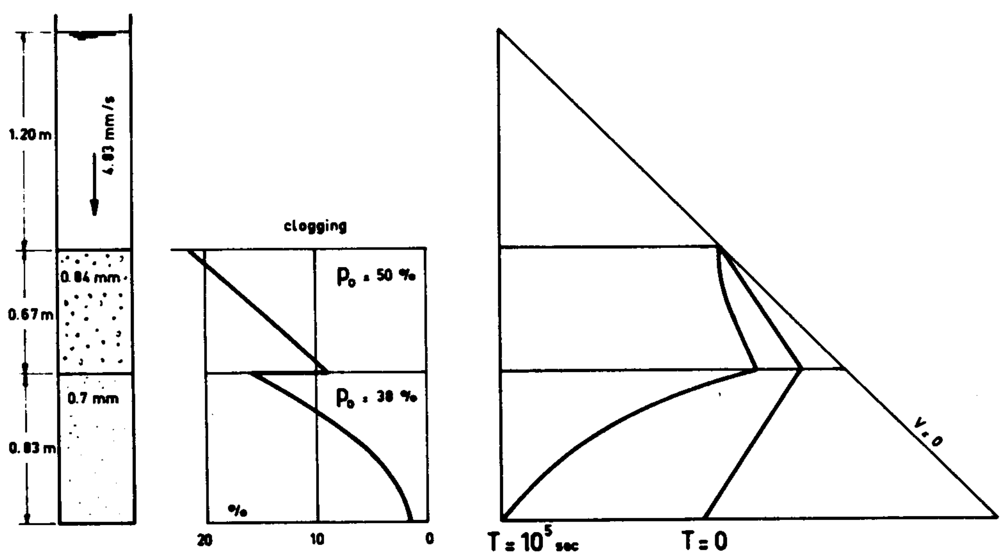



Fig. 7.6 Distribution of pressure and clogging in a double-bed filter

When considering fig. 7.5, it should never be forgotten that the possibilities for double-bed filtration shown here are calculated for the assumptions mentioned in the text. In practice other assumptions have to be made, leading to quite different results.

Double bed filtration is often used to upgrade existing plants. Replacing the upper part of the filterbed by anthracite with a larger grain size, does lower the filter resistance, but filtration efficiency will be less, deteriorating effluent quality. When this is not acceptable or an improvement in effluent quality is required, the lower part of the filterbed must also be replaced, now with sand of a smaller grain size.

## 7.4. Back-washing

The head loss during back-washing again equals the submerged weight of the filterbed. According to the formula in the centre of page 74.

$$H = (1-p_a)L_a \frac{\rho_a-\rho_w}{\rho_w} + (1-p_s)L_s \frac{\rho_s-\rho_w}{\rho_w}$$

and for the situation of fig. 7.6

$$H = (1-0.50)(0.67)\frac{1500-1000}{1000} + (1-0.38)(0.83)\frac{2650-1000}{1000}$$

$$H = 0.17 + 0.85 = 1.02 \text{ m}$$

To calculate the required back-wash rate, the effective grain sizes, mentioned in fig. 7.6 for filtration must first be transformed in those for back-washing.

filtration $\quad d' = \phi' s$

backwashing $\quad d = \phi\, s = \phi \dfrac{d'}{\phi'}$

and with the shape factors mentioned at the end of section 7.1

$$d_a = (0.85)\,\frac{0.84}{0.7} = 1.02 \text{ mm}$$

$$d_s = (0.70)\,\frac{1.0}{0.9} = 0.78 \text{ mm}$$

For this case with a rather low grain size ratio, section 7.2 advises

anthracite $E_{max} = 30\%$, $E_{av} = 17\%$
sand $E_{min} = 10\%$, $E_{av} = 18\%$

From the average amounts of filterbed expansion, the porosity during back-washing can be calculated as

$$p_e = \frac{p+E}{1+E}$$ with p as the porosity of the filterbed at rest

$$p_{ea} = \frac{0.50 + 0.17}{1 + 0.17} = 0.573$$

$$p_{es} = \frac{0.38 + 0.18}{1 + 0.18} = 0.475$$

The back-wash rate can now be calculated with the formula in the beginning of section 7.2. For $t = 10^oC$, $\nu = (1.31)10^{-6}\ m^2/s$ this gives

$$v_a = v_s = 7.7 \text{ mm/s} = 28 \text{ m/hour}$$

With regard to the small grain sizes, an auxilliary air scour is now certainly required, but it should be practised with care. According to fig. 7.6, the pore space in the sand bed just below the interface is severely reduced. The air has difficulty in passing this layer and tends to lift the overlying anthracite bed, promoting loss of this material. To prevent this piston action from occurring, back-washing should start with water at a low rate, say 10 m/hour. The liberated cloggings finally are flushed away with water, at the rate of 28 m/hour calculated above. During this back- washing some mixing between anthracite and sand will occur, but these materials can be segregated again by terminating the water wash slowly.

# 8. DRY FILTRATION

## 8.1. Construction

Dry filtration is the process whereby the water to be treated flows in downward direction through a bed of granular material, accompanied by a downward or upward flow of air of about the same magnitude. This combined flow of air and water through the filterbed has several advantages.

a. the oxygen consumed during treatment can be replenished directly from the accompanying air. Even with high ammonia contents in the raw water, the filter effluent is nearly saturated with oxygen;

b. the pores are only partially filled with water by which the real water velocities are much higher than with wet filtration. This creates turbulent flow conditions which promote the hydrodynamic transport of impurities from the flowing interstitial water to the filter grain surfaces where they become fixed by adsorbtion;

c. the large velocity at which the water flows past the grains, creates a streaming potential which increases the negative charge of the sand surface. This promotes the adsorbtion of positively charged particles such as colloïdal flocs of iron;

d. in particular with a counter current flow of air, the dissolved gaseous and volatile substances such as $CO_2$, $CH_4$, $H_2S$ and taste and odour producing organic compounds are removed by stripping.

Dry filtration is not a new process. It has already been applied for a long time in drinking water industry as perforated trays filled with coke for aeration (fig. 8.1 ) and in sewage treatment for aerobic oxidation of organic impurities in trickling filters (fig. 8.2). New features are the use

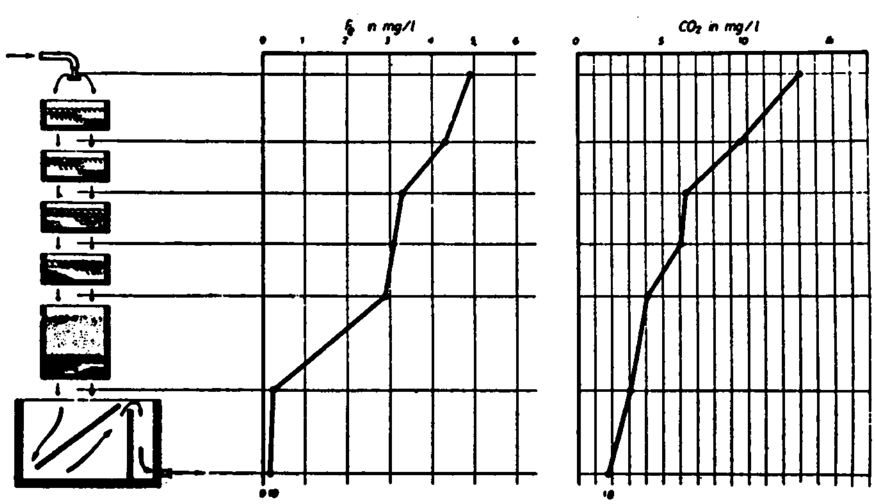


Fig. 8.1 Multiple tray aerator, the upper four filled with coke and the lower one filled with gravel

of much finer material, 1 to 2 mm for instance, and a periodic cleaning of the filterbed by back-washing with water and air.

The simplest construction of a dry filter is shown in fig. 8.3, where the water to be treated is distributed over the full area of the filterbed as evenly as possible with the help of stationary nozzles, producing fine droplets. These droplets strike the filterbed every time at the same place and a better solution would therefore be the use of a rotating sprinkler with 4 to 6 arms and 20 to 30 revolutions per minute, provided with perforations of 3 to 4 mm diameter, and staggered in such a way that every $m^2$ of the filterbed receives the same amount of water. For good results, however, the filter should now have a circular or 8-sided cross-section. The filterbed itself has a thickness of 1.5 to 2 m and is composed of rounded sandgrains, say 1-2 mm. Sometimes broken gravel of the same size is used to obtain a larger surface area. The filterbed is supported by a false floor with long stem filter nozzles for back-washing with water and air. The filtered water level is some distance below this floor, allowing a separate abstraction of air and water. The water flows by gravity to the clear well, while in the effluent line an air ejector is installed to maintain a simultaneous flow of air through the filterbed. This arrangement, however, has two disadvantages. On one hand it brings stale air in the clear well, imparting bad taste and odour to the filtered water, while on the other hand the air flow is quite limited, 0.2-0.5 $m^3$ of air per $m^3$ of water. This limits allowable filtration rates to 1-2 mm/s and raw water ammonia contents to 3-5 $g/m^3$.

When larger filtration rates are desired and/or higher ammonia contents must be dealt with, the flow of air through the filterbed must be increased by mechanical means. In fig. 8.4 on the left, a suction ventilator is applied with a maximum vacuum of 0.8-1m water column, producing an air flow of 1-2 $m^3$ per $m^3$ of water. This ventilator handles moist and agressive air,

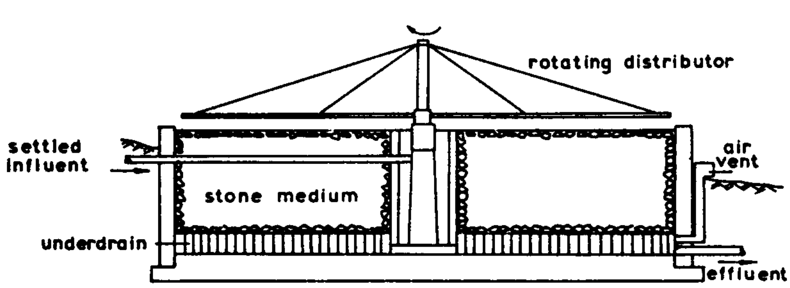



Fig. 8.2 Trickling filter

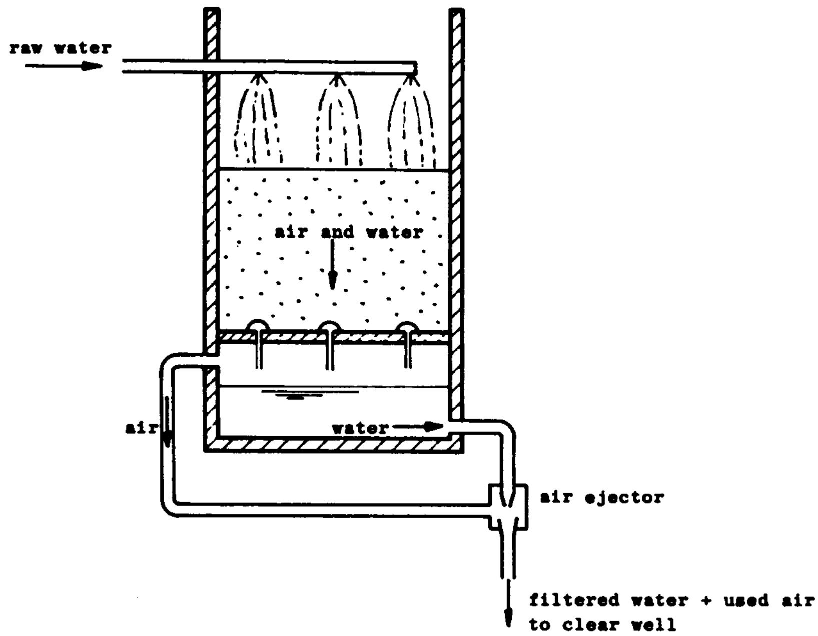



Fig. 8.3 Dry filter with air ejector

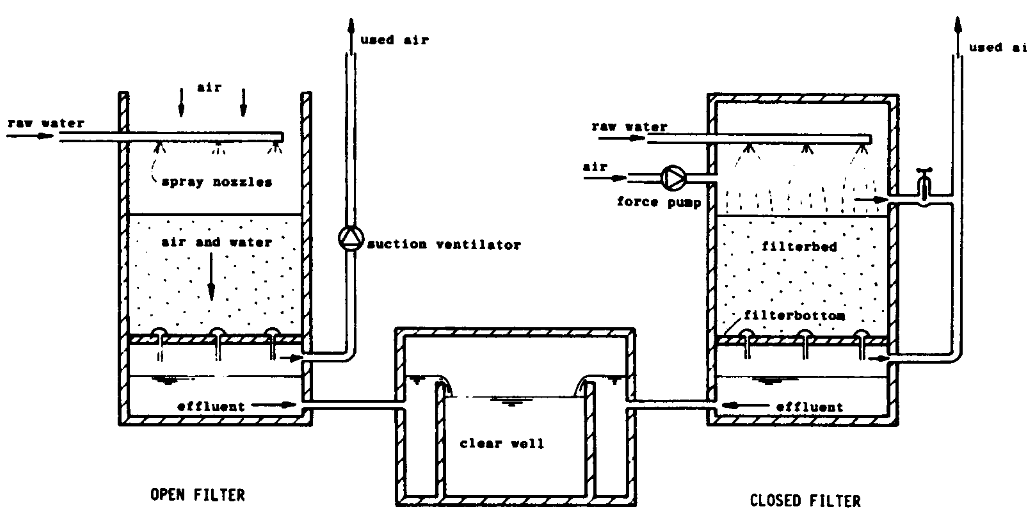



Fig. 8.4 Dry filter with downward flow of air

shortening its useful length of life. A better solution in this respect is shown in fig. 8.4 on the right where the filter is covered and the air flow is maintained by a force pump, producing higher pressures, up to 30 m water-column and larger air volumes,up to 5 $m^3$ per $m^3$ of water. Part of this air is discharged above the filterbed, to remove the gases liberated by spray aeration. When stripping is a major objective of dry filtration, the flow of air can better be reversed as shown in fig. 8.5. In this way $CO_2$ contents can be reduced by 90%, $CH_4$ and $H_2S$ by 60% and taste and odour by 50%. This means on the other hand that the air should be discharged some distance above the covered filter building where the wind assures adequate dilution. This upflow aeration gives sometimes difficulties by moving the settled out impurities to above.

## 8.2. Operation

Without any doubt groundwater is the best source for human consumption, but some groundwaters are difficult to treat by wet filtration. This is particularly the case when the water has a high colour, a high permanganate consumption and contains larger amounts of carbon dioxide and ammonia, next to hydrogen sulfide and methane. The oxidation of ammonia requires large amounts of oxygen which are not available in a wet filter, while the iron is present in a complex of positively charged ferrous ions and negatively

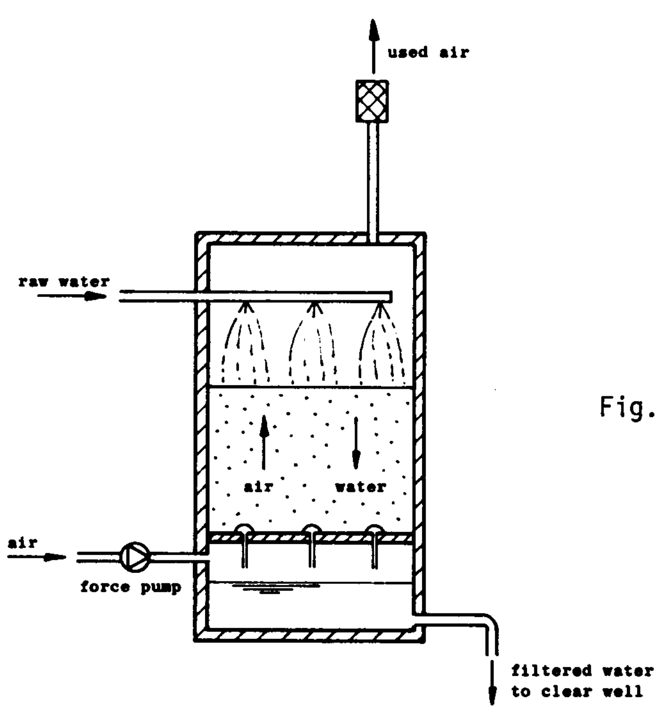



Fig. 8.5 Dry filter with counter current flow of air

charged organic groups which are difficult to break down. The sulfur compounds finally poison the catalytic action of previously deposited ferric hydroxide. In dry filters on the other hand the gaseous pollutants are removed by stripping and oxidation($H_2S$), the oxygen content of the water remains high allowing large amounts of ammonia to be oxidized, while the same factor promotes the break-down and oxidation of organic iron compounds.

Dry filtration as described above is primarily meant for the treatment of groundwater having high contents of ammonia, next to iron and manganese, while the suspended matter content is low. Purification starts with iron removal, which takes place in the upper part of the filterbed, down to a depth of 0.5-1.5 m, larger as the filtration rates are higher. It consists of an oxidation from ferrous to ferric iron according to the equation

$$4Fe^{++} + O_2 + (2x + 4)H_2O \rightarrow 2(Fe_2O_3.xH_2O) + 8H^+$$

which process is accelerated catalitically by previously deposited $Fe(OH)_3$ and promoted by a high oxygen content of the water. For $p_H < 8$, as is normally the case, the colloidal iron flocs produced have a positive charge and are adsorbed on the negatively charged sand grain surfaces. After this process has been completed, manganese and ammonia removal occur more or less simultaneously. Manganous ions are oxidized to manganic ones

$$2Mn^{++} + (x-1)O_2 + (2y + 2)H_2O \rightarrow 2(MnO_x.yH_2O)+ 4H^+$$

which process is catalitically promoted by previously formed deposits of $MnO_2$ and in particular $Mn_2O_3$. Again the positively charged manganic flocs adhere to the sand grain surfaces by electrical attraction. Both with the oxidation of iron and manganese, the hydrogen ions formed react with the bicarbonate ones present in the raw water

$$H^+ + HCO_3^- \rightarrow H_2O + CO_2$$

Oxidation of ammonia finally is a bio-chemical process

$$2NH_4^+ + 3O_2 + \text{nitrosomonas} \rightarrow 2\,NO_2^- + 2H_2O + 4H^+$$

$$2NO_2^- + O_2 + \text{nitrobacter} \rightarrow 2NO_3^-$$

$$4H^+ + 4HCO_3^- \rightarrow 4H_2O + 4CO_2$$

together

$$2NH_4^+ + 4O_2 + 4HCO_3^- \rightarrow 2NO_3^- + 6H_2O + 4CO_2$$

This process is strongly temperature dependent, giving good results at $16^0C$, fair at $5^0C$, while below $2^0C$ it nearly stops completely. Oxygen requirements are small for iron (0.14g/gFe) and manganese (0.15-0.29 g/gMn ) and high for ammonia (3.56 $g/gNH_4^+$) . In wet filters iron gives rather fluffy deposits, easy to remove by back-washing, but in dry filters the deposits have a higher mass density, are of a more granular nature, requiring elevated back-wash rates. The bacteria necessary for ammonia oxidation finally are present in a coating around the filter grains. In the surface layer of this coating so much oxygen may be consumed that below this layer anaerobic conditions prevail. Here nitrate is reduced to nitrogen gas

$$5C + 4NO_3^- + 2H_2O \rightarrow 2N_2 + CO_2 + 4HCO_3^-$$

The gas escapes with the air and the nitrogen balance will show a deficit.

The filtration rates to be applied may be chosen higher as the raw water is less polluted, the filterbed thickness is larger and the air-water ratio is higher. Common values are between 2 and 8 mm/s, but the exact value can only be determined by operating an experimental plant. In case a break-through of iron occurs, the filterbed thickness must be increased or the filtration rate lowered. A better solution would be the use of 2 filters in serie, the first one for deferrization and the second one for demanganization and ammonia removal. The first filter can now be back-washed vigorously to remove the heavy iron deposits which have penetrated to great depths. The second filter needs a much more gentle back-wash, not to loose the catalytic action of the manganic oxide deposits, neither the nitrosomonas and nitrobacter bacteria responsible for oxidation of ammonia. Dry filters are not meant for clarification, the suspended matter in the raw water resulting in a rapid clogging of the filterbed. When a turbid river water must be dealt with, pre-treatment for instance by settling with or without chemical coagulation and flocculation is required. Dry filters are seldom the last step in the treatment process. Commonly they are followed by wet filtration, for polishing purposes and for retaining the higher forms of biological live that thrives in a dry filter.

From the foregoing it will be clear that back-washing disturbs the purification process, in particular ammonia removal. For this reason filter runs must be rather long, 2-5 days. Cleaning begins by filling the box with water to the top of the wash-water troughs. This should be done gently, not to disturb the clogged filterbed. Back-washing proper starts with air at 5-15 mm/s and water at 2-3 mm/s. After the cloggings have been liberated from the filter grain surfaces, they are flushed away with water alone. This should be done at high rates to remove iron deposits, but at medium to low rates not to endanger nitrification and manganese retention. The compromise lies somewhere between 5 and 15 mm/s, preferably near the lower end of this range and for extended periods, up to 20 minutes.