DELFT UNIVERSITY OF TECHNOLOGY
Department of civil engineering
Division of sanitary engineering

RAPID FILTRATION
Part 1

Prof. ir L. Huisman

1979

DELFT UNIVERSITY OF TECHNOLOGY

Department of civil engineering

Division of sanitary engineering

RAPID FILTRATION
Part 1

Prof.ir.L.Huisman

tgave 73	2e herdruk october 1979		N4.K	753090					*f* 4,30

L.S.

Bij de samenstelling van elk diktaat wordt er uiteraard naar gestreefd om fouten te voorkomen en de inhoud zo overzichtelijk mogelijk aan te bieden.
Niettegenstaande dat kunnen toch onduidelijkheden voorkomen en kunnen fouten zijn ingeslopen.
Indien U dan ook bij de bestudering van dit diktaat:
- onjuistheden ontdekt
- op onduidelijkheden stuit
- of gedeelten ontmoet, die naar Uw mening nadere uitwerking behoeven, verzoeken de samenstellers U dringend hen daarvan mededeling te doen.

Bij de volgende drukken kunnen dan op- en aanmerkingen worden verwerkt ten gerieve van toekomstige gebruikers.
Zonodig kan ook nog in de lopende cursus voor verduidelijking worden gezorgd.

CONTENTS

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)
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)

α	- coefficient in filtration theory (sec^{-1})
ϕ_1	- shape factor of non-spherical grains during filtration
ϕ	- shape factor of non-spherical grains during backwashing
λ, λ_o	- coefficient in filtration theory (m^{-1})
ν	- kinematic viscosity (m^2/sec)
ρ	- mass density (kg/m^3)
ρ_d	- mass density of impurities as deposited in the filterbed (kg/m^3)
σ	- gravimetric concentration of impurities in filterbed (g/m^3)
σ_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

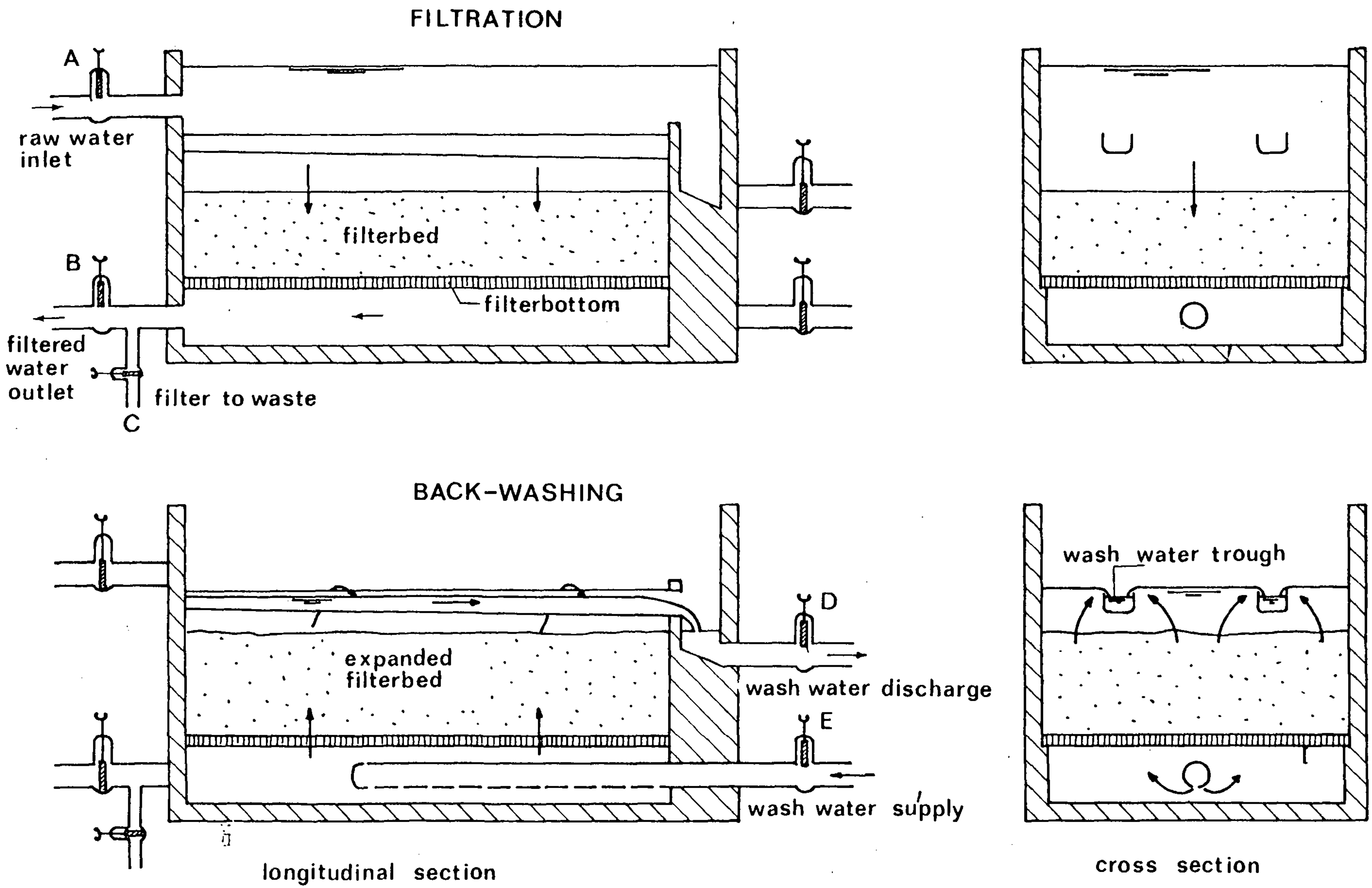


Fig. 1.1 Free-surface rapid gravity filter.

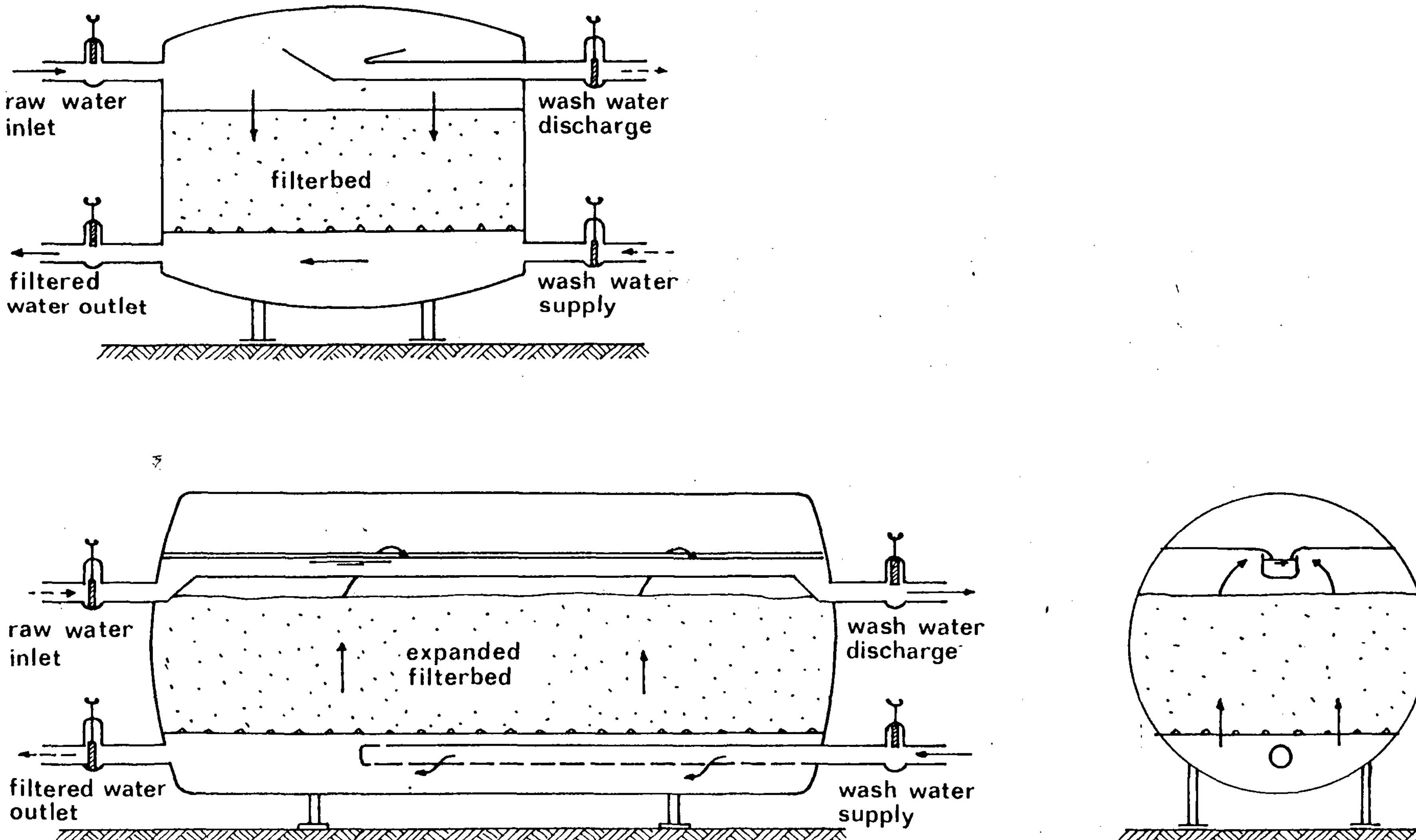


Fig. 1.2 Pressure filters.

desired, this depth could be increased, but this asks for a greater height of the filterbox, 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

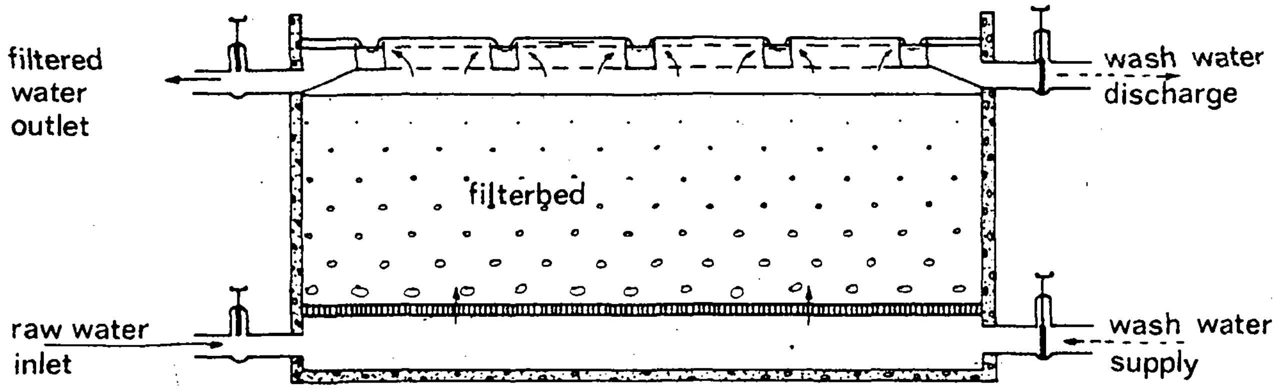
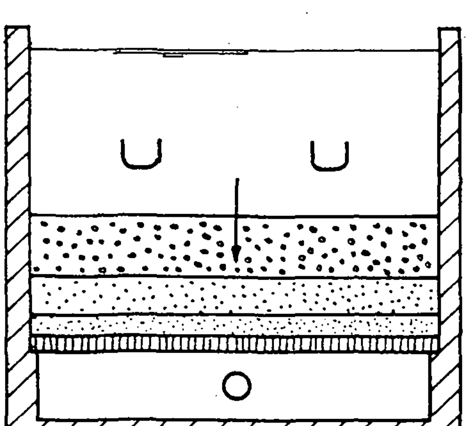


Fig. 1.3 Upflow filtration.

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, ϕ 1.6mm, ρ_a/ρ_w : 1.5
0.4 m sand, ϕ 0.8mm, ρ_a/ρ_w : 2.6
0.2 m garnet, ϕ 0.5mm, ρ_a/ρ_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° C to 8 g/m^3 at 30° 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.

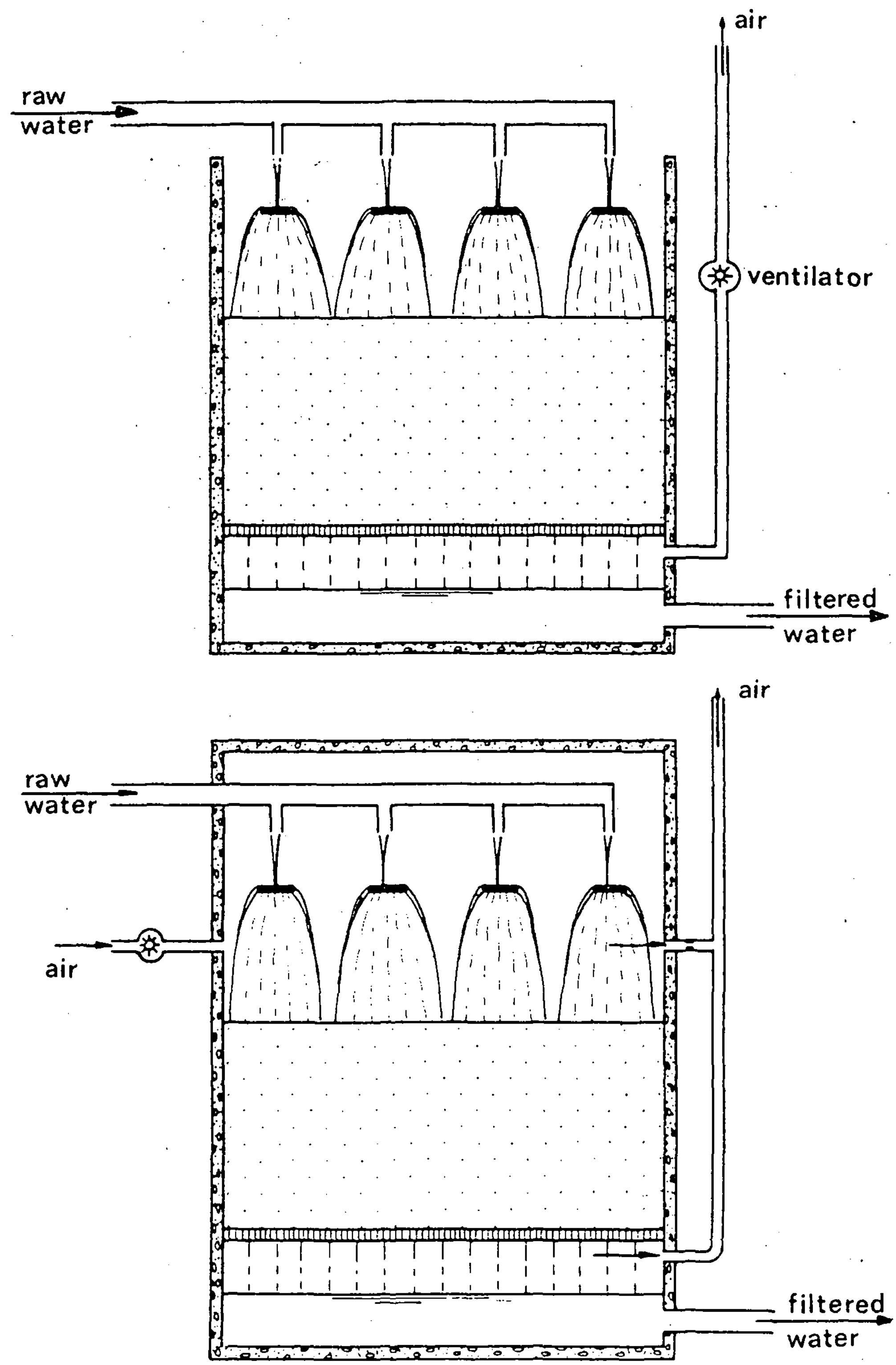


Fig. 1.5 Dry filters.

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

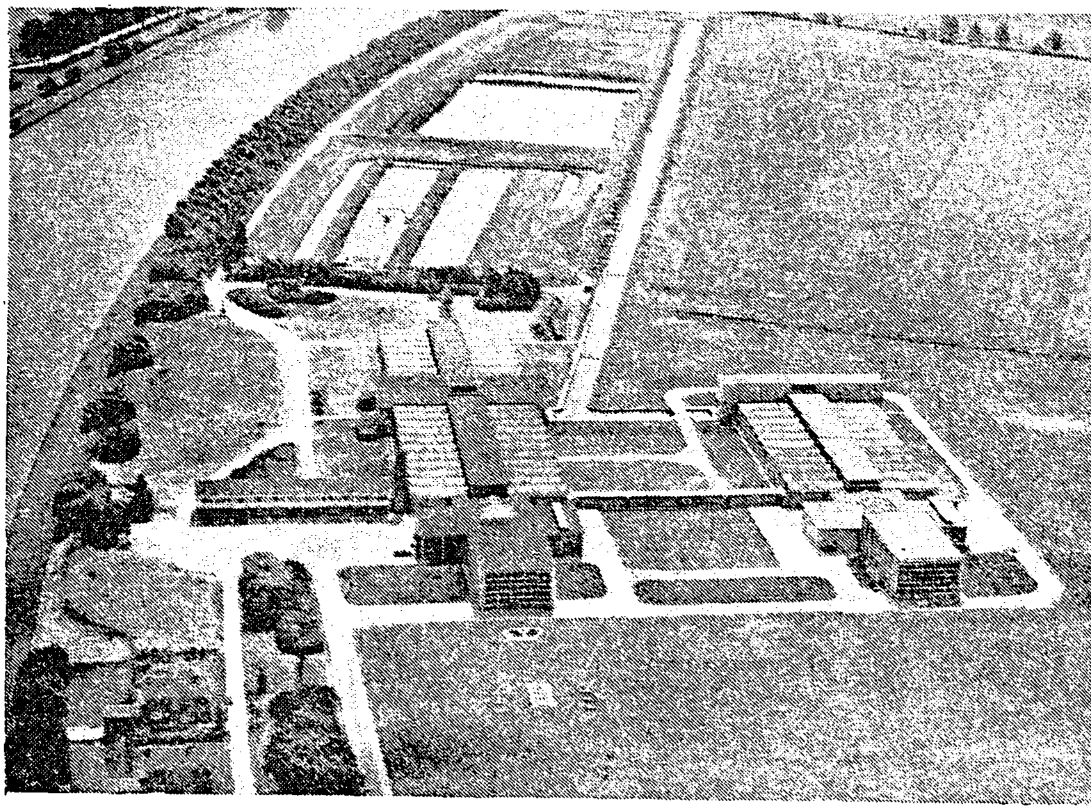


Fig. 1.6 Rapid filtration plant of the N.V. Watertransportmaatschappij Rijn-Kennemerland at Jutfaas, Netherlands.

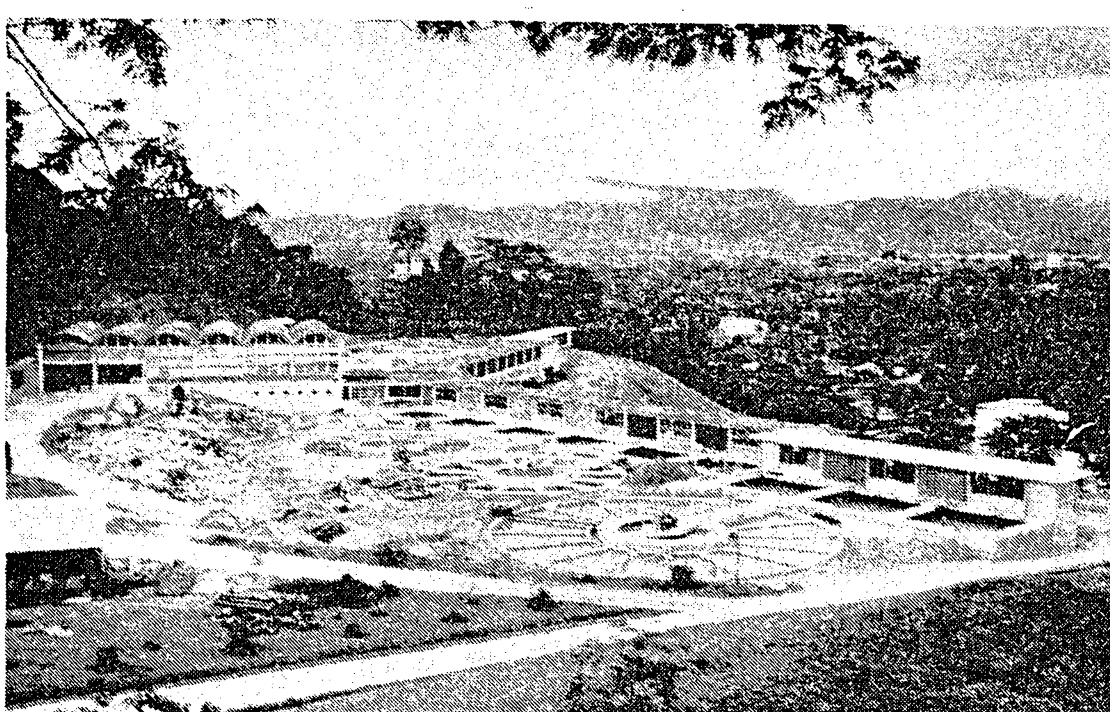


Fig. 1.7 Bukit Nanas treatment plant at Kualalumpur, Malaysia.

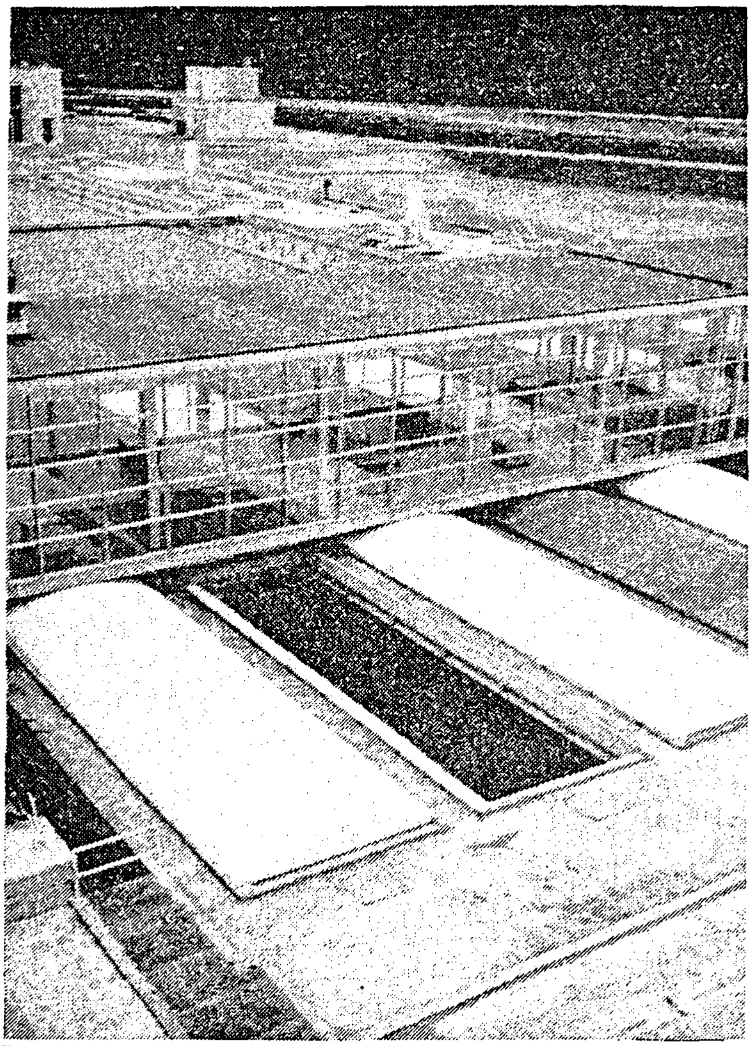


Fig. 1.8 Rapid filtration plant of the Antwerp Waterworks at Oelegem, Belgium.

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-

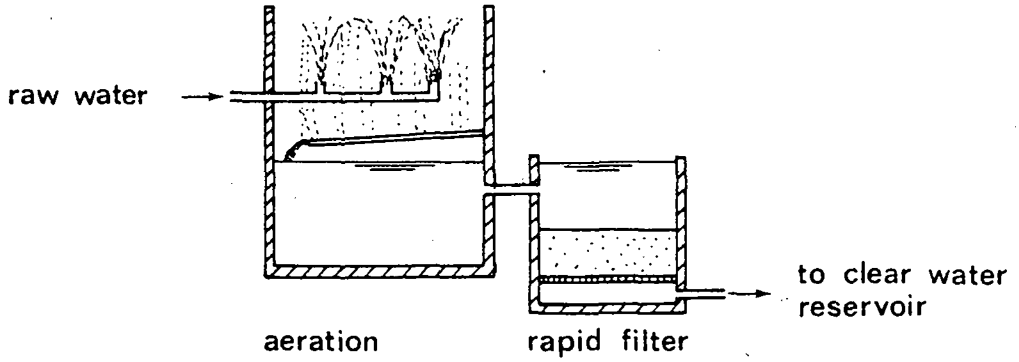


Fig. 1.9 Treatment system for deep anaerobic groundwater.

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.

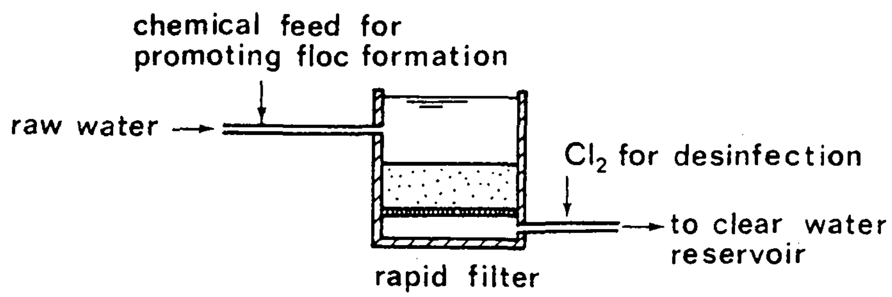


Fig. 1.10 Treatment system for slightly polluted surface water.

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

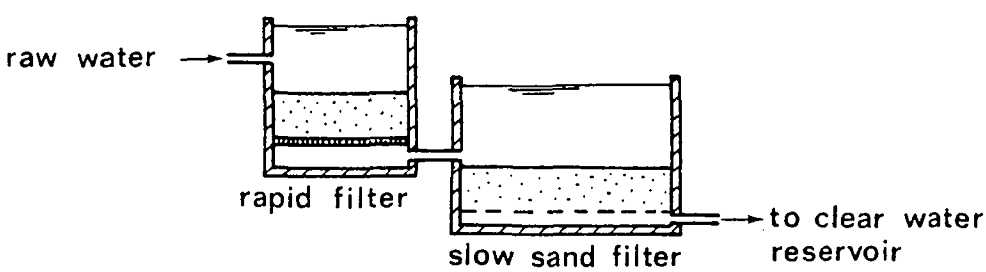


Fig. 1.11 Biological purification of surface water.

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.

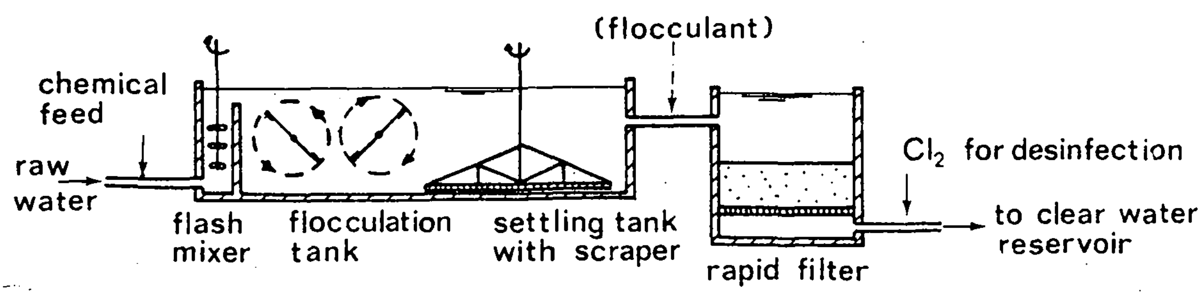


Fig. 1.12 Mechanical purification of surface water.

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

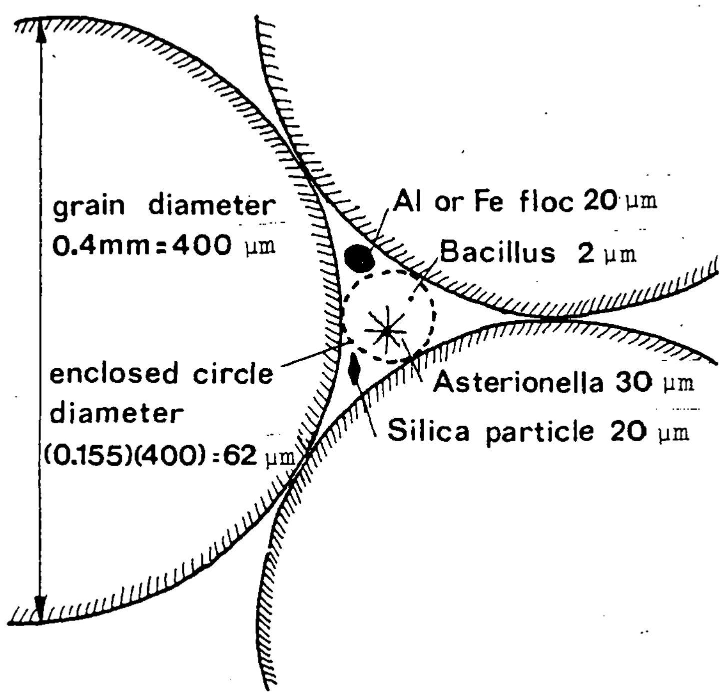


Fig. 2.1 Size of pore openings and suspended particles.

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, ρ and $\rho + \Delta\rho$ the mass density of water and suspended matter respectively, g the gravity constant (9.81 m/sec^2) and ν the kinematic viscosity of the fluid. For water at 10^oC, $\nu = (1.31)10^{-6}$ 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

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

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

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 settles 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 colloidal 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

$$C_5H_7O_2N + 5O_2 \longrightarrow H_2O + 4CO_2 + NH_4^+ + HCO_3^-$$

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

$$C_5H_7O_2N + 7O_2 \longrightarrow 3H_2O + 5CO_2 + NO_3^- + H^+$$

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 bicar- tonate 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 ⟶ nitrite ⟶ 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 environment 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 definite and detailed answers, indispensable for the design of a rapid filtration plant, while also 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 to be carried out simultaneously, in this way excluding the influence of (seasonal) changes in raw water quality as much as possible. 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 height of 2 - 4 m and an inner diameter of 0.1 - 0.3 m and sometimes

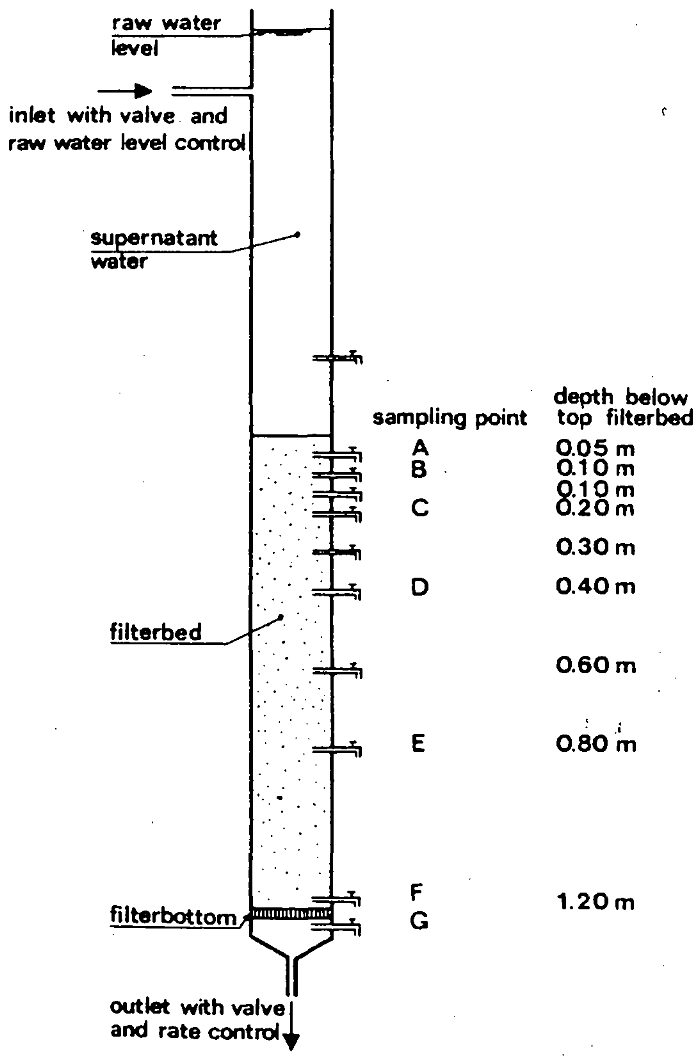


Fig. 2.2 Experimental filter.

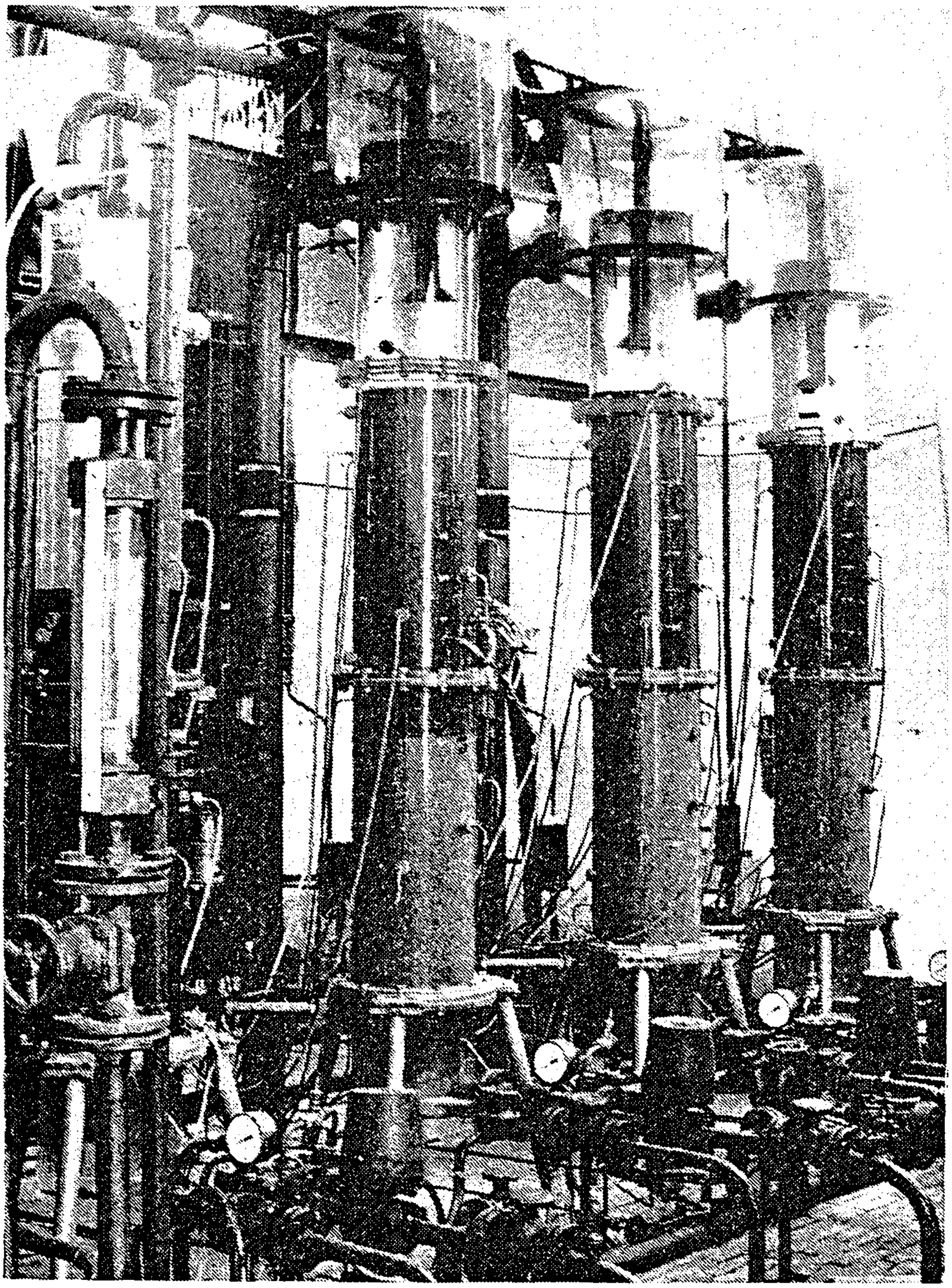


Fig. 2.3 Inside view of a pilot plant.

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.

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 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 filter

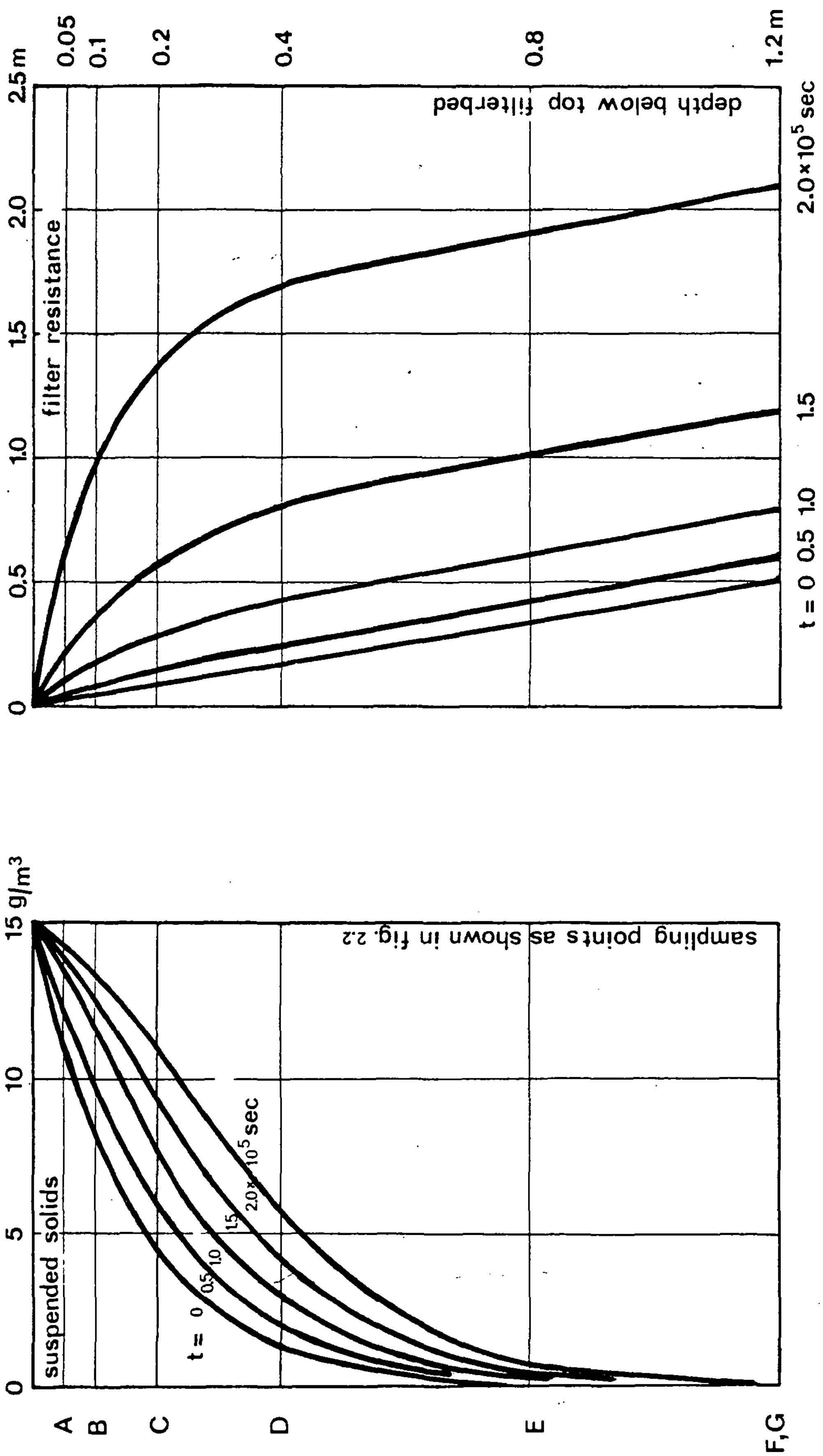


Fig. 2.4 Results of an experimental filterrun.

in parallel, each provided with a different depth of filtering material, for instance 0.4, 0.6, 0.9 and 1.5 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 intuitive method of experimentation mentioned above, it is wise to recall that the purpose of rapid filtration is to improve water quality, transforming for instance a turbid river water into a clear water with a suspended matter content less than 0.5 g/m^3. This quality requirement can easily be fulfilled by filtering the raw river water at a low rate, through a thick bed of fine grained filtering material. By the application of a low rate, however, large and expensive filters are necessary, while the use of fine filtering material and a great bed thickness moreover, means a rapid and strong increase of filter resistance with time, again increasing the cost of construction and operation. In practice another approach is therefore preferred, applying coarser grained filtering materials in smaller bed thicknesses and augmenting the filtration rate as much as possible. The increase of filter resistance with time will now be smaller, but next to this a deterioration of effluent quality with time must be expected. This means 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 to 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.

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 finally is 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/sec and fairly fine 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 =	2.12	1.17	< 0	× 10^5 sec
T_r =	1.60	2.34	> 3	× 10^5 sec

From these data the following conclusions can be drawn

a. the finest filtering material, d = 0.7 mm, satisfies all requirements. The lengths of filterrun, however, are rather great, meaning that also a higher rate of filtration could be considered;

b. with the middle grainsize of 0.8 mm the minimum length of filterrun, $T_q = (1.17)10^5$ sec = 32 hours, is still adequate, but this set-up has the serious disadvantage that effluent quality deteriorates below the set standard long before the filter resistance 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

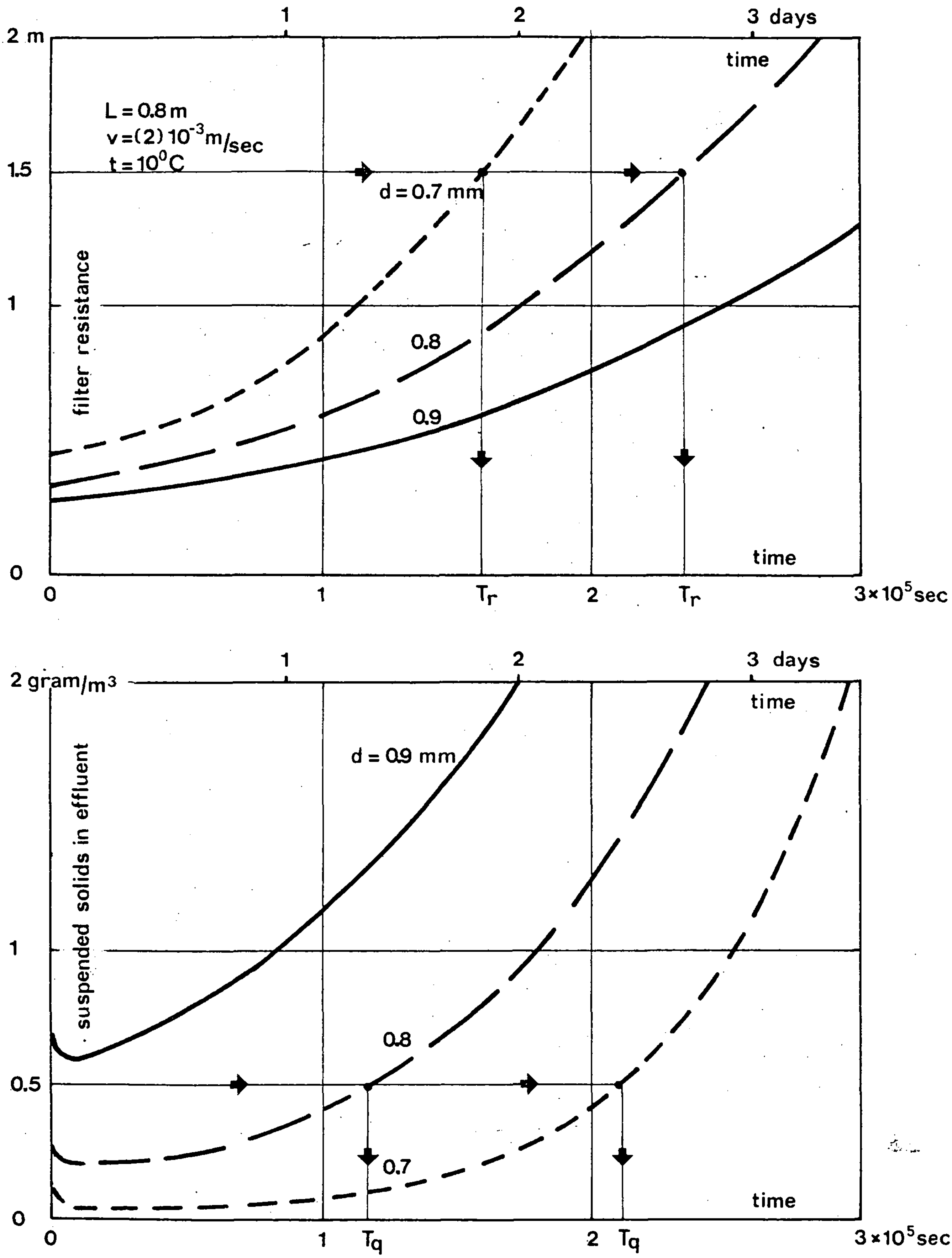


Fig. 2.5 Results of experimental filterruns using different grainsizes.

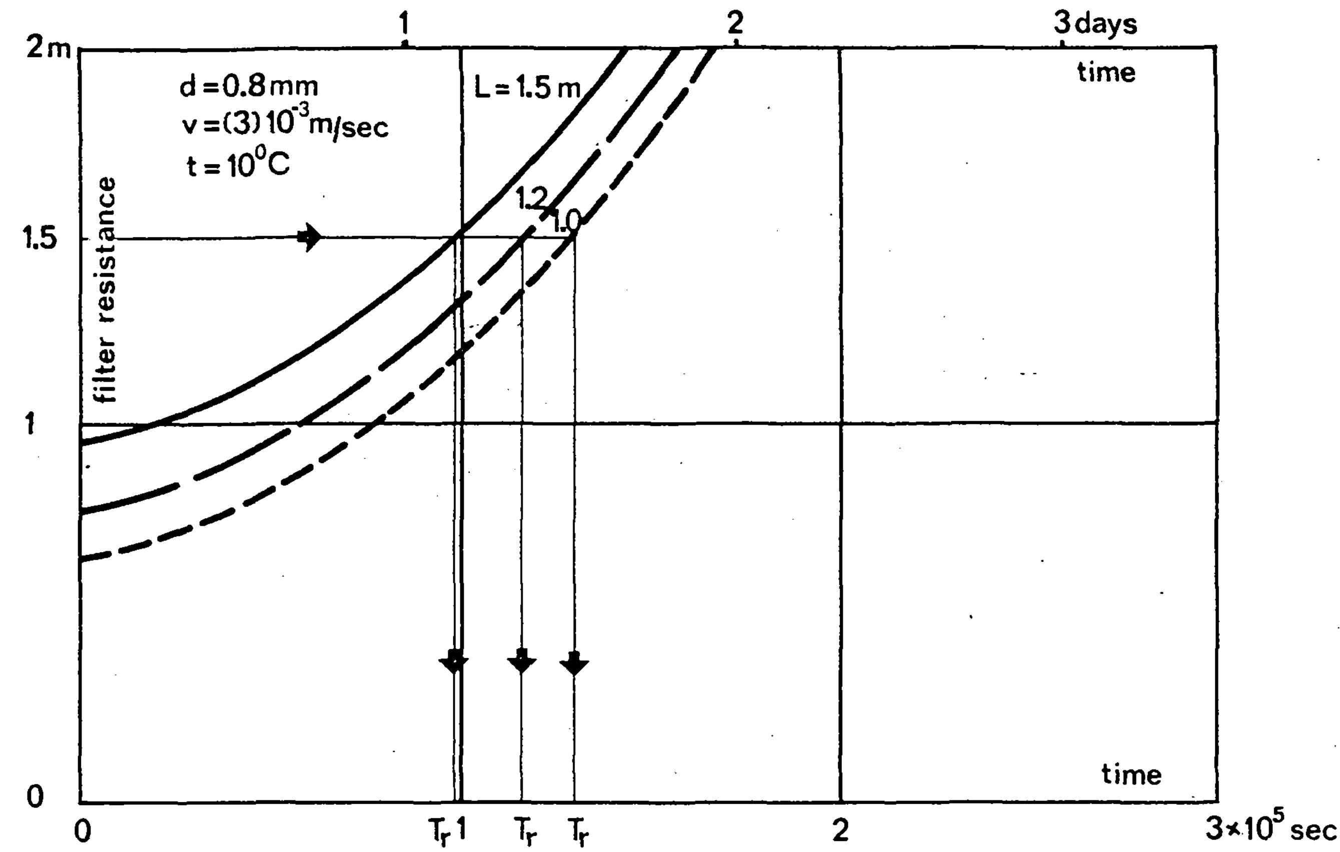
be obtained by increasing the filterbed thickness by which T_q will be greatly enlarged and T_r slightly reduced. Both lengths of filterrun will now be so long that again a higher rate of filtration could be contemplated;

c. with a filterbed thickness of 0.8 m, a grainsize of 0.9 mm is not able to satisfy the chosen effluent standards.

Going out from these conclusions, 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 shown below can be composed

L =	1.0	1.2	1.5	m
T_q =	0.52	1.18	2.18 $\times 10^5$	sec
T_r =	1.30	1.16	0.98 $\times 10^5$	sec

According to these data, a filterbed thickness of 1.2 m gives excellent results. Some economy in the cost of construction and operation, however, may still be obtained by considering that in practice a length of filter-run 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 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 1.15 m and a filter resistance not surpassing a value of 1.2 m. The latter value in the meanwhile is still rather 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 in the meanwhile 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.



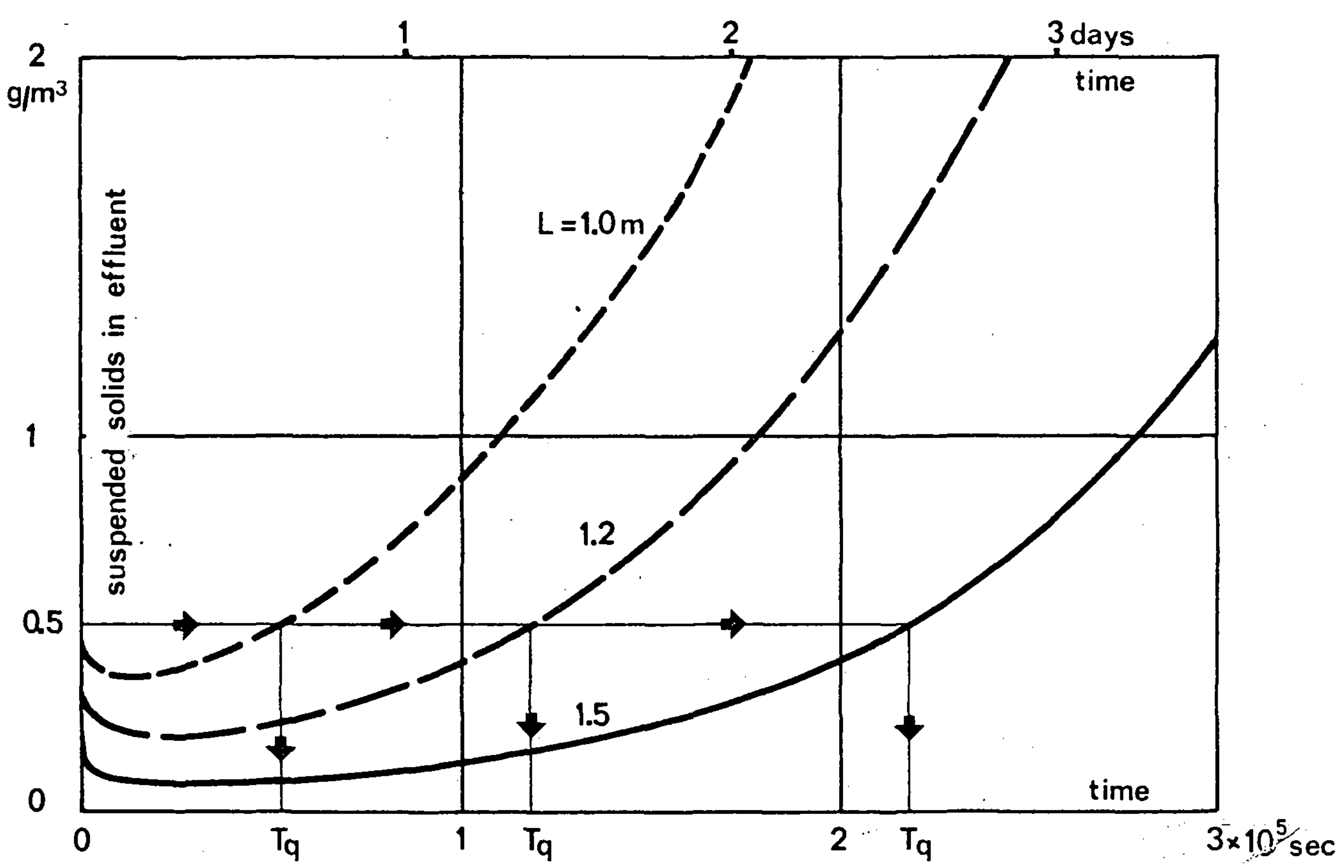


Fig. 2.6 Results of experimental filterruns using various bed thicknesses.

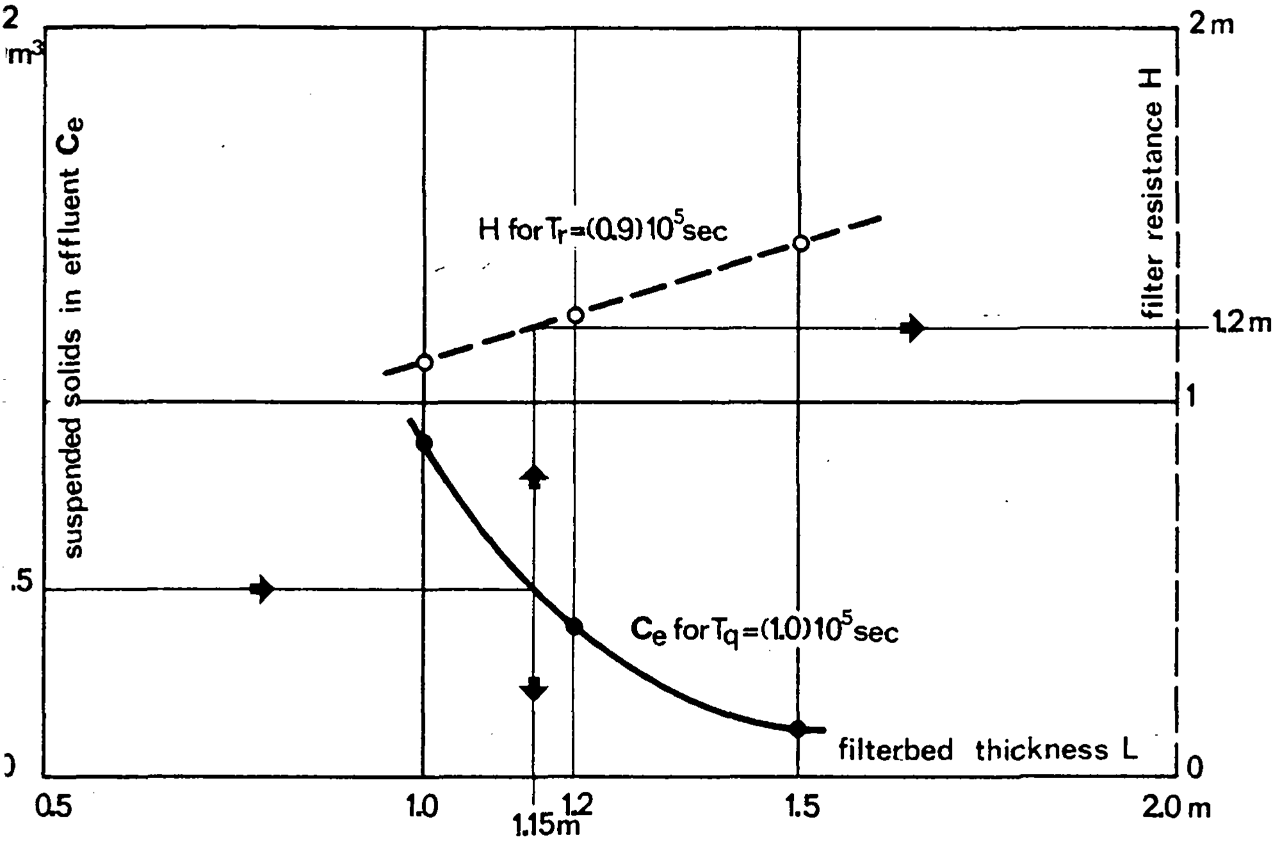


Fig. 2.7 Selection of bed thickness and filterresistance from the filtration results of fig. 2.6.

2.3 Dynamics of filtration

During the filtration process, impurities are removed from the downward flowing water. This means on one hand an improvement in water quality and on the other hand a clogging of the pores between the filtergrains with a subsequent increase in filter resistance. To study these phenomena mathematically, it is assumed that the clean filterbed has a thickness L and is composed of spherical grains with a diameter d_o and a porosity p_o (fig. 2.8, left). The accumulation of impurities in the filterbed will leave the thickness L unchanged, but the diameter of the grains will change from d_o to d and the porosity from p_o to p. The water to be treated approaches the filterbed at a velocity v, carrying a (gravimetric) concentration of impurities c_o. At a depth y below the top of the filterbed, the filtration rate still equals v, but the concentration of impurities has dropped to c. With regard to the random distribution of the impurities over the body of the water flowing through the pores of the filterbed, this reduction is brought about by a probability process with the removal ratio directly proportional to the concentration still present. In formula

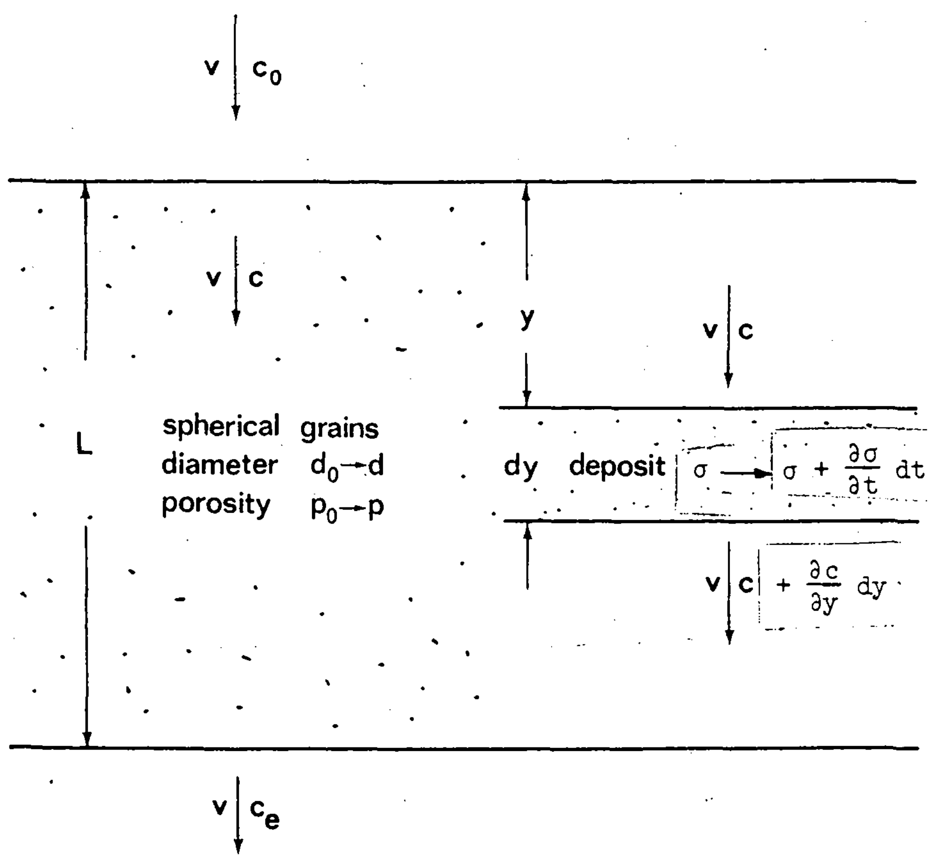


Fig. 2.8 Mathematics of filtration.

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

with λ as proportionality constant, the so-called coefficient of filtration. When it is provisionaly assumed that λ is constant, equal to λ_o, direct integration is possible. With the boundary conditions

$y = 0$, $c = c_o$, this gives

$$c = c_o e^{-\lambda_o y}$$

According to this equation the concentration of impurities still present in the downward moving water decreases logarithmically with depth, the upper part of the filterbed doing most of the work and the lower part relatively little (fig. 2.9). The quality of the effluent is given by

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

With for instance L = 0.75 m, $\lambda_o = 6\ m^{-1}$ and $c_o = 15\ g/m^3$

$$c_e = (15)e^{-(6)(0.75)} = \frac{15}{90.0} = 0.17\ g/m^3 \text{ (fig. 2.10)}$$

The impurities removed from the water during filtration are transferred to the filterbed, where they accumulate on and between the

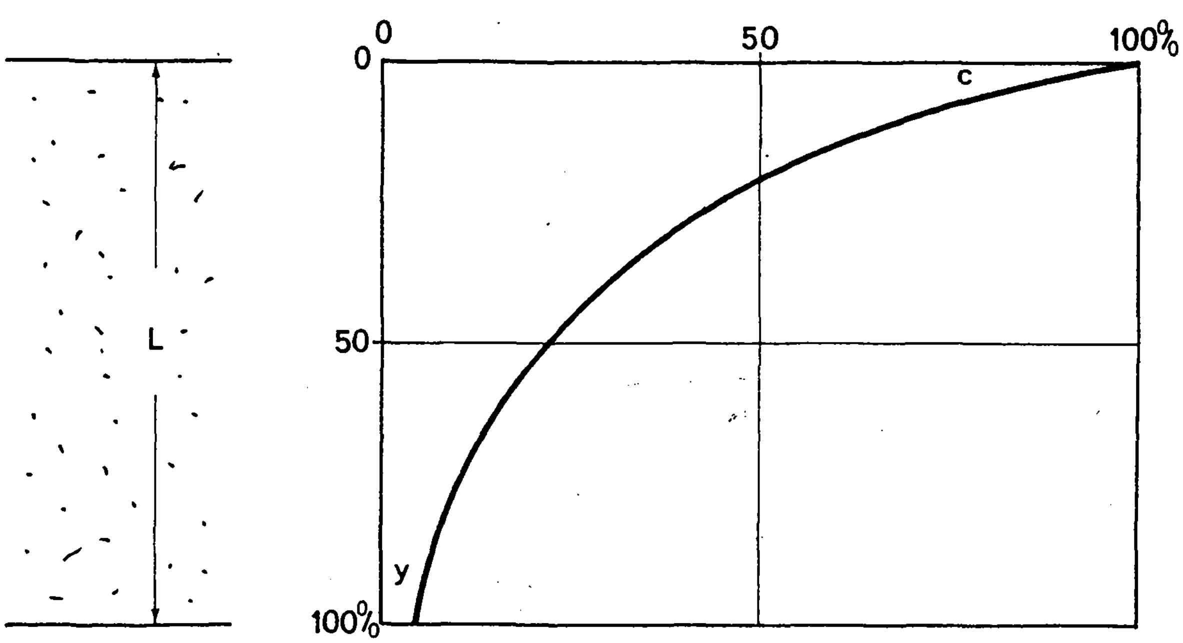


Fig. 2.9 Purification accompanying filtration.

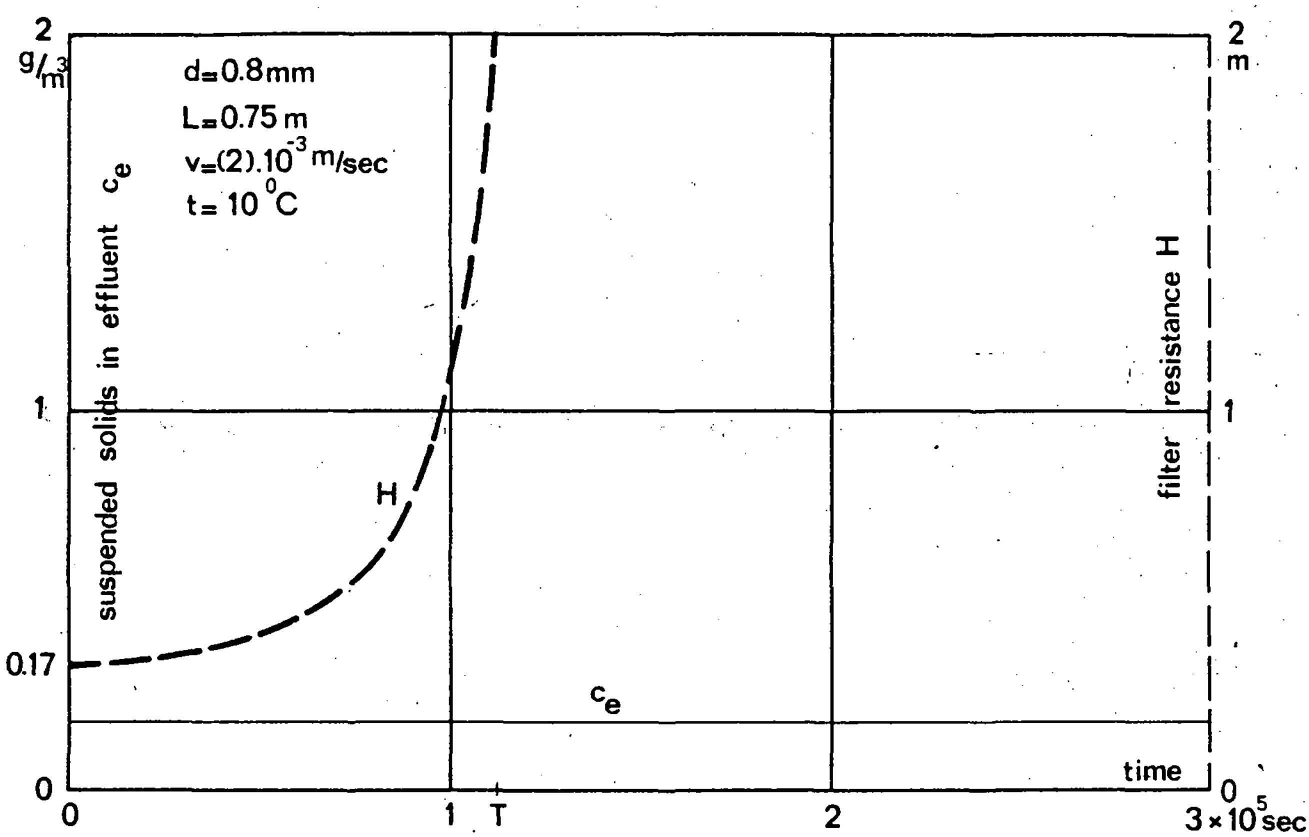


Fig. 2.10 Results of a calculated filterrun assuming a constant value of the filtration coëfficient $\lambda = \lambda_o$.

filtergrains. The rate of deposition can be determined by considering an element of the filterbed with thickness dy at a depth y below the top (fig. 2.8 at the right). At the time t under consideration, impurities from the raw water have accumulated here in a concentration σ. During the next period dt, this concentration will increase by an amount $d\sigma = \frac{\partial \sigma}{\partial t}\, dt$. According to the continuity equation, this increase must equal the net inflow of impurities, carried by the water

deposition = inflow - outflow, in formula per unit area of filterbed

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

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

In case the filtration coefficient λ is constant, equal to λ_o, the concentration of impurities in the pore water has been found at

$$c = c_o e^{-\lambda_o y} \quad \text{giving} \quad \frac{\partial c}{\partial y} = -\lambda_o c_o e^{-\lambda_o y}$$

Substituted

$$\frac{\partial\sigma}{\partial t} = v\ \lambda_o c_o e^{-\lambda_o y}$$

Integration with the boundary condition $t = 0$, $\sigma = 0$ now yields

$$\sigma = v\ \lambda_o c_o e^{-\lambda_o y}\ t$$

that is to say a logarithmic decrease with depth y and a linear increase with time t.

To determine the increase in filter resistance due to the deposition of impurities in the filterbed as calculated above, recourse may be taken to the Carman-Kozeny equation for laminar flow (compare chapter 12). For a clean filterbed this equation gives as headloss

$H = I_o L$ with I_o as slope of the piezometric level equal to

$$I_o = \left(\frac{dz}{dy}\right)_o = \frac{180\ \nu}{g}\ \frac{(1-p_o)^2}{p_o^{\ 3}}\ \frac{v}{d_o^{\ 2}}$$

with ν as kinematic viscosity and g as gravity constant and the other factors as defined above. For the subsequent considerations, another form of this equation is better suited, replacing the diameter d_o by the total grain surface S_o per unit volume of material as present in the filterbed

$$S_o = \frac{6}{d_o}(1 - p_o).$$ Substituted

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

By accumulation of impurities in the filterbed, the porosity will drop from p_o to p and the specific surface will change from S_o to S, augmenting the slope of the piezometric level to

$$\left(\frac{dz}{dy}\right)_t = \frac{5\nu}{g}\ \frac{S^2}{p^3}\ v \quad \text{or}$$

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

in which I_o is a constant for specified operating conditions. The decrease in porosity accompanying filtration is easy to calculate.

When the impurities accumulate in a concentration σ and a density ρ_d, their volumetric concentration equals

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

This concentration causes a drop in pore space

$$\sigma_v = p_o - p \qquad \text{or} \qquad \left(\frac{p_o}{p}\right)^3 = \left(\frac{p_o}{p_o - \sigma_v}\right)^3$$

The change in specific surface is less exact to determine, the outcome being heavily dependent on the mathematical model visualized for the flow of water through a bed of granular material. Here the capillary model will be applied, assuming that per unit surface and unit depth of the clean filterbed n capillaries of internal diameter t_o and length l will be present. This gives for the clean filterbed

$$p_o = n \frac{\pi}{4} t_o^2 l, \qquad S_o = n \pi t_o l$$

Clogging is supposed to reduce the internal diameter t_o uniformaly to t

$$p = n \frac{\pi}{4} t^2 l, \qquad S = n \pi t l$$

From both sets of equation follows

$$\left(\frac{S}{S_o}\right)^2 = \left(\frac{t}{t_o}\right)^2 = \frac{p}{p_o} = \frac{p_o - \sigma_v}{p_o}$$

Substitution of these results in the Carman-Kozeny equation gives

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

Still supposing λ to be constant at λ_o, the value of σ_v is given by

$$\sigma_v = \frac{\sigma}{\rho_d} = \frac{v \lambda_o c_o}{\rho_d} e^{-\lambda_o y} t \qquad \text{and with}$$

$$\alpha = \frac{v \lambda_o c_o}{\rho_d \, p_o}$$

$$\sigma_v = \alpha\, p_o\, t\, e^{-\lambda_o y}$$

Substituted

$$\left(\frac{dz}{dy}\right)_t = I_o \frac{1}{(1-\alpha t\, e^{-\lambda_o y})^2}$$

This gives as head loss over the full depth L of the filterbed at time t

$$H = \int_0^L \left(\frac{dz}{dy}\right)_t dy = \int_0^L I_o \frac{dy}{(1-\alpha t e^{-\lambda_o y})^2} \qquad \text{or}$$

$$H = \frac{I_o}{\lambda_o} \left\{ \frac{\alpha t}{1-\alpha t}\, \frac{e^{\lambda_o L}-1}{e^{\lambda_o L}-\alpha t} + \ln\left(\frac{e^{\lambda_o L}-\alpha t}{1-\alpha t}\right) \right\}$$

Assuming for instance $L = 0.75$ m, $d = 0.8$ mm $= (0.8)10^{-3}$ m, $p_o = 0.4$, $v = (2)10^{-3}$ m/sec, $c_o = 15$ g/m^3 $= (15)10^{-3}$ kg/m^3, $t = 10^\circ$C, $\nu = (1.31)10^{-6}$ m^2/sec, $\rho_d = 50$ kg/m^3 and λ_o equal to 6 m^{-1}, the slope I_o at $t = 0$ equals

$$I_o = \frac{(180)(1.31)10^{-6}}{9.81}\; \frac{(0.6)^2}{(0.4)^3}\; \frac{(2)10^{-3}}{(0.8)^2\, 10^{-6}} = 0.423$$

With

$$\alpha = \frac{(2)10^{-3}\,(6)(15)10^{-3}}{(50)(0.4)} = (9)10^{-6}\ \text{sec}^{-1} \qquad \text{and}$$

$$e^{\lambda_o L} = e^{(6)(0.75)} = e^{4.5} = 90.0$$

the resistance of the filterbed after time t equals

$$H = \frac{0.423}{6} \left\{ \frac{(9)10^{-6}t}{1-(9)10^{-6}t}\; \frac{89}{90-(9)10^{-6}t} + \ln \frac{90-(9)10^{-6}t}{1-(9)10^{-6}t} \right\}$$

simplified

$$H = 0.070 \left\{ \frac{t}{(1.11)10^5 - t} \quad \frac{(99)10^5}{(100)10^5 - t} + \ln \frac{(100)10^5 - t}{(1.11)10^5 - t} \right\}$$

This gives as increase of filterresistance with time

t = 0	0.25	0.5	0.75	1.0	1.11	x 10^5 sec
H = 0.32	0.35	0.41	0.55	1.11	∞	m

According to the graphical representation of fig. 2.10, the filter resistance first increases slowly with time, but later on at ever increasing rapidity. This is due to the circumstance that the clogging of the pores

$$\sigma_v = \alpha p_o \, t \, e^{-\lambda_o y} = (9)10^{-6}(0.4) \, t \, e^{-6y} = (3.6)10^{-6} t \, e^{-6y}$$

mainly occurs at the top of the filterbed (fig. 2.11). At the end of the filterrun at

$$T = (1.11)10^5 \text{ sec}$$

the amount of deposition at the top of the filterbed (y = 0) equals

$$\sigma_v = (3.6)10^{-6}(1.11)10^5(1) = 0.40$$

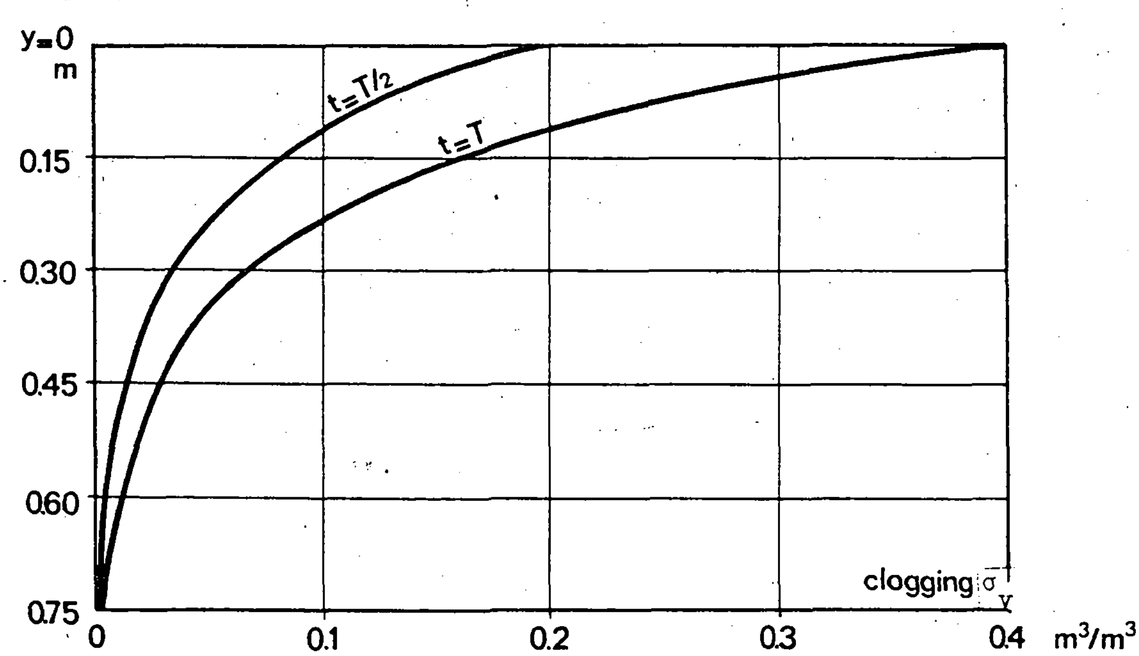


Fig. 2.11 Clogging of the pores for the filterrun of fig. 2.10.

reducing the original pore space $p_o = 0.40$ to zero and causing an infinite resistance. Even at t = T, however, the average amount of deposition is still quite small

$$\bar{\sigma}_v = \frac{1}{L}\int_o^L \sigma_v dy = \frac{1}{L}\int_o^L \alpha p_o T e^{-\lambda_o y}\, dy = \frac{\alpha p_o T}{L\lambda_o}\left(1 - e^{-\lambda_o L}\right)$$

$$\bar{\sigma}_v = \frac{(9)10^{-6}(0.4)(1.11)10^5}{(0.75)(6)}\left(1 - e^{-(6)(0.75)}\right) = 0.088 \text{ m}^3/\text{m}^3 \text{ and}$$

$$\bar{\sigma} = \rho_d \sigma_v = (50)(0.088) = 4.4 \text{ kg/m}^3$$

The latter value may also be calculated from the reduction in the suspended load carried by the water. With c_o and c_a constant

$$\sigma = \frac{(c_o - c_e)vT}{L} = \frac{(15 - 0.17)10^{-3}(2)10^{-3}(1.11)10^5}{0.75} = 4.4 \text{ kg/m}^3$$

2.4 Mathematical theories of filtration

The filtration results obtained in the preceding section and shown graphically in fig. 2.10 are clearly at fault. In actual practice a constant effluent quality is never encountered, neither such a rapid increase of filter resistance with time. For the larger part these deviations between theory and practice are due to the erroneous assumption of a constant value λ_o for the coefficient of filtration. In reality λ will decrease with time, allowing a larger part of the impurities in the raw water to travel to greater depths in the filterbed. This means deep bed filtration, accompanied by a decrease of effluent quality and a lower increase of filter resistance with time. As second drawback of the calculations in the preceding section must be mentioned that no indication has been given about the interrelation between λ and operational factors such as filterbed thickness and grain size distribution, rate of filtration, raw water quality, etc.

During the last decades, many investigators all over the world have tried to establish the two relationships governing filtration efficiency

$$\lambda_o = f(L, d_o, p_o, v, c_o, \nu, \rho_d, \text{etc}), \qquad \lambda = \lambda_o f(t)$$

The first interrelation depends on the transport and removal mechanisms mainly responsible for purification. The mathematical model and its results will consequently vary strongly whether straining, sedimentation, diffusion, van der Waals' or Coulomb forces, etc, are thought to be the prime factors. The second relationship is even more complicated. During the first part of the filterrun, λ will grow as for instance the efficiency of straining increases by deposition of settled out material, thus improving effluent quality. As filtration goes on, however, and deposits constrict the pores of the filterbed, increasing the interstitial velocity of flow, scouring will take place, reducing for instance settling efficiency and lowering the value of λ. With the large number of variables involved, it will not astonish that great differences of opinion between the various research workers exist and that such widely divergent filtration theories have emerged.
As results of the most important theories may be mentioned

Ison $\lambda_o \sim \frac{\nu^{1.4} e^{0.3}}{d_o^{1.4} v^4}$

with e as suspended particle size

Ives $\lambda_o \sim \frac{1}{d_o^{n} \nu^2 v}$ $\lambda = \lambda_o \left(1 + \beta_1 \frac{\sigma_v}{p_o} - \beta_2 \frac{\sigma_v^2}{p_o(p_o - \sigma_v)}\right)$

with n (between 1 and 3) and β as experimental constants

Iwasaki $\lambda = \lambda_o \left(1 - \beta \frac{v}{p_o} \right)$

Lerk $\lambda_o \sim \frac{(1 - p_o)p_o}{d_o^3 \nu v}$ $\lambda = \lambda_o \left(1 - \frac{\sigma_v}{p_o} \right)$

Mackrle $\lambda = \lambda_o \left(1 + \beta \frac{\sigma_v}{p_o}\right)^{n_1} \left(1 - \frac{\sigma_v}{p_o}\right)^{n_2}$

with β, n_1 and n_2 constants to be determined by experiment

Maroudas $\lambda = \lambda_o \left(1 - \frac{\sigma_v}{\sigma_v^*} \right)$

with σ_v^* as maximum possible accumulation of deposits, again to be determined experimentally

Mintz $\lambda_o \sim \frac{v}{d_o}$ $\lambda = \lambda_o \left(1 - \frac{c_o}{c} \frac{\sigma_v}{\sigma_v^*} \right)$

Mintz/Lerk $\lambda = \lambda_o \left(1 - \frac{\sigma_v}{\sigma_v^*} \right)$

Shekhtman/Lerk $\lambda = \lambda_o \left(1 - \frac{\sigma_v}{p_o} \right)$

Sholje $\lambda_o \sim \frac{1}{\nu^2 v}$

Most theories do not allow an analytical solution and require a computer to make the necessary calculations. When moreover a number of variables have to be determined by experiment, the advantage compared with the empirical approach of the preceding section is only slight. In this section a rather simple theory will therefore be used, taking the value of λ_o from Lerk's theory

$$\lambda_o \sim \frac{(1 - p_o)p_o}{d_o^3 \nu v}$$

and the value of λ from Iwasaki and Maroudas

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

with n smaller than unity. With these assumptions the basic equations of filtration become

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

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

resistance $$H = \int_o^L I_o \left(\frac{p_o}{p_o - \sigma_v}\right)^2 dy$$

With the boundary conditions

$$y = 0 \qquad c = c_o$$
$$t = 0 \qquad \sigma_v = 0$$

and $$\alpha = \frac{v\, c_o \lambda_o}{n\, \rho_d p_o}$$

these equations have as solution

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

$$\sigma_v = n\, p_o \frac{e^{\alpha t} - 1}{e^{\lambda_o y} + e^{\alpha t} - 1}, \qquad \bar{\sigma}_v = n p_o \left(1 - \frac{1}{\lambda_o L} \ln \frac{e^{\lambda_o L} + e^{\alpha t} - 1}{e^{\alpha t}}\right)$$

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

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

and for n arbitrarely chosen at 0.75

$$H = \frac{I_o}{\lambda_o} \left\{ 16\, \lambda_o L - 36 \frac{(e^{\lambda_o L} - 1)(e^{\alpha t} - 1)}{(4e^{\lambda_o L} + e^{\alpha t} - 1)(e^{\alpha t} + 3)} - 15 \ln \frac{4e^{\lambda_o L} + e^{\alpha t} - 1}{e^{\alpha t} + 3} \right\}$$

With the same assumptions as made in the preceding section

$L = 0.75$ m , $d = 0.8$ mm $= (0.8)10^{-3}$ m, $p_o = 0.4$, $v = (2)10^{-3}$ m/sec,
$c_o = 15$ g/m^3 $= (15)10^{-3}$ kg/m^3 , $t = 10\ ^0C$, $\nu = (1.31)10^{-6}$ m^2/sec
$\lambda_o = 6$ m^{-1} , $\rho_d = 50$ kg/m^3 , $I_o = 0.423$ and

$$\alpha = \frac{v\ c_o\ \lambda_o}{n\ \rho_d\ p_o} = \frac{(2)10^{-3}(15)10^{-3}(6)}{(0.75)(50)(0.4)} = (12)10^{-6}\ \text{sec}^{-1}$$

the filtration results now become

t = 0	0.5	1.0	1.5	2.0	2.5	3.0 x 10^5	sec
c_e = 0.17	0.30	0.54	0.96	1.65	2.77	4.37	g/m^3
$\bar{\sigma}_v$ = 0.000	0.039	0.078	0.116	0.153	0.187	0.218	m^3/m^3
H = 0.32	0.41	0.57	0.82	1.18	1.62	2.13	m

and are shown graphically in fig. 2.12. Comparing these results with those of fig. 2.10 shows that at the beginning of the filterrun the conditions are exactly the same. Later on, however, great deviations occur, the effluent quality deteriorating with time, while the increase of filter resistance with time is much less pronounced. According to fig. 2.13 impurities from the raw water moreover are deposited to much greater depths in the filterbed.

When for the filtration results of fig. 2.12 the effluent standard is set at a suspended solids content of 0.5 g/m^3 and the maximum allowable filter resistance at 1.5 m, the lengths of filterrun equal

$$T_q = (0.93)10^5 \text{ sec} = 26 \text{ hours} \qquad T_r = (2.36)10^5 \text{ sec} = 64 \text{ hours}$$

These values could be brought closer together by the use of a finer grained filtering material. According to the theory at hand

$$\lambda_o \sim \frac{(1-p_o)p_o}{d_o^{\ 3}\quad \nu\ v} \qquad \alpha = \frac{v\ c_o\ \lambda_o}{n\ \rho_d\ p_o} \qquad I_o \sim \frac{(1-p_o)^2}{p_o^{\ 3}}\frac{v}{d_o^{\ 2}}$$

Lowering the grain size d_o from 0.8 to 0.7 mm changes these parameters to

$$\lambda_o' = \left(\frac{0.8}{0.7}\right)^3\ 6 = 8.956\ \text{m}^{-1}$$

$$\alpha' = \left(\frac{0.8}{0.7}\right)^3(12)10^{-6} = (17.913)10^{-6}\ \text{sec}^{-1}$$

$$I_o' = \left(\frac{0.8}{0.7}\right)^2(0.423) = 0.552$$

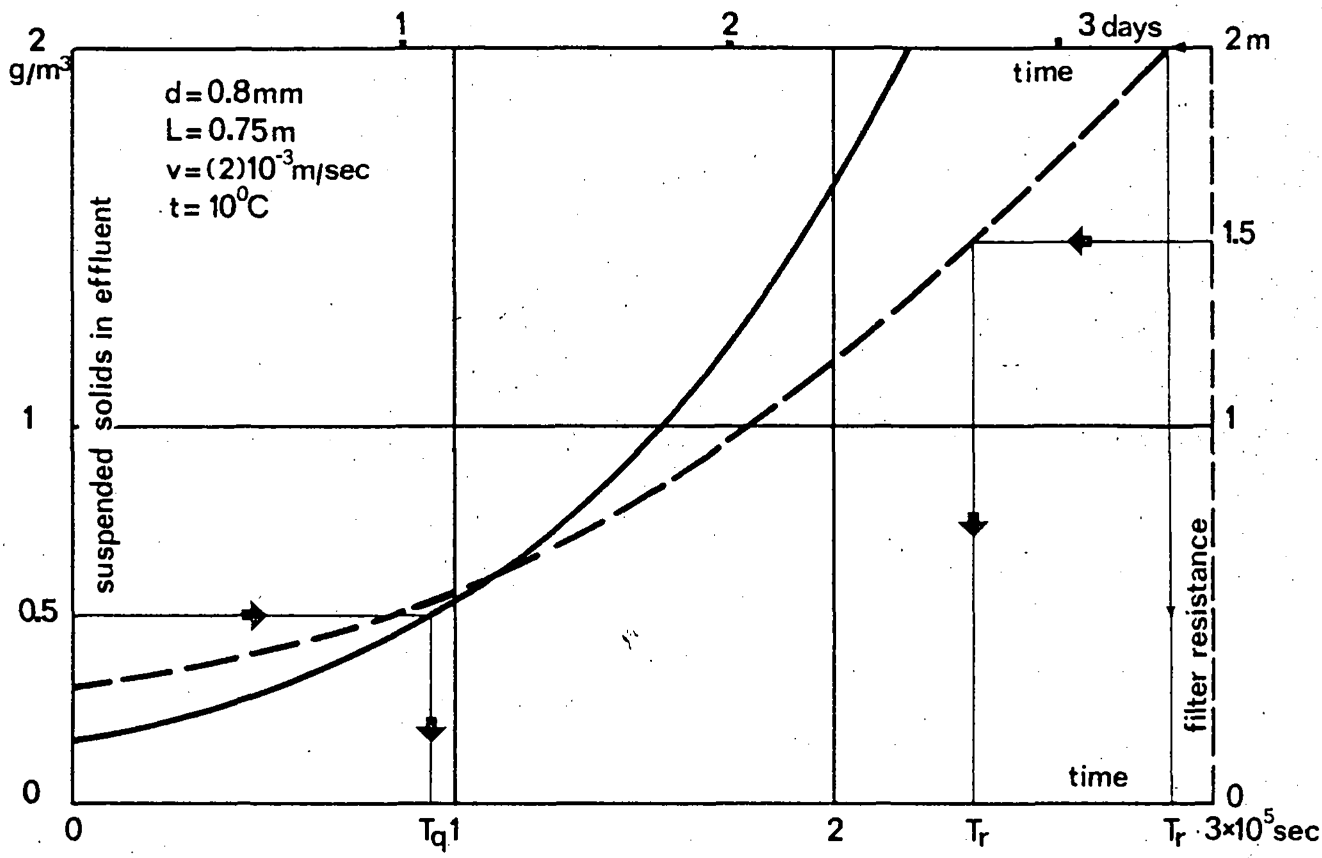


Fig. 2.12 Results of a calculated filterrun going out from Lerk's modified filtration theory.

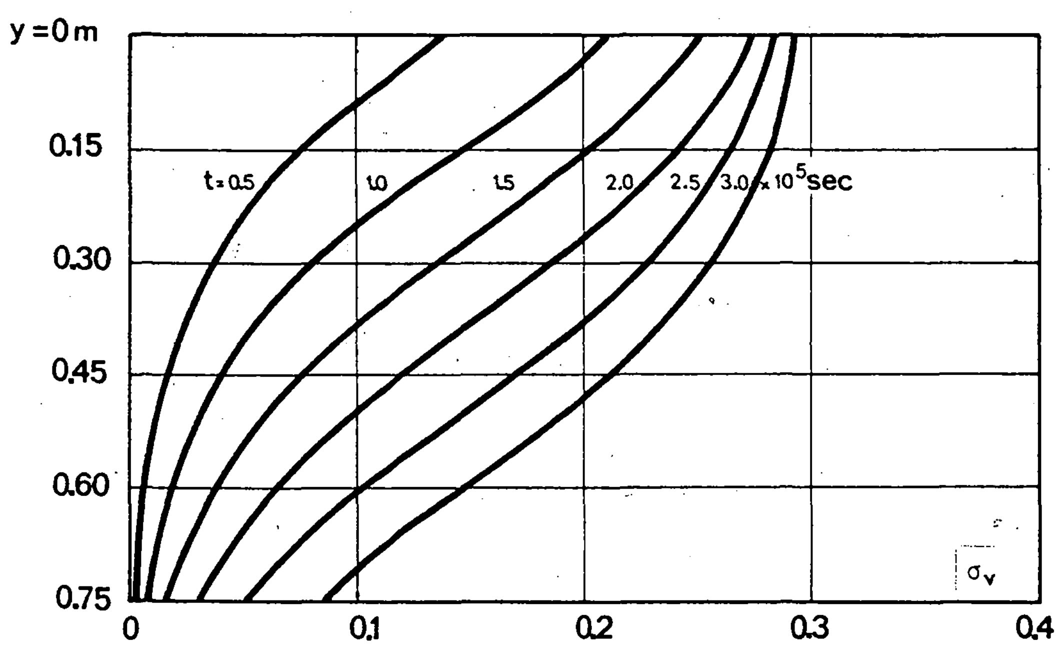


Fig. 2.13 Clogging of the pores for the filterrun of fig. 2.12.

With the same filterbed thickness L = 0.75 m and filtration rate v = $(2)10^{-3}$ m/sec, the filtration results are now

t	= 0	0.5	1.0	1.5	2.0	2.5	3.0 × 10^{-5} sec
c_e	= 0.02	0.04	0.11	0.26	0.63	1.45	3.11 g/m^3
H	= 0.41	0.55	0.86	1.36	2.02	2.76	3.55 m

According to the graphical representation of these results in fig. 2.14, the lengths of filterrun equal

$$T_q = (1.85)10^5 \text{ sec} = 51 \text{ hours}, \quad T_r = (1.62)10^5 \text{ sec} = 45 \text{ hours}$$

This satisfies all practical requirements, but considered by itself, the length of filterrun is rather long. A higher rate of filtration could thus be applied, allowing a smaller filterbed area and a subsequent saving in the cost of construction. In its turn, however, such a higher rate of filtration might require a coarser grained filtering material and a greater bed thickness, the latter factor increasing the cost of construction. With the mathematical theory of filtration, this optimization

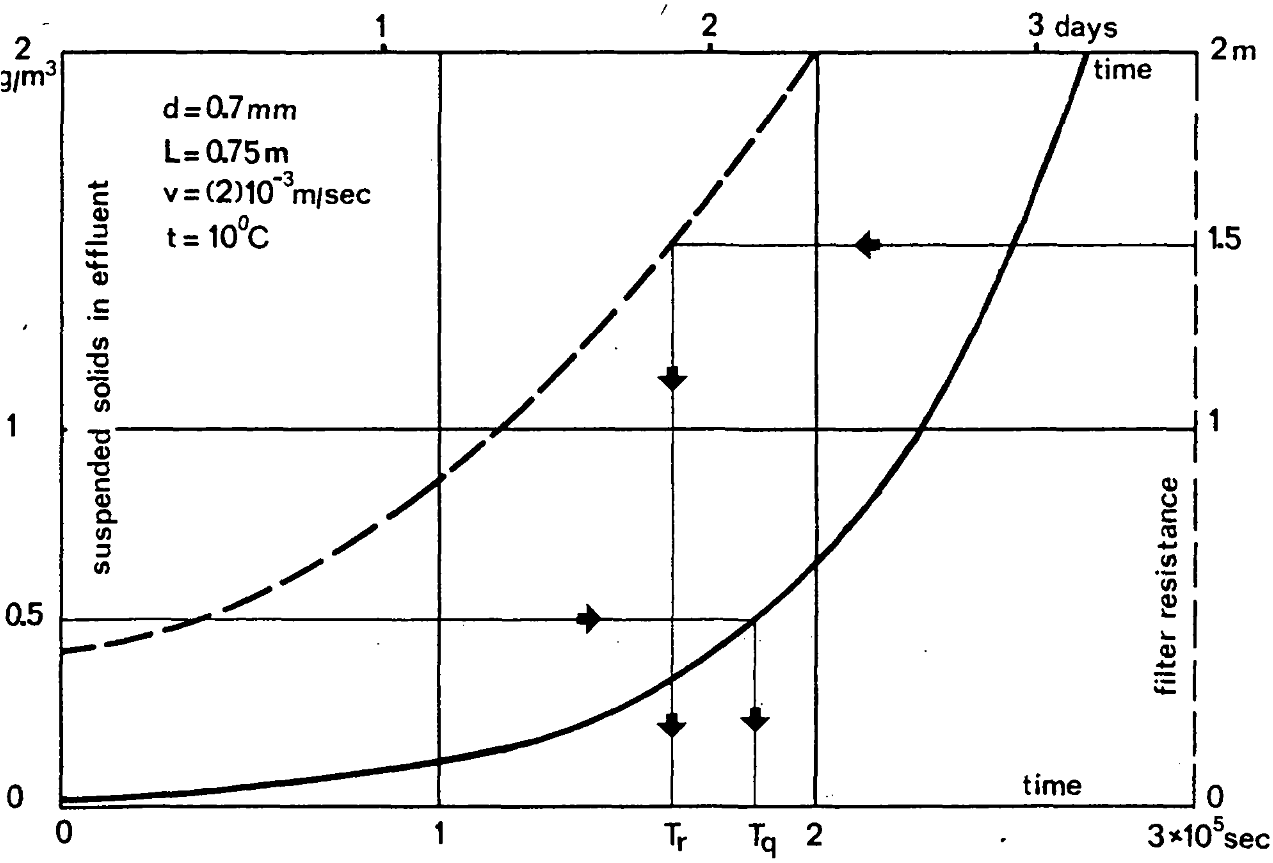


Fig. 2.14 Filtration results of fig. 2.12 recalculated for a grainsize of 0.7 in stead of 0.8 mm.

problem can easily be solved, provided that the desired results of the filtration process in terms of effluent quality and length of filterrun are chosen in advance.

Quality requirements depend on the subsequent use that is made of the effluent and will vary from one case to another. The length of filter-run on the other hand is more or less standard, equal to about 1 day or $(0.9)10^5$ sec. Care, however, must be exercised to assure that the maximum allowable filter resistance is reached before effluent quality deteriorates beyond the chosen valve. Going out from the filtration parameters as used in fig. 2.12 and 2.14 and setting as standards

$$c_e < 0.5 \text{ g/m}^3, \qquad T_r = (0.9)10^5 \text{ sec}, \qquad T_q = (1)10^5 \text{ sec}$$

all possible combinations are shown in fig. 2.15. The cost of construction will be more or less proportional to the volume of the filterbox, that is to say to its area and to its depth. The area of the filterbox is inverse proportional to the filtration rate v, while to prevent negative heads (section 2.7) and to accomodate the filterbottom (section 4.5) the depth will roughly equal (0.3 L + H + 1)m. This means that the most economical construction is obtained when the optimization factor

$F_o \sim \dfrac{0.3\ L + H + 1}{v}$ reaches its minimum value. According to the table below

	v = 2	2.5	3	3.5	4 x 10^{-3} m/sec
d = 0.7 mm	F_o = 100	98	100	104	110
0.8	96	92	92	95	98
0.9	96	91	90	92	94
1.0	99	95	94	94	97

this is the case when the raw water under consideration is filtered at a rate of $(3)10^{-3}$ m/sec through material with a grain size of 0.9 mm in a bed thickness (fig. 2.15) of 1.5 m and with a maximum allowable filter resistance of nearly 1.1 m. Without an appreciable loss in economy, however, also other combinations can be used, a grain size of 0.8 mm and rates of $(2.5)10^{-3}$ and $(3.0)10^{-3}$ m/sec or a grain size

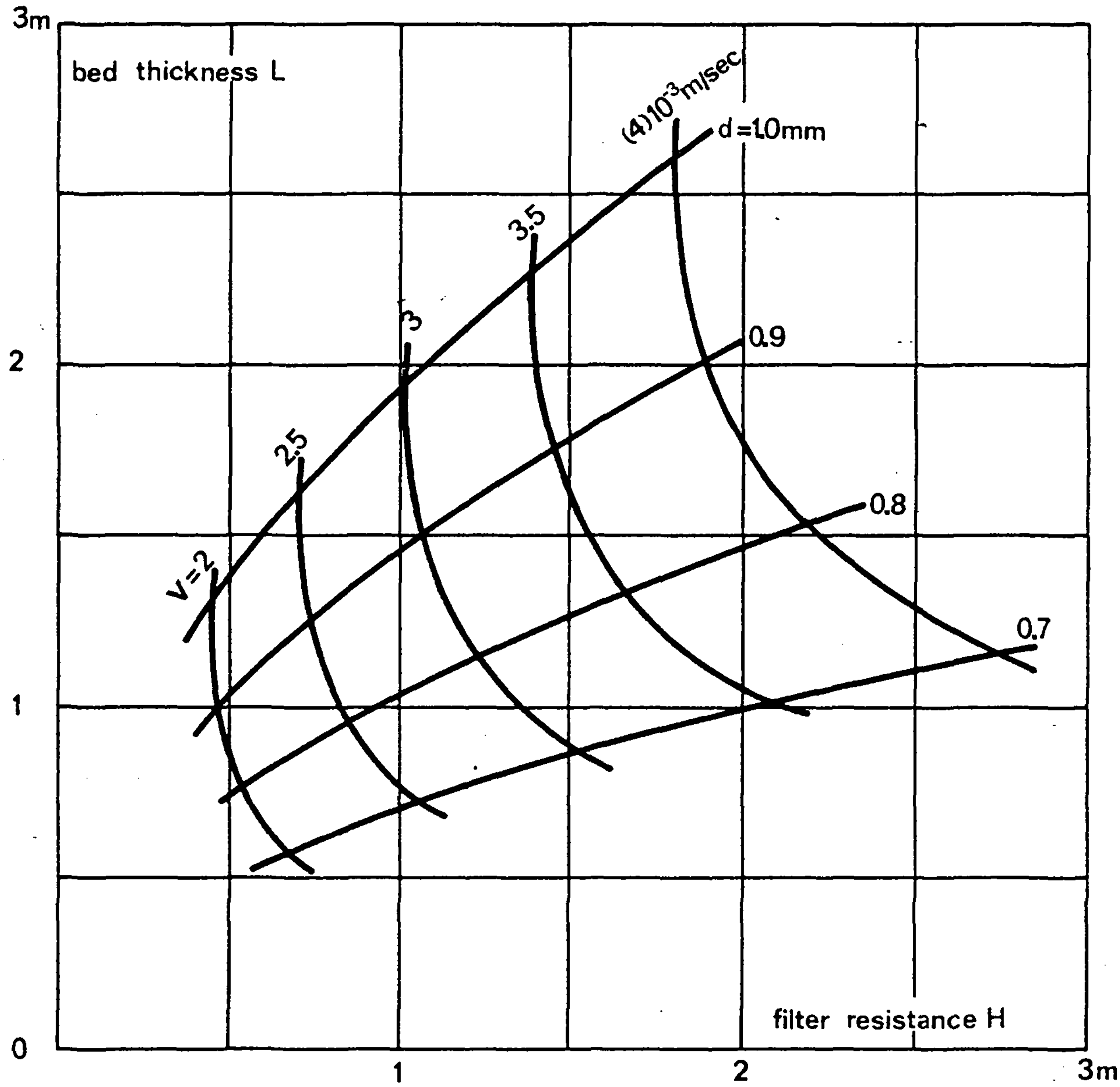


Fig. 2.15 Possibilities for rapid filtration of the water dealt with in fig. 2.12 , 2.13 and 2.14 at 10°C, going out from the requirements $c_e < 0.5$ gram/m^3, $T_r=(0.9)\ 10^5$sec , $T_q=(1.0)\ 10^5$sec.

of 0.9 mm and rates between $(2.5)10^{-3}$ and $(3.5)10^{-3}$ m/sec. Compared with the set-up of fig. 2.14 with d = 0.7 mm and v = $(2)10^{-3}$ m/sec, the cost of construction can thus be reduced by about 10%. This saving is interesting, but not very impressive.

A rapid filtration plant serving a public supply will always consist of a number of filtering units. In the clear well, the effluent from the various units is mixed and the decisive factor is consequently the quality of the mixed effluent. When a larger number of filtering units are present, this mixed quality is constant, equal to the average quality over the full length of the filterrun. With effluent quality changing with time according to

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

this average quality equals

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

in which T is the length of filterrun. During this run, the effluent quality of a single unit varies from the minimum value

$$t = 0, \quad c_{min} = c_o \frac{1}{e^{\lambda_o L}}$$

to the maximum value

$$t = T, \quad c_{max} = c_o \frac{e^{\alpha T}}{e^{\alpha T} + e^{\lambda_o L} - 1}$$

giving with good approximation as average effluent quality

$$c_a = \frac{c_{max} - c_{min}}{\ln \frac{c_{max}}{c_{min}}}$$

According to fig. 2.14, $T = T_r = (1.62)10^5$ sec and $c_{min} = 0.02$ g/m^3, $c_{max} = 0.33$ g/m^3 from which follows

$$c_a = \frac{0.33 - 0.02}{\ln \frac{0.33}{0.02}} = 0.11 \text{ g/m}^3$$

or only 22% of the maximum allowable value assumed at 0.5 g/m^3.

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. In this respect it is good to realize that the assumptions underlying the mathematical theory of filtration are highly speculative, while with other assumptions completely different results will be obtained. Just to mention one example, attention may be drawn to fig. 2.14 where the results of fig. 2.12 have been recalculated for a grain size of 0.7 instead of 0.8 mm. This recalculation has been based on Lerk's adsorbtion theory, according to which the filtration coefficient λ_o is inverse proportional to the third power of the grain size. Assuming sedimentation to be the prime factor in purification on the other hand, would make λ_o inverse proportional to the first power of the grain size, while according to Ives with a combination of both processes λ_o is inverse proportional to a power of the grainsize somewhere between 1 and 3. The power chosen, however, has a large influence on filtration results, as shown in the table below

	t =	0	0.5	1.0	1.5	2.0	2.5	3.0	x 10^5 sec
n = 3	c_e =	0.02	0.04	0.11	0.26	0.63	1.45	3.11	g/m^3
2		0.04	0.09	0.20	0.43	0.91	1.86	3.54	
1		0.09	0.17	0.34	0.66	1.25	2.30	3.97	
n = 3	H =	0.41	0.55	0.86	1.36	2.02	2.76	3.55	m
2		0.41	0.54	0.81	1.25	1.85	2.55	3.30	
1		0.41	0.54	0.77	1.15	1.69	2.33	3.04	

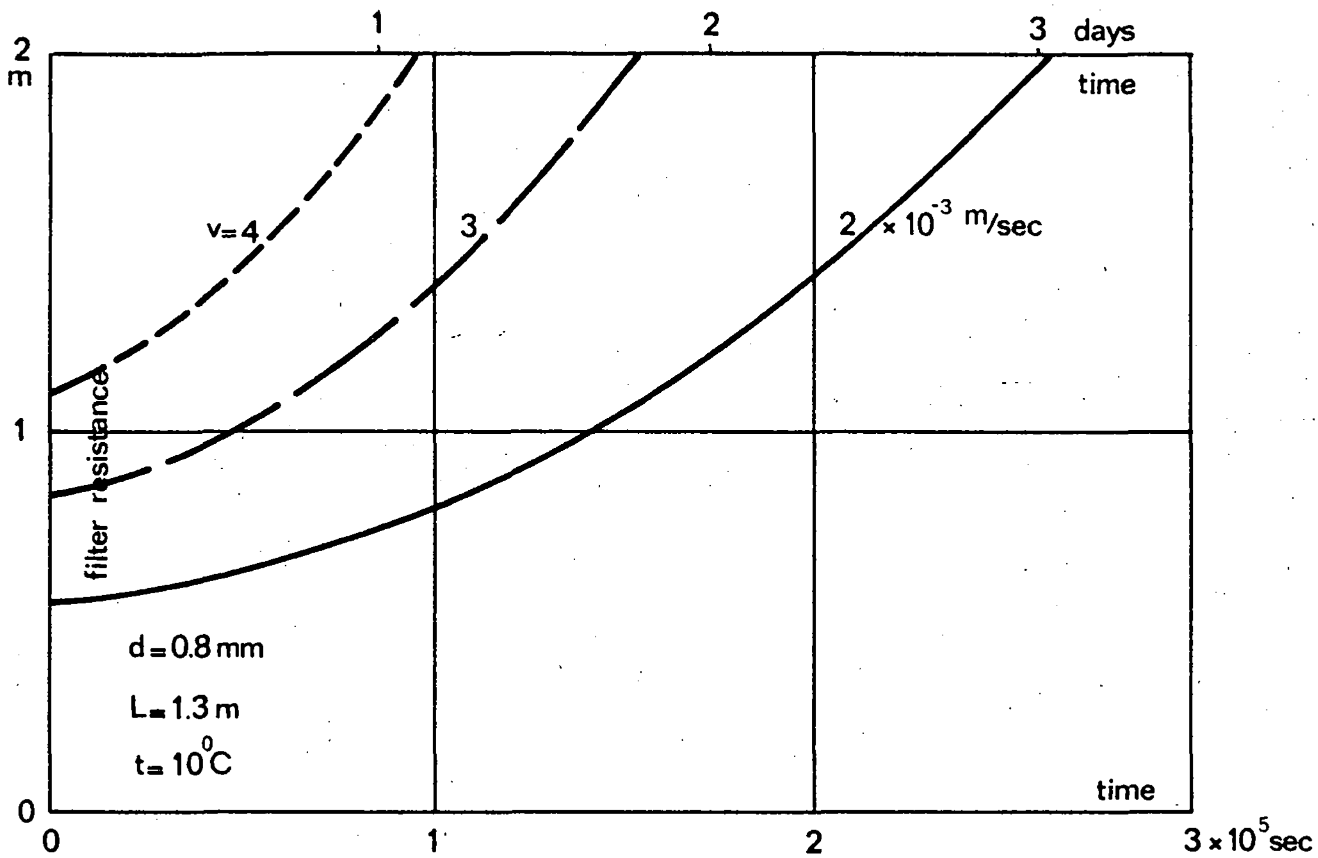
From n = 3 to n = 1, effluent quality deteriorates by a factor 1.5 to 2, while filter resistance drops by a factor 1.1 to 1.2. As a consequence, optimum filter design in terms of filtration rate, thickness and grain size distribution of the filterbed and length of filterrun will depend strongly on the value selected for n. Which value must be chosen in a specified case, however, is still and will long remain unknown, reducing the mathematical theory of filtration more or less to an elaborate, impressive, but not so very reliable calculation exercise. Even when in future the mechanisms of filtration are fully understood, experiments or practical experience remain necessary to determine the parameters of filtration, the ever changing value of factors such as λ_o, α, ρ_d and so on.

Summing up, it must be said that the main value of the mathematical theory of filtration is in improving the insight, in helping to understand this complicated process. In practice it allows slight extrapolation of observed data, predicting for instance the effect of a slightly smaller grain size on the length of filterrun T_q and especially T_r. The mathematical theory of filtration is of great help when carrying out filtration tests in a pilot plant, indicating what type of experimental model should be used under the prevailing conditions, which observations are meaningful and should be made, in which direction research can be applied to yield results of practical value (for instance recycling as a means to improve effluent quality), and so on. For the filter design proper it is still unsuited and as before, this must be based on practical experience and for larger installations on the results of a pilot plant. Notwithstanding all efforts, filtration is still more an art than a science.

2.5. Changes in operating conditions

In the preceding section constant operating conditions have been assumed. In reality, however, these conditions change continuously, due to variations in the physical and chemical characteristics of the water to be treated and due to variations in water demand, necessitating an adjustment of the filtration rate.

Deep groundwater has a constant temperature and a constant chemical composition, meaning that the filtration rate is the only variable in the day-to-day operation of the plant. With public water supplies and an adequate amount of clear water storage, the plant load usually varies between 70 and 140% of average daily demand, requiring a variation in filtration rate by a factor 2, for instance between $(2)10^{-3}$ and $(4)10^{-3}$ m/sec. Assuming the same data as applied before and taking into account the information supplied by fig. 2.15, a grain size of 0.8 mm and a bed thickness of 1.3 m could be chosen for this particular case. Filterruns for various filtration rates can now be calculated (fig. 2.16) from which after choosing the maximum allowable effluent quality and the maximum allowable filter resistance, the lengths of filterrun T_q and T_r may be deduced. According to the graphical representation of fig. 2.17, the variation in filtration rate (dotted line) is accompanied by a much stronger variation in the lengths of filterruns. At the average rate of filtration, $v = (3)10^{-3}$ m/sec, these lengths equal $T_q = 42$ hours for $c_e < 0.5$ g/m^3 and $T_r = 31$ hours for $H < 1.5$ m. This is quite satisfactory the decisive length of filterrun (T_r) being a little over one day while at the end of the run effluent quality is still quite good, according to the fig. 2.16 a value of c_e equal to 0.32 g/m^3 or only 64% of the maximum allowable value. During periods of below average demand, $v < (3)10^{-3}$ m/sec, the lengths of filterrun are larger, but under all circumstances $T_r < T_q$ while the maximum value of 58 hours or a little below 2.5 days is not excessive. Difficulties seem to arise when the filtration rate is above normal. The lengths of filterrun are now rather short with minimum values for T_r and T_q of 16 and 12.5 hours respectively and above all T_q drops below T_r, meaning an unacceptable lowering of effluent quality during the last part of the filterrun, before the rise in filter resistance makes cleaning of the filterbed necessary. It must be realised, however, that peak demands are rare, the maximum anticipated value, corresponding in this case with $v = (4)10^{-3}$ m/sec, occuring only once every 10 to 100 years. Under such extreme conditions back-washing a filter twice a day



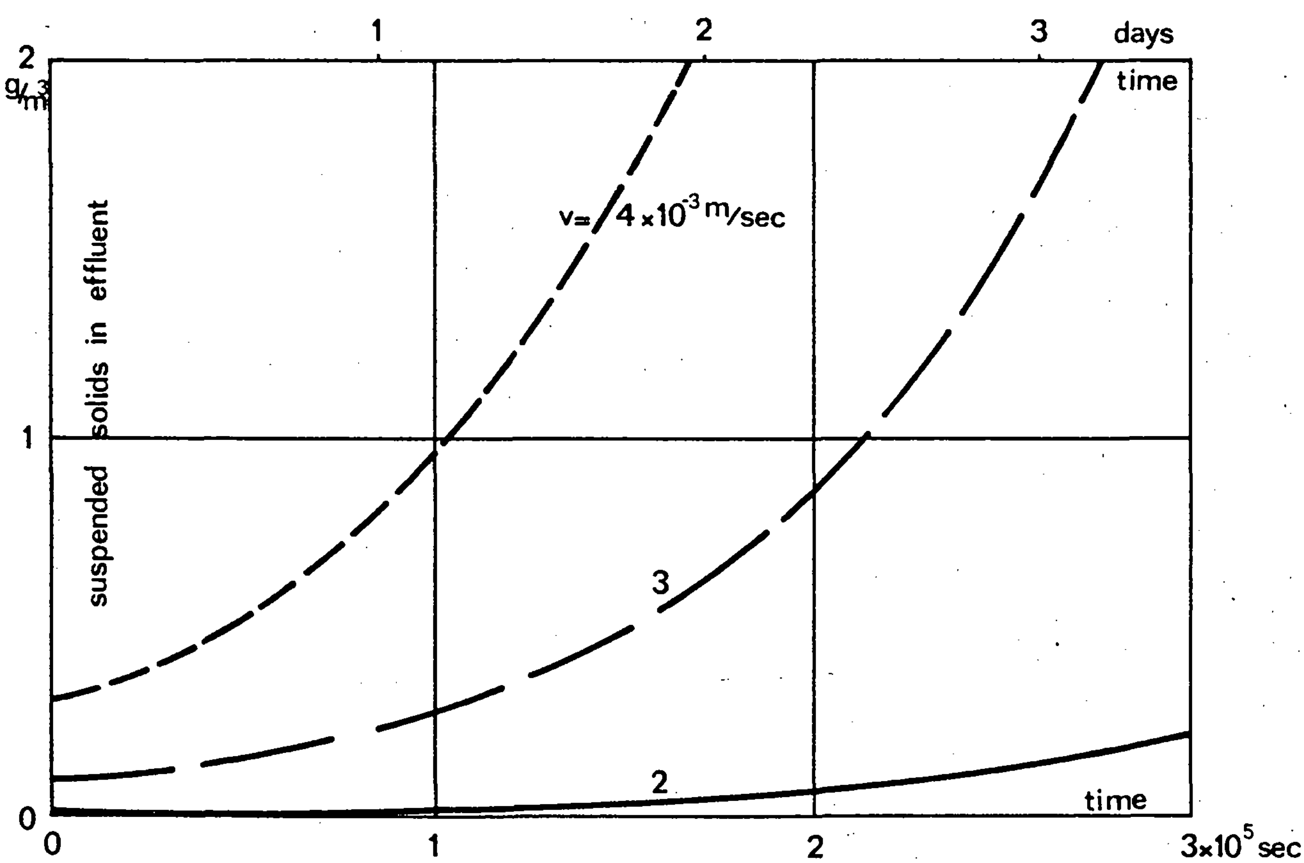


Fig. 2.16 Influence of variatien in filtration rate.

is not objectionable. With regard to the feared for deterioration of effluent quality, according to fig. 2.16 a suspended solids content increasing from 0.31 to 0.56 g/m^3 or from 62 to 112% of the maximum allowable value, it may be recalled that in section 2.4 the quality as average over the whole length of filterrun has been calculated at

$$C_a = \frac{c_{max} - c_{min}}{\ln \frac{c_{max}}{c_{min}}}, \qquad \text{in this case}$$

$$c_a = \frac{0.56 - 0.31}{\ln \frac{0.56}{0.31}} = \frac{0.25}{0.59} = 0.42 \text{ g/m}^3$$

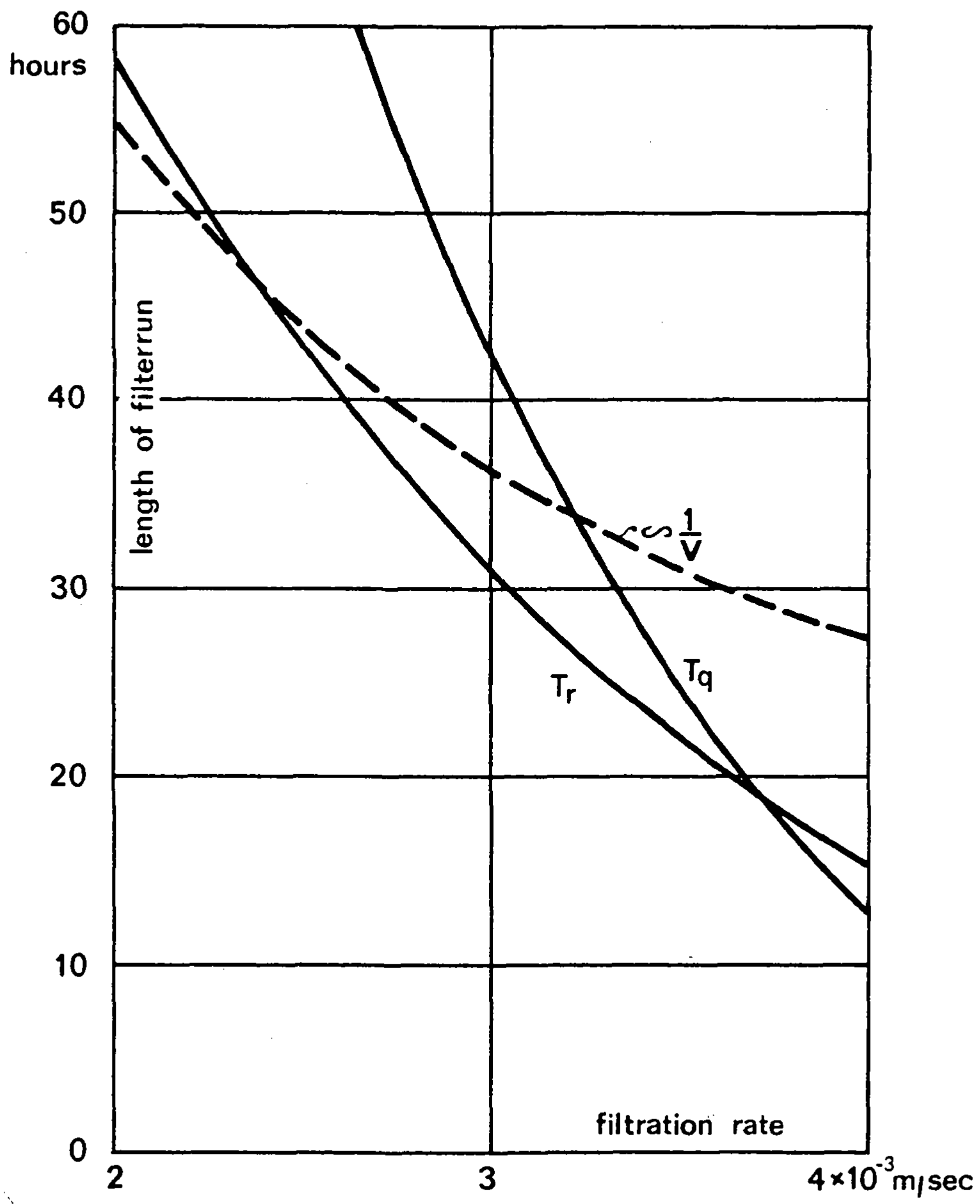


Fig. 2.17 Length of filterrun calculated from fig. 2.16 for c_e < 0.5 g/m^3 and H < 1.5 m.

This value is quite acceptable and may be expected as the (constant) quality of the mixed effluent when a larger number of filtering units are present.

In moderate climates, even surface water of the most constant quality will show a variation in water temperature. According to the table below

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

this means a strong variation in the value of the kinematic viscosity ν, changing the value of the filtration coefficient λ_o and the magnitude of the filter resistance for a clean bed. Going out from the same data as mentioned above, the subsequent variations in the lengths of filterrun T_q and T_r may again be calculated. According to fig. 2.18, the change in T_r is smaller and in T_q stronger than that of ν with temperature. In this particular case all requirements are satisfied as long as the water temperature is above 3 ^0C. Below this value two objections could again be made, $T_q < T_r$ and both values less than one day. During the time T_r, however, effluent quality has been found to vary from 0.34 to 0.61 g/m^3 (not shown) giving as average value

$$c_a = \frac{0.61 - 0.34}{\ln \frac{0.61}{0.34}} = \frac{0.27}{0.58} = 0.46 \text{ g/m}^3$$

This is still acceptable when a larger number of filtering units are present, while the minimum value of T_r at 20 hours is even quite good.

For the same data as used above, fig. 2.19 finally shows the variation in lengths of filterrun with the suspended solids content c_o in the raw water to be treated. For all practical purposes T_r is now inverse proportional to c_o, while the drop in T_q is stronger than corresponds with the rise in c_o. Taking T_r as the deciding factor in the filtration process, gives no difficulties in this particular case as long as $c_o < 24$ g/m^3. For $c_o = 30$ g/m^3, T_r drops to 16 hours, a quite acceptable value, giving a variation in effluent quality (not shown) from 0.17 to 0.63 g/m and

$$c_a = \frac{0.63 - 0.17}{\ln \frac{0.63}{0.17}} = \frac{0.46}{1.31} = 0.35 \text{ g/m}$$

which is even quite good.

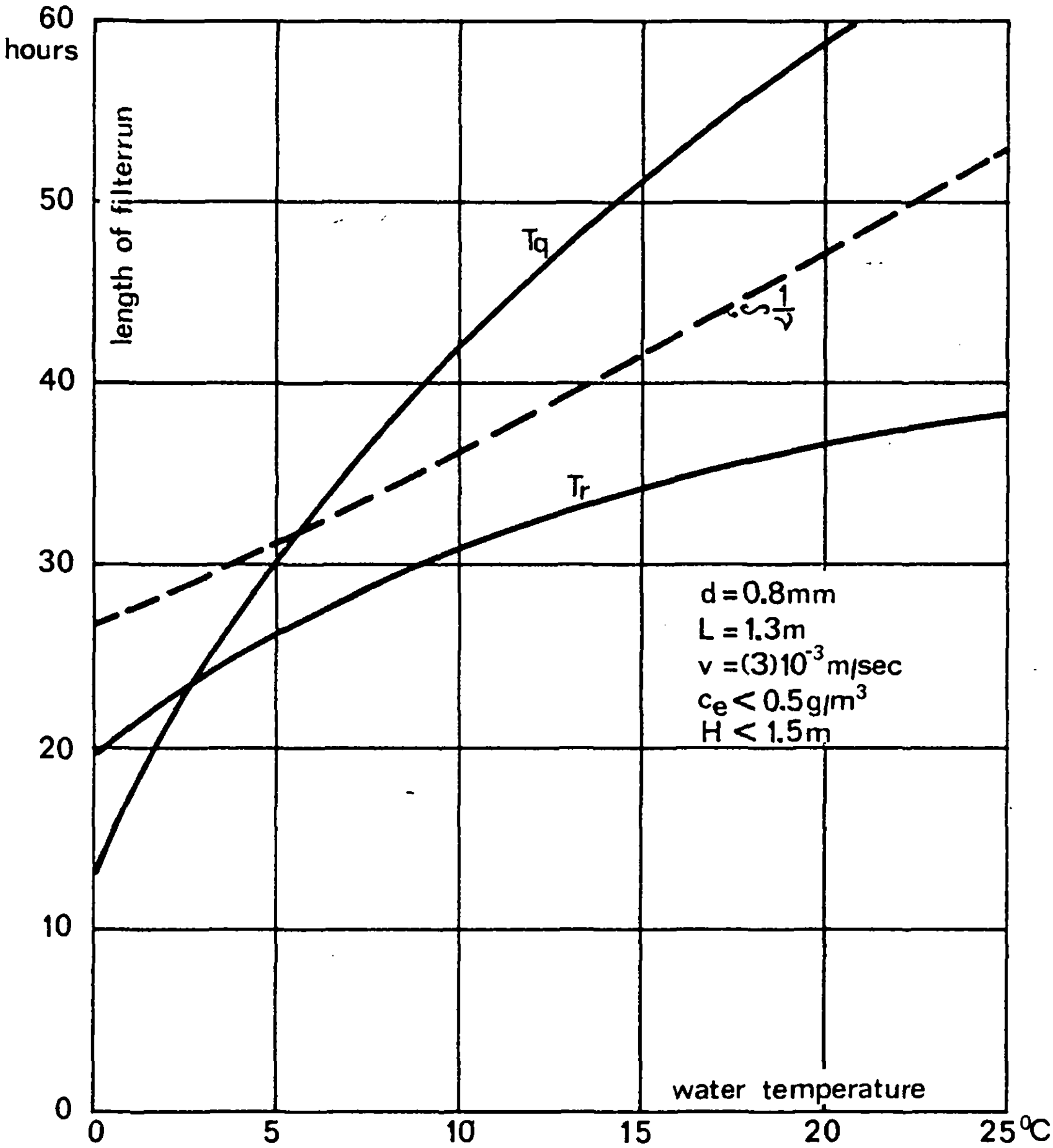


Fig. 2.18 Length of filterrun as functie of water temperature.

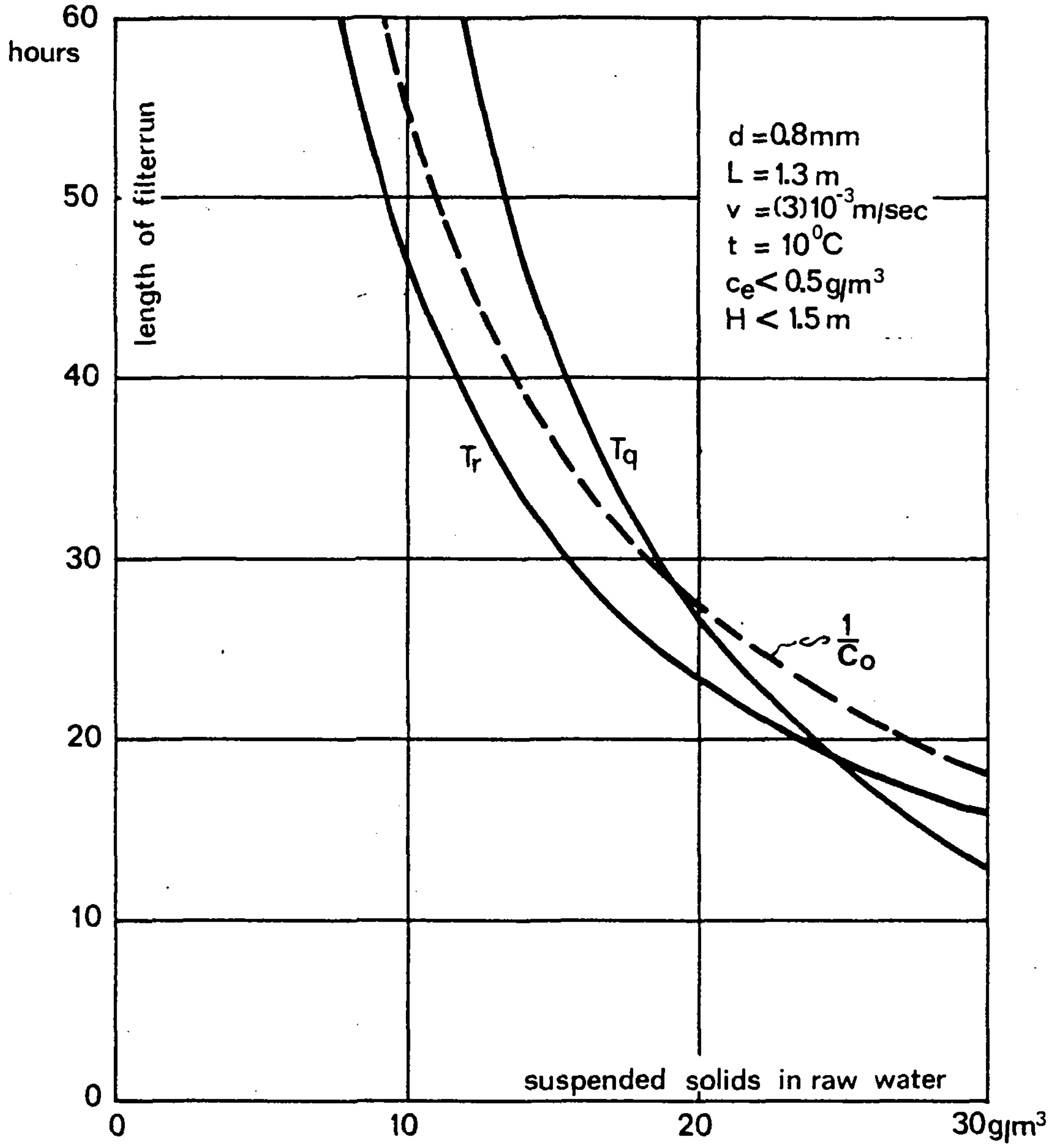


Fig. 2.19 Length of filterrun as functie of raw water quality.

2.6. Non-spherical and non-uniform filtergrains

In all the preceding calculations, the filterbed was supposed to consist of spherical grains with a uniform diameter d_o when clean. Such filtering materials do exist, but they are made artificially from steel, glass, plastic and so on. Their price is consequently very high and this is the main reason that in practice natural filtering materials are commonly preferred. The grain shape will now deviate more or less from the spherical one, while under all circumstances the grain size will vary be between certain limits. In practice moreover, the grain size is not determined with a micrometer or by counting 1000 grains and weighing them above and below water, but by separating the filtering material with the help of sieves into various fractions.

The most practical approach to this problem can 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

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

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

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

Non-spherical grains have a larger area to volume ratio, while square woven wire sieves of clear opening s pass spheres of volume $\frac{\pi}{6} s^3$ and non-spherical grains of a larger volume. Both factors are taken into account simultaneously by a shape factor ϕ, defined by the relationship

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

With filtering materials of varying shapes, however, the value of ϕ can not be calculated from the relation given above. It is determined by equating the measured resistance H_o of a clean filterbed to the value following from the Carman-Kozeny formula

$$H_o = \frac{180\nu}{g} \frac{(1-p_o)^2}{p_o^3} \frac{v}{d_h^2} L$$ and with

$$d_h = \phi_i \sqrt{s_i s_j}$$ as so-called hydraulic diameter.

In case the grain-size distribution of the filtering material covers a wider range, a sample of weight w can be separated by sieves with clear openings s_1, s_2, ... s_n, s_{n+1} into fractions of weight w_1, w_2, ... w_n. The hydraulic diameter satisfying the Carman-Kozeny formula now equals

$d_h = \phi d_s$ and can be calculated from

$$\frac{w}{\phi d_s} = \frac{w_1}{\phi_1 \sqrt{s_1 s_2}} + \frac{w_2}{\phi_2 \sqrt{s_2 s_3}} + \ldots\ldots + \frac{w_n}{\phi_n \sqrt{s_n s_{n+1}}}$$

with ϕ as average shape factor and d_s as so-called specific diameter

$$\frac{w}{d_s} = \frac{w_1}{\sqrt{s_1 s_2}} + \frac{w_2}{\sqrt{s_2 s_3}} + \ldots\ldots \frac{w_n}{\sqrt{s_n s_{n+1}}}$$

The value of ϕ is always smaller than unity. According to Fair, Geyer and others

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

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 ϕ 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 ϕ =	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	

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 ϕd_s to be used in the calculations of the preceding section follows from

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

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

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

A deviation from the spherical shape certainly reduces the reliability of the mathematical calculations as for a random but otherwise uniform material the value of the shape factor ϕ is difficult to determine with a good accuracy. With a judicious selection of grain sizes, however, adverse effects on the process of filtration do not need to be feared. This on the other hand is always the case when a variation in grain size is present. When during back-washing the filterbed is fully expanded, as it ought to be, a hydraulic classification occurs, bringing the finer grains to the top and the coarser grains to the bottom of the filterbed. In the beginning of the filterrun, the top layer of fine grains has a high filtration efficiency, the value of the filtration coefficient λ_o according to Lerk's theory being inverse proportional to the third power of the grain size. This also means, however, a rapid accumulation σ_v of impurities removed from the passing water, greatly increasing filter resistance and above all decreasing the value of the filtration coefficient $\lambda = \lambda_o(1 - \sigma_v/np_o)$. Already after a short time filtration efficiency will consequently drop, allowing impurities carried by the raw water to penetrate the filterbed to greater depths. By the hydraulic stratification, however, only coarser grains are present here. They have a much lower filtration efficiency with as consequence that only part of the impurities are removed from the passing water, thus deteriorating effluent quality. To demonstrate this phenomenon, the filter run of fig. 2.14 with uniform spherical grains of 0.7 mm diameter will be recalculated for the non-uniform filtering material

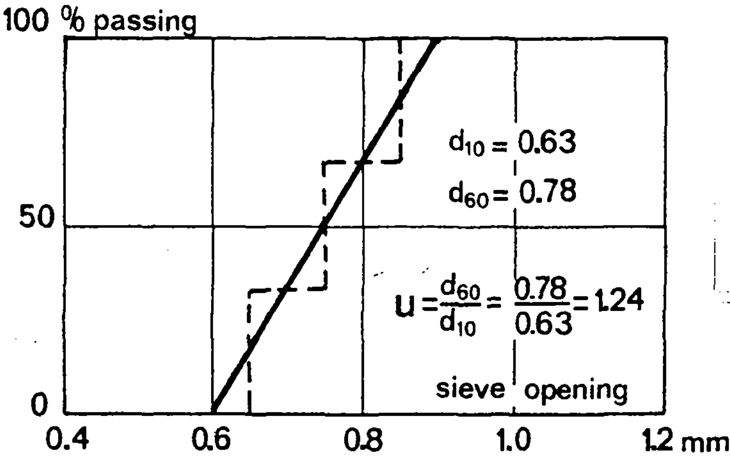


Fig. 2.20 Grainsize distribution of filtering material.

of fig. 2.20. Here the grain size varies linearly from 0.6 to 0.9 mm, giving for the specific diameter d_s

$$\frac{100}{d_s} = \int_0^{100} \frac{dp}{0.6 + (0.9 - 0.6)\frac{p}{100}} = \frac{100}{0.3} \ln \frac{0.9}{0.6} = \frac{40.55}{0.3} \text{ or}$$

d_s = 0.740 mm. With the shape factor ϕ assumed at 0.946, the hydraulic diameter $d_h = \phi . d_s$ again equals 0.7 mm. By hydraulic classification, however, this size will now vary from 0.57 mm at the top to 0.85 mm at the bottom of the filterbed. To enable these calculations to be made by hand, the continuous distribution of fig. 2.20 is replaced by the dotted line. After backwashing, the filterbed with a thickness of 0.75 m is now composed of 3 distinct layers, each 0.25 m thick, with hydraulic diameters of 0.615, 0.710 and 0.804 mm respectively. Using Lerk's theory to translate the influence of grainsize and raw water composition on the mechanism of filtration, the effect of each layer on water quality and filter resistance can easily be determined. In terms of effluent quality and total filter resistance, the results of these calculations are shown in fig. 2.21, while fig. 2.22 renders the purifying effect of each layer separately. Compared with the results of fig. 2.14 for uniform filtering material of the same effective size, stratification reduces the length of filterrun T_q from $(1.85)10^5$ sec to $(1.19)10^5$ sec and the length of filterrun T_q from $(1.62)10^5$ sec to $(1.13)10^5$ sec. The magnitude of these reductions, about 30%, are somewhat surprising in the meanwhile, as in practice the material of fig. 2.20 with a coefficient of uniformity d_{60}/d_{10} according to Allen Hazen of only 1.24, would be considered rather uniform. For best results, a lower value is clearly preferable, as low as can be obtained.

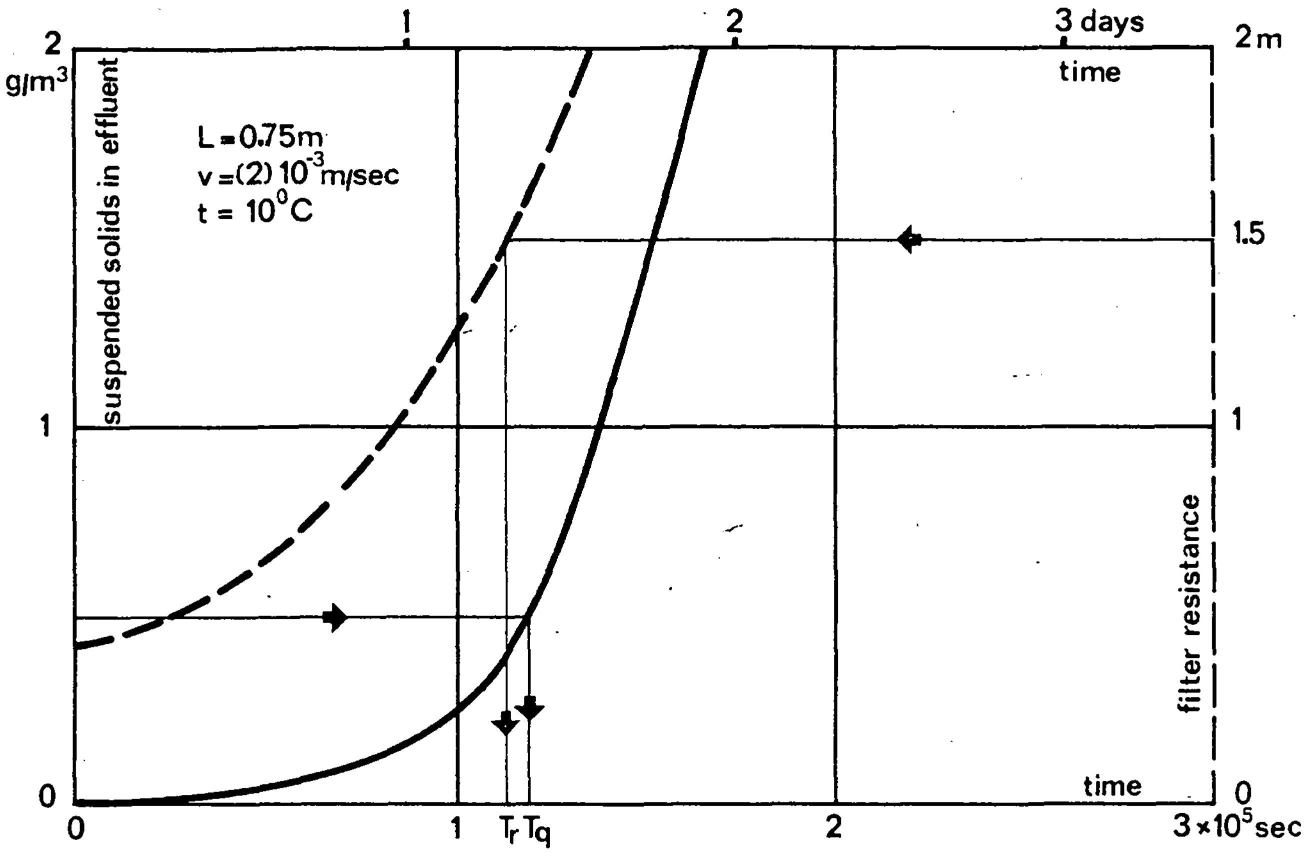


Fig. 2.21 Filtration results of fig. 2.14 recalculated for the filtering material of fig. 2.20.

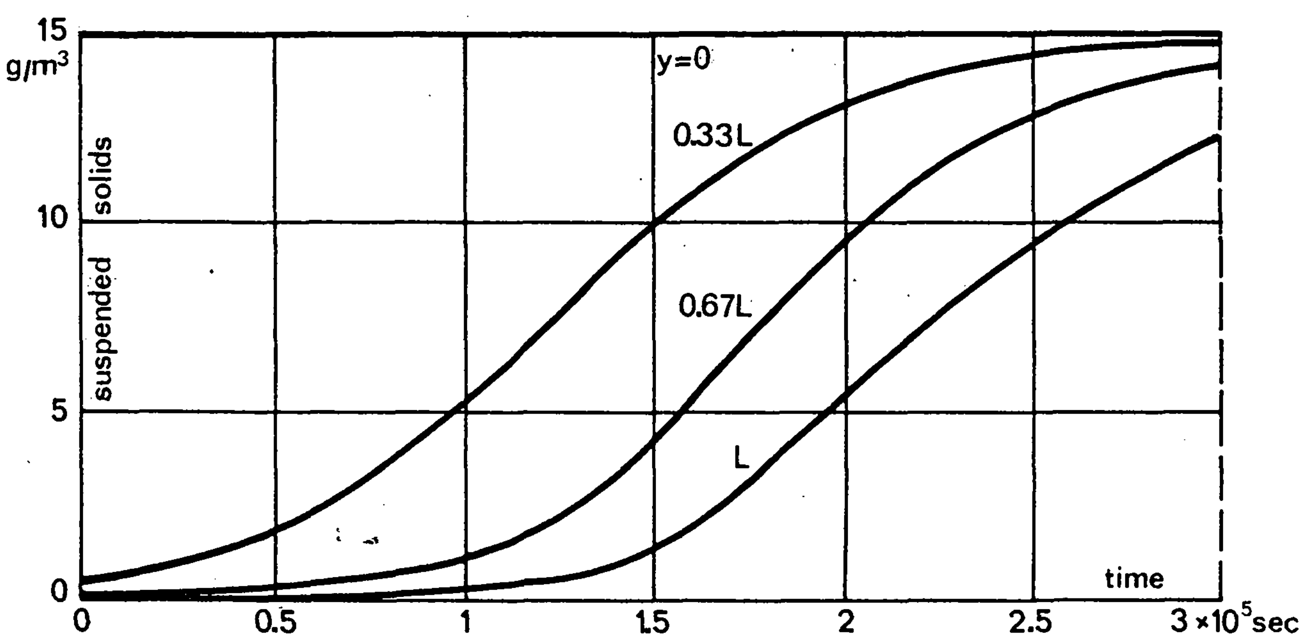


Fig. 2.22 Penetration of impurities into the filterbed of fig.2.21.

7. Design considerations

As demonstrated in the preceding sections, the results of rapid filtration may be expressed in two parameters, being effluent quality and filterresistance. Both deteriorate with time, limiting the length of filterrun to T_q and T_r respectively, the smallest value of which may be considered the real parameter of the rapid filtration process. In their turn these parameters depend on three variables, on raw water quality, composition of the filterbed and filtration rate applied. When provisionally pre-treatment and the use of filtration aids are disregarded, raw water quality is a fact and must be accepted with all its seasonal fluctuations, leaving only the filterbed and the filtration rate to be chosen at will.

With respect to the composition of the filterbed in the meanwhile, three sub-factors may be distinguished, namely filterbed thickness, grain-size distribution and the kind of filtering material. To keep the cost of construction down, the thickness of the filterbed should not be chosen larger than necessary. This may entail the use of fine grained filtering materials, which are difficult to keep clean by back-washing alone. Many filterbed troubles will now result and when the specific grainsize tends to drop below 0.8 mm, the choice of a larger bed thickness and coarser grains is certainly advisable. Normal combinations nowadays are 0.6 - 0.8 m with a specific size of 0.6 - 1.0 mm for final treatment after coagulation and sedimentation, 0.8 - 1.2 m with a specific size of 0.8 - 1.2 mm for pre-treatment preceding slow sand filtration and 1.5 - 3 m with a specific size of 1 - 3 mm for deferrisation of groundwater, while for the future an increase in both filterbed thickness and specific grainsize might be expected. The grainsize distribution of the filtering material should be chosen as uniform as can be obtained, with a coefficient of uniformity according to Allan Hazen under all circumstances below 1.5 and preferably between 1.2 and 1.3, depending on availability. In this respect it is good to realize that the actual work of the filter is done in the filterbed and the cost of even the best quality filtersand seldom exceeds 2 - 3% of the total cost of the filterplant. Requirements to the grading of filtersand are best given as maximum and minimum allowable percentages of material passing through various standard sieves. For better visualizations they may be plotted on a graph of which fig. 2.23 gives an example. With regard to the type of filtering material, it must be admitted that there are cases indeed where other materials as sand give better results. When the back-wash system has

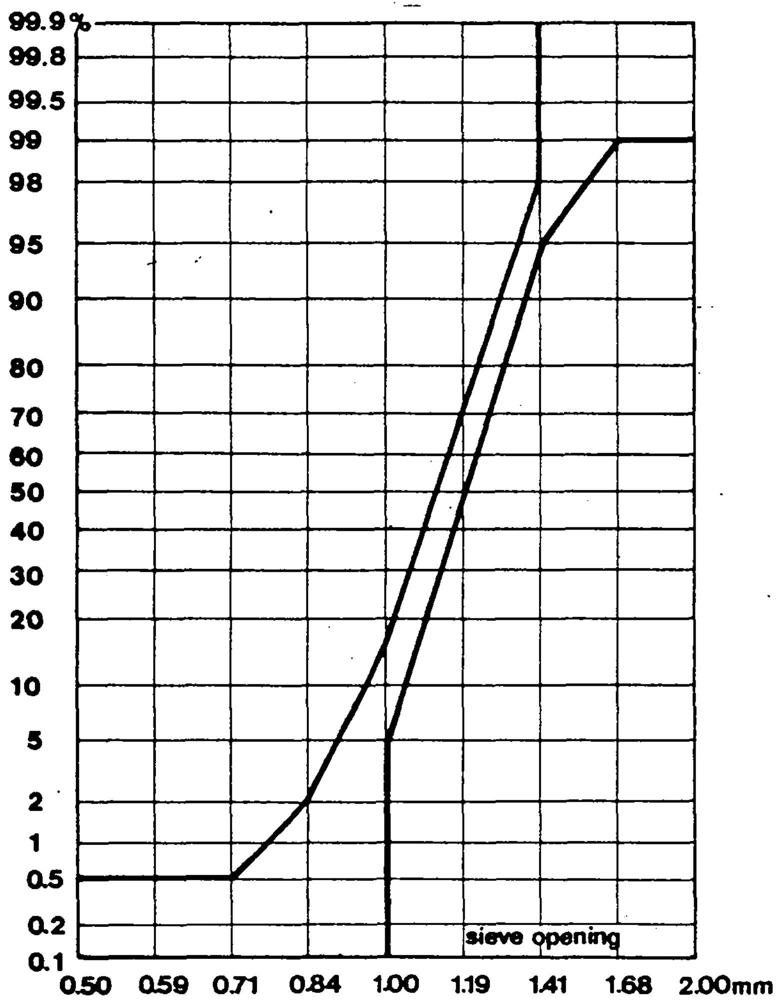


Fig. 2.23 Specifications of filtersand for pre-treatment of river water.

insufficient capacity to expand the filterbed, materials of lower specific gravity such as anthracite are advantageous. In the reverse case of a large sandbed expansion accompanying low rate backwash of fine grained materials, the filterbed can be kept in better condition by the use of heavier materials such as magnetite or garnet. For deferisation of groundwater, the catalytic action of burned dolomite or the electrostatical potential of broken material and the electrokinetical potential of sharp material may promote filtration efficiency. These examples, however, are exceptions and in 99% of all cases sand gives just as good, or even better results, while its price is only a fraction of that of any other filtering material. Needless to say that whatever filtering material is applied, it should be clean and durable, as much as possible free of clay, loam, dust, dirt or organic matter and able to resist mechanical, chemical or biological attack.

As regards the rate of filtration, a value of $(1.4)10^{-3}$ m/sec or 5 m/hour has become the standard one for conditions of average daily demand. This standard rate has been used for the past 80 years, all over the world under widely different circumstances and generally has given reasonable results. This, however, is not due to the versatility of the standard rate, but only because pre- or post-treatment were able to take up any deficiencies the rapid filtration process might show. When rapid filters are used for polishing purposes, the preceding treatment by coagulation and sedimentation can easily be brought to such a level, that the rapid filtrate satisfies normal requirements. In case rapid filters serve to lessen the load on subsequent slow sand filters, the effluent of the rapid filters has mostly a much better quality than required, while next to this the enormous reserve capacity of slow filters enable them to deal with any shortcomings of the rapid filtration process during short periods. These possibilities for compensation are absent when rapid filtration constitutes the sole treatment to which the raw water is subjected and especially here it should always be realized that the standard rate of filtration is certainly not a guarantee for success. In contrary, various types of raw water are difficult to treat properly and ask for a lower rate of filtration to satisfy required effluent standards during acceptable lengths of filterrun. When designing a small installation for which no experiments can be carried out, the rate of filtration should therefore not be chosen higher than the standard one or better be limited to $(2)10^{-3}$ m/sec on the maximum day. Instead of a lower filtration rate as an added factor of safety, the filterbed thickness may also be increased or the grain size lowered, preferably in this order for the reasons already noted above. It goes without saying that there are also many waters easy to treat, allowing a much higher rate of filtration than the standard one. This is especially so when effluent standards are not very strict, as often is the case when rapid filters are used as pre-treatment or for industrial purposes. As the required filterbed area is inverse proportional to the design rate of filtration, a higher value means a smaller installation, reducing the cost of construction. This does not mean to say, however, that the designer should take undue risks. Indeed, the price of water produced is a factor, but especially with public water supplies, a good quality water under all conditions carries a much greater weight. High-rate filtration may therefore only be applied after operation of a pilot plant during all seasons has demonstrated the feasibility beyond reasonable doubt. The saving in cost obtained with a higher rate of filtration is moreover usually not impressive. In the Netherlands and for larger installations, the cost of construction nowadays (1970) amounts to about *f* 4000 (US$ 1500) per m^2 of filterbed area. With interest at 8%, depreciation at 3%, maintenance and operation at 1.2%, this means a yearly cost of 10% or *f* 400 per m^2. At the standard rate of $(1.4)10^{-3}$ m/sec during the average day, the same m^2 produces 44000 m^3/year, corresponding with a total price of 0.9 Dutch cent/m^3 (0.33 US$ cent per m^3). Raising the filtration rate will undoubtedly lower this price, but even when the filtration rate is doubled, it will not drop below 0.5 Dutch cent/m^3, giving a saving of 0.4 Dutch cent/m^3. This in the meanwhile is not more than 0.5 - 1% of the price as delivered to the customers, hardly worthwhile to endanger effluent quality, even not during short periods with an extreme low quality of the raw water. From the same calculation may be gathered, that pretreatment to increase the filtration rate beyond the standard one, is seldom able to give attractive financial returns. An exception to this rule must be made when otherwise much lower rates of filtration are necessary. In case a rate of only $(0.7)10^{-3}$ m/sec on the average day is acceptable, pre-treatment doubling this rate may cost as much as 0.7 Dutch cent/m^3, giving in many cases an economic proposition.

Filter design in the meanwhile also depends on the desired length of filterrun, that is on the required effluent quality and on the maximum allowable filter resistance. When rapid filters constitute the final treatment for drinking water purposes, effluent quality may not be tampered with and should satisfy strict standards under all conditions. It must be realised, however, that such standards change during the years, mostly becoming more severe as time goes on. In the past turbidity standards in the USA have been raised from 1 J.T.U. 40 years ago to 0.2 J.T.U. at present while as goal a value of 0.1 J.T.U. is set by the A.W.W.A. In the affluent society of tomorrow, this trend may even accelerate and extend to other not injurious but still objectionable impurities such as iron, manganese, ammonia, organic matter and so on. When those standards cannot be obtained by a better pre-treatment of the water to be filtered, lower filtration rates allowing finer filtering material in greater depths must be used, giving another reason for choosing present-day filtration rates no too high. In case rapid filters are used as 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 he most economical solution.

The maximum allowable filterresistance depends primarily on outside factors, in principle being equal to the difference in level at which the raw water is received and the filtered water must be discharged. In practice this drop in level varies between about 1.5 and 4 m. The resulting energy loss in the meanwhile is negligeable, even with 4 m head loss not more than 0.01 Dutch cent per m^3 and this is certainly not a factor to save on the maximum allowable filterresistance. With the same raw and filtered water levels, however, two different filter constructions are still possible, operating with excess or reduced pressure. For operation under excess pressure, a large depth of water on top of the filterbed is necessary, usually between 1 and 1.5 m, while for operation under reduced pressure a depth of 0.25 - 0.4 m is already sufficient. As shown in fig. 2.24, the latter set-up allows a much smaller depth of the filter-box, appreciably reducing the cost of construction, but it brings with it the danger of air binding. To explain this phenomenon, the same figure shows the pressure distribution in the filterbed at various time intervals after the respective filterrun started. With the mathematical theory of filtration, this pressure distribution can be calculated, but a much more reliable way is measuring it directly, using piezometers installed at various depth below the top of the filterbed. When the maximum allowable filterresistance is not excessive, a large depth of supernatant water will prevent the occurrence of negative heads, but when this depth is small negative heads will arise soon after filtration starts, in this way reducing the solubility of the gases contained in it. The point being that the saturation concentration of gases in water is direct proportional to the partial pressures of the gases with which the water is in contact.

Assuming the water on top of the filter to be saturated with atmospheric air, this means in reverse that the sum of the gaspressures, including that of watervapour, 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

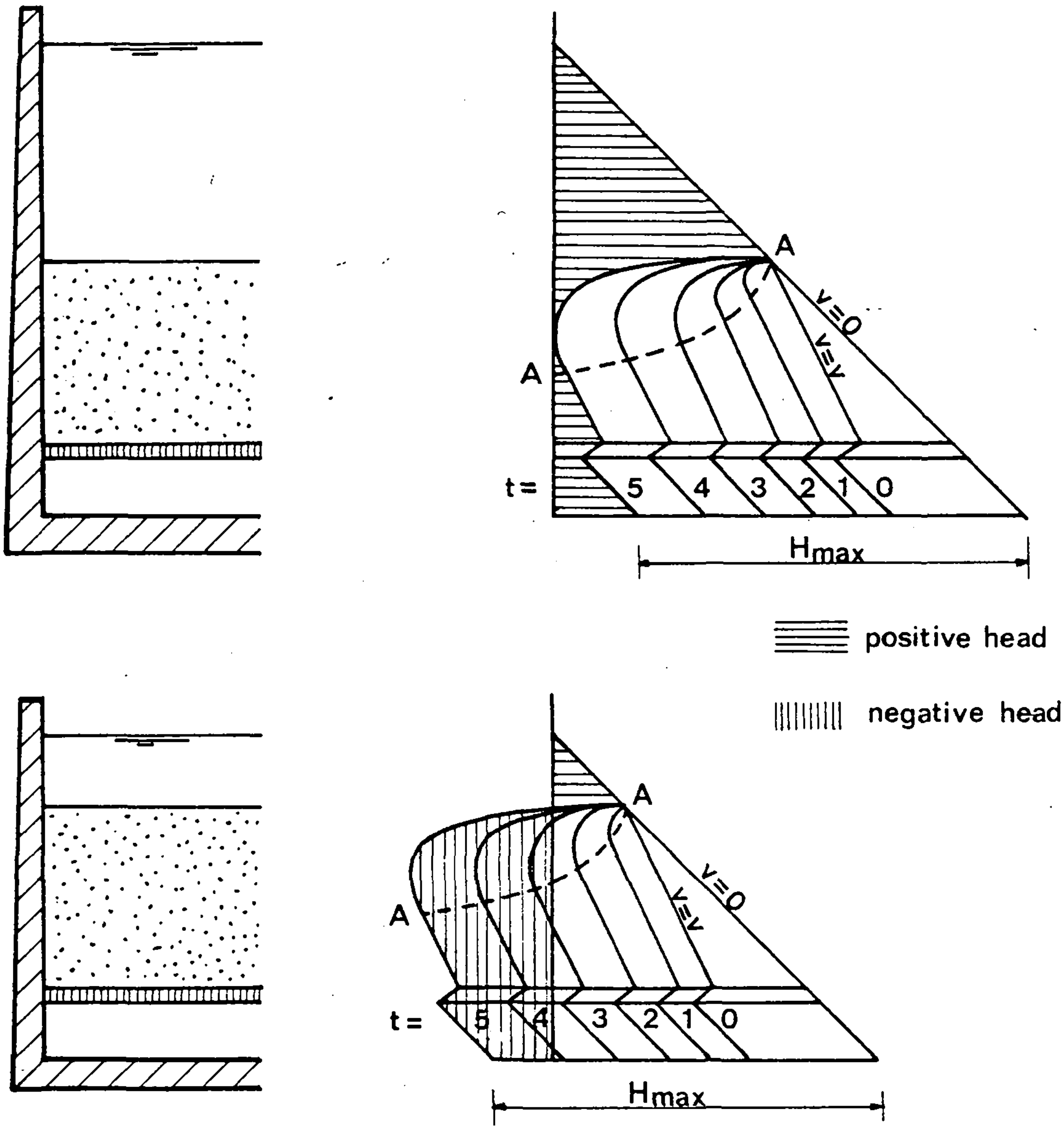


Fig. 2.24 Filter operation with excess and reduced pressure.

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. In most cases in the meanwhile, oxygen will be consumed during filtration, reducing the gaspressure, while the reaction products formed are so highly soluble that they do no 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 ^{0}C the volumetric composition of atmospheric air in rural areas equals

$$N_2 = 76.2\%, \quad O_2 = 20.6\%, \; H_2O = 2.3\%, \; \text{argon and other gases} = 0.9\%$$

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

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

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

For various temperatures, the relation between the amount of oxygen consumed and the allowable negative head is shown in fig. 2.25. 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

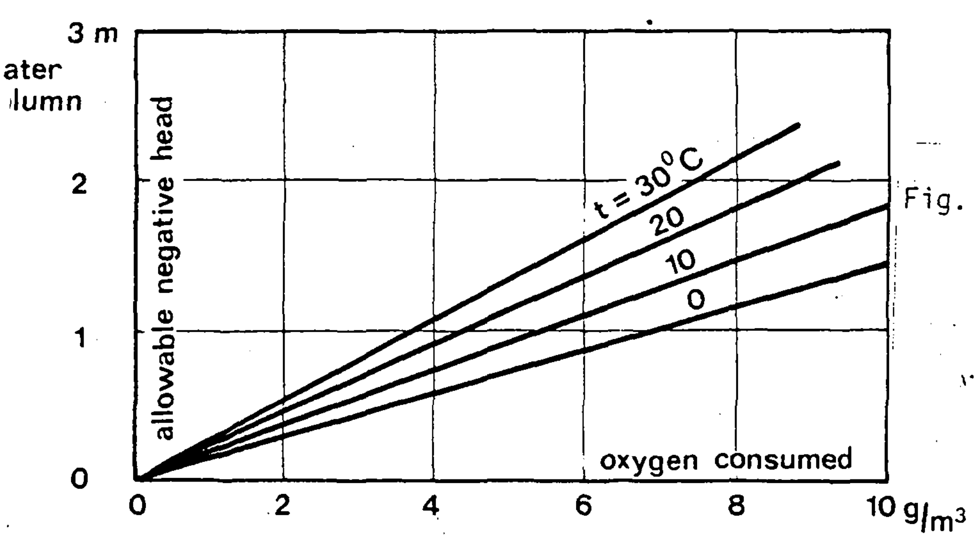


Fig. 2.25 Relation between oxygen consumed and allowable negative head.

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 less serious. For deferrisation and demanganisation of groundwater, however, oxygen consumption is very low, while such waters are often heavily aerated to remove excessive carbon-dioxide. With pressure aeration, the water may even enter the filterbed supersaturated with atmospheric gases and airbinding may now take place long before filterresistance has reduced the waterpressure to atmospheric.

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. Incidentally, the pressure curves of fig. 2.24 are a mighty aid in understanding the complicated process of filtration. Especially important is the locus of points A, below which the slope of the pressure line has remained unchanged, equal to that for t = 0. These points consequently indicate the deepest penetration of impurities into the filterbed.

Which length of filterrun considered by itself is most attractive, is again a compromise between conflicting interests. A great length of filterrun, several days for instance, asks for coarse filtering material in a large bed thickness and percolated at low filtration rates, both factors appreciably increasing the cost of construction. A short length of filterrun, some hours, on the other hand will result in a deterioration of average effluent quality and will give rise to serious operational difficulties. 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. 2.26 shows this phenomenon with regard to the quality of the effluent from fil-

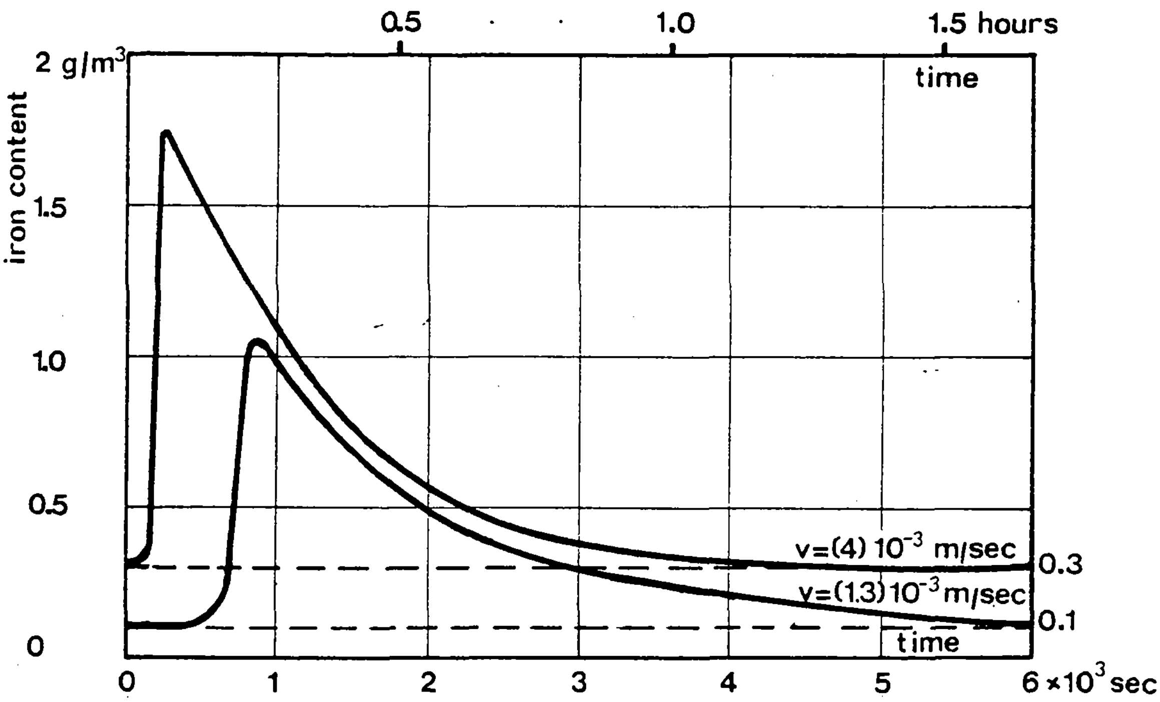


Fig. 2.26 Effluent quality of rapid filters treating aerated groundwater with iron content of 5 g/m^3.

ters 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 the normal value of 0.1 g/m^3. With a length of filterrun equal to $(140)10^3$ sec or about 39 hours, the higher iron content during the first 2 hours corresponds with an average increase of the iron content by 12% from 0.1 to over 0.11 g/m^3. Cutting the length of filterrun in half, would give no operational difficulties, but a raise in average iron content by 25% has then to be accepted. At a rate of $(4)10^{-3}$ m/sec, a length of filterrun equal to $(50)10^3$ sec of 14 hours and a steady-state iron content of 0.3 g/m^3, the average increase is again 12%, raising the average iron content by no less then 0.04 g/m^3. In principle, the lowering of effluent quality 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 constuction, 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.

As regards the operational difficulties referred to above, its must be remembered that filters should always be backwashed before the maximum allowable filterresistance is reached.

In fig. 2.27 it is assumed, that the filterresistance increases linearly with time and that the filters are backwashed at equal intervals. With H_a as maximum allowable resistance to be obtained at time T, the dirtiest filter is cleaned at time T' when its resistance has reached a value H_w. At this moment the filter next in degree of clogging has a resistance

$$H_w - \frac{H_w - H_i}{n} = \frac{n-1}{n} H_w + \frac{1}{n} H_i.$$

when n filters are present and the initial resistance amounts to H_i. During the period τ of backwashing, refilling and filter-to-waste operation when present, this resistance increases by an amount

$$\frac{\tau}{T'} (H_w - H_i) = \frac{\tau}{T} (H_a - H_i)$$

Most filterplants operate at constant capacity, meaning that backwashing one filter increases the rate and resistance of the remaining filters by a factor $\frac{n}{n-1}$ to in total

$$\frac{n}{n-1} \{(\frac{n-1}{n} H_w + \frac{1}{n} H_i) + \frac{\tau}{T} (H_a - H_i)\}$$

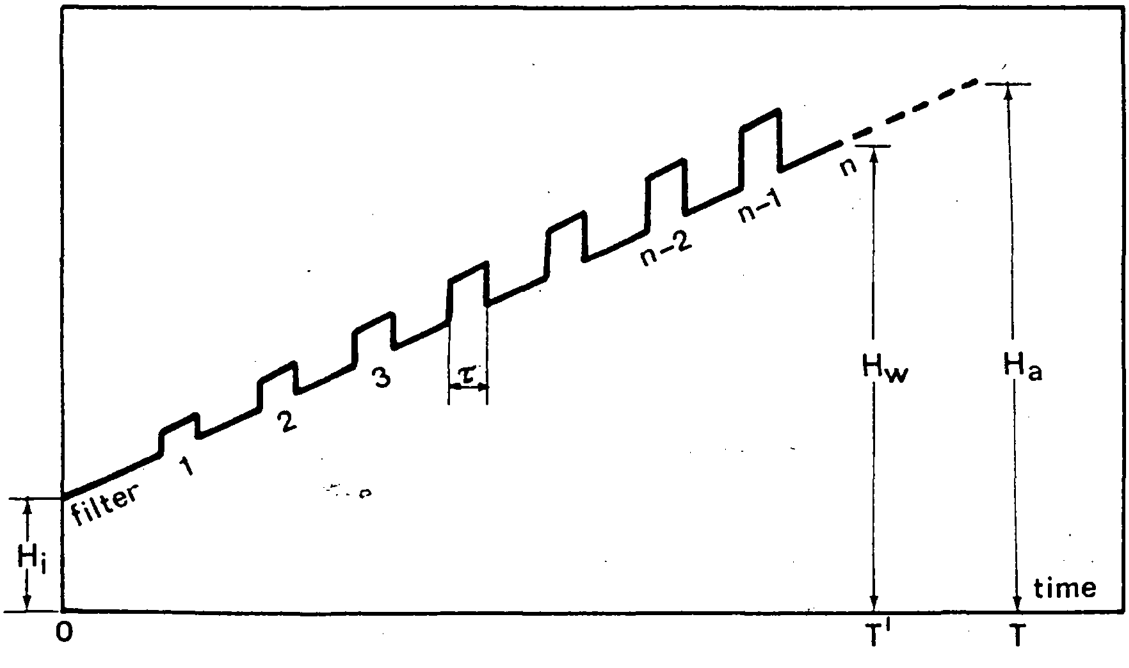


Fig. 2.27 Change in filter resistance during backwashing one filter.

This resistance must be smaller as H_a giving as requirement for the length of filterrun

$$T > \frac{n}{n-1} \frac{\tau}{1 - \frac{T'}{T} - \frac{1}{n-1} \frac{H_i}{H_a - H_i}}$$

With an allowable reduction in length of filterrun of 20% ($\frac{T'}{T} > 0.8$) this gives with for instance n = 20, H_i = 0.4 m and H_a = 2.0 m

$$T > 6\ \tau \qquad \text{or} \qquad T > (10 \text{ to } 30)10^3 \text{ sec,}$$

depending on local circumstances. In reality, however, th clogging will not be distributed equally over all units and with 20 filters as mentioned above, cleaning of 3 filters simultaneously must be anticipated. This gives as requirement

$$\frac{n}{n-3} \{(\frac{n-1}{n} H_w + \frac{1}{n} H_i) + \frac{\tau}{T} (H_a - H_i)\} < H_a$$

or

$$T > \frac{n}{n-3} \frac{\tau}{1 - \frac{n-1}{n-3} \frac{T'}{T} - \frac{3}{n-3} \frac{H_i}{H_a - H_i}}$$

With the same assumptions as used above this gives

$$T > 19\ \tau \qquad \text{or} \qquad T > (0.4 \text{ to } 1)10^5 \text{ sec}$$

These calculations in the meanwhile suppose an automatic actuated backwashing process and no objections to filter cleaning during the small hours of the night. With manual operation there is a strong preference for backwashing during the day shifts only, requiring a length of filterrun T in the neighbourhood of $(1)10^5$ sec. In periods of good raw water quality, the length of filterrun will greatly increase. To prevent a deep penetration of impurities, however, it is good practice to backwash the filters at least every 3 days.

3. CLEANING

3.1. Elements of rapid filter cleaning

When during filtration the hydraulic resistance attains its maximum allowable value or the quality of the effluent drops below the set standard, cleaning of the filter is necessary to restore its capacity and/or to improve the quality of the filtered water. With a deep penetration of the impurities from the raw water into the filterbed, handcleaning using jetting tubes is cumbersome, while the short lengths of filterrun commonly applied, makes this system very expensive in terms of labour (fig. 3.1). These are

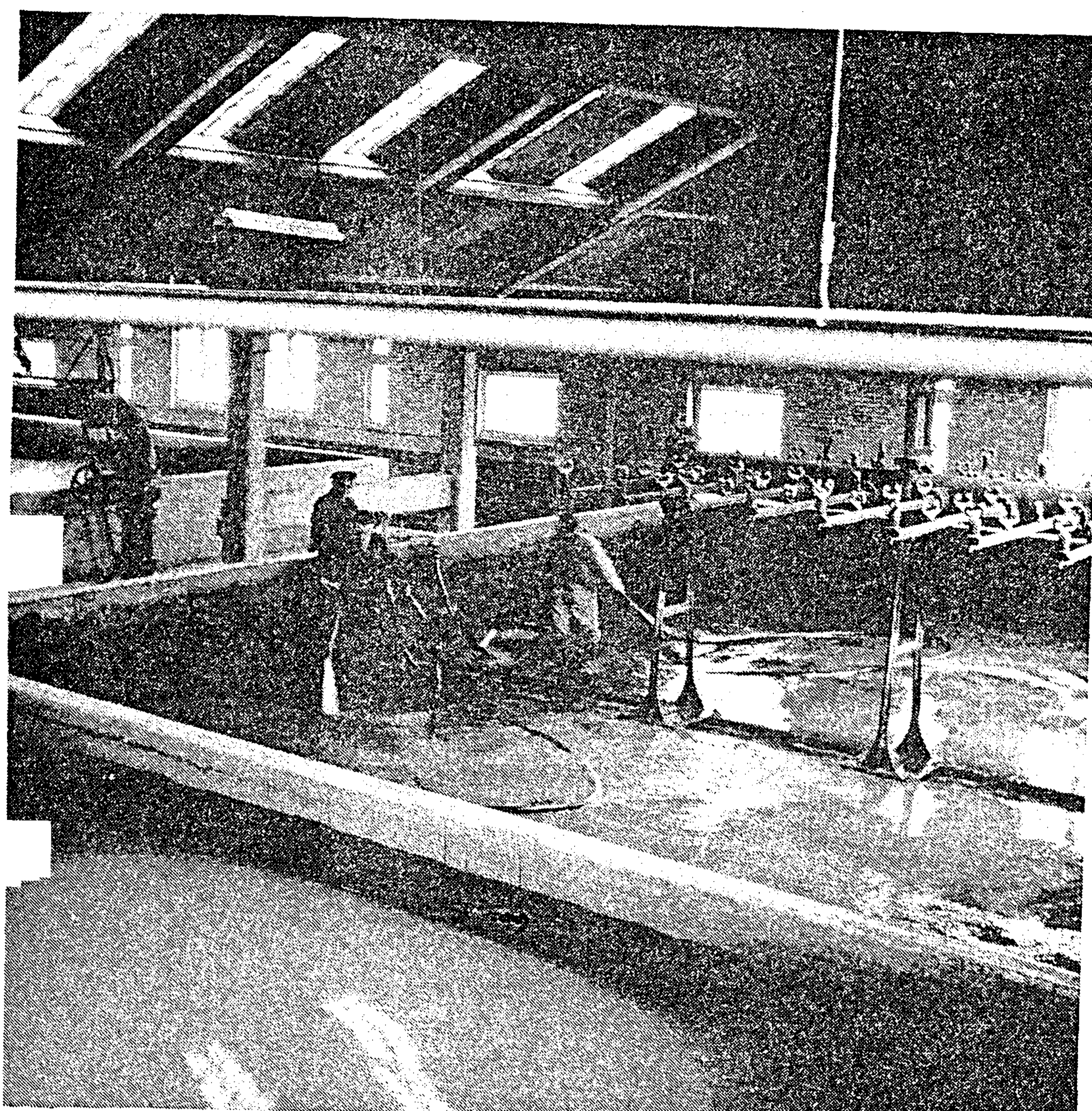


Fig. 3.1 Manual cleaning of a rapid filter.

the reasons that nowadays practically without exception mechanical cleaning is used, effected by reversing the direction of flow, admitting washwater to the underside of the filterbed. 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 washwater troughs and gutters are present to convey it to a drain leading outside the filter (fig. 1.1 and fig. 3.2). Impurities adhering to the filtergrain surfaces must first be dislodged before they can be flushed upward. Primarily this is accomplished by the shearing action of the rising washwater, flowing at high rates past the stationary grains. When the washwater rates are further increased and fluidisation of the filterbed sets in, the increase in hydraulic scour will slow down. It is now supplemented, however, by a mechanical scour when the filtergrains, dancing in the rising washwaterstream, bounce and tumble against each other. The filterbed expansion accompanying fluidisation also enlarges the pore channels, allowing the loosened impurities to escape more freely with the washwater.

With fairly heavy filtergrains, the backwashing process described above gives excellent results. High washwater velocities with correspondingly large hydraulic scour are necessary to obtain expansion, during which process the heavy grains rub with great forces against each other and already in a few minutes the desired amount of cleaning can be obtained. With light grains on the other hand, of small size or low mass density, just the reverse will happen. Already a low rate of backwash will expand the sandbed, while the bumping together of light-weight grains will give rise to small forces only. Application of a large amount of sandbed expansion will certainly increase the amount of hydraulic scour, but it will

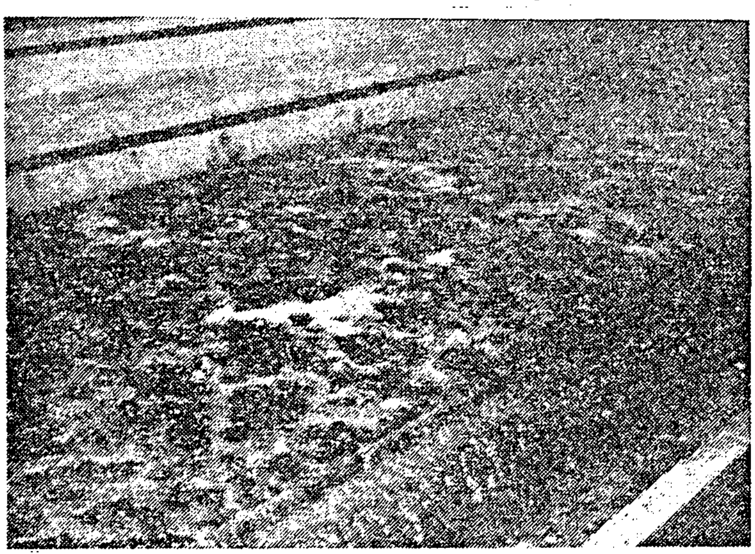


Fig. 3.2 Backwashing of a rapid filter.

also separate the grains and thus reduce the mechanical scour with as result that the over-all cleaning action remains about the same. Backwashing of light-weight grains must therefore be carried out for prolonged periods, say 5 to 10 minutes. Even in this way it is often impossible to keep filterbeds clean on the long run. A persistent fouling will now occur, resulting in many filterbed troubles. Ultimately the filtering material must be removed for washing outside the filter, a cumbersome and expensive process. To some degree, but not altogether, these troubles may be prevented by supporting the backwash process with an additional scour, by stirring the expanded sandbed mechanically with rates, pneumatically with air or hydraulically with jets of water that are directed from the surface downward into the expanded bed.

In various cases the purification accompanying filtration depends on deposits previously formed on the filtergrains, such as a cover of manganese oxide compounds (Graveland,doctoral thesis,Delft,1971)for the removal of manganese, a coating of nitrifying bacteria for the oxydation of ammonia, etc. In such instances great caution must be exercised in backwashing, using low rates during short periods to prevent a too large removal of these active deposits.

3.2. Hydraulics of backwashing

As already mentioned in section 2.3, the head loss z 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

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

with g as gravity constant and ν 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

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

and shows indeed the inverse proportionality between z 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^{o} C, $\nu = (1.31)10^{-6}$ 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

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

and after substitution of the value of Re

$$z = 130 \frac{\nu^{0.8}}{g} \frac{(1-p)^{1.8}}{p^3} \frac{v^{1.2}}{d^{1.8}} L$$

The use of this formula is demonstrated in fig. 3.3, indicating for a sand-bed L = 1.2 m thick with porosity p = 0.4 and various grain sizes d, the relation between the head loss z and the backwash rate v at a temperature of 10° 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 z equals the submerged weight of the filterbed

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

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

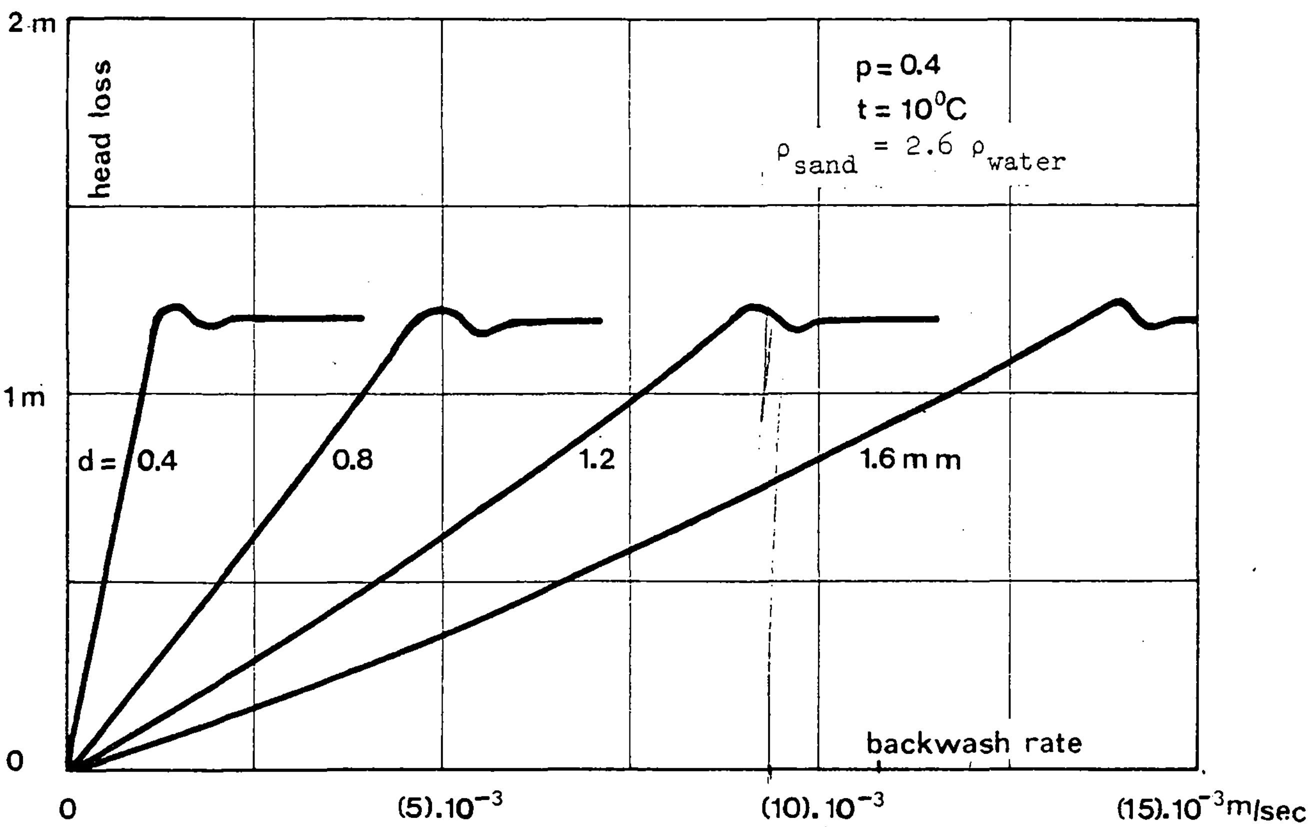


Fig. 3.3 Head loss of a sandbed 1.2 m thick, grainsize d, during backwashing.

with ρ_w and ρ_f as mass densities of water and filtering material respectively. With spherical sand grains of one size,

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

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 ρ_f, resulting on the other hand in a lower value of the maximum head loss.

The backwash rate v at which expansion starts may be found by equating the two values of z calculated above

$$(1 - p) \; L \; \frac{\rho_f - \rho_w}{\rho_w} = 130 \; \frac{\nu^{0.8}}{g} \; \frac{(1 - p)^{1.8}}{p^3} \; \frac{v^{1.2}}{d^{1.8}} \; L$$

Increasing v above this value will not result in a larger head loss, but will expand the filterbed, increasing the porosity from p to p_e and the thickness of the filterbed from L to L_e This changes the formula above to

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

With the amount of filtering material constant (fig. 3.4)

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

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

Solving this equation for the porosity p_e of the expanded filterbed gives

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

With good approximation

$$\frac{p_e^{\,3}}{(1 - p_e)^{0.8}} = (2.63)p_e^{\,3.6}, \text{ substituted}$$

$$p_e^{\,3.6} = \frac{130}{2.63} \frac{\nu^{0.8}}{g} \frac{\rho_w}{\rho_f - \rho_w} \frac{v^{1.2}}{d^{1.8}} \qquad \text{or}$$

$$p_e = 2.95 \frac{\nu^{1/4.5}}{g^{1/3.6}} \left(\frac{\rho_w}{\rho_f - \rho_w}\right)^{1/3.6} \frac{v^{1/3}}{d^{1/2}}$$

When water has to be filtered, the value of the kinematic viscosity ν as function of the temperature

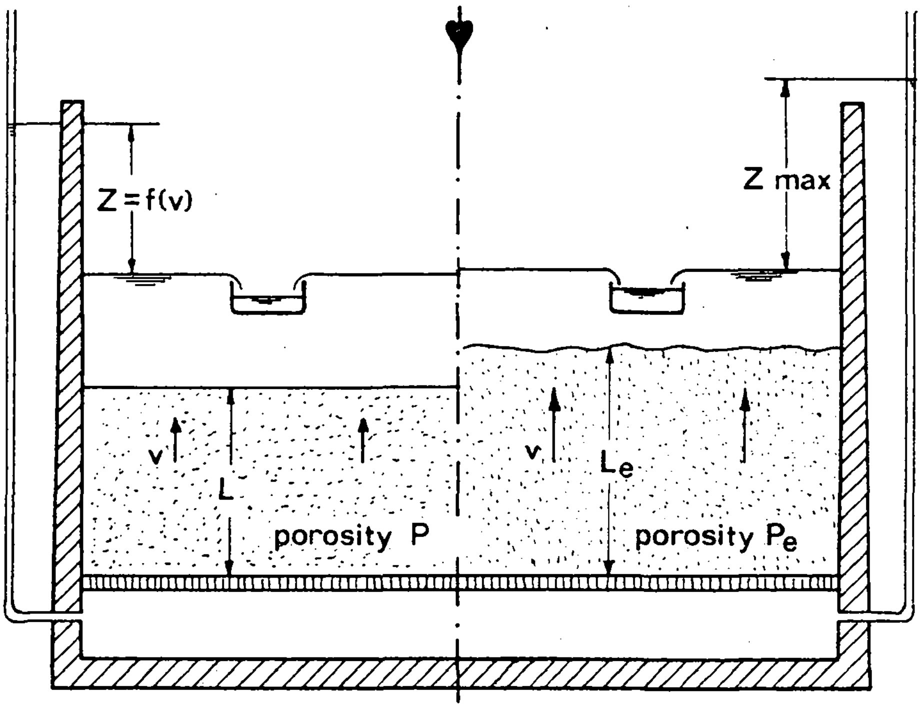


Fig. 3.4 Backwashing a filter with and without sandbed expansion (head loss of filterbottom neglected).

t =	0	5	10	15	20	25	30	°C
ν =	1.792	1.519	1.310	1.146	1.011	0.898	0.804	x 10^{-6} m^2/sec

may be approximated by

$$\nu^{1/4.5} \approx \frac{1}{(20.2)(0.94 + 0.006\ t)}$$

With sand as filtering material

$$\left(\frac{\rho_w}{\rho_f - \rho_w}\right)^{1/3.6} = \left(\frac{1}{1.6}\right)^{1/3.6} = 0.877\text{, giving with}$$

$$g^{1/3.6} = (9.81)^{1/3.6} = 1.877 \text{ after substitution}$$

$$p_e = \frac{0.068}{(0.94 + 0.006\ t)} \frac{v^{1/3}}{d^{1/2}}$$

when v is expressed in m/sec, d in m and t in degree centigrade

Filterbed expansion is commonly expressed as the percentage increase of filterbed thickness

$$E = 100 \frac{L_e - L}{L} = 100 \frac{p_e - p}{1 - p_e}$$

For initial porosities p at 0.35 and 0.4 and for water temperatures of 0 and 20°C, fig. 3.5 gives the sandbed expansion E as function of backwashrate v and grainsize d. Clearly this figure shows the enormous influence of water temperature on the results obtained. In more detail this effect can be read from the table below.

water temperature	0	5	10	15	20	25	30	°C
backwashrate	81	91	100	109	119	129	139	%

In the Netherlands, surface water temperatures nowadays vary between 0 and 25°C, meaning that for the same amount of sandbed expansion, the backwashrate in summer must be a factor 129/81 = 1.59 larger then in winter, requiring a backwash installation which is easily adjustable. The original porosity has also a fair influence with lower values increasing the amount of material present per unit volume of the filterbed at rest and thus resulting in a larger amount of expansion for the same backwash rate. Next to the type of filtering material, this porosity depends on the backwash procedure, a slow and carefull closing of the washwater supply valve resulting in a higher porosity. In practice this factor is difficult to determine with the required accuracy. The amount of filterbed expansion itself can be measured with the device of fig. 3.6, which is self-explanatory.

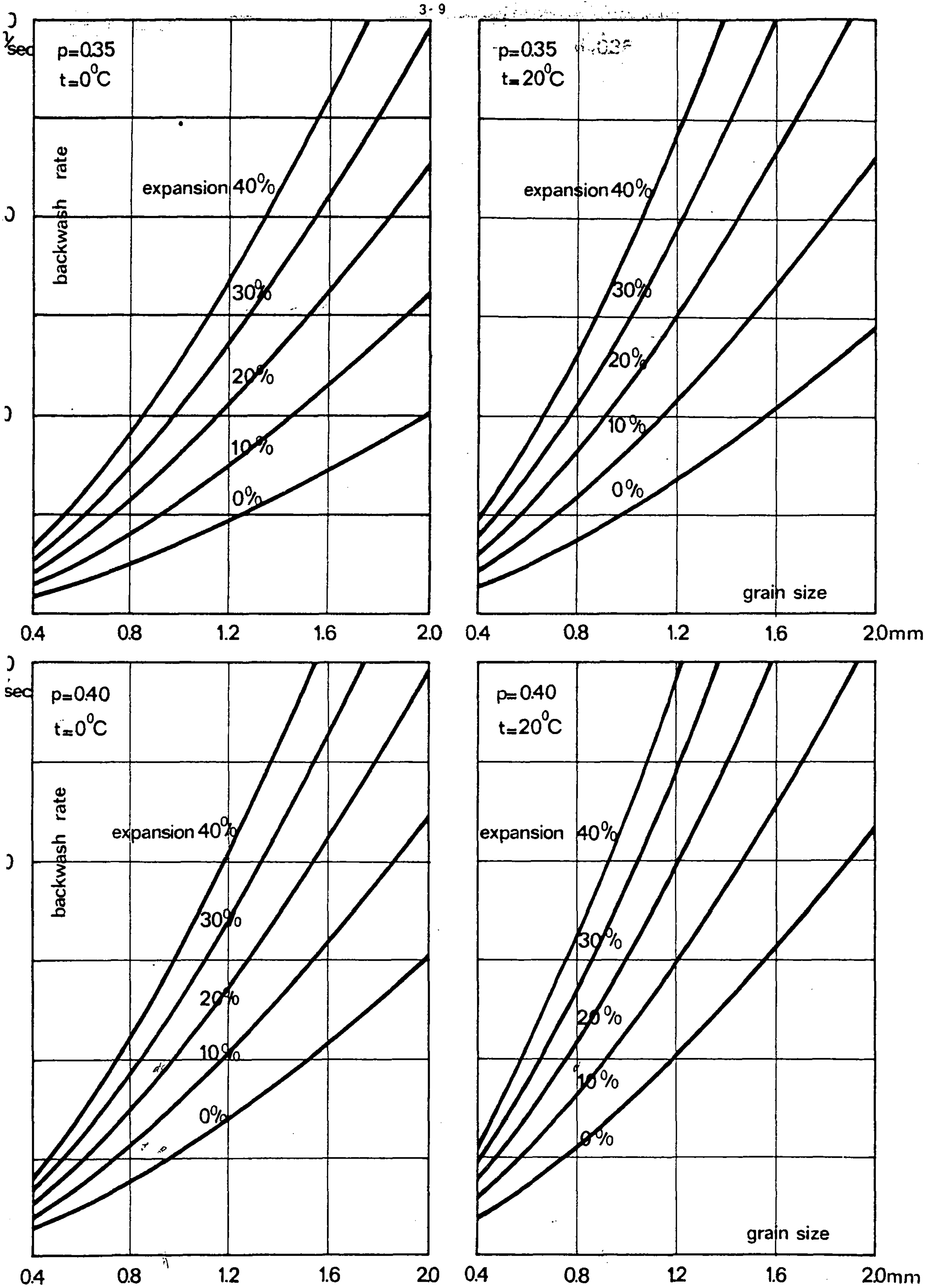


Fig. 3.5 Filterbed expansion for uniform spherical sandgrains.
($\rho_f = 2.6\ \rho_w$)

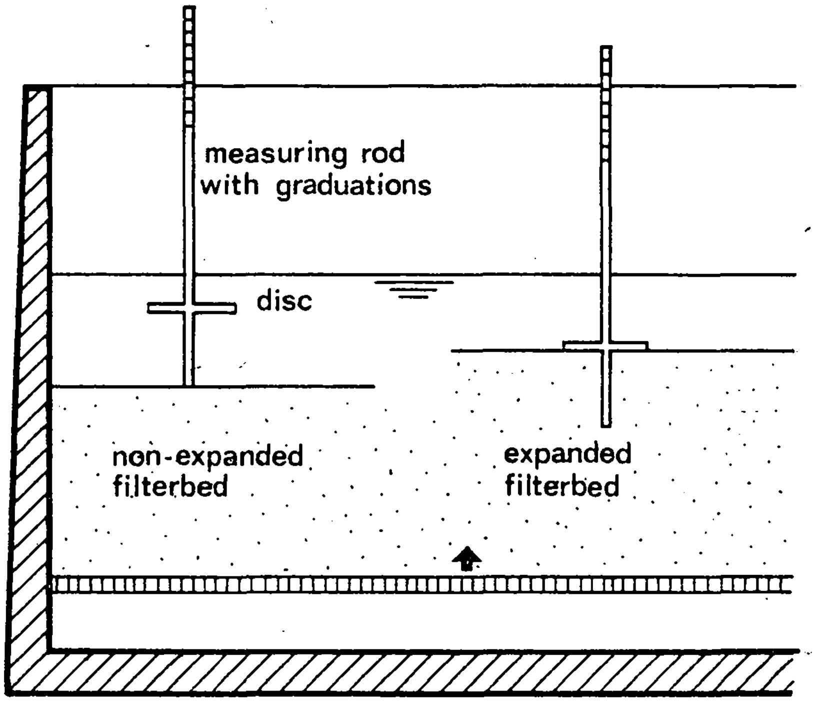


Fig. 3.6 Measurement of filterbed expansion during backwashing.

The calculated results of fig. 3.5 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 resistence 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 ϕ's with s as the clear opening of square woven wire sieves which just passes the grains and with ϕ' 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 here 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 fig. 3.5, a 15% expansion of the uniform spherical material with 0.9 mm diameter, requires at 20^{o}C and 40% original porosity a backwash rate of $(11)10^{-3}$ m/sec. For the upper portion of the non-uniform bed with 0.7 mm diameter, this means an expansion of 31%, 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 $(13)10^{-3}$ m/sec is necessary. Judged by itself this has little disadvantages, but the expansion of the upper layer now rises to 39%. With sandbeds of 1.2 m thickness, the expansions at the latter backwash rates applied increase the thickness of the bed with uniform material by 0.18 m, while for the bed of non-uniform material this increase amounts to 0.28 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. 3.18 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 Allan Hazen coefficient of uniformity at least below 1.5 and preferable below 1.3.

The selection of the backwash rate best suited in a particular case is a compromise between conflicting interests. A high rate of backwash and a correspondingly large amount of sandbed expansion increases the shearing action of the rising washwater (fig. 3.7) and allows the liberated cloggings to float more easily to above, thus preventing as much as possible a persistent fouling of the filterbed to occur. With large sandbed expansions on the other hand, the mechanical scour produced by the rubbing together of the grains is less, while more material will be lost during backwashing, es-

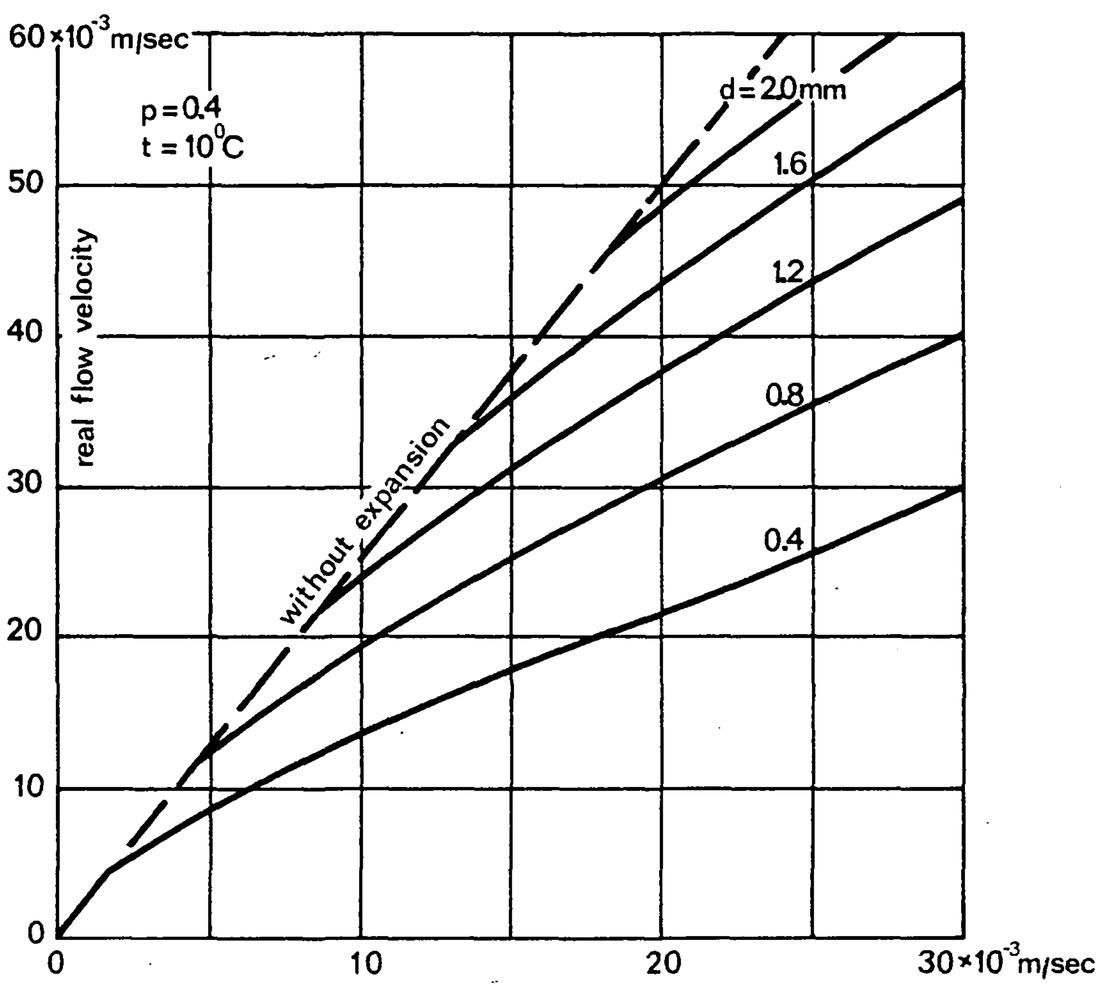


Fig. 3.7 Flow velocities in the pores of a filterbed during backwash.

pecially important when expensive filtering materials are used. Although not a factor of any importance, smaller sandbed expansions also slightly reduce washwater consumption. In the past, the amount of sandbed expansion applied was fairly standard, with values as high as 50% for finishing filters with fine grains and 30% for pre-filters with coarser material. During the last years, however, a marked tendency of smaller sandbed expansions is noticeable, going down as far as 15% with grains of 0.8 mm and 10% with grains of 1.2 mm size. With grains over 2 mm diameter, for deferrization of groundwater for instance, commonly no sandbed expansion is provided. For good results backwashing must now be carried on over extended periods, appreciable increasing washwater consumption.

In particular when dealing with groundwater, deposits are often formed on the filtergrain surfaces, augmenting their diameter and increasing or decreasing their average mass density. Such deposits are often difficult to remove ($CaCO_3$ for instance), while in other cases they are strictly necessary to keep up the purifying capacity of a ripened filterbed. In all cases, however, the backwash rate needed for the desired amount of filterbed expansion will change with time. In particular when in future larger rates are required, this phenomenon should already be anticipated during the design stage.

3.3. Equality of washwater distribution

Fig. 3.4 showed the head losses accompanying the upward flow of washwater, 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. 3.8 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. 3.9. When the sandbed is supported by graded lay-

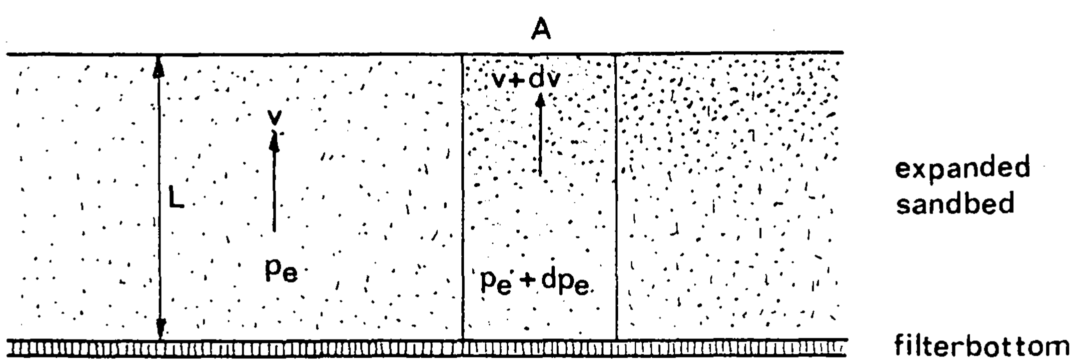


Fig. 3.8 Unequal washwater distribution.

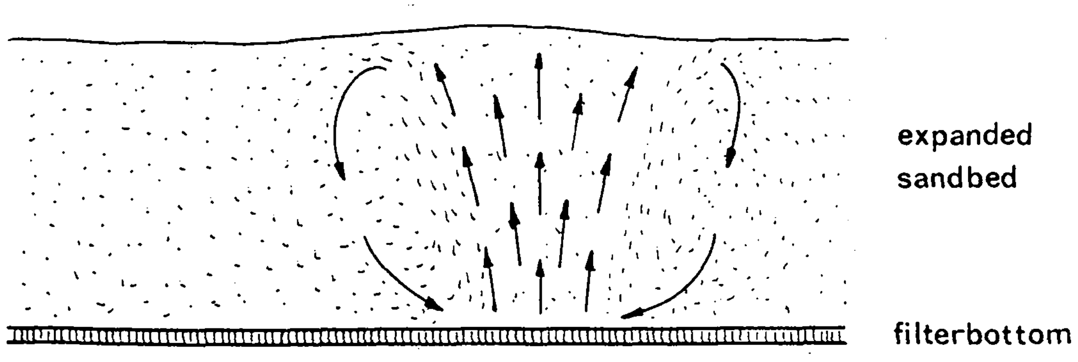


Fig. 3.9 Sandboil.

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. 3.10). 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. 3.11, 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. 3.11, the difference between the heads at which the washwater is supplied and discharged, varies between H and H + dH,

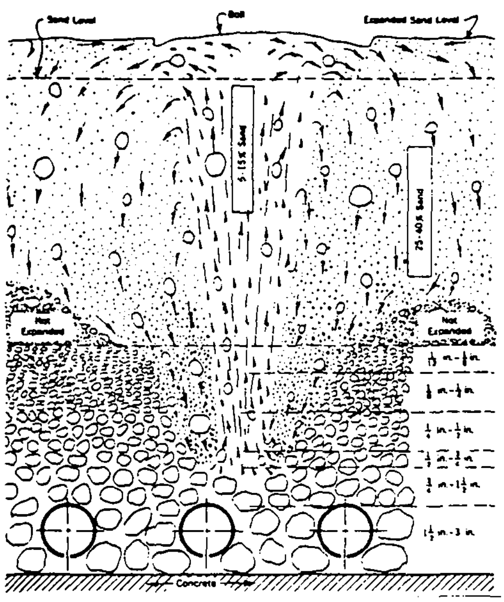


Fig. 3.10 Sandboil disturbing underlying rgavel layers(after Baylis).

with

$$dH = d_1H + \text{ or } -d_2H,$$ 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

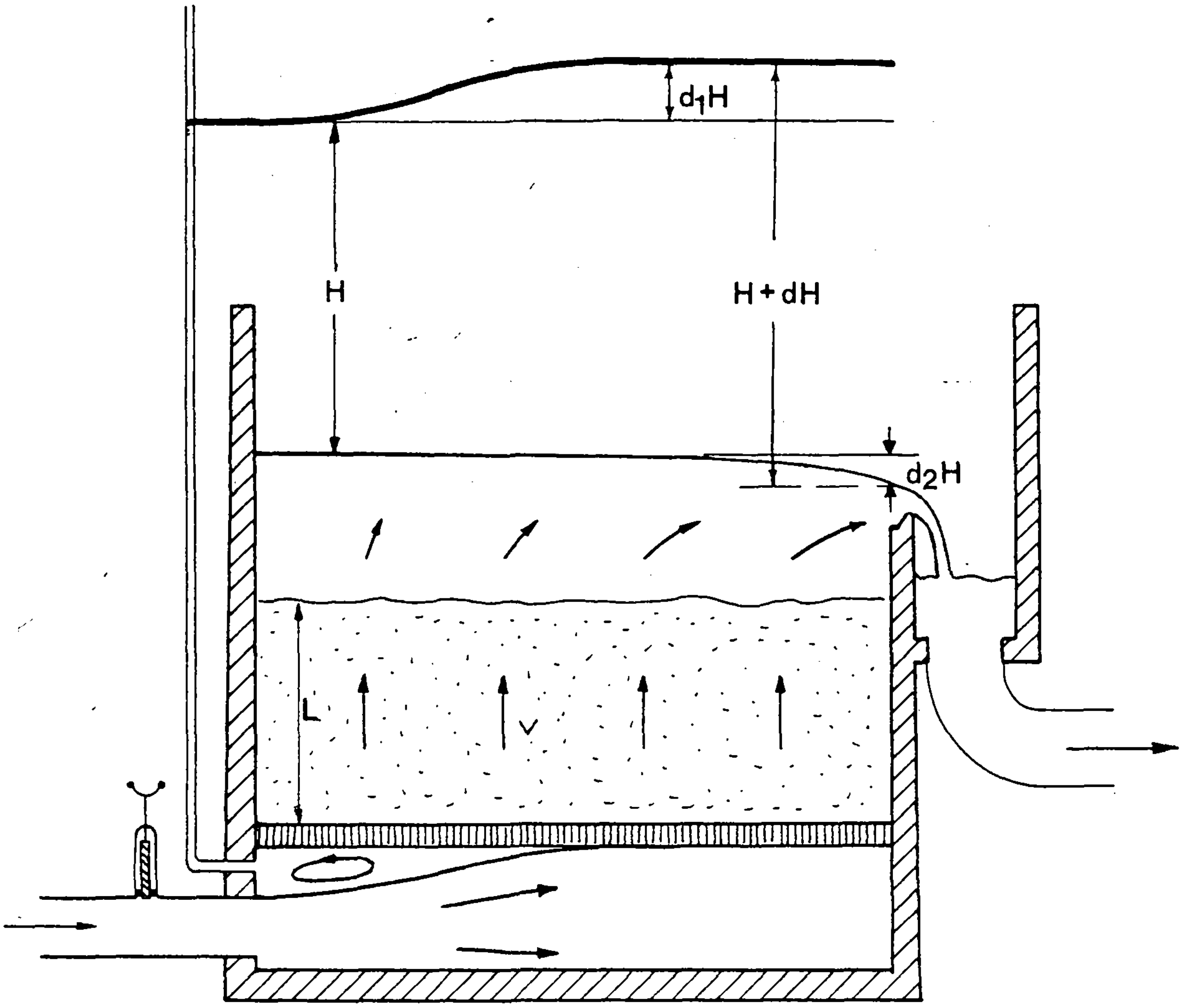


Fig. 3.11 Pressure distribution during backwashing.

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

with v as backwashrate and α 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 20% filterbed expansion may further be simplified to about

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

With sand as filtering material, the resistance of the bed against backwashing is about equal to the bed thickness say 1.2 m. Allowing further a 2% variation in backwashrate, $dv = 0.02\,v$ and assuming $dH = 0.05$ m gives finally

$$H_{bottom} = (0.15)(1.2) + \tfrac{1}{2}(50)(0.05) = 0.18 + 1.25 = 1.43\ \text{m}$$

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 taking suction from 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.

To prevent sudden drops in system pressure when taking backwash water from the distribution system, a large number of filters is necessary. With n filters of unit area A, filterrate v and backwash-rate mv, the ratio between backwash capacity and total production equals

$$\frac{Q_{wash}}{Q_{filters}} = \frac{m\ v\ A}{n\ v\ A} = \frac{m}{n}$$

Limiting this ratio to 0.2 gives with m somewhere between 4 and 7 a minimum number of filters equal to 20 or 35, a large installation indeed. With the system pressure usually much higher than the head required for backwashing an appreciable loss of energy will moreover occur and 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 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, 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. 3.12). 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, 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 is poor practice and 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

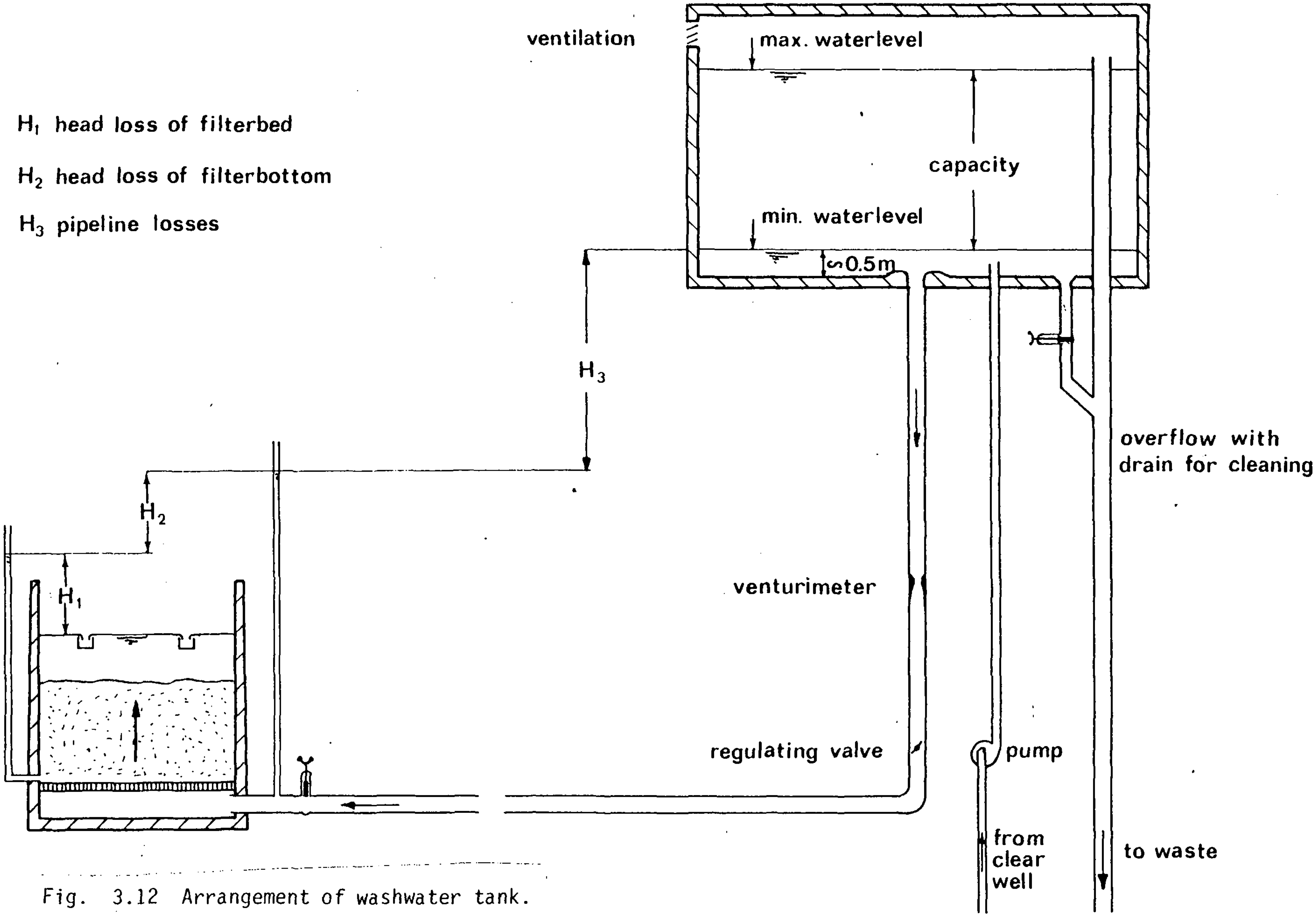
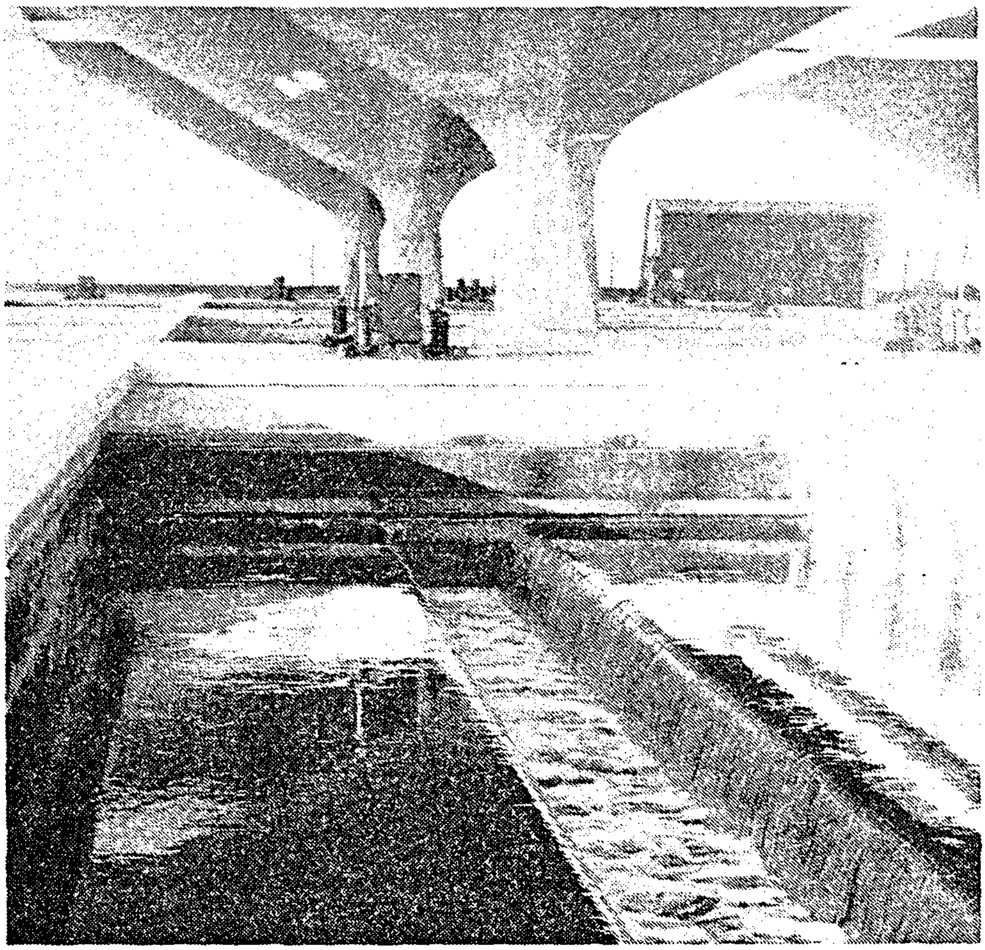


Fig. 3.12 Arrangement of washwater tank.

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 (fig. 3.13), 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!



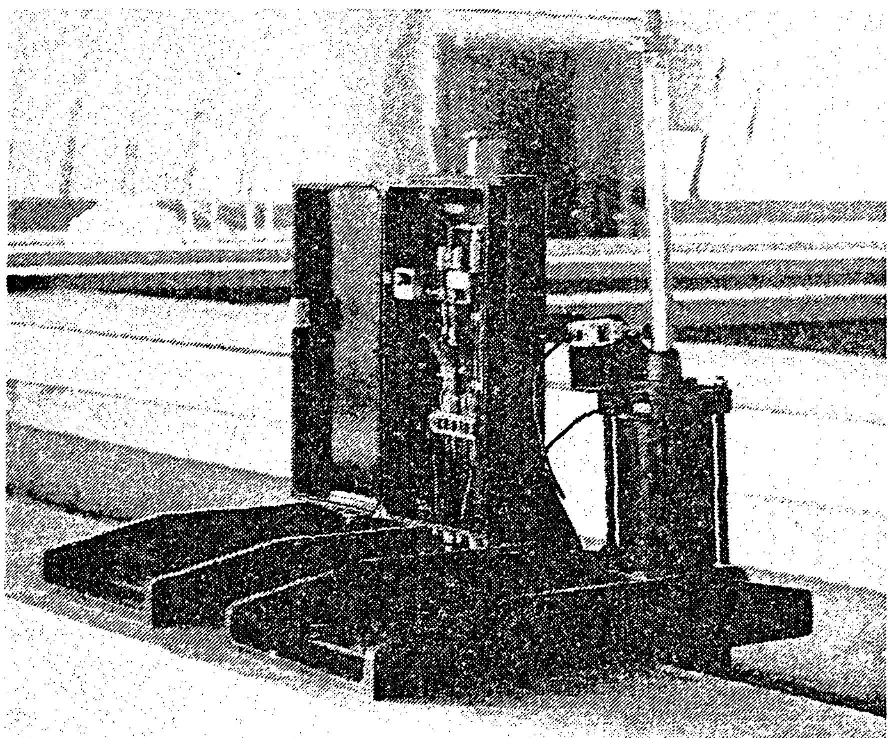


Fig. 3.13 Automatic backwash.

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, 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. 3.14. 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. 3.15. With this so-called water sweep, the length

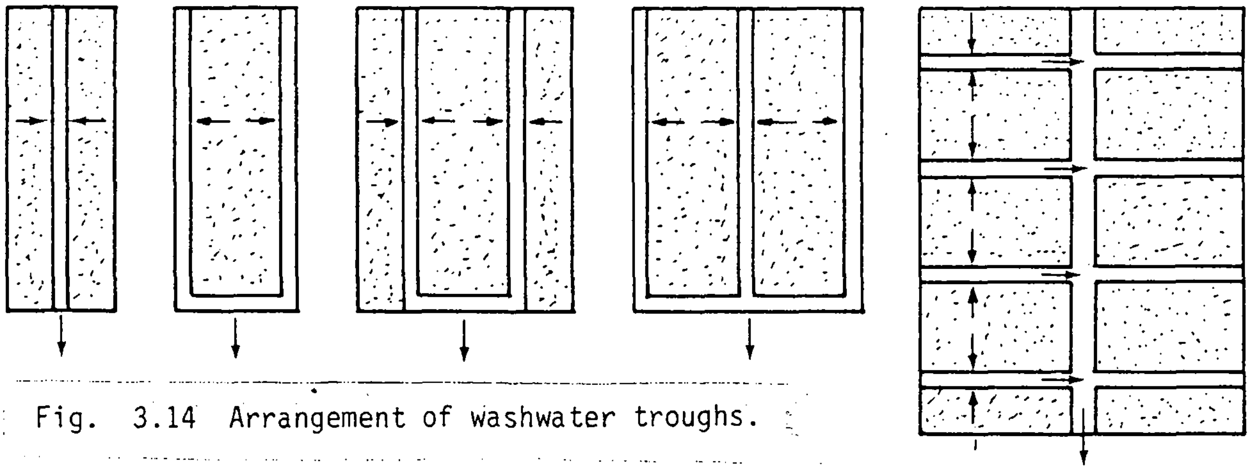


Fig. 3.14 Arrangement of washwater troughs.

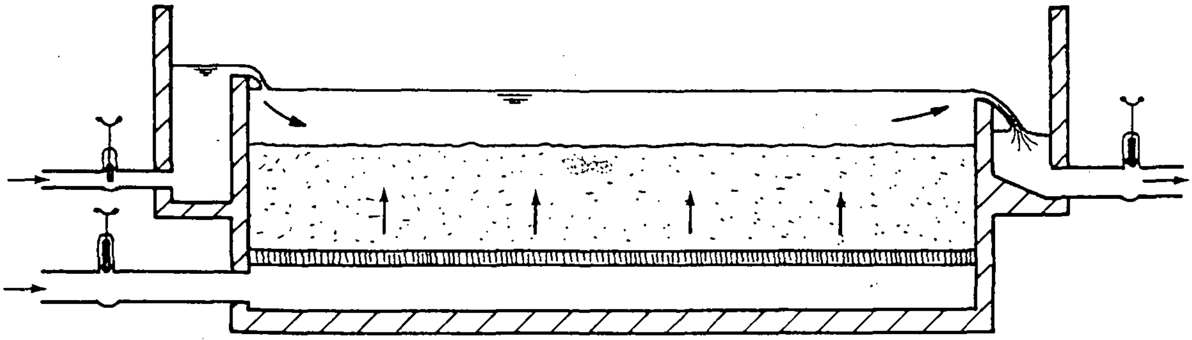


Fig. 3.15 Water sweep.

of horizontal travel may be increased to 10 m and sometimes even more. It has the disadvantage, however, of changing the head available for back-washing (Δ_2H in fig. 3.11), 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. 3.15 by a collapsible one (fig. 3.16) or by a syphon (fig. 3.17), 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. 3.18). With a

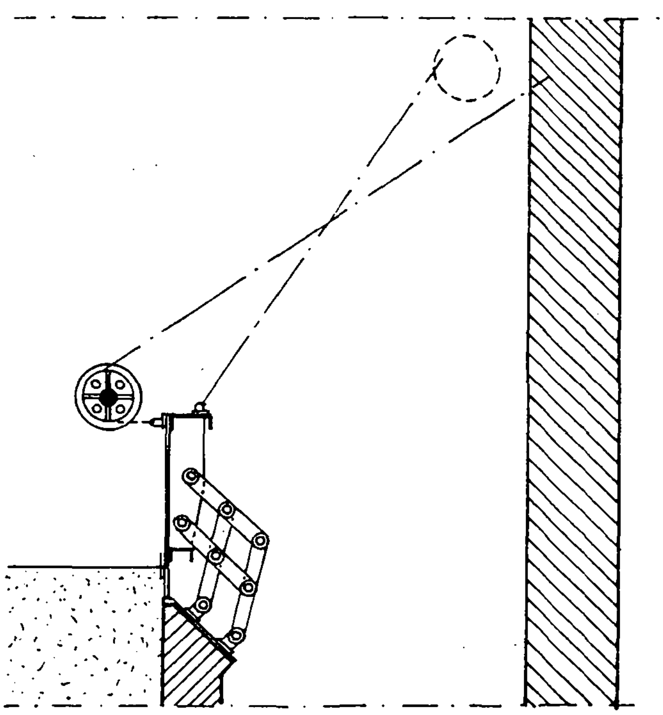


Fig. 3.16 Collapsible weir for washwater removal.

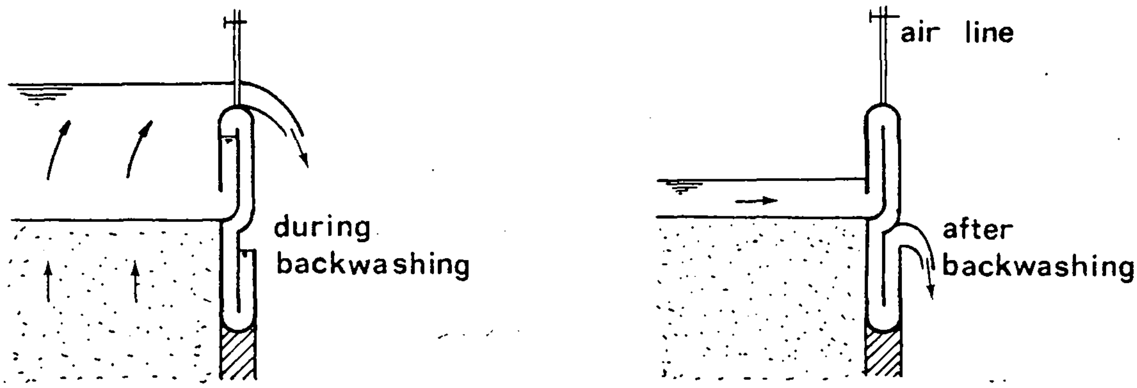


Fig. 3.17 Syphon for washwater removal.

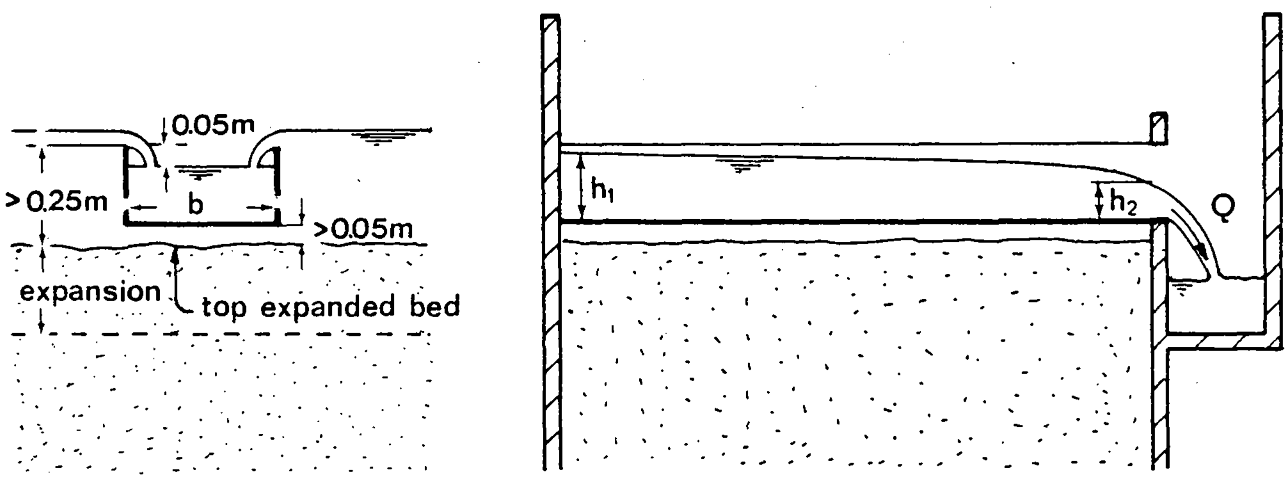


Fig. 3.18 Washwater gutter.

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

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

With free discharging troughs, h_2 closely approximates critical depth

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

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.

.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. 3.19) 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. 3.20). 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-

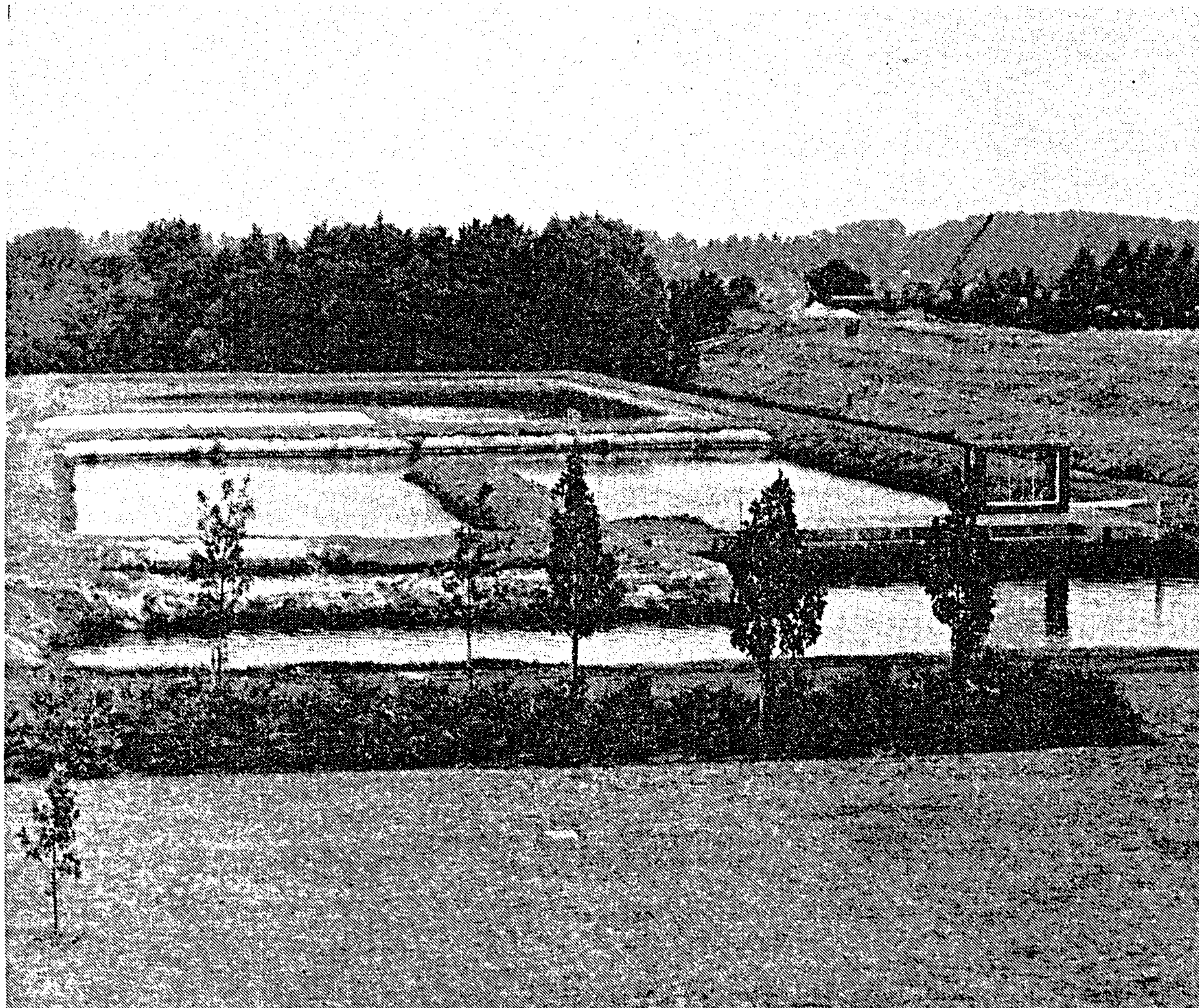


Fig. 3.19 Simple dug basins for washwater purification by settling.

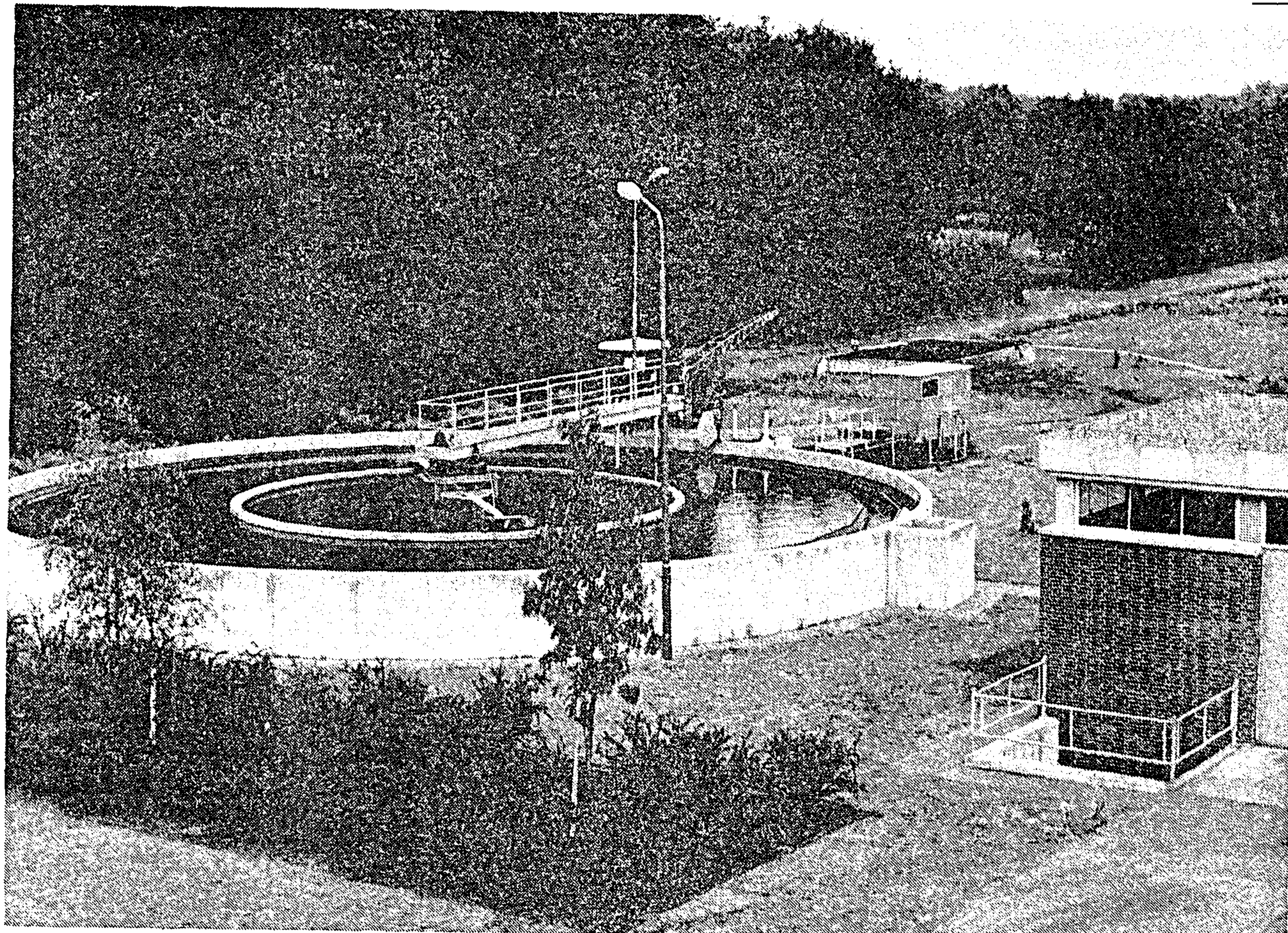


Fig. 3.20 Coagulation supported upward flow sedimentation of washwater at the dune-water treatment plant of Amsterdam municipal water works.

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. 3.21) and one of the many systems for natural or artificial sludge drying (fig. 3.22 and 3.23). In case the sludge is of mineral, inorganic origin only, it may subsequently be dumped, using it for land-fills

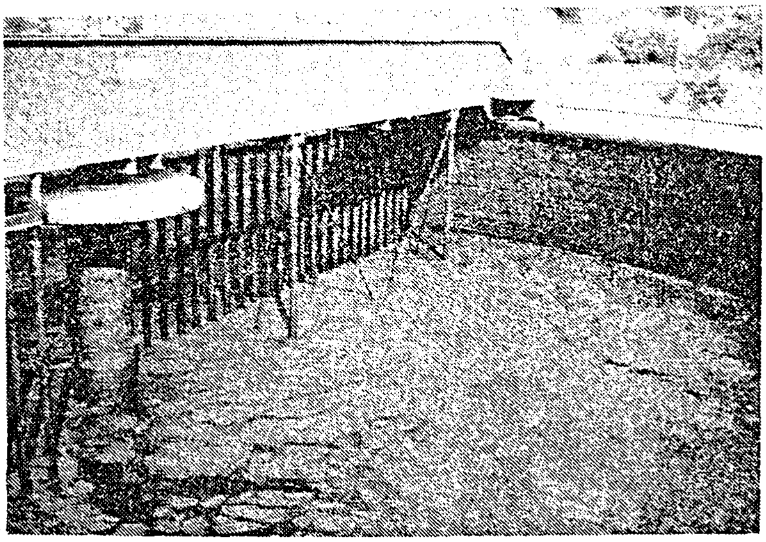


Fig. 3.21 Sludge thickening.

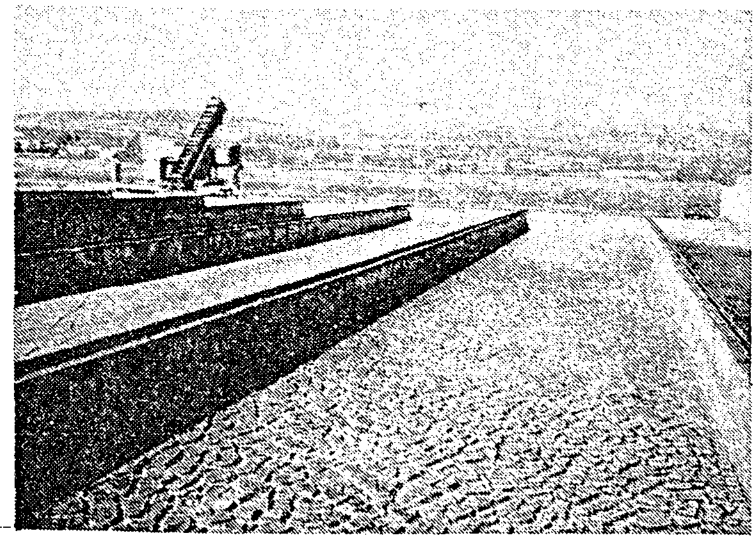


Fig. 3.22 Sludge drying beds (Wiesbaden)

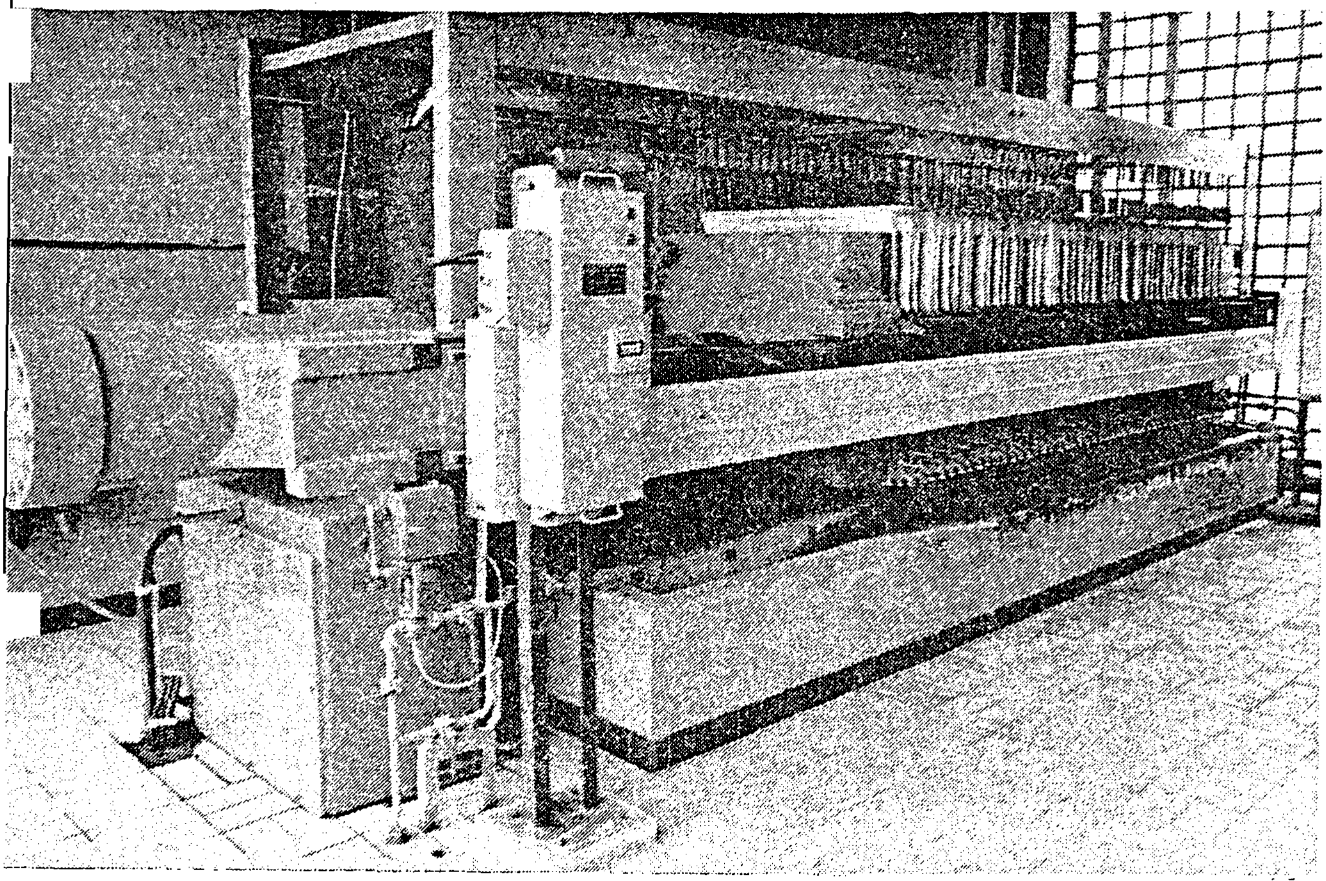


Fig. 3.23 Mechanical sludge drying.

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 as will be described in greater detail in chapter 8. 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. 3.24), which must be taken up by the grain pressures below, with a compression of this layer as unavoidable result. As already explained in sections 3.1 and 3.2, fine grained filtering materials are difficult to keep clean by backwashing with water alone. In many cases they have a sticky gelatinous coating by which the compression mentioned above forms a tough crust, which during backwashing is not disintegrated 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. 3.25). 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. 3.26). As well during backwash as during filtration,

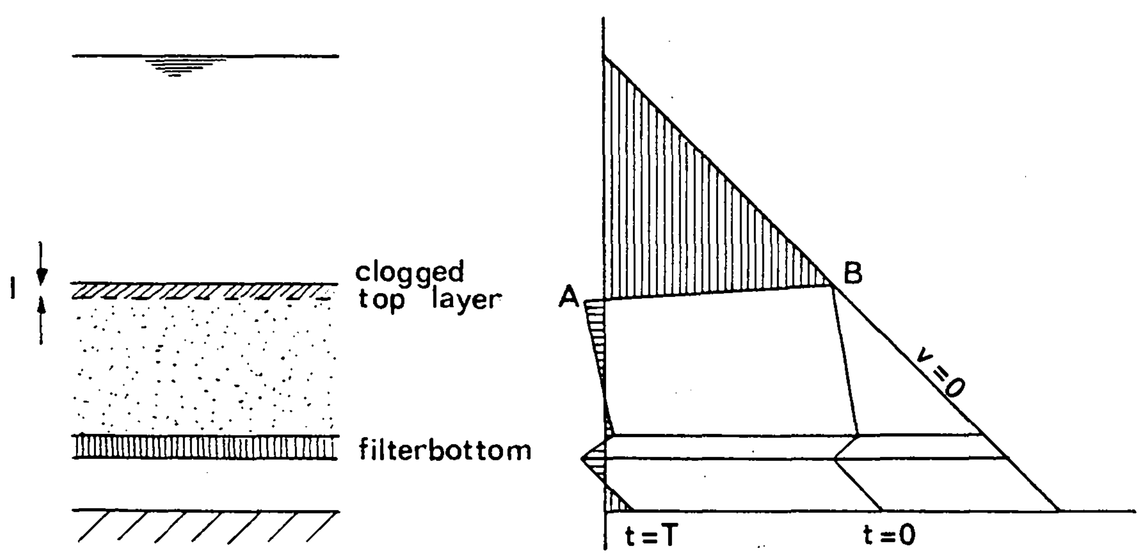


Fig. 3.24 Pressure distribution with surface filtration.

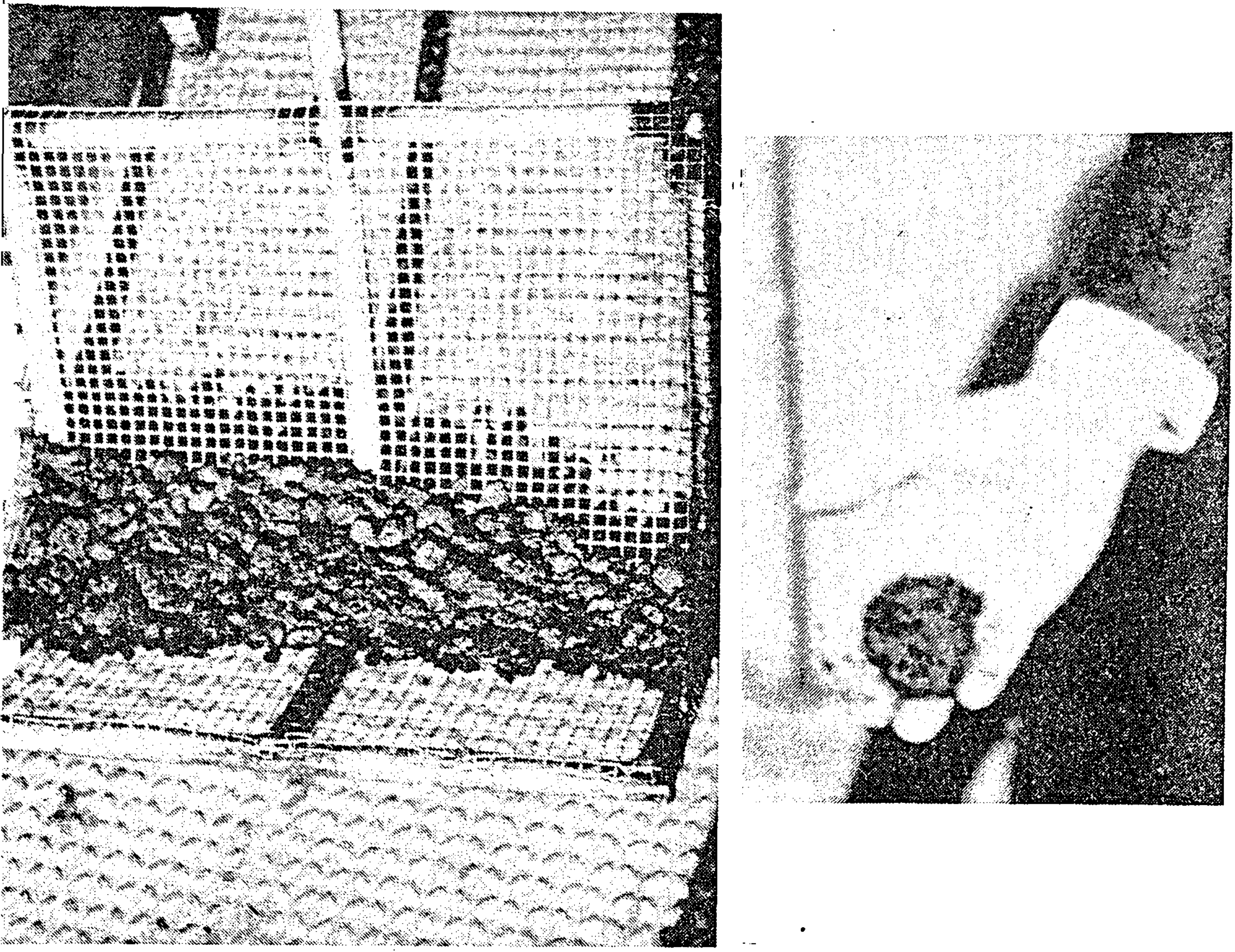


Fig. 3.25 Mud balls on top of a rapid filter.

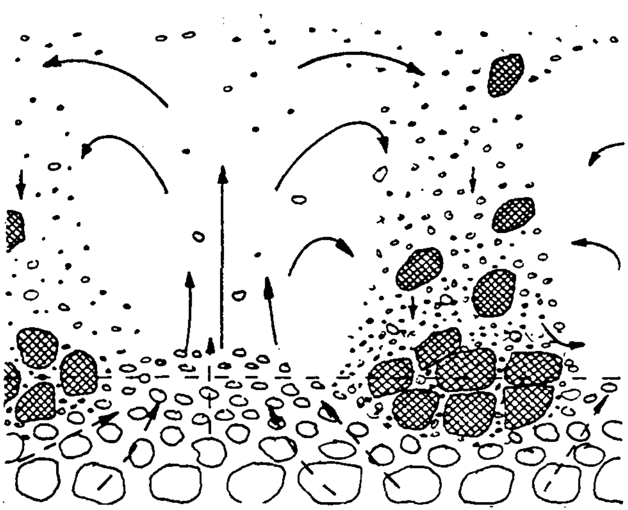


Fig. 3.26 Mud balls clogging a filterbottom.

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. 3.27), 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. 3.28) 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,

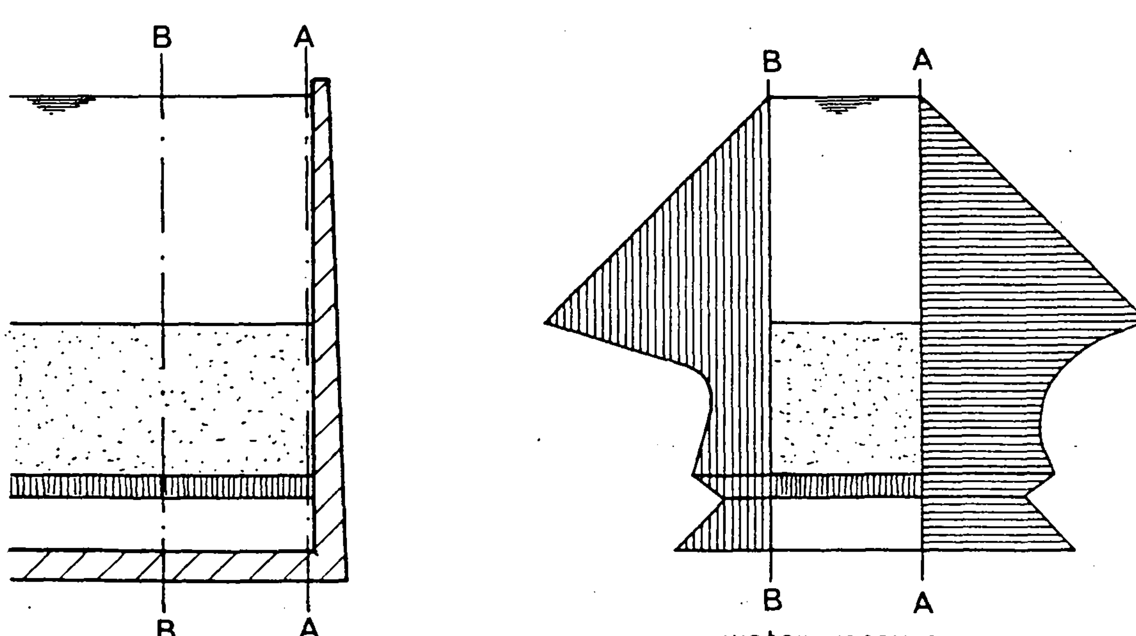


Fig. 3.27 Wall effect.

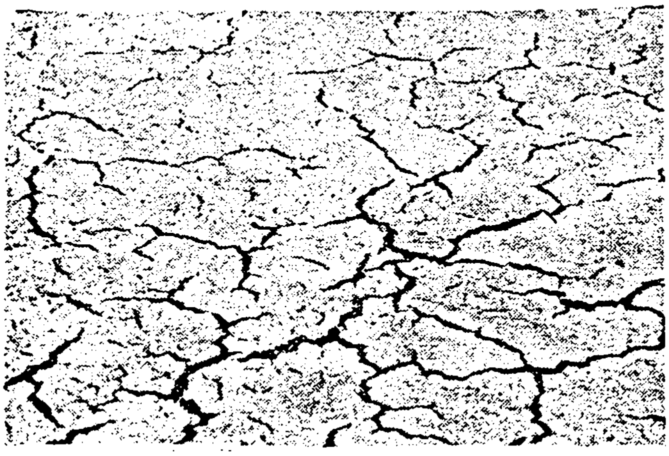


Fig. 3.28 Filter cracks.

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. 3.29.

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. 3.30). 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

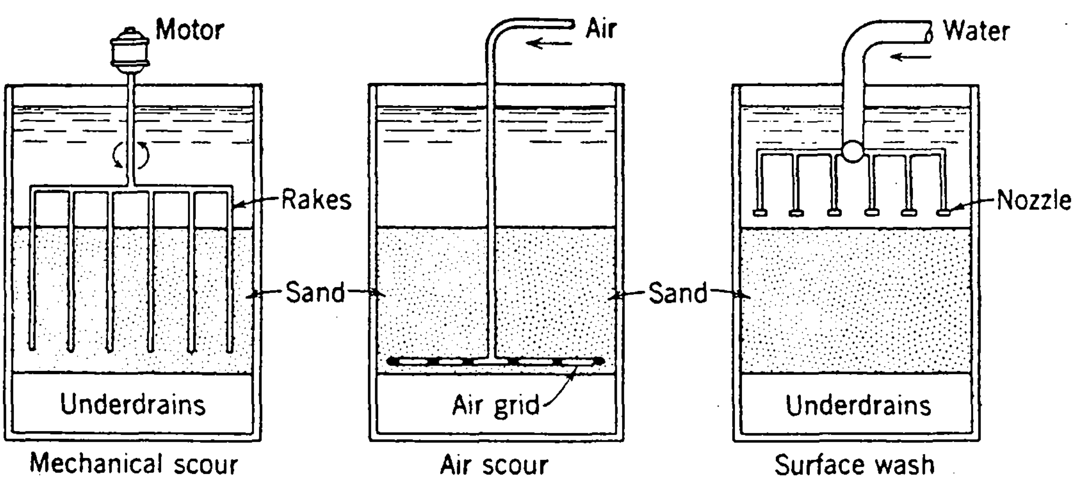


Fig. 3.29 Auxiliary scour.

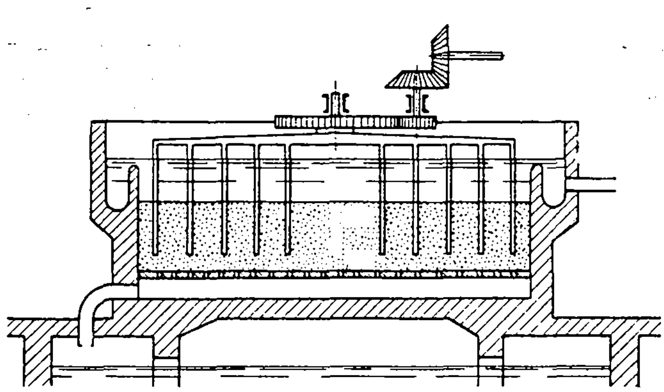


Fig. 3.30 Mechanical rakes for added agitation during backwashing.

cost of construction (fig. 3.31). 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. 3.32). 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

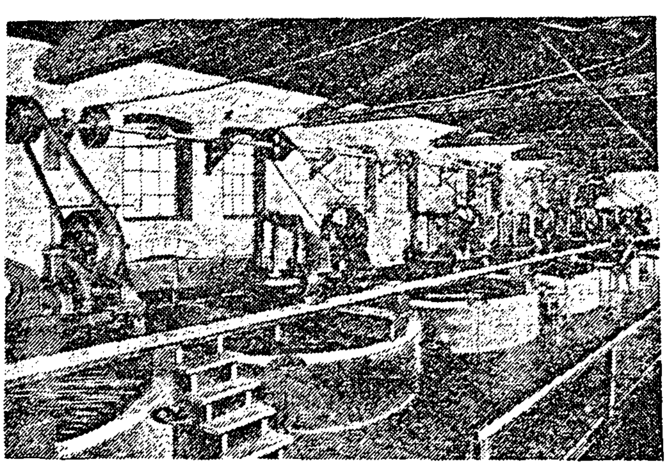


Fig. 3.31 Battery of circular filters with mechanical rakes.

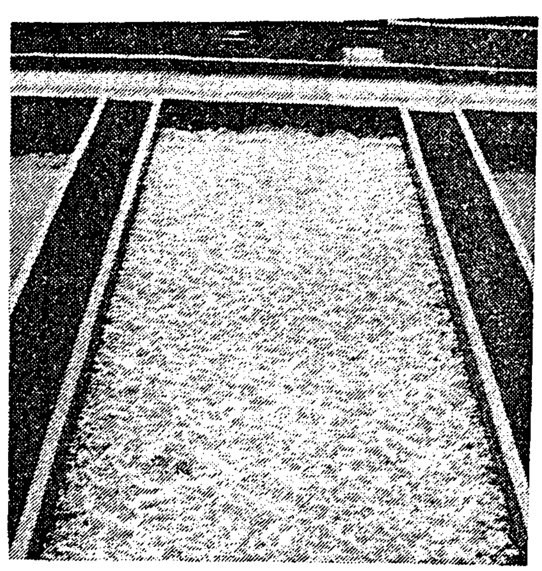


Fig. 3.32 Air wash prior to water wash.

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. 3.33). 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. 3.34). Air-wash in the mean-

Fig. 3.33 Air wash combined with water wash.

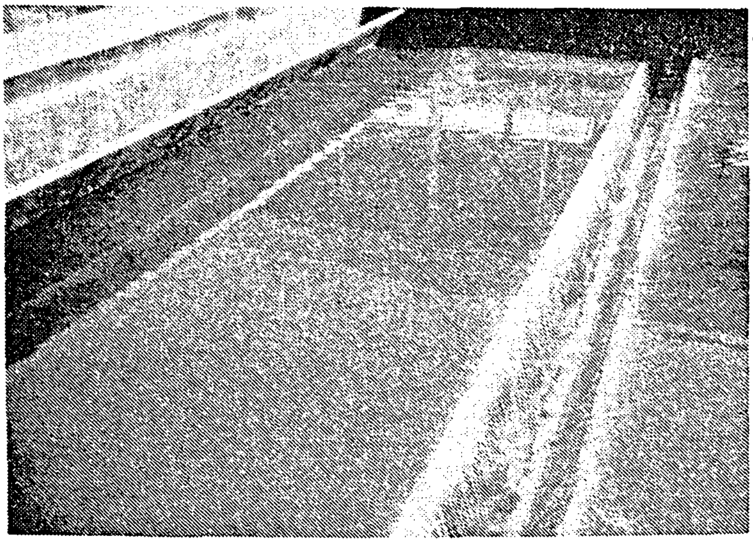


Fig. 3.34 Water wash following air wash.

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. 3.35). 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

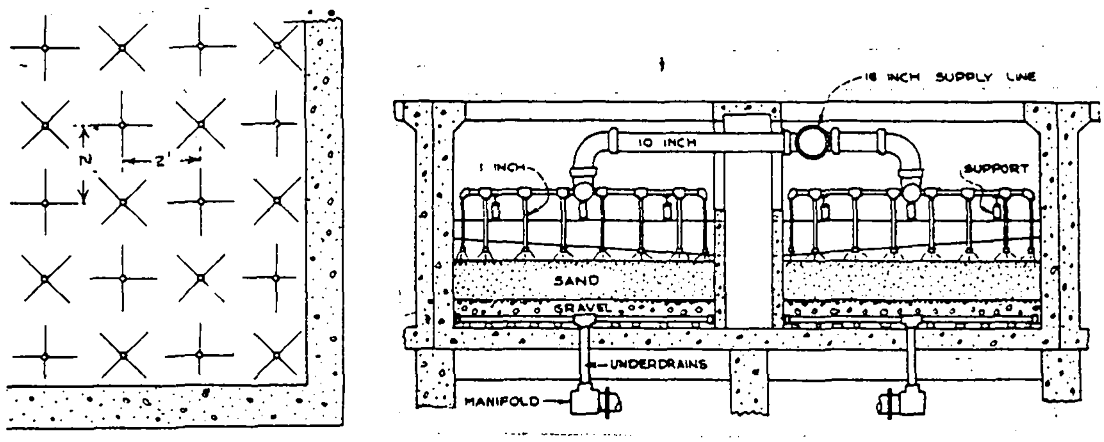


Fig. 3.35 Surface wash with stationary nozzles according to Baylis.

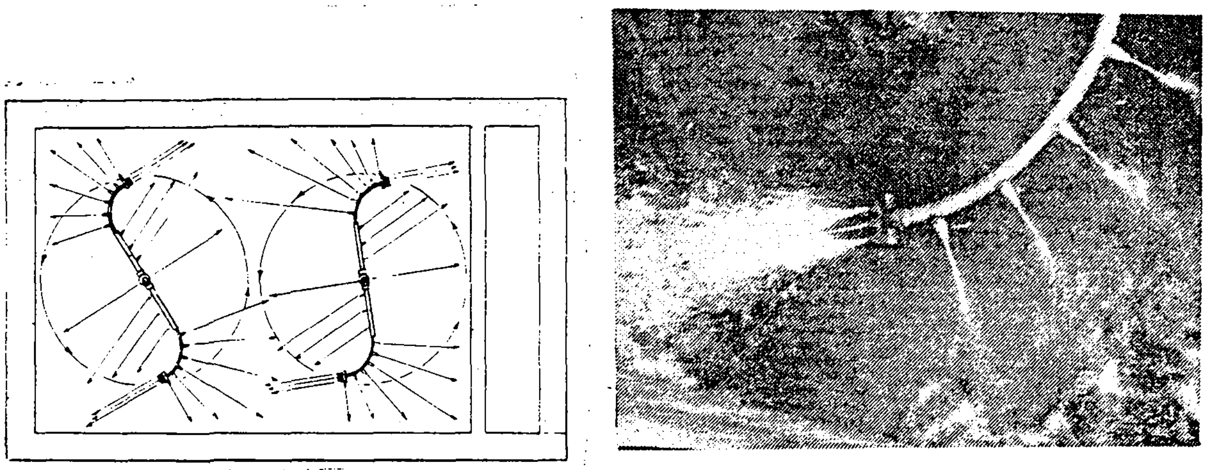


Fig. 3.36 Palmer sweep.

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. 3.36 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. 3.37), 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.

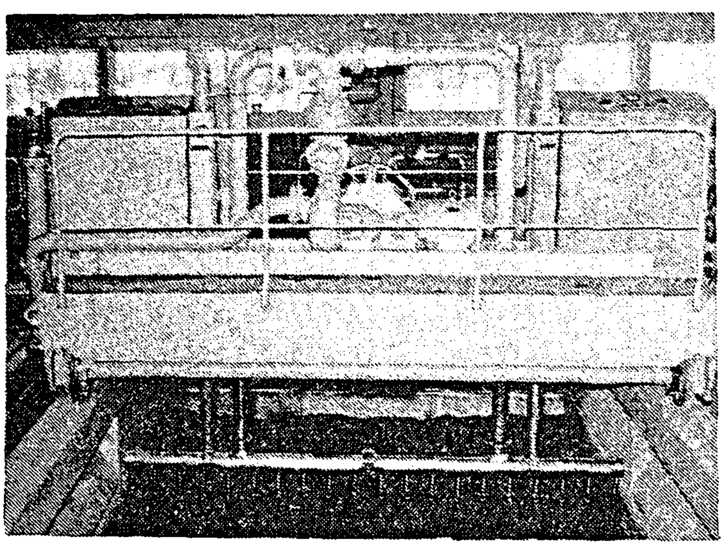


Fig. 3.37 Surface wash mounted on a moving bridge.

. CONSTRUCTION OF A RAPID GRAVITY FILTER PLANT

.1. Plant size

Primarily the size of a filtration plant depends on the total filter-bed area A, which in principle equals the quotient between the amount Q of water to be treated and the filtration rate v to be applied

$$A = \frac{Q}{v}$$

Both factors Q and v are not constant, but show many variations. Generally speaking water consumption will be higher in summer than in winter, higher on working days than in the weekend (fig. 4.1) and higher in daytime than during the small hours of the night (fig. 4.2). The magnitude of these variations will change from one community to another, being smaller as the climate is more uniform and industrial consumption is relativily larger. Filtered water demand also depends on the amount of clear water storage provided. With no storage at all, the capacity of the filtration plant must satisfy the maximum momentarily consumption of 2 to 4 times the average one, while with a large amount of storage a capacity of 1.1 or 1.2 times the average consumption will suffice. As an example of the equalizing influence of clear water storage, fig. 4.3. shows the reduction in peak demand for the Municipal Waterworks of Amsterdam in the Netherlands during the 10 year period 1955-1964. Average water demand finally will go up as years pass by, as well by an increase in the number of population served as by a rise in the per capita per day consumption. Filtration plants are commonly built with a capacity sufficient for the next 10 or 15 years to come. To take care of unforeseen developments, ample opportunities for extension should furthermore be provided.

The maximum allowable filtration rate depends on influent water quality, including temperature, the composition of the filterbed and the desired effluent quality. Especially with surface water supplies, raw water composition may change strongly during the year, showing for instance with the rive Rhine a variation in suspended matter content from 15 to over 100 g/m^3, while water temperature varies from 25°C in summer to 0°C in winter. Going down from 25 to 0°C, however, increases the kinematic viscosity of water by a factor 2, with a corresponding decrease in sedimentation efficiency. The

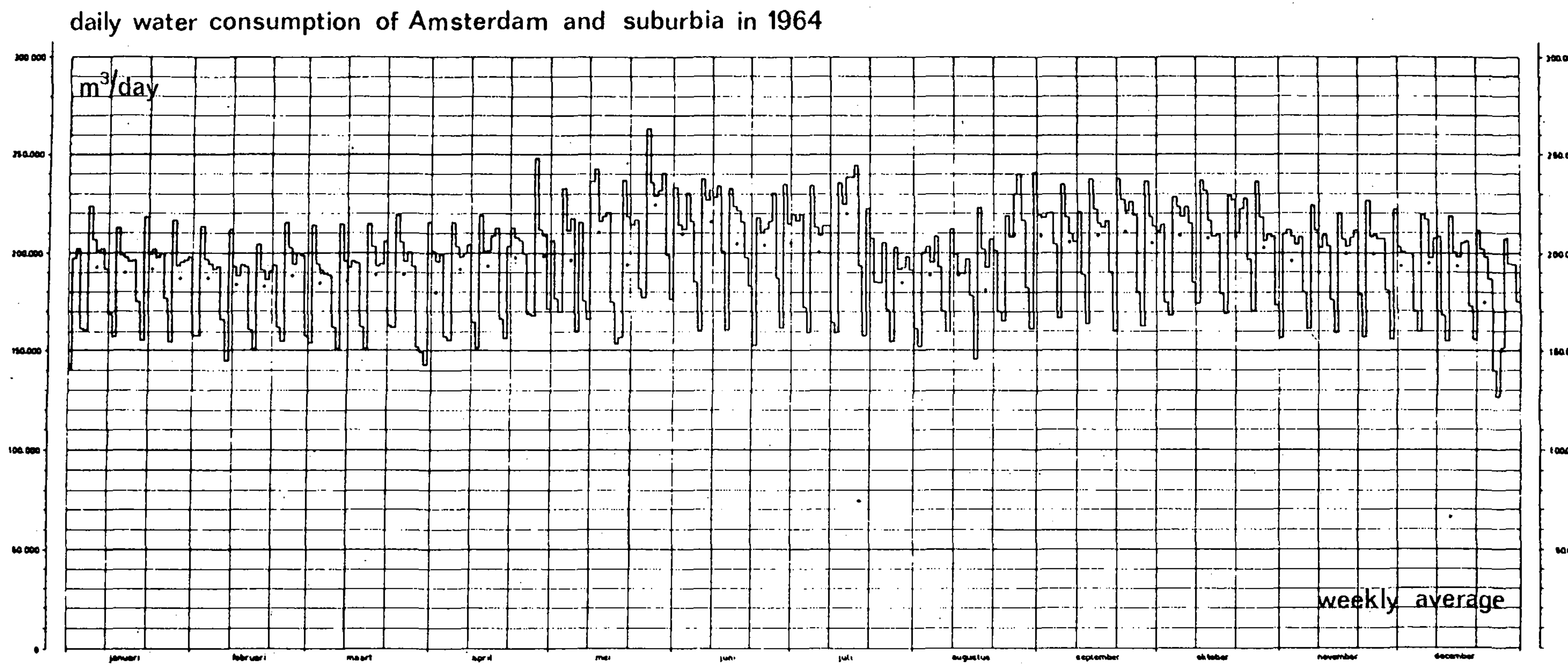


Fig. 4.1 Daily water consumption in Amsterdam and suburbia in 1964.

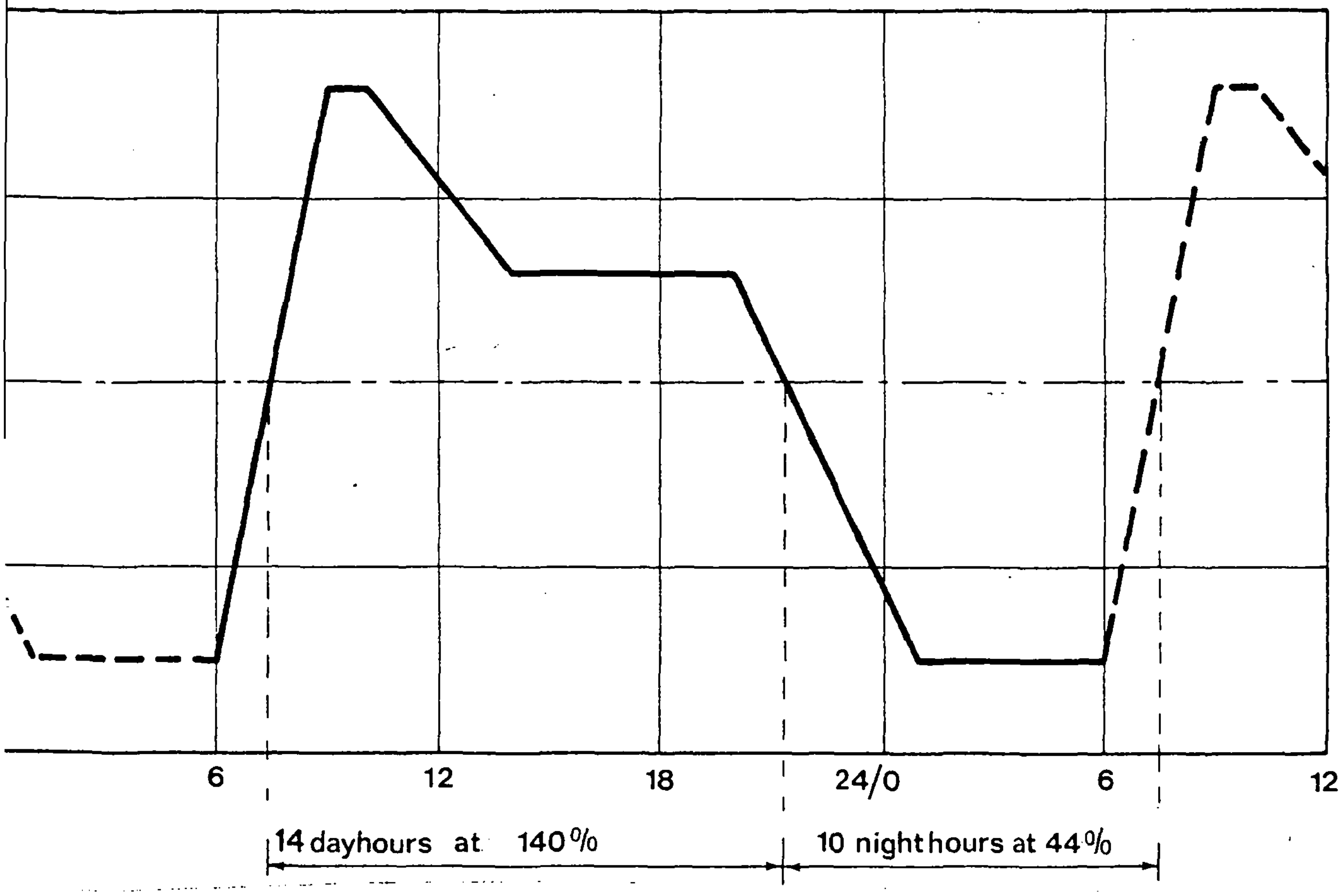


Fig. 4.2 Schematized hourly water consumption in Amsterdam in $^{0}/_{0}$ of average daily consumption.

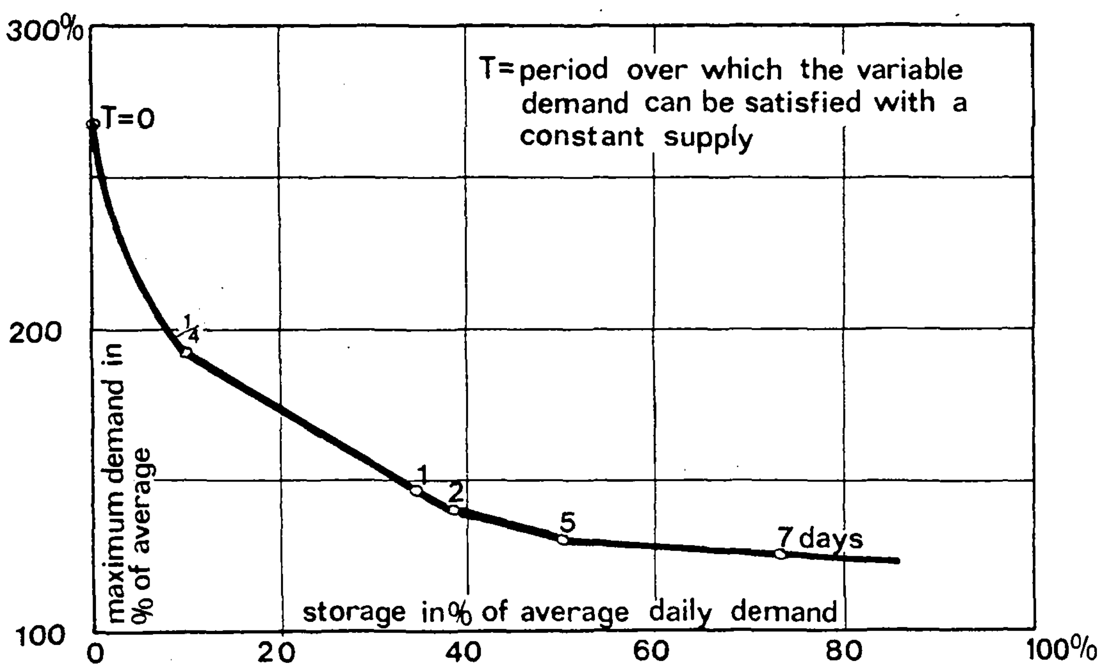


Fig. 4.3 Relation between water demand and storage for the municipal waterworks of Amsterdam during 1955-1964.

same drop in temperature lowers the rate of chemical reactions by a factor 5, while below $4^{o}C$ biological activity ceases nearly completely. With water from the river Rhine, the maximum allowable filtration rate in winter will be only a fractionof the one in summer and here conditions during the cold season will be decisive.

Taking into account the variations in filtered water demand Q and maximum allowable filtration rate v as described above, it will be clear that the required filterbed area A must be re-defined as the maximum value of the ratio between Q and v for matching periods. Assuming a clear water storage sufficient to take care of the hourly fluctuations in water consumption gives for instance

		summer	winter
$\frac{\text{maximum}}{\text{average}}$	daily demand	1.5	1.2
$\frac{\text{maximum}}{\text{average}}$	allowable filtrationrate	2	0.7
required filterbed area A		$\frac{1.5Q}{2v} = 0.75\ \frac{Q}{v}$	$\frac{1.2Q}{0.7v} = 1.7\ \frac{Q}{v}$

with Q and v as long term averages.

With ground-water supplies and constant raw water characteristics, the maximum allowable filtration rate v also is constant and the required filterbed area A only depends on variations in filtered water demand. For the example quoted above, this gives

$$A = \frac{1.5\ Q}{v} = 1.5\ \frac{Q}{v}$$

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 (compare fig. 2.17). 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

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

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).

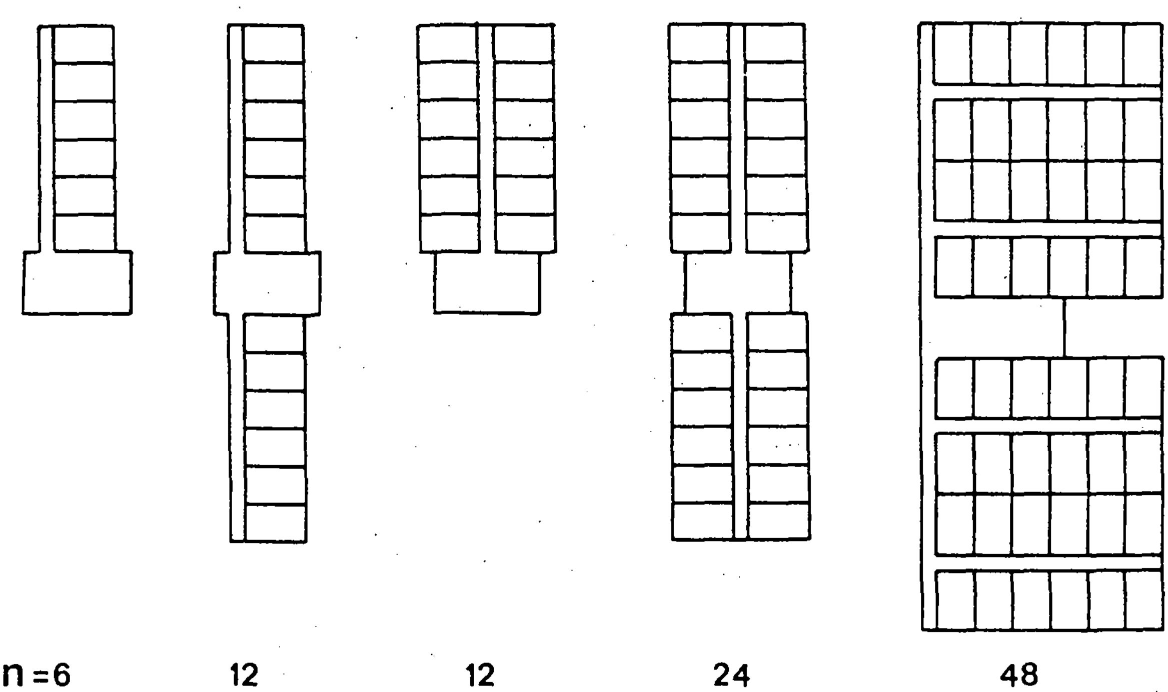


Fig. 4.4 Filter arrangements for n units.

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

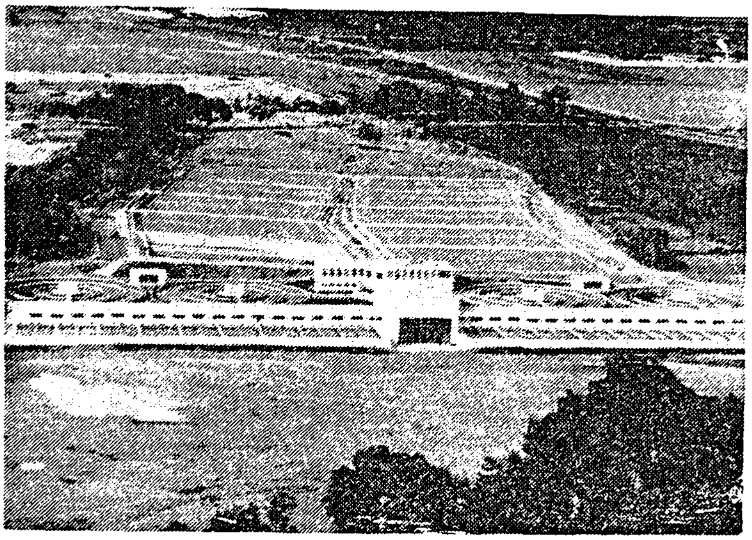


Fig. 4.5 Rapid filtration plant of Lima in Peru.

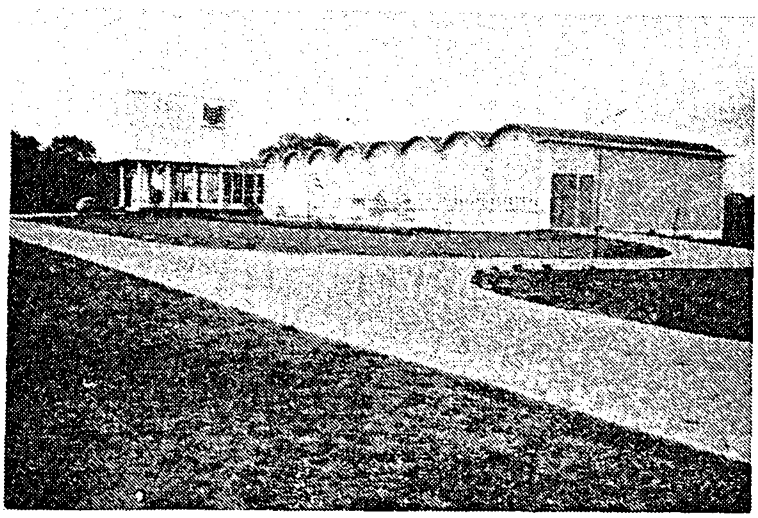


Fig. 4.6 Rapid filtration plant of Amsterdam in Leiduin.

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

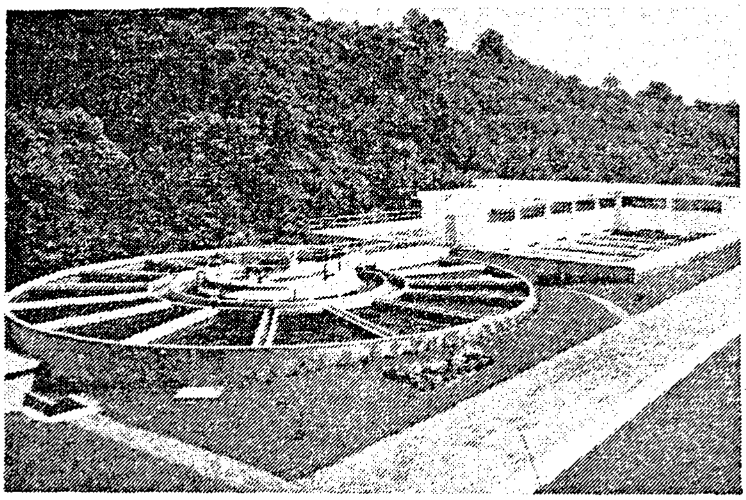


Fig. 4.7 Rapid filtration of the Syndicat Intercommunal de la Forêt de Mervar

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

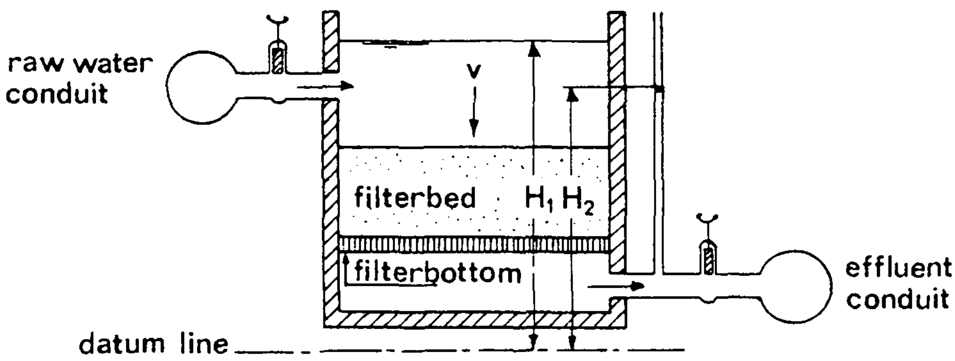
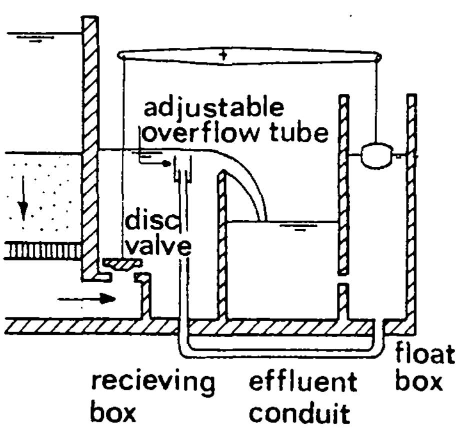


Fig. 4.8 Basic factors in filter control.

of the proportionality constant α 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 α and requiring a larger difference in head $(H_1 - H_2)$ to keep the filtration rate at the intended value. At any time, however, α 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



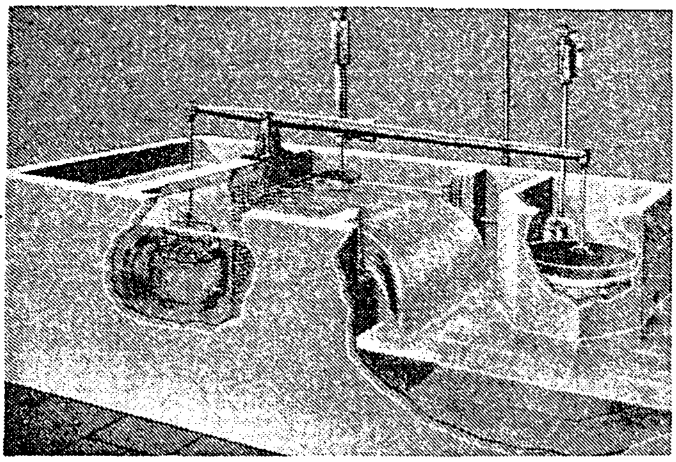


Fig. 4.9 Open filter rate controller (Paterson engineering co LTD,London).

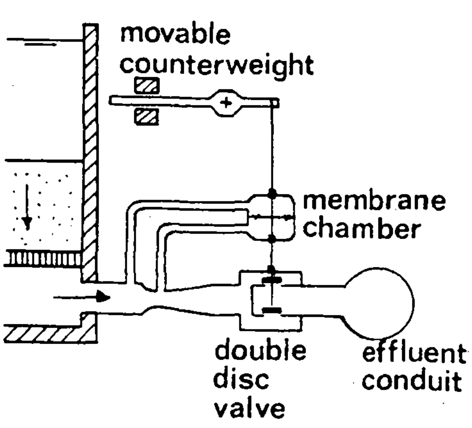
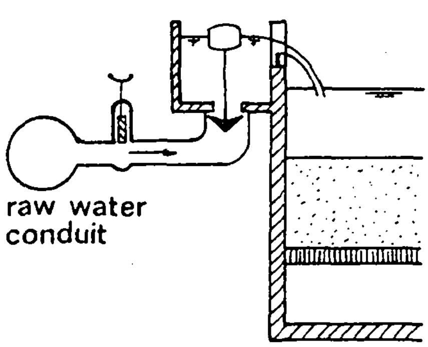


Fig. 4.10 Venturi rate controller with mechanical operation.

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



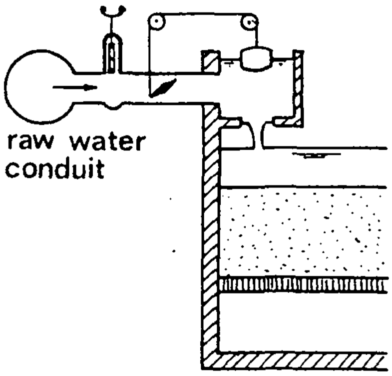


Fig. 4.11 Open filter controllers in raw water influent lines.

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

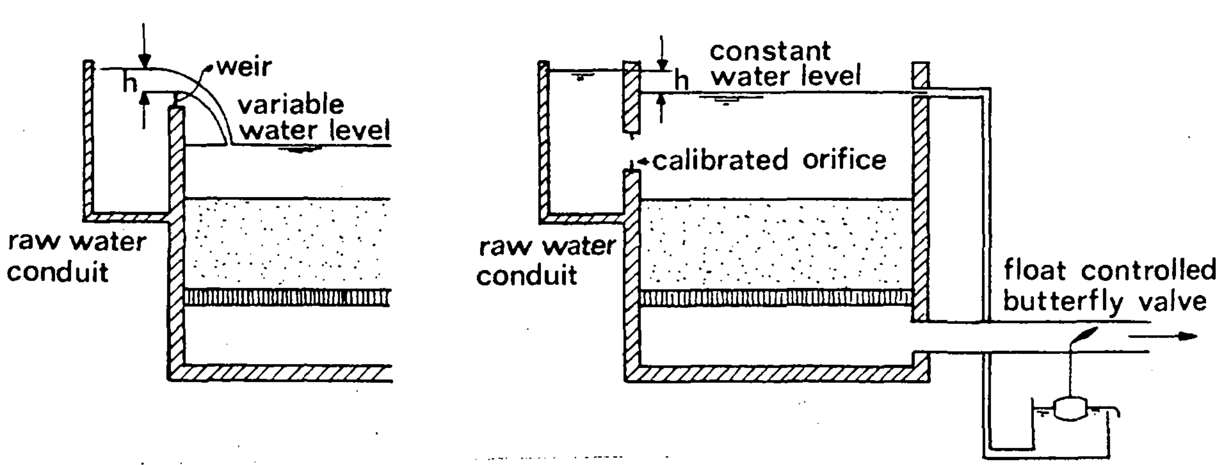


Fig. 4.12 Equal supply of raw water to all filtering units.

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

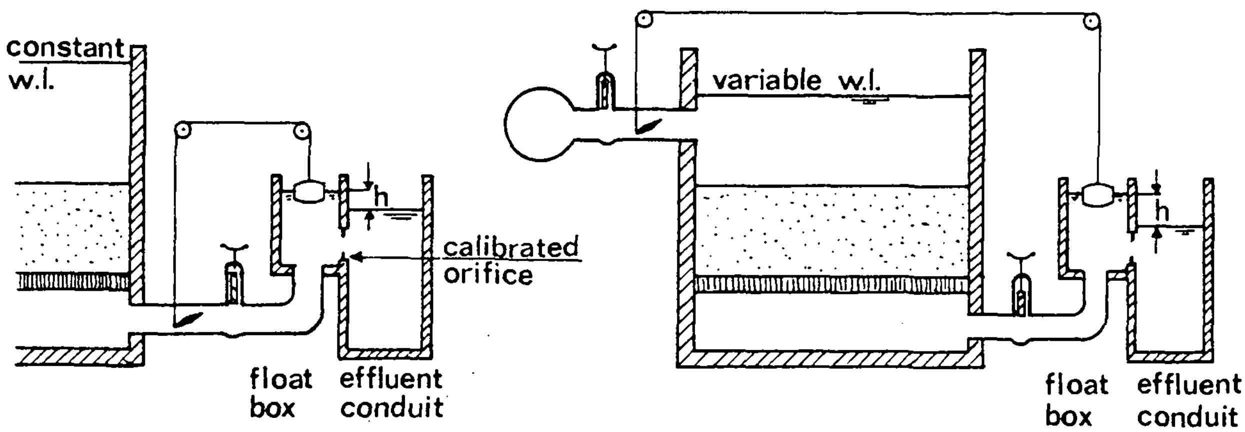


Fig. 4.13 Equal abstraction of filtered water from all filtering units.

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 of varying lengths of filterrun and deteriorating effluent quality are not too serious. According to fig. 2.17 a + or - 10% variation in a filtration rate of $(3)10^{-3}$ m/sec changes the length of filterrun T_r from 31 to 25 or 37 hours. This is not yet objectionable, but a further variation would hinder the proper operation of the plant. From fig. 2.16 the average effluent quality over the full length of filterrun T_r may be calculated and plotted against the filtration rate v. This is done in fig. 4.14 from

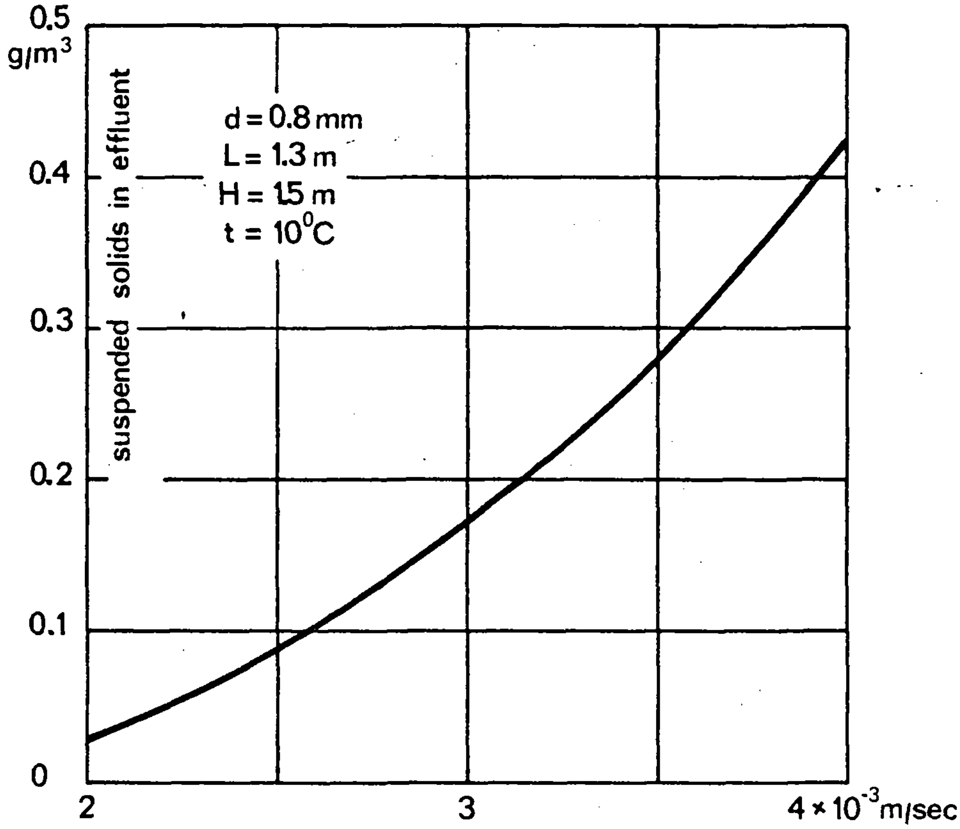


Fig. 4.14 Average effluent quality for the filterruns of fig. 2.16.

which may be taken that the same variation in filtration rate as mentioned above changes the suspended solids content in the effluent from 0.17 to 0.12 or 0.24 g/m^3, in average 0.18 g/m^3. This lowering of the effluent quality by roughly 5% is again not yet objectionable but on the other hand a further reduction is unwanted. It should be realized moreover, that the figures mentioned above are taken from examples. In other cases a 10% variation in filtration rate may certainly be objectionable and this is the reason that in most cases this variation is limited to + and - 5%. When in fig. 4.12 on the left, the water level in the raw water conduit varies from one unit to the other by a maximum of 0.010 m, the overflow height h should be calculated from

$$\frac{dv}{v} = 1.5\,\frac{dh}{h}\,, \qquad 0.05 = 1.5\,\frac{0.01}{h}$$

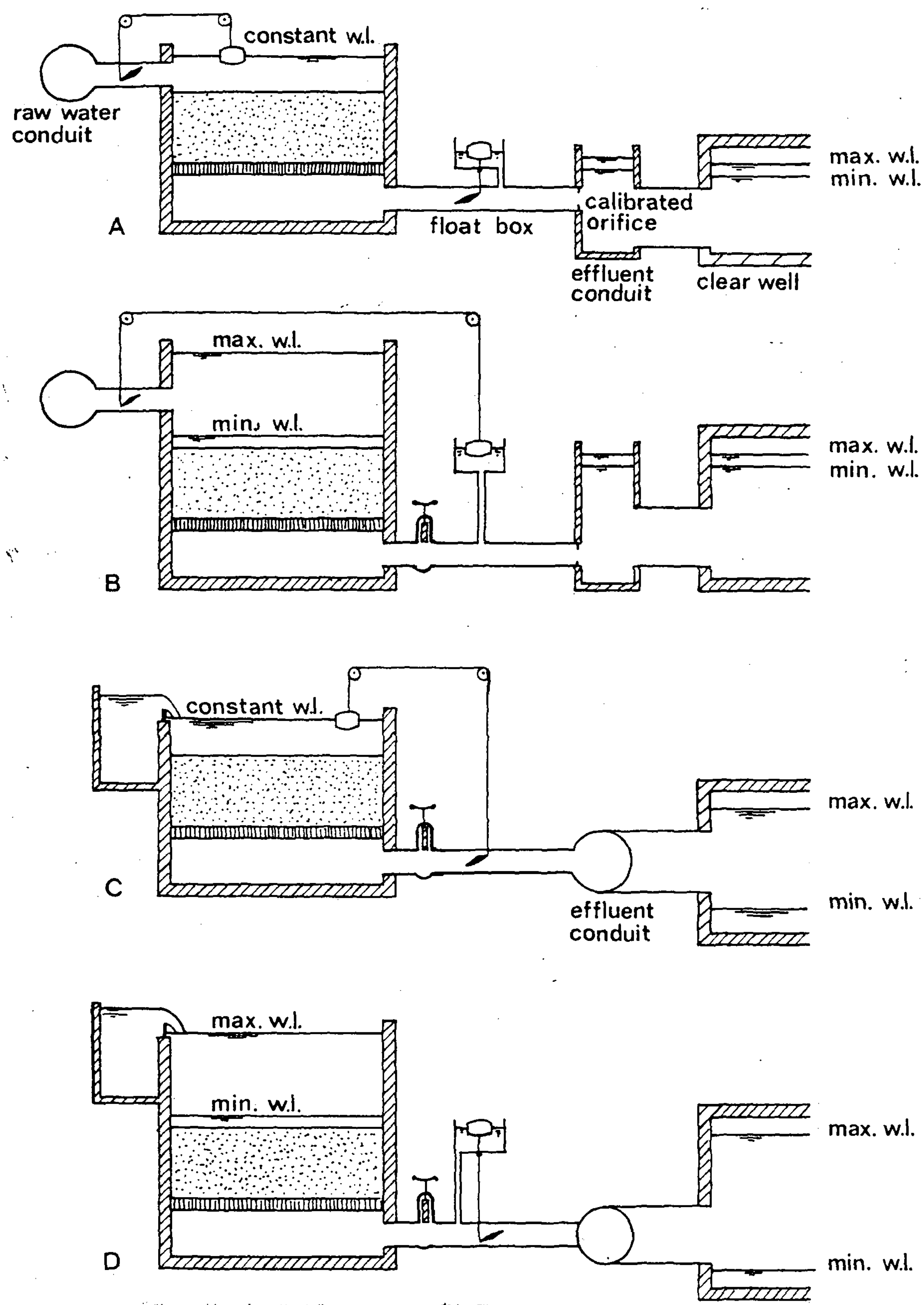
or h = 0.30 m, a rather large value asking for every effort to reduce the value of dh assumed at 0.01 m.

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 water-level 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. 4.15.

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



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. 4.15 Systems of filter control.

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 filterbed 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

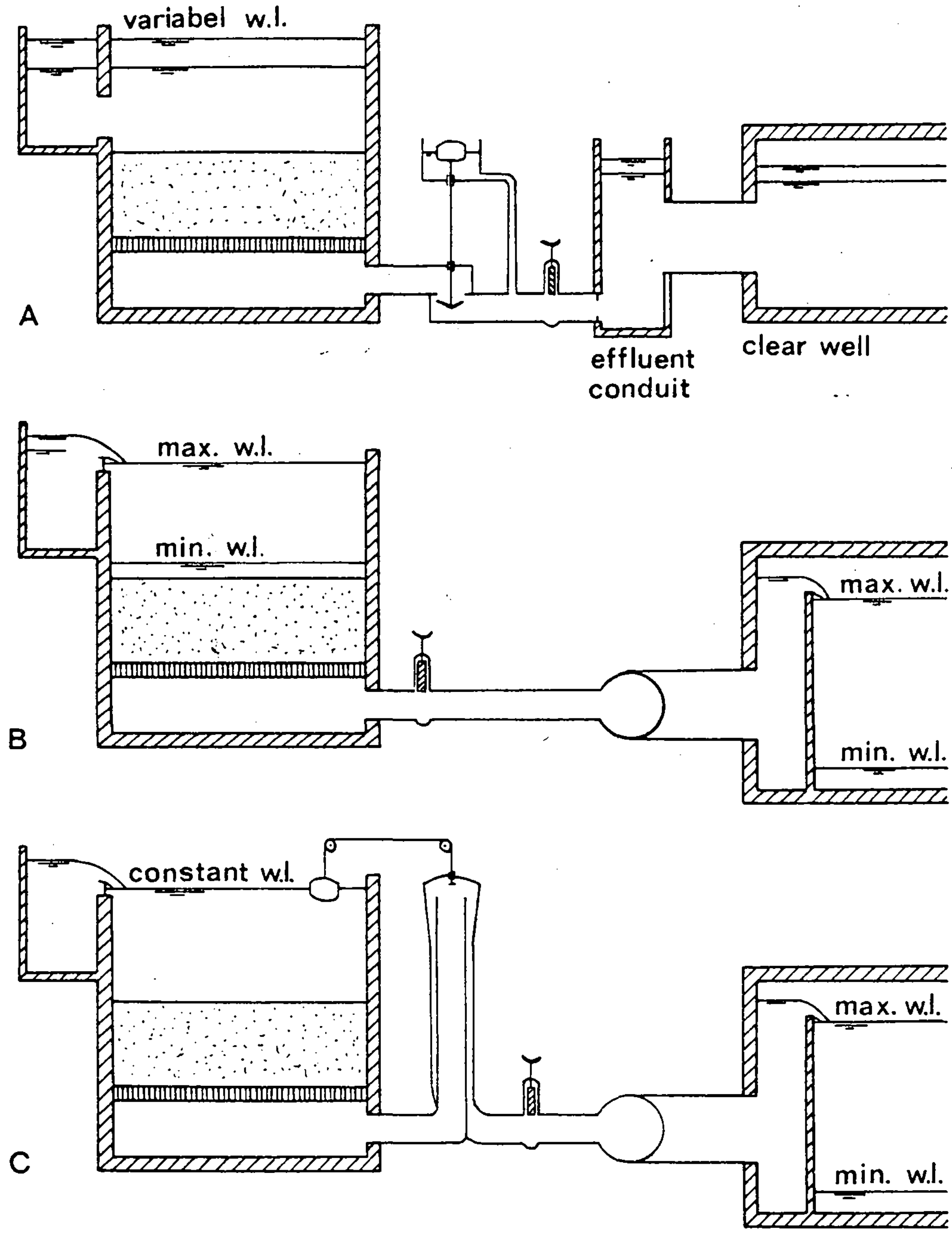


Fig. 4.16 Systems of filter control.

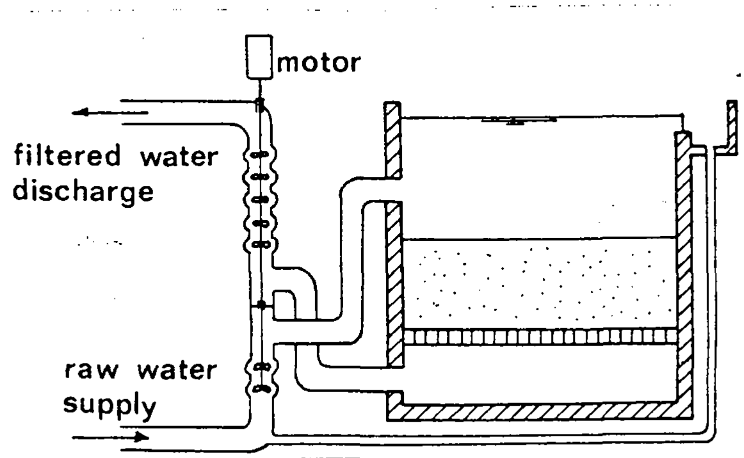


Fig. 4.17 Pump control of a rapid filter.

flow velocities, as well as the application of a variable raw water level, leaving the construction of fig. 4.15 A as only possibility. Still better results may now be obtained with the construction of fig. 4.16 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. 4.15 D by a fixed weir in the effluent line, as shown in fig. 4.16 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. 4.15 C or fig. 4.16 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 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 large, a great depth of raw water on top of the filterbed is necessary. The construction of fig. 4.15 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. 4.15 and fig. 4.16 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. 4.15, but an exception may be made for the pump control of fig. 4.17. 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 with the low cost of energy to-day, the increase in operating cost is next to negligeable.

In the set-up of fig. 4.16 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. 4.18 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

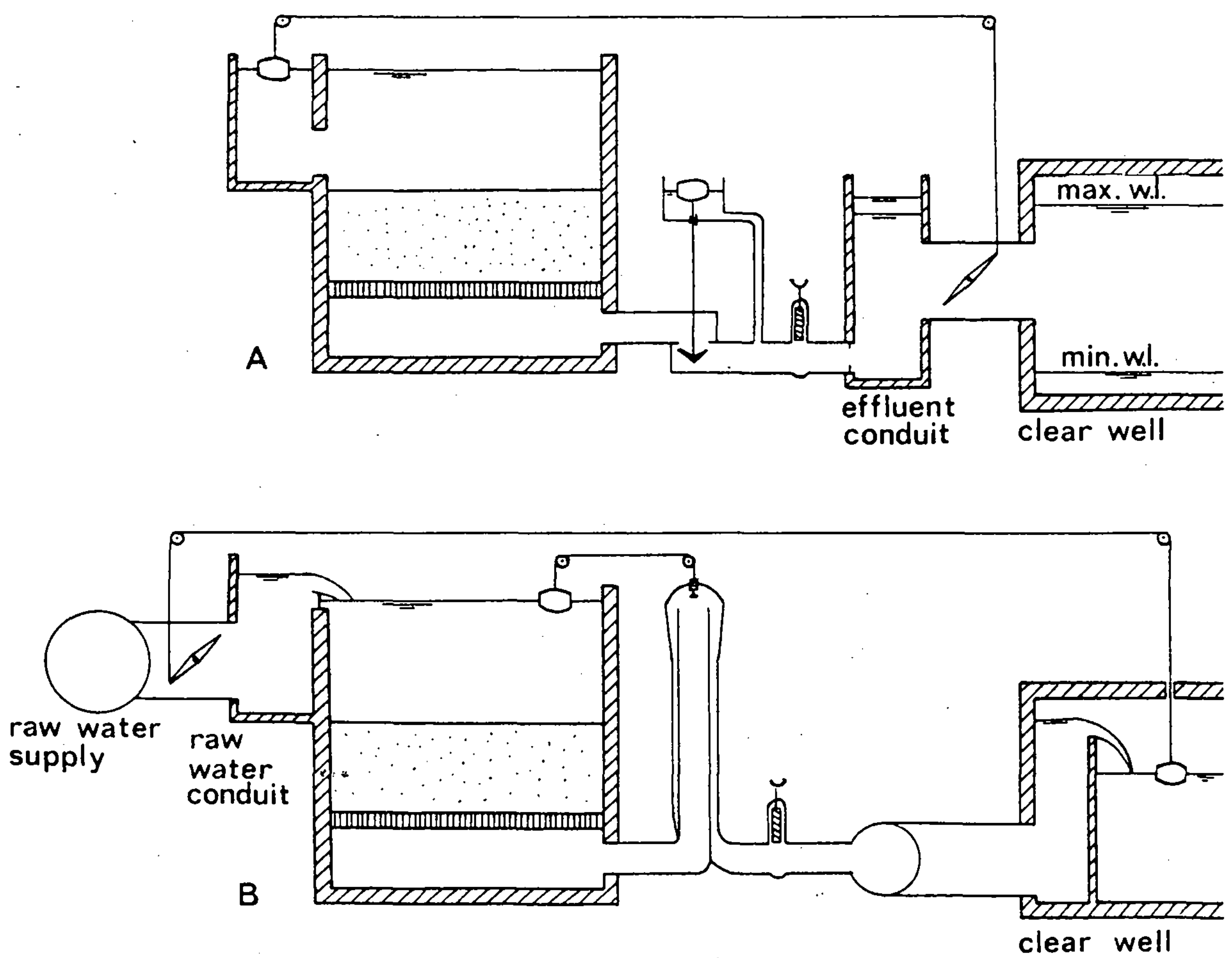


Fig. 4.18 Systems of filter control.

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. 4.16 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. 4.18 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.

4.4. Variable and declining rate filtration

The control systems of the preceding section serve to keep the filtration rate constant and although many of them are quite simple and not expensive, they do limit the designing engineer's freedom of action. Better set-ups in this respect could be obtained with a variable filtration rate, declining as filtration continues and impurities removed from the raw water accumulate in the pores of the filterbed. An example of such a system is shown in fig. 4.19 where the raw water level is kept constant at an equal elevation for all filtering units by manipulating the raw water supply pumps and the piezometric level of the filtered water upstreams of the clear well is kept constant by a fixed weir. The difference between both levels gives the constant head loss H, necessary to overcome the linear resistance of the filterbed and the quadratic resistance of filterbottom and effluent piping

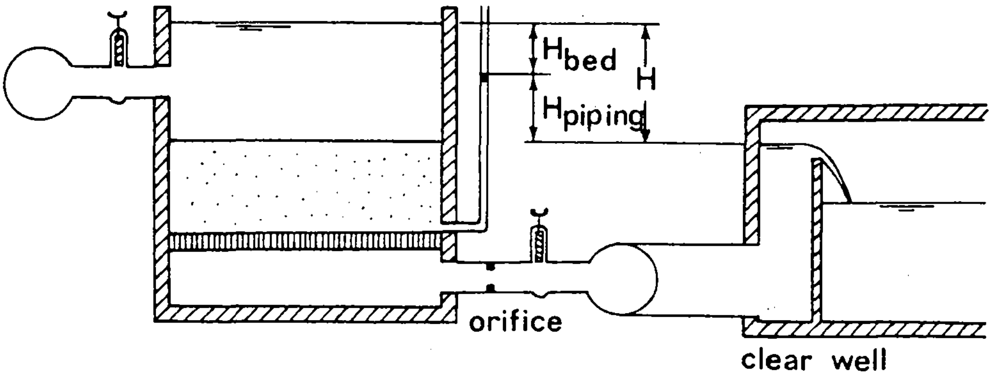


Fig. 4.19 Declining rate filtration.

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

For a specified constructions, β_2 is constant but β_1 will increase with time as the filterbed becomes clogged by deposits from the raw water. This means, however, that the filtration rate will be high in the beginning of the filterrun, declining afterwards continuously. A high rate of filtration in the meanwhile gives a low value of the filtration coefficient. According to the filtration theory of Lerk (section 2.4.)

$$\lambda_o \sim \frac{(1 - p_o)\, p_o}{d_o^3\, \nu\, v}$$

meaning a high amount of impurities in the filtered water at the start of the filterrun

$$t = 0, \qquad\qquad c_e = \frac{c_o}{e^{\lambda_o L} - 1}$$

To demonstrate this phenomenon, the data underlying the calculated filter-run of fig. 2.14 will be used. There the rate of filtration was constant at $(2)10^{-3}$ m/sec, giving an initial resistance of the filterbed of 0.41 m. The maximum resistance of this bed was assumed at 1.5 m, to which must still be added a head loss of about 0.2 m to overcome losses of friction and turbulence in filter bottom and effluent piping. With declining rate filtration and all other factors unchanged, the filterrun starts with a filtration velocity v determined by

$$1.5 + 0.2 = 1.7 = \frac{v}{(2)10^{-3}}\, 0.41 + \left(\frac{v}{(2)10^{-3}}\right)^2 0.2$$

from which follows

$$v = (4.13)10^{-3} \text{ m/sec.}$$

This lowers the value of the filtration coefficient λ_o from 8.956 m^{-1} (section 2.4.) to

$$\lambda_o^1 = \frac{(2.00)\ 10^{-3}}{(4.13)\ 10^{-3}}\ 8.956 = 4.337\ m^{-1}$$

giving with a filterbed thickness of 0.75 m and a raw water turbidity of 15 g/m^3 an initial effluent quality of

$$c_f = \frac{15}{e^{(4.337)\ (0.75)} - 1} = 0.60\ g/m^3$$

or much higher than the maximum allowable value assumed at 0.5 g/m^3. To improve this situation, an orifice must be installed in the discharge pipe, increasing the resistance of filter bottom and effluent piping from 0.2 m to for instance 0.5 m water column at a filtration rate of $(2)10^{-3}$ m/sec. The initial filtration rate v now follows from

$$2.0 = \frac{v}{(2)10^{-3}}\ 0.41 + \left(\frac{v}{(2)10^{-3}}\right)^2\ 0.5 \qquad \text{or}$$

$$v = (3.26)10^{-3}\ \text{m/sec}, \qquad \lambda = \frac{(2.00)10^{-3}}{(3.26)10^{-3}}\ 8.956 = 5.494\ m^{-1}$$

$$c_e = \frac{15}{e^{(5.494)\ (0.75)} - 1} = 0.25\ mg/l$$

or well below the maximum allowable value.

As filtration continues, clogging of the filterbed occurs, decreasing the value of the filtration coefficient λ

$$\lambda = \lambda_o \left(1 - \frac{\sigma_v}{0.75\ p_o}\right)$$

while at the same time the resistance increases, lowering the filtration rate and augmenting the value of λ_o. The result of both actions is a more gradual deterioration of effluent quality with time than shown in fig. 2.14. With a judicious design it is even possible to increase the length of filterrun T_q, while by a more even distribution of impurities over the depth of the filterbed also the length T_r will be larger, improving operating conditions. The average effluent quality, however, is less and the mixed effluent from all the filtering units will carry a higher amount of impurities, although well below the maximum allowable value.

In fig. 2.14 the filterrun T was broken off at $t = T_r$, when with a constant rate of $(2)10^{-3}$ m/sec the resistance of the filterbed reached a value of 1.5 m watercolumn. With declining rate filtration and using an orifice to increase the resistance, operating conditions at this moment are given by

$$2.0 = \frac{v}{(2)10^{-3}}\,1.5 + \left(\frac{v}{(2)10^{-3}}\right)^2 0.5 \quad \text{or} \quad v = (2.00)10^{-3} \text{ m/sec}$$

meaning that back-washing of the filter is only necessary when the combined capacity of the plant is insufficient to take care of filtered water demand or raw water supply. The most simple operating system may now be obtained by adapting raw water supply to the combination of filtered water demand and the amount of filtered water storage, while back-washing of the filters operating at the lowest rate is necessary when the raw water level of the filters tends to surpass the maximum allowable elevation.

As mentioned in the beginning of section 4.3, a sudden increase in filtration rate might cause previously deposited material to be resuspended and flushed through the filterbed, thus impairing filtered water quality. According to practical experience, the filtration rate should therefore not increase by more than 3% per minute or

$$\frac{dv}{v} < \frac{0.03}{60}\,dt \quad \text{and}$$

$$\frac{dv}{dt} < (0.5)10^{-3} v \text{ m/sec}^2$$

This condition if fulfilled automatically with the declining rate filtration of fig. 4.19. Suppose that here the raw water supply is increased suddenly by p%, resulting in an increase in filtration rate from v_o at $t = o$ to v at $t = t$. According to the continuity equation, the amount of water supplied in time dt equals the amount of water filtered in this period plus the amount of water stored by a rise of the raw water level. Per unit area

$$(1 + \frac{p}{100})v_o\,dt = v\,dt + dH \quad \text{or} \quad \frac{dH}{dt} = (1 + \frac{p}{100})v_o - v$$

Under all circumstances, the loss of head H equals the resistance of the filterbed together with the resistance of filterbottom and effluent piping

$$H = \beta_1 v + \beta_2 v^2$$

$$\frac{dH}{dt} = \beta_1 \frac{dv}{dt} + 2\beta_2 \, v \frac{dv}{dt} \, , \quad \text{together}$$

$$\frac{dH}{dt} = \frac{2H - \beta_1 v}{v} \; \frac{dv}{dt} \qquad \text{and}$$

$$\frac{dv}{dt} = \frac{v}{2H - \beta_1 v} \; \{ \, (1 + \frac{p}{100}) v_o - v \, \}$$

This differential quotient reaches its maximum value at $t = o$, $v = v_o$, at the beginning of the filterrun when β_1 has it smallest value.
According to the figures given above

$$H = 2.0 \text{ m} \qquad \beta_1 v = \frac{(3.26)10^{-3}}{(2.00)10^{-3}} \, 0.41 = 0.67 \text{ m.}$$

Substituted

$$\frac{dv}{dt} = \frac{(3.26)10^{-3}}{4.00 - 0.67} \, \frac{p}{100} \quad v_o$$

giving as requirement

$$\frac{(3.26)10^{-3}}{4.33} \; \frac{p}{100} \; v_o < (0.5)10^{-3} \; v_o \qquad \text{or}$$

$$p < 66$$

that is to say a sudden increase in raw water supply by 66%! With the variable raw water level of fig. 4.16 B the allowable increase is much smaller, about 10% only but still higher as mostly used in actual practice for larger filtration plants, where raw water supply commonly follows filtered water demand rather closely. Real difficulties arise with those control systems where the raw water level is kept constant. Here a careful check of the maximum expectable increase in filtration rate is necessary, limiting the increase in raw water supply or asking for additional damping devices.

With regard to sudden changes in filtration rate and their injurious effect on effluent quality, the control systems of the preceding section even have many disadvantages. When for instance the system of 4.16 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, great 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 4.16 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.

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 are nowadays 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. 4.20. Through these openings the washwater is directed downward, either vertically or under an angle of 30° to 45° with the vertical (fig. 4.21). 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

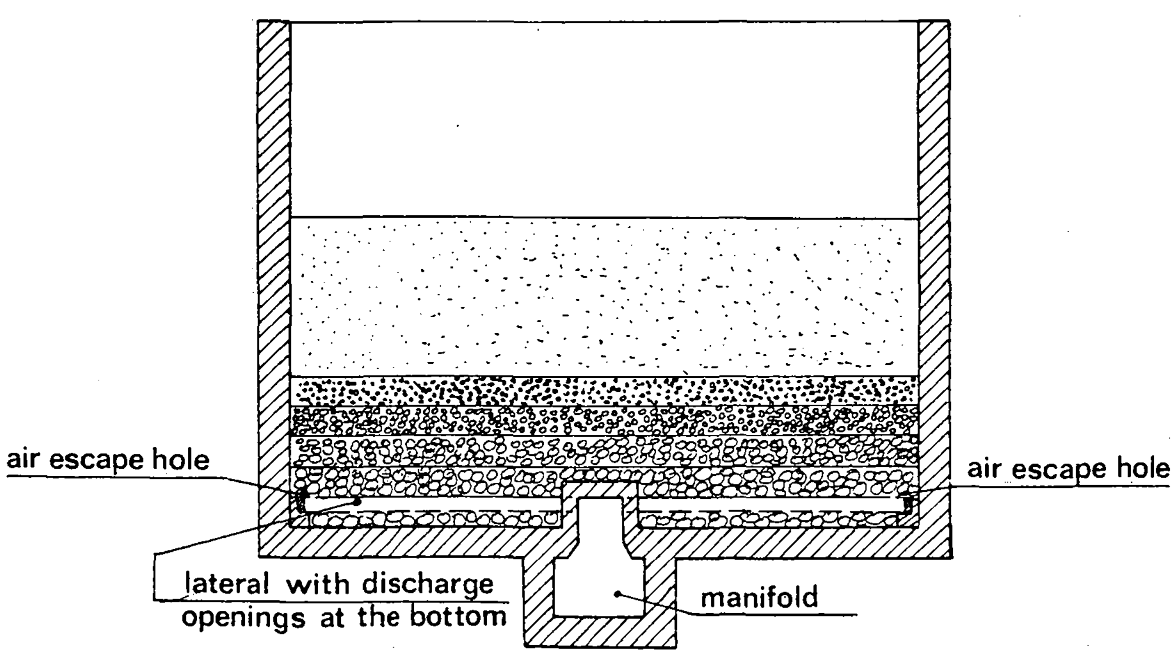
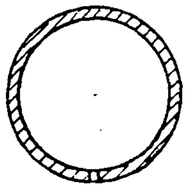
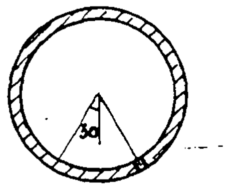


Fig. 4.20 Perforated lateral underdrain system for backwashing with water.



holes vertically down for instance 10mm ϕ at 100mm on centers



holes under 30^{o} with vertical for instance 10mm ϕ at 100mm on centers staggered

Fig. 4.21 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 ϕ = 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. 4.22 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

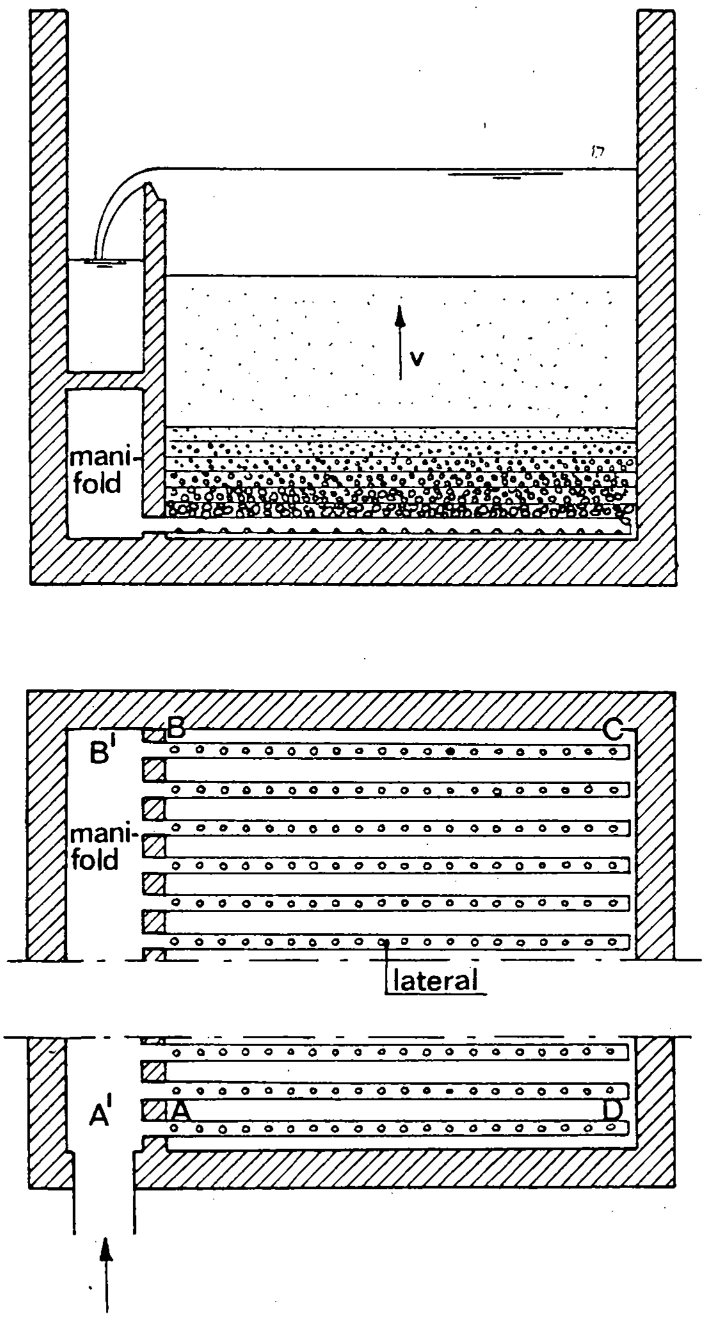


Fig. 4.22 Plan and cross-section of a rapid filter.

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

$$\Delta_{B-C} = \frac{v_1^2}{2g} - \frac{1}{3} \lambda_1 \frac{L_1}{D_1} \frac{v_1^2}{2g} - \frac{1}{n_1} \frac{v_1^2}{2g} = \alpha_1 \frac{v_1^2}{2g}$$

in which L is the length, D the (hydraulic) diameter, λ the friction coefficient, v the entrance velocity and n the number of outflows of manifold (index m) and lateral (index 1) respectively. The total variation in piezometric level thus equals

$$\Delta_{A-C} = \alpha_m \frac{v_m^2}{2g} + \alpha_1 \frac{v_1^2}{2g}$$

with α_m and α_1 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

$$H_{bottom} = 0.18 + 25\ dH$$

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 Δ_{A-C} calculated above, while the water level variations above the filterbed may be neglected. This gives finally

$$\frac{v_o^2}{2g} = 0.18 + 25 \left(\alpha_m \frac{v_m^2}{2g} + \alpha_1 \frac{v_1^2}{2g}\right)$$

Large design values of v_m and v_1 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 μ 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

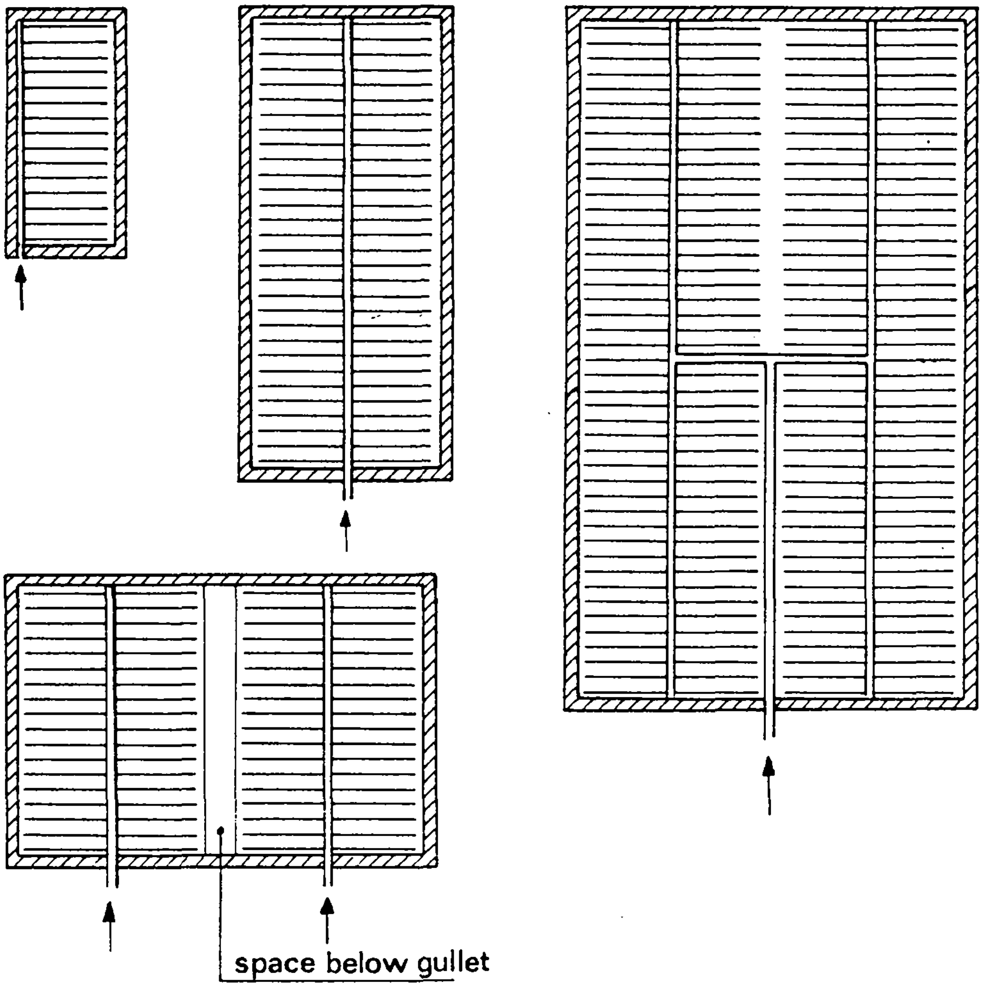


Fig. 4.23 Common arrangements of perforated pipe underdrains.

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 back-washing. 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 dis-lodged 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. Fig. 4.24 on the right shows a system meant to prevent expansion of the upper gravel layer and subsequent dispersion through the filterbed. When this danger is feared, a better solution is to compose the upper gravel layer of heavier grains, for instance from garnet or magnetite.
When this is too expensive, the grain size distribution of the upper layer should be taken as coarse as possible and the thickness increased, for instance to 0.15 m. Such a larger thickness also greatly helps in obtaining an equal distribution of the washwater. The hydraulic resistance of the gravel system may be calculated with the Carman-Kozeny equation of section 2.3. or set at a value of 0.4 m for a backwash rate of $(15)10^{-3}$ m/sec.

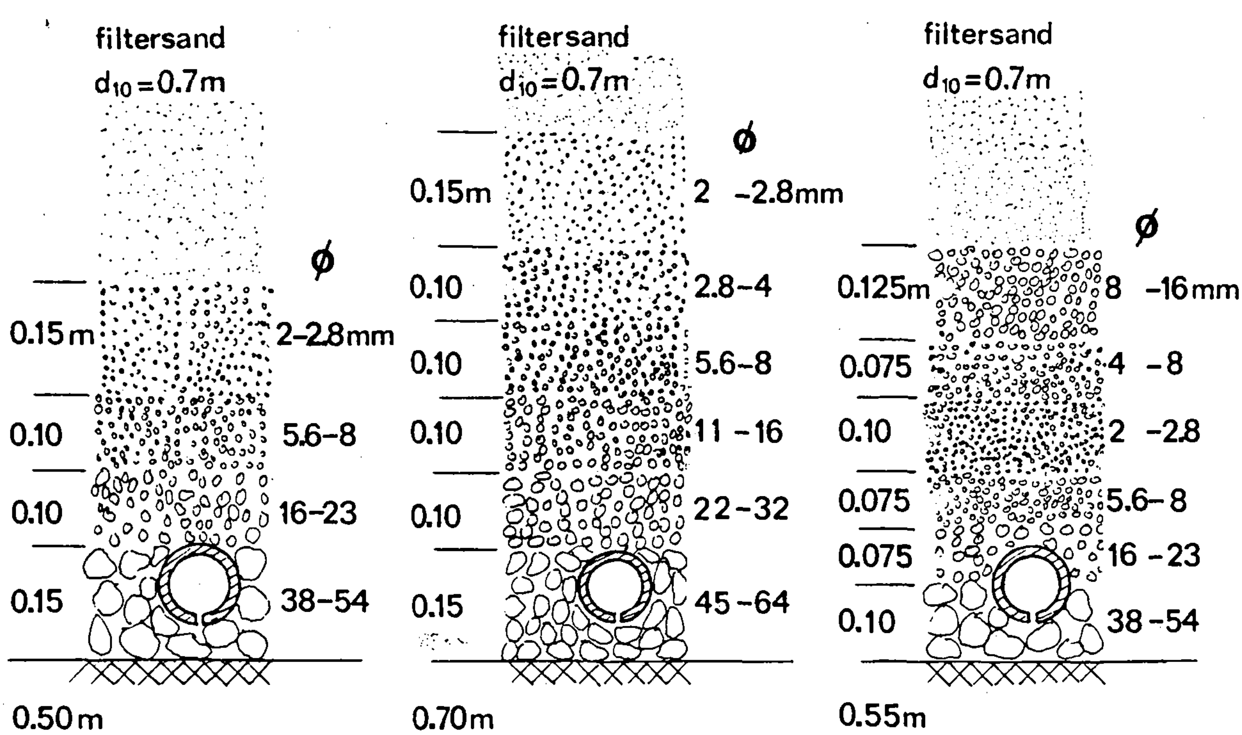


Fig. 4.24 Composition of gravel layers in rapid filters.

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.

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

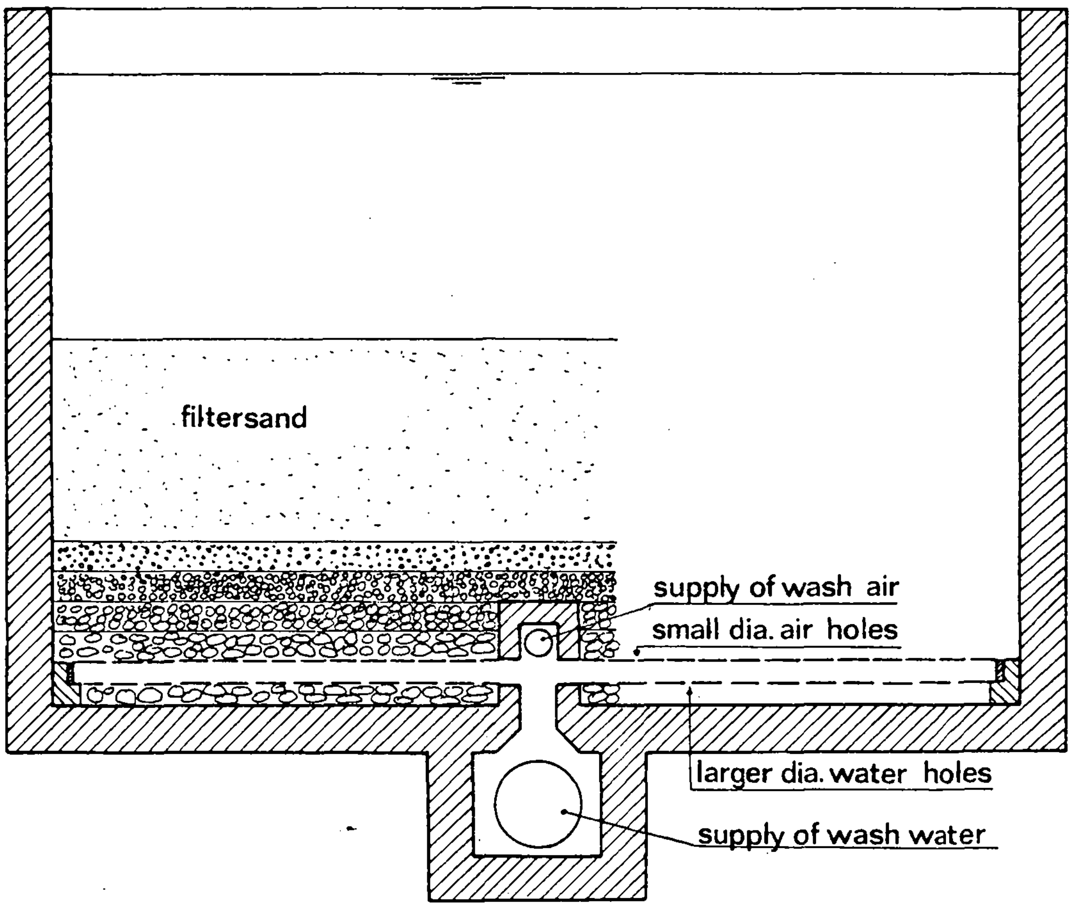


Fig. 4.25 Perforated lateral underdrains for backwashing with water and air.

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, 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

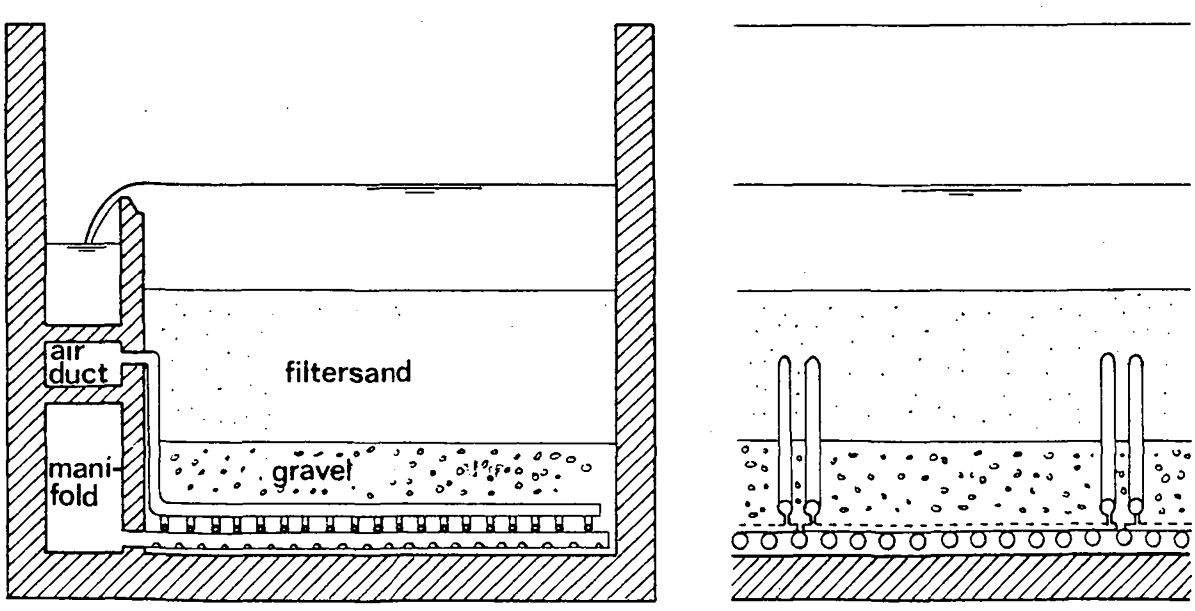


Fig. 4.26 Seperate system of perforated laterals for backwashing with water and air.

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 materials. As absolute 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. Especial-

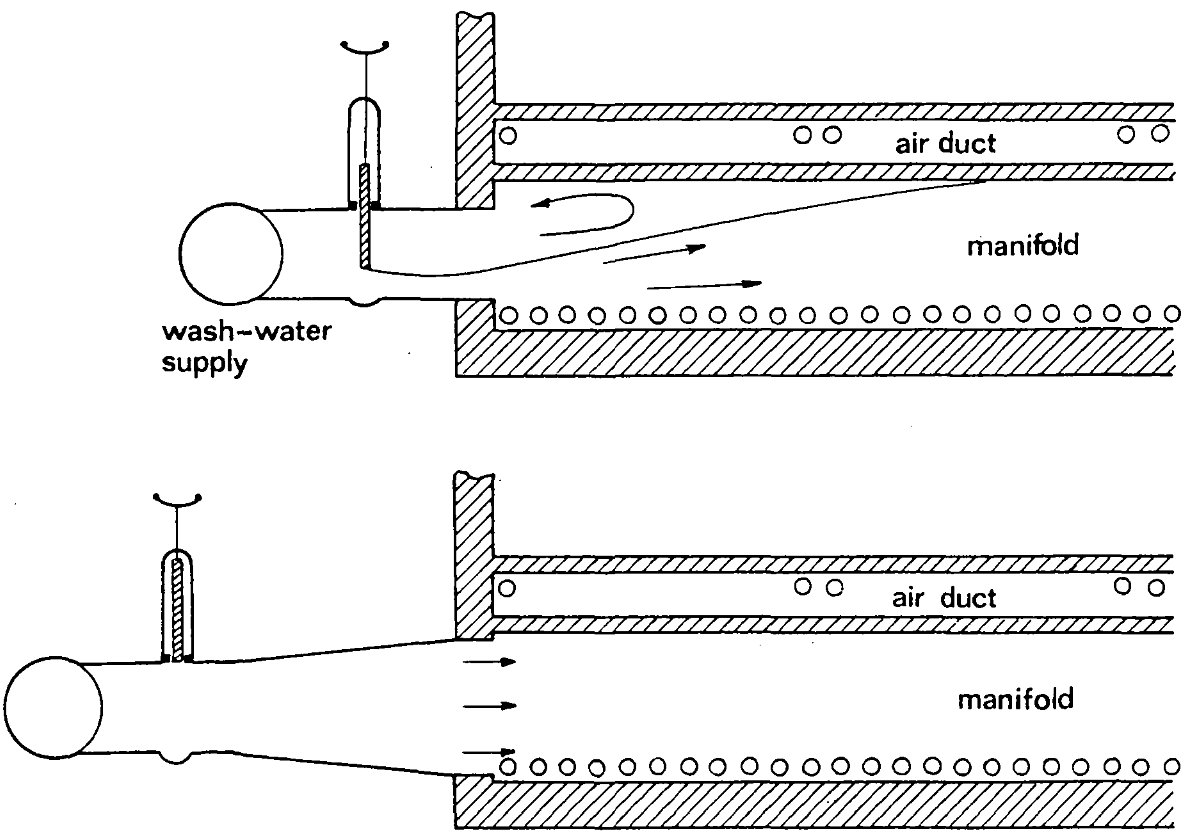


Fig. 4.27 Washwater connection to manifold.

ly 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. 3.12), 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. 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 real disadvantage of the perforated pipe underdrainage system is certainly 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,

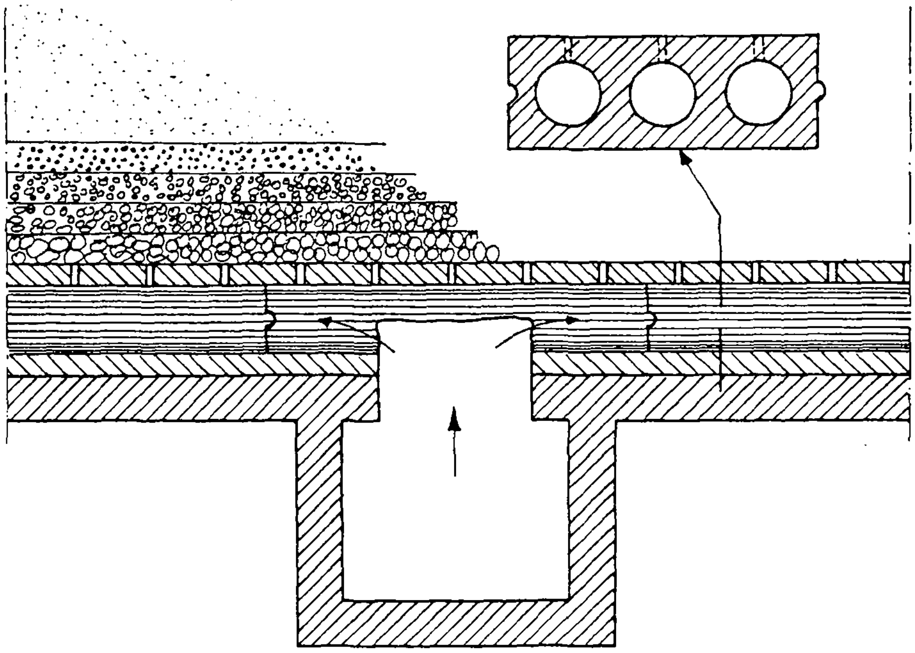


Fig. 4.28 Modern design of perforated pipe underdrain system.

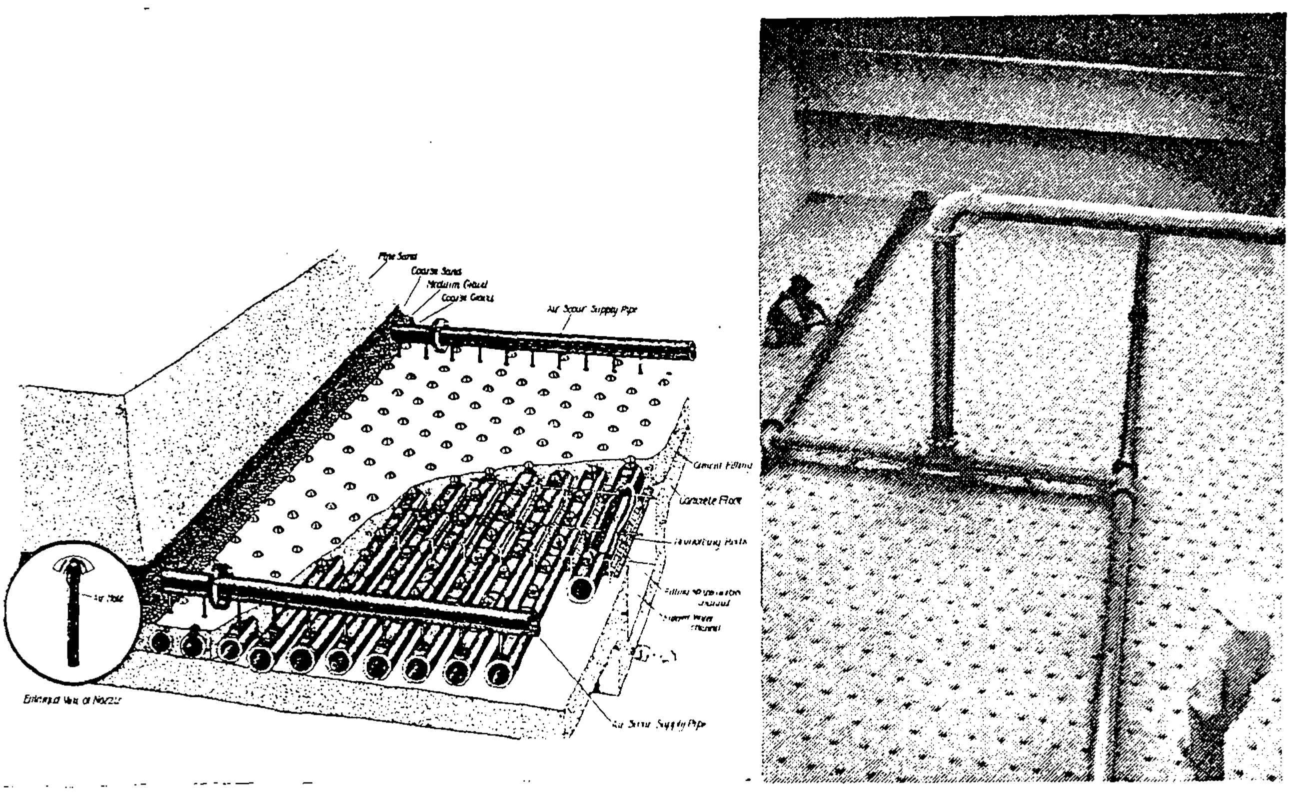


Fig. 4.29 Pipe-and strainer underdrainage system.

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 ϕ 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 70 to 100 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.

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,

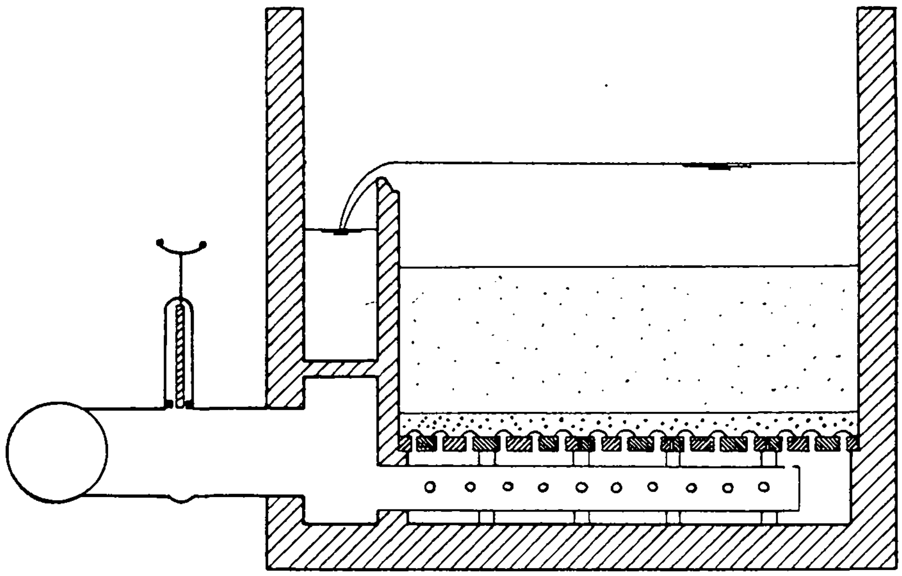


Fig. 4.30 False bottom and strainer underdrainage system for backwashing with water.

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 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 of the rising washwater and at the same time some protection against mechanical 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

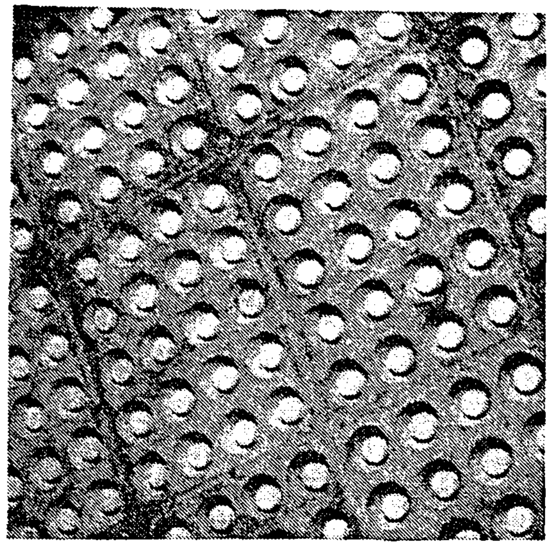
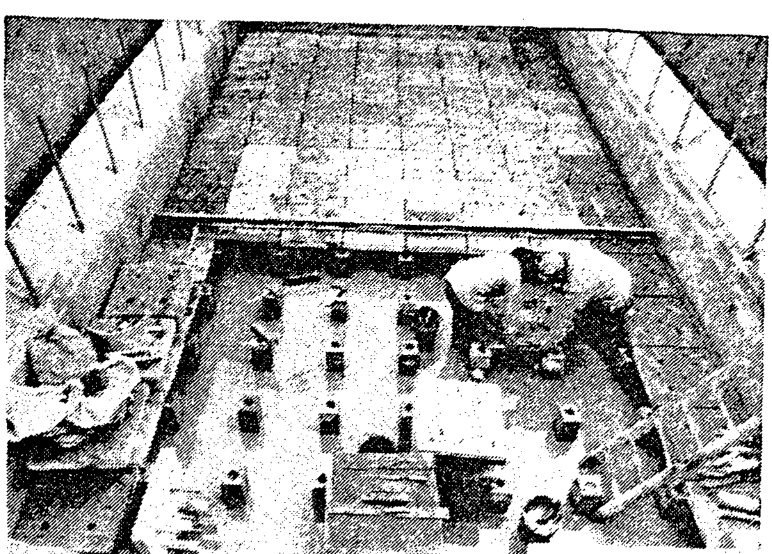


Fig. 4.31 Countersink mounting of strainers in a false bottom.



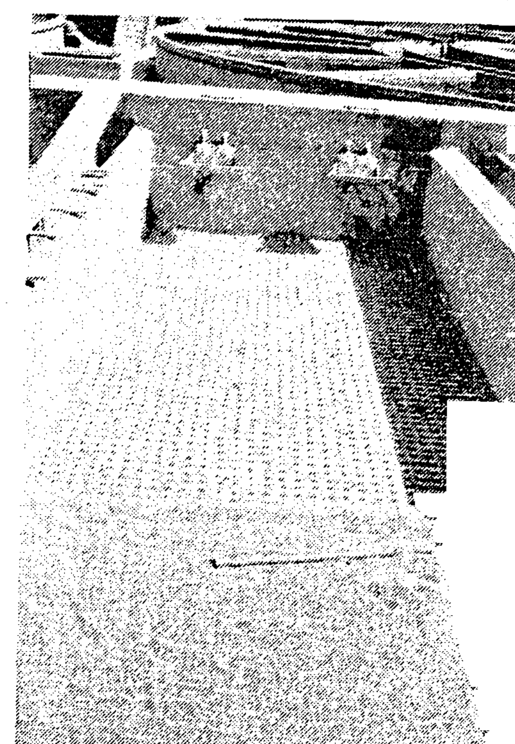


Fig. 4.32 Construction of false bottom and strainer underdrainage system.

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 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

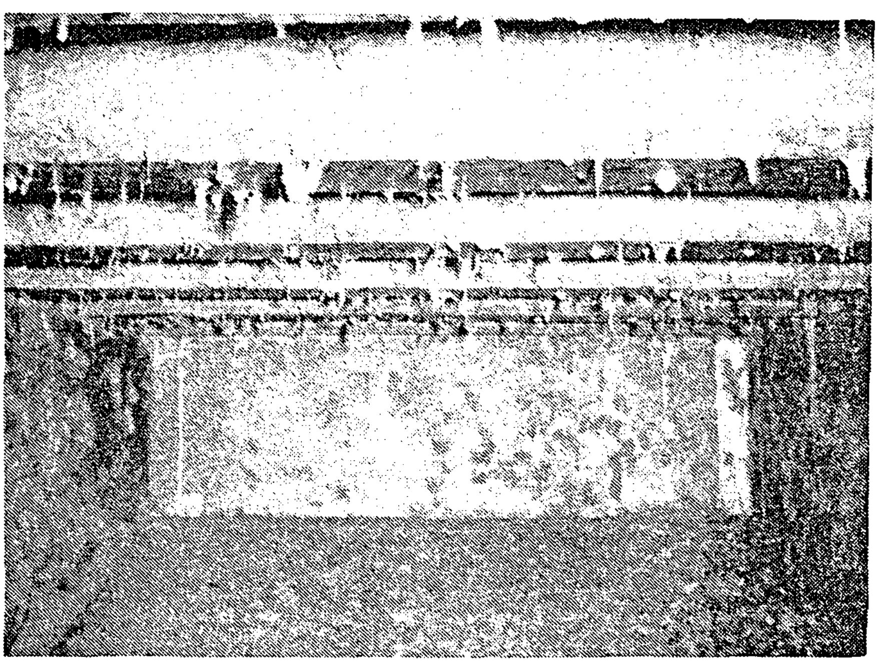


Fig. 4.33 Fouling of the space below a false bottom by aquatic growth.

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 dispersion 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

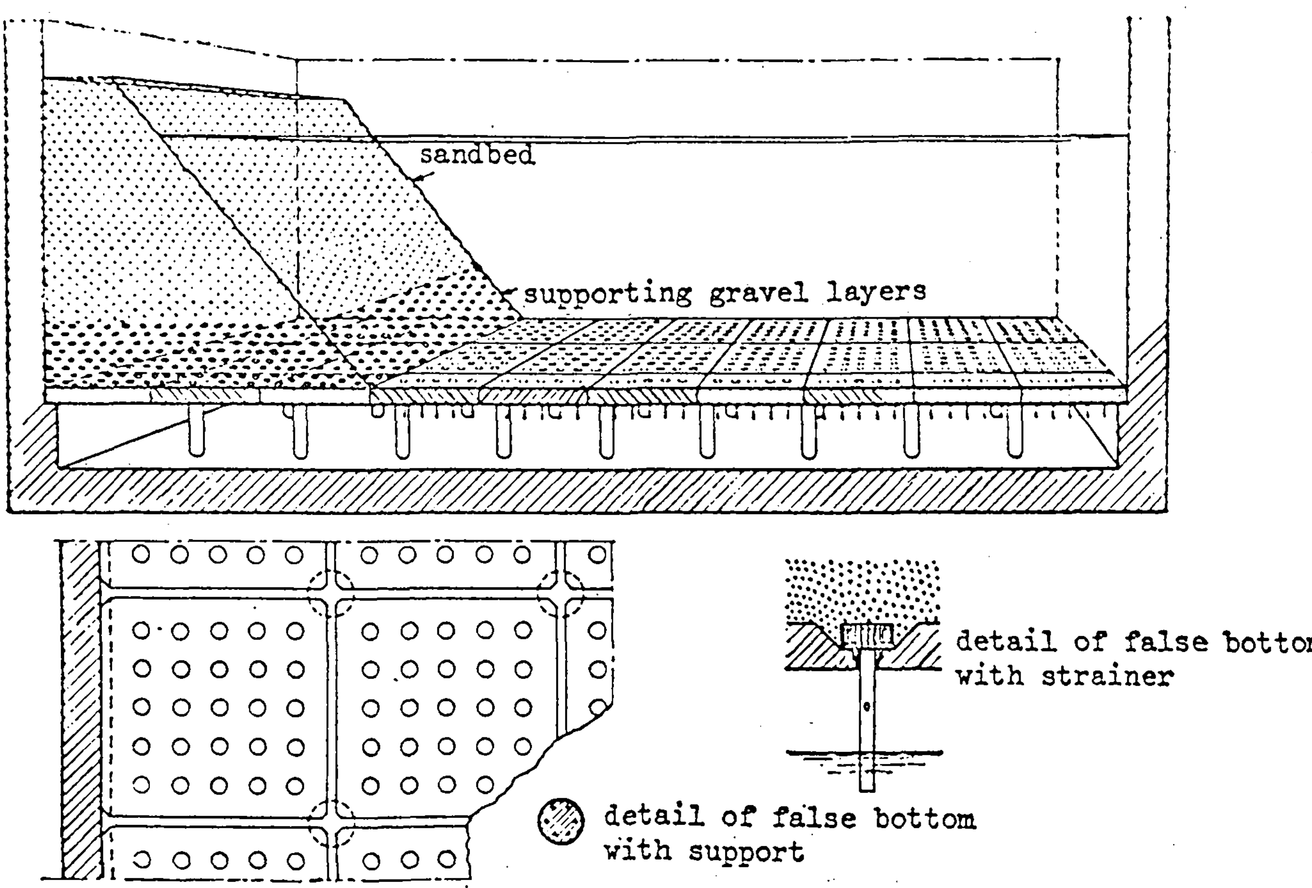


Fig. 4.34 False bottom and strainer underdrainage system for backwashing with water and air.

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 purpose, 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

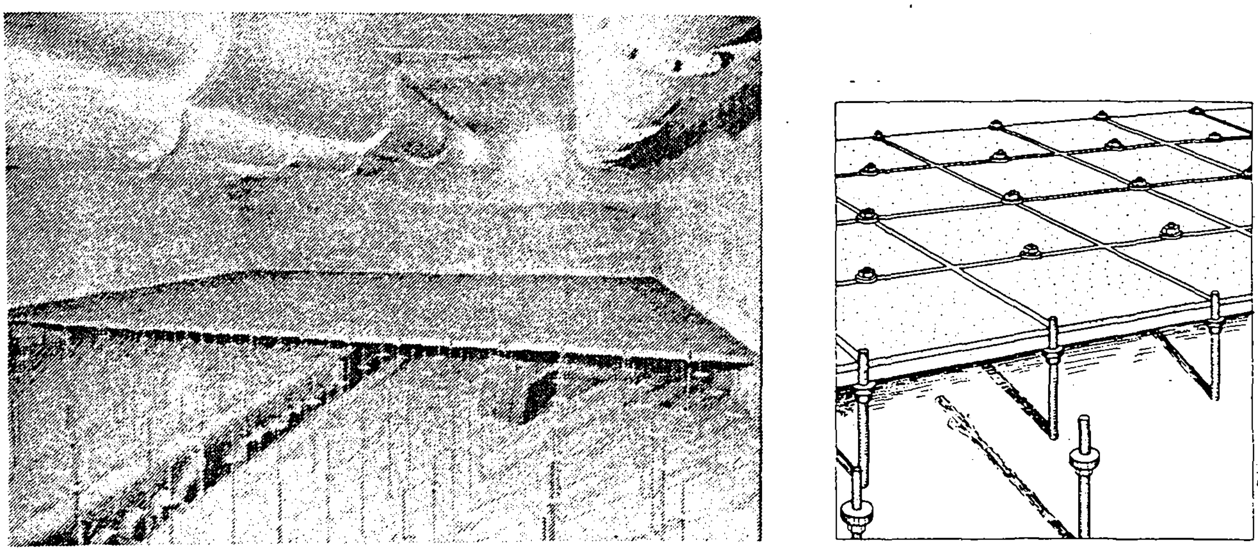


Fig. 4.35 Porous plate filter bottom.

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.

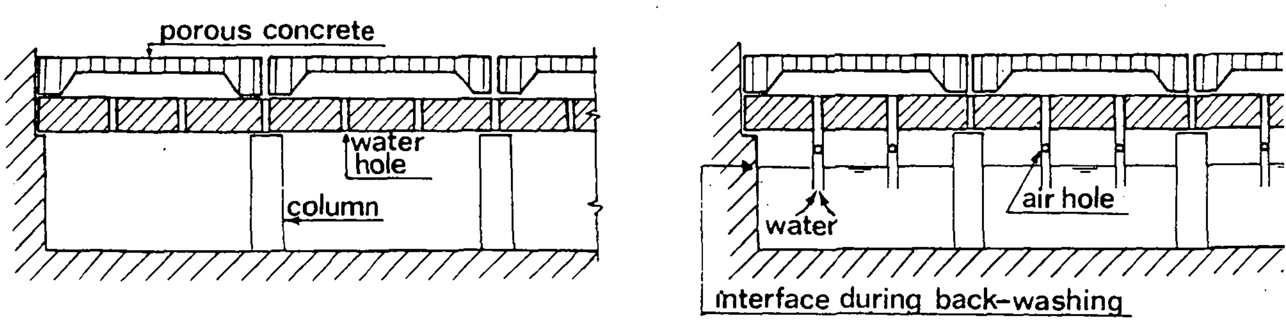


Fig. 4.36 Double false bottom underdrains.

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!

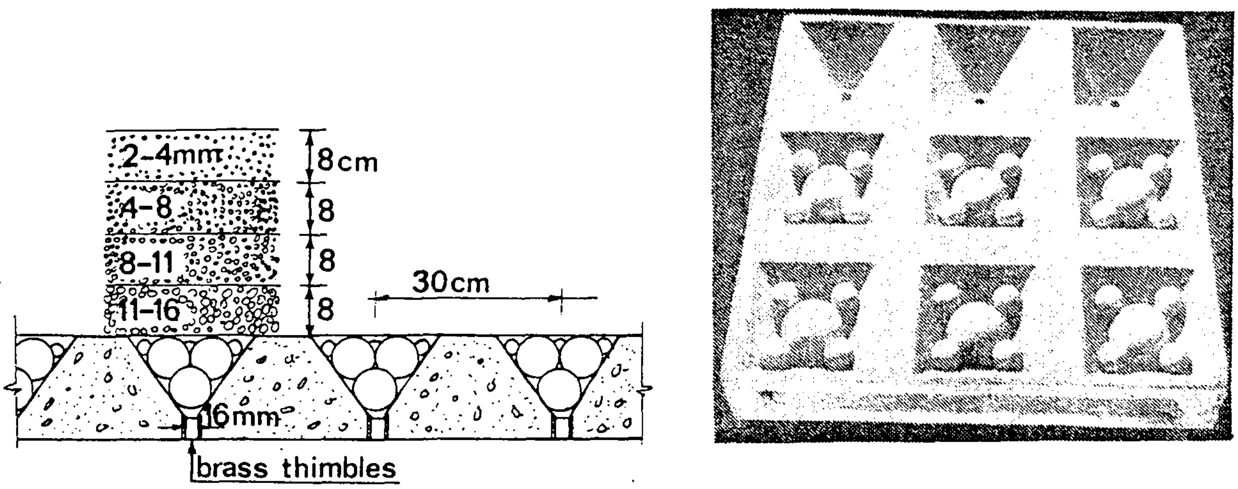


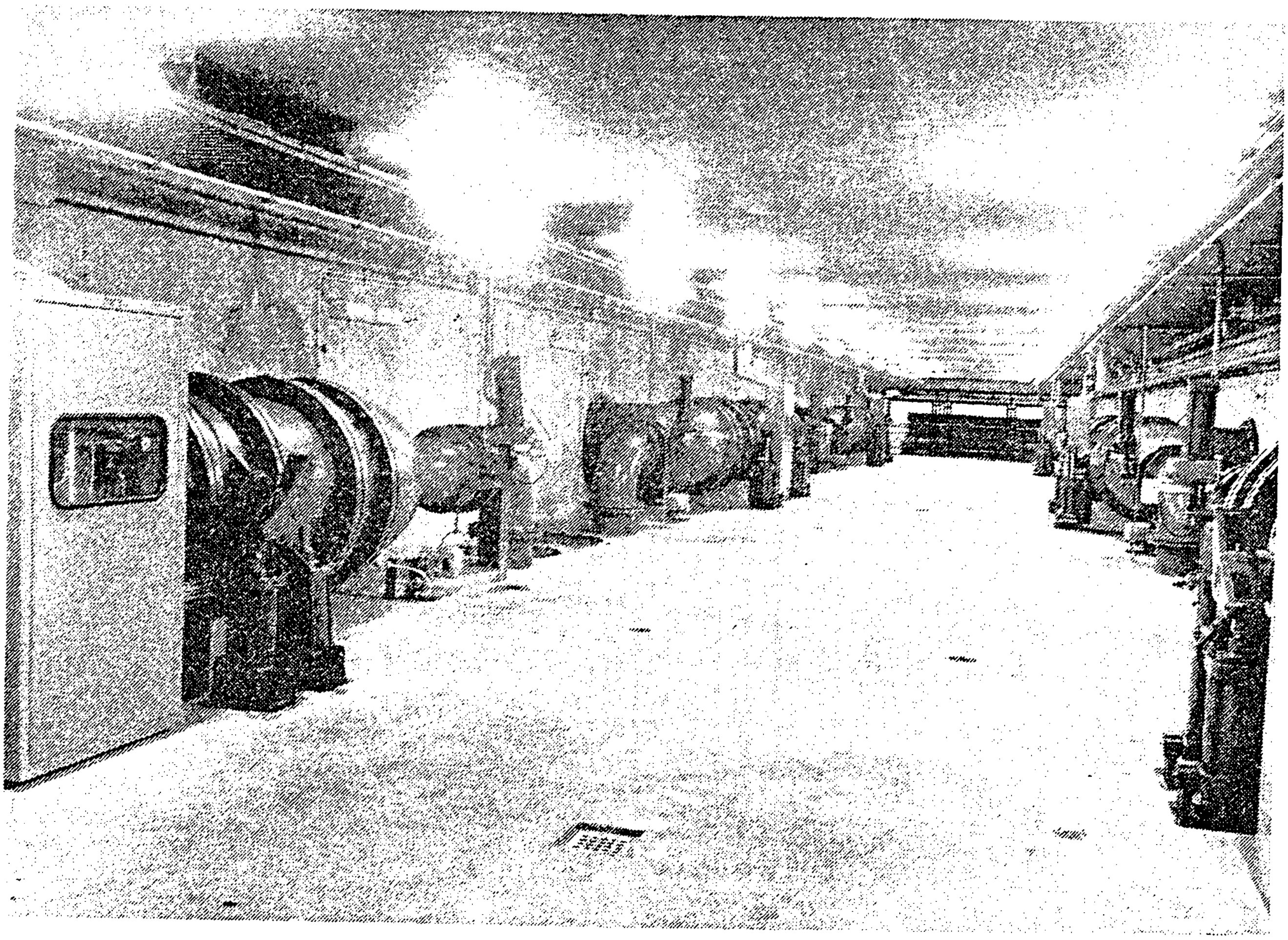
Fig. 4.37 Wheeler false filter bottom

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.



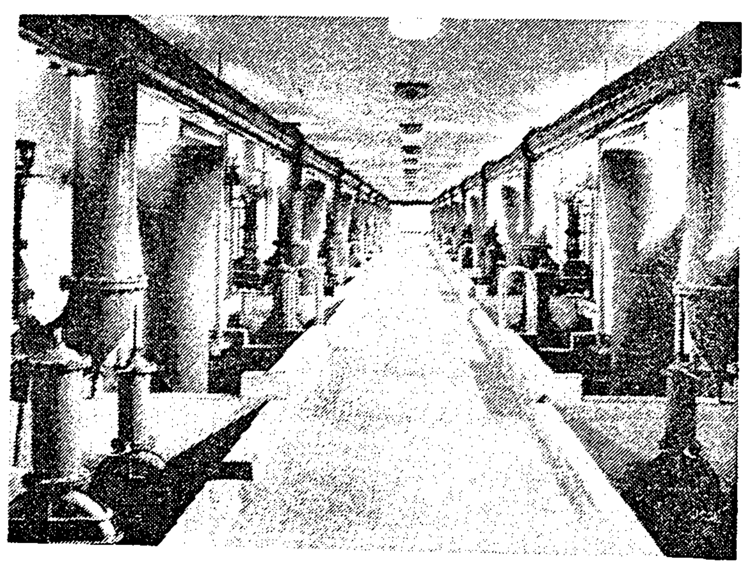
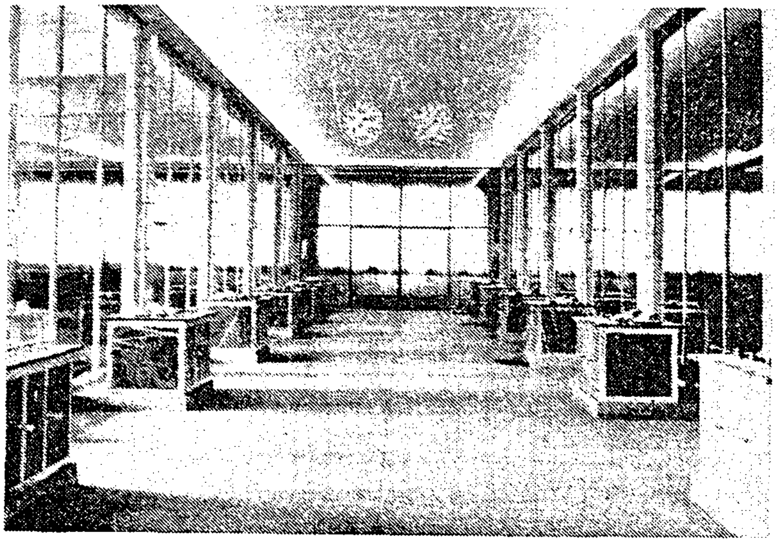


Fig. 4.38 Pipe galleries.

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.



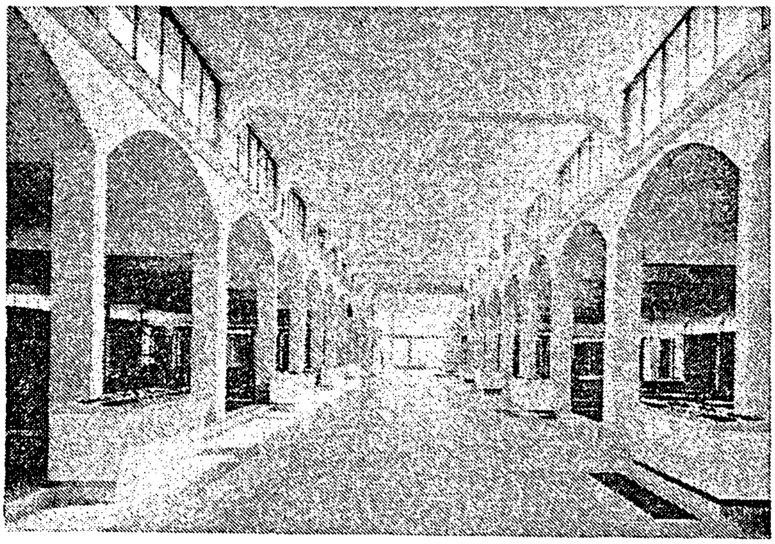


Fig. 4.39 Operating floors.

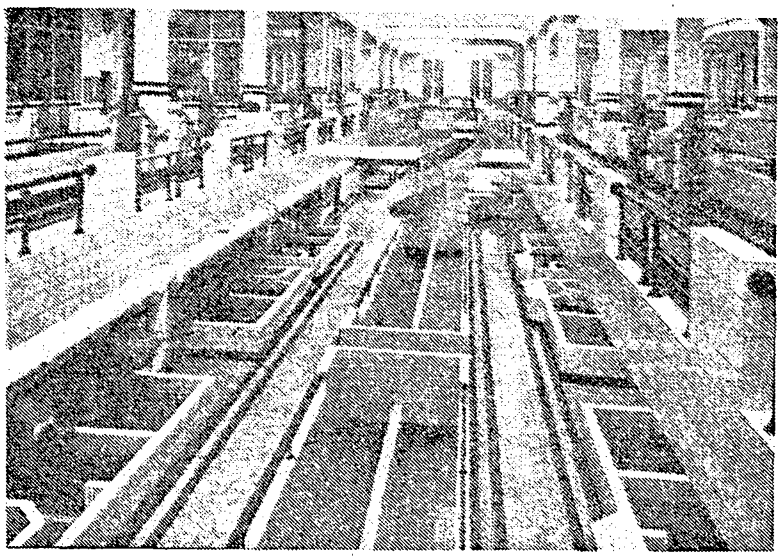
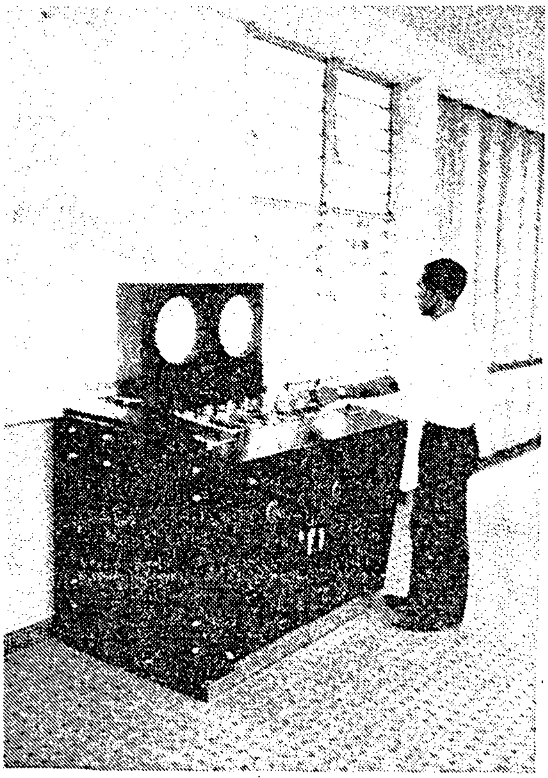


Fig. 4.40 Direct access from operating floor to pipe gallery.



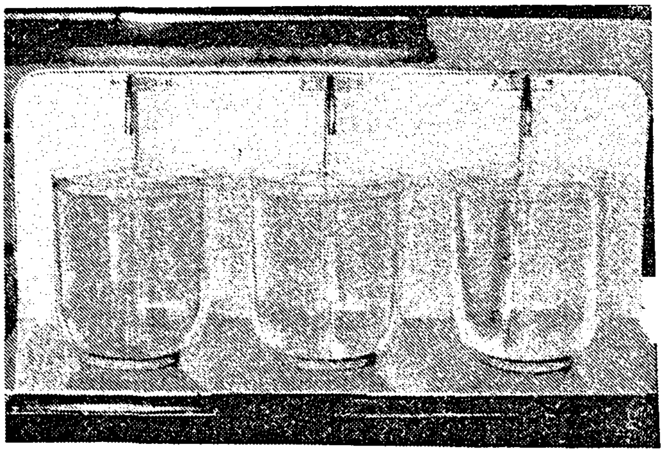


Fig. 4.41 Operating tables.

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

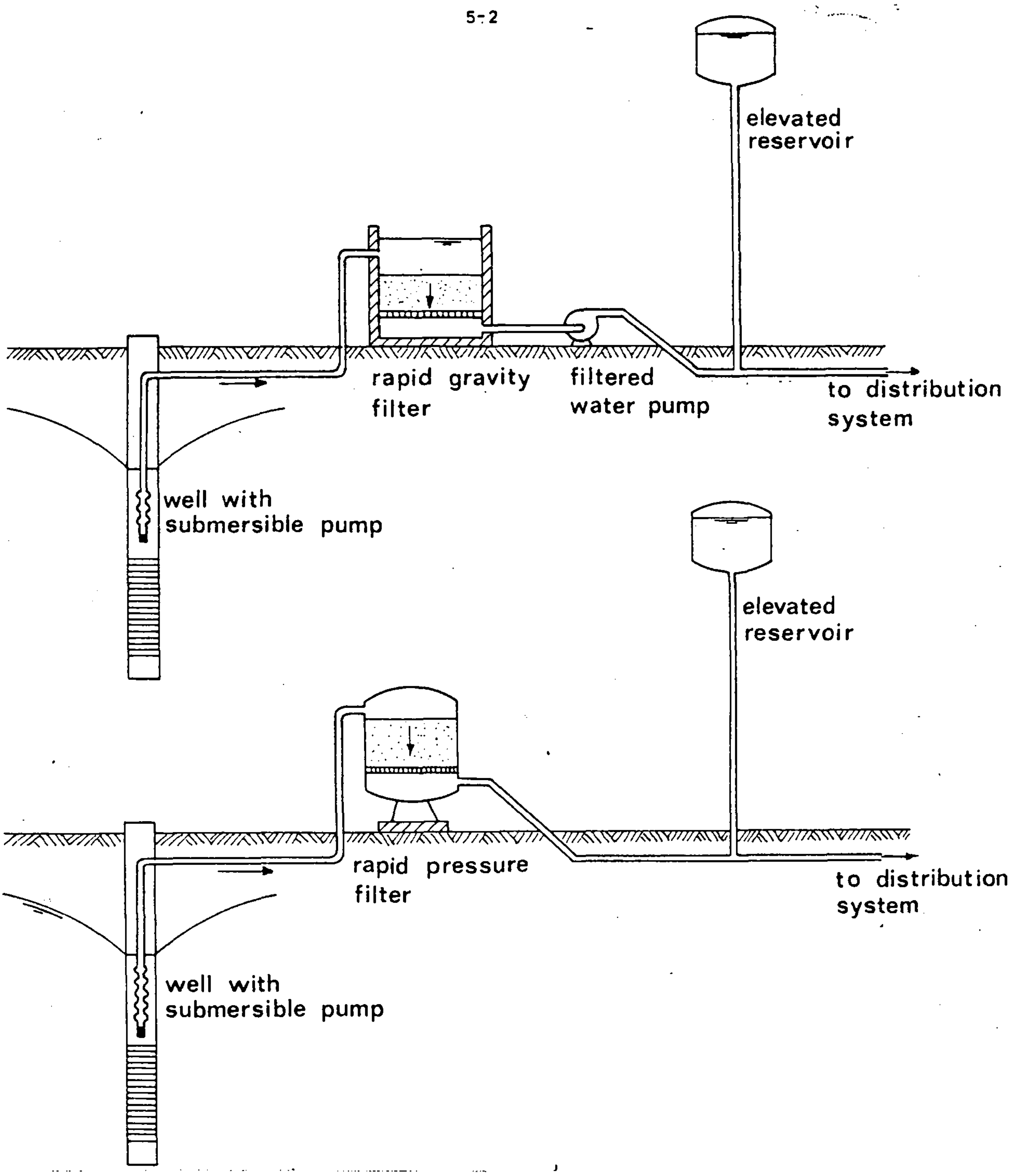


Fig. 5.1 Use of pressure filters to avoid double pumping.

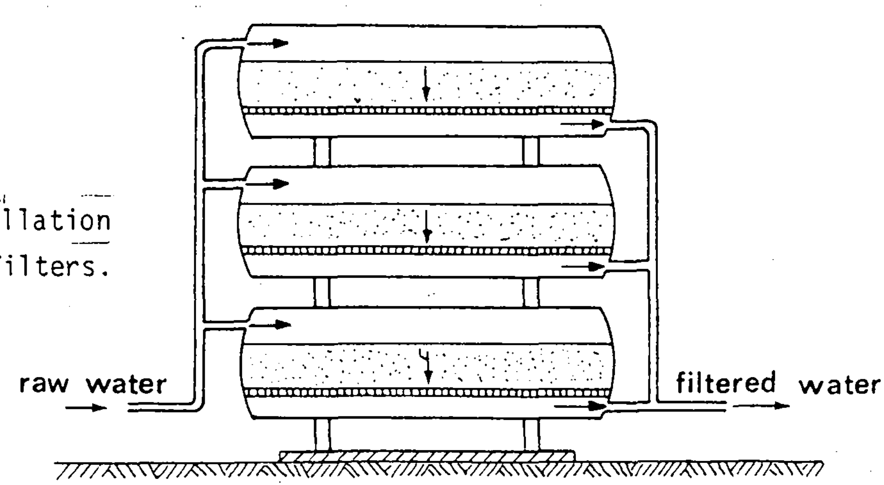


Fig. 5.2 Compact installation of pressure filters.

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

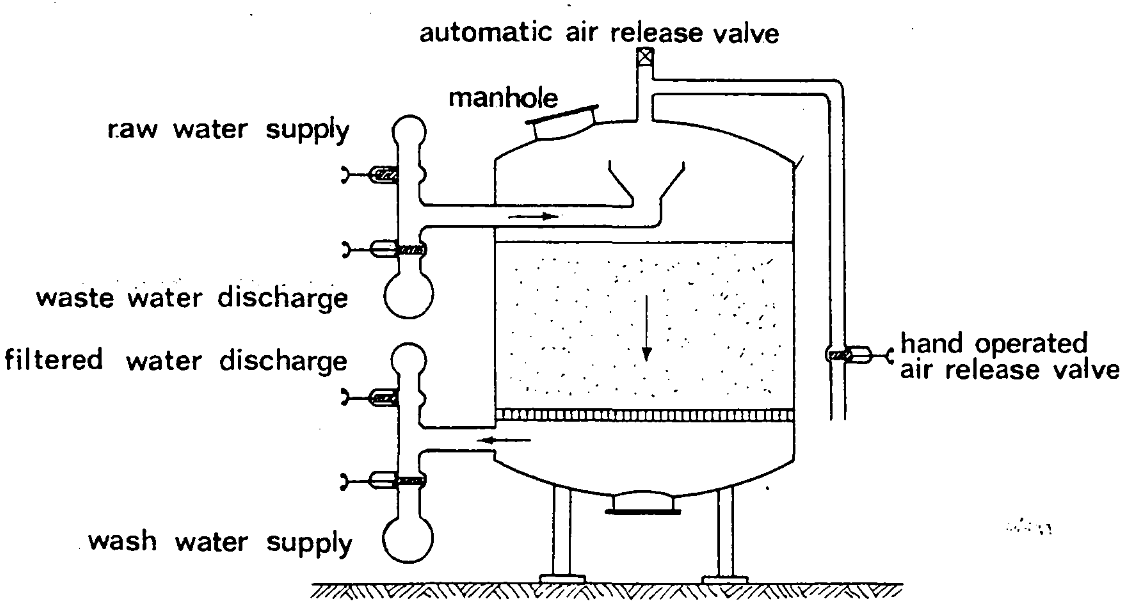


Fig. 5.3 Vertical pressure filter(during filtration).

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.

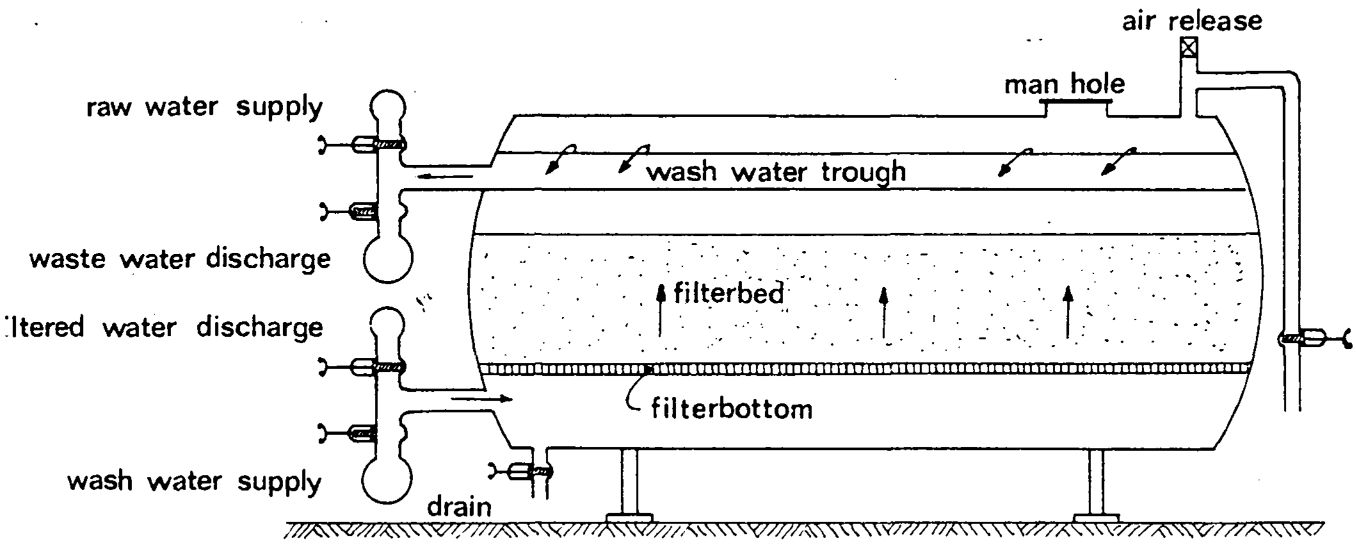


Fig. 5.4 Horizontal pressure filter(during backwashing).

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

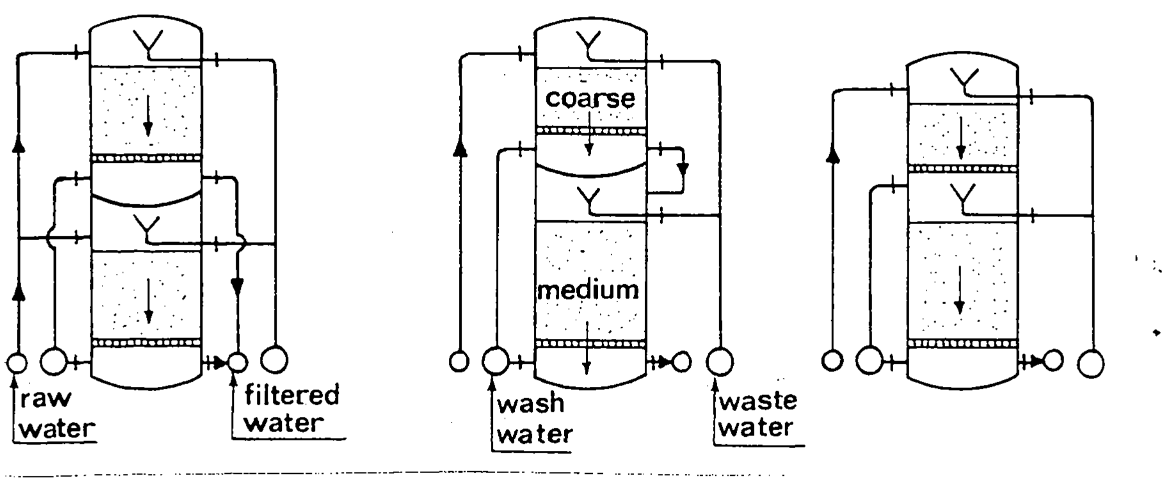


Fig. 5.5 Double pressure filters,in parallel or in series.

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

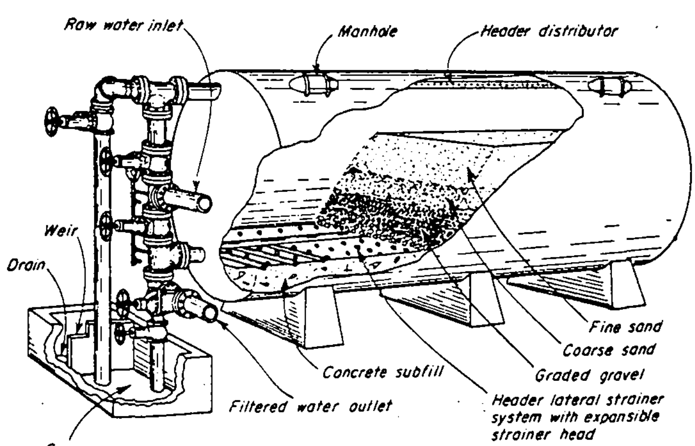
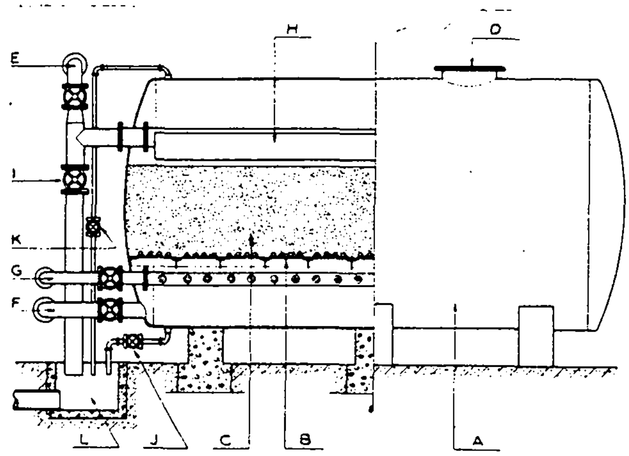


Fig. 5.6 Pressure filter with perforated pipe lateral underdrainage system. (Permutit Co)

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)

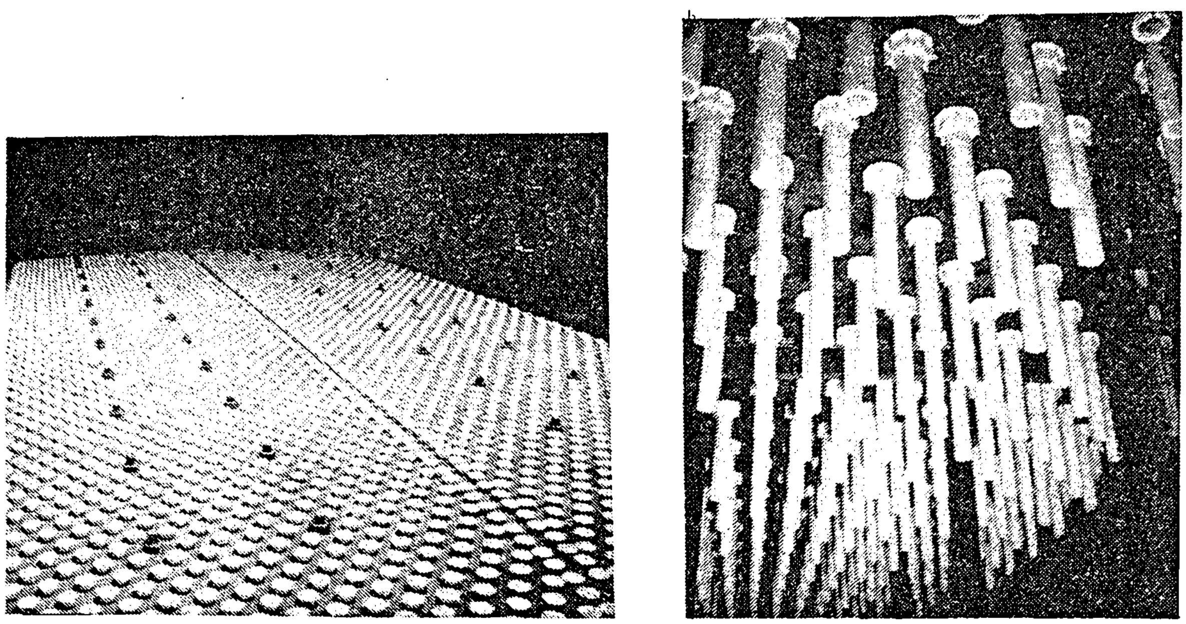


Fig. 5.8 False bottom-and strainer underdrains.

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.

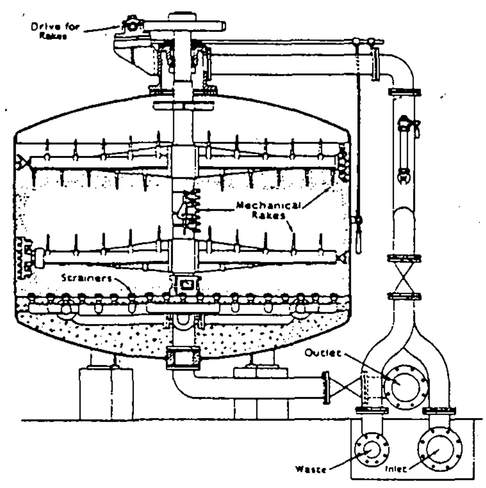


Fig. 5.9 Rake-cleaned pressure filters. (Bell Brothers)

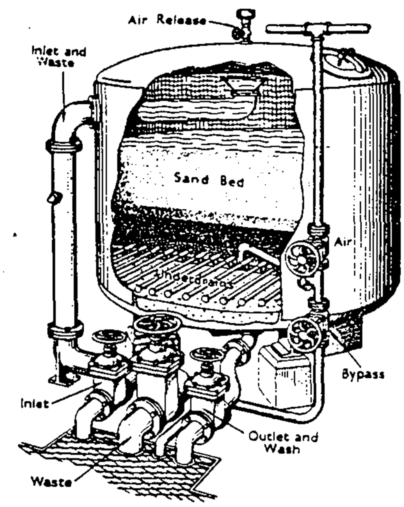
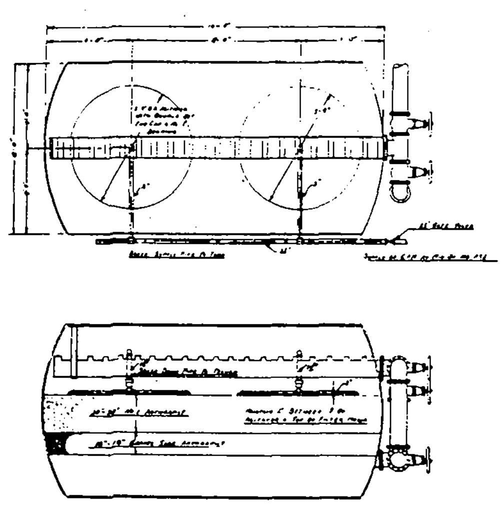
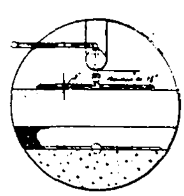


Fig. 5.10 Air cleaned pressure filters. (Paterson Engineering Co)





Agitator bearings & anthrafilt only supplied by PFE 'Co on order

Agitator & Ball equipped bearings constructed of brass & bronze to standard specifications

Use of brass or galv. pipe Inside filter preferred

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,

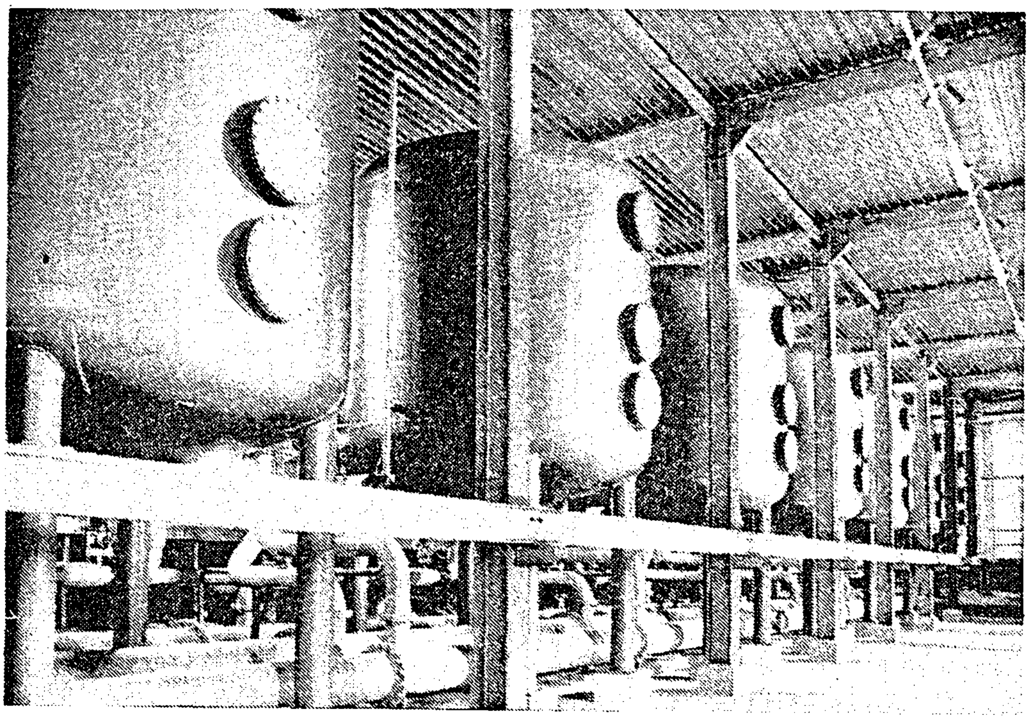


Fig. 5.12 Pumping station,Braakman.
(Public Water Supply of Zeeland)

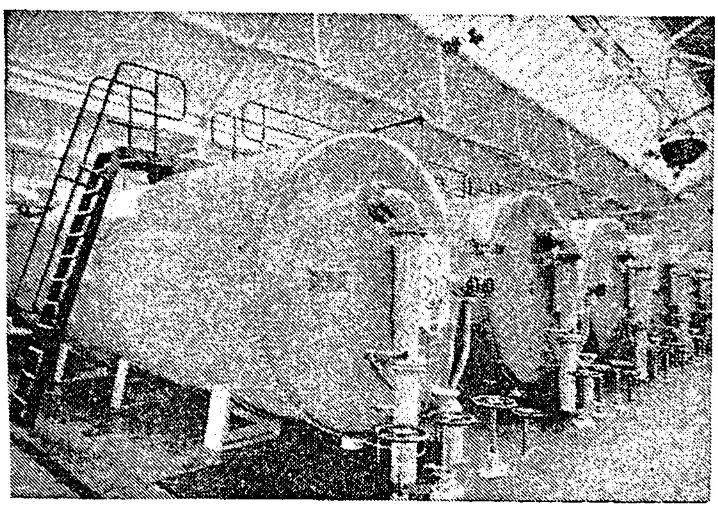


Fig. 5.13 Housing of horizontal pressure filters.
(Candy filter Co)

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).

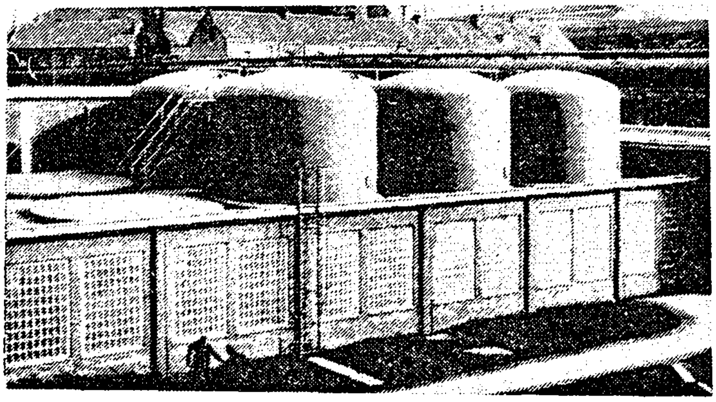


Fig. 5.14 Vertical pressure filters in the open air.
(Pintsch Bamag)

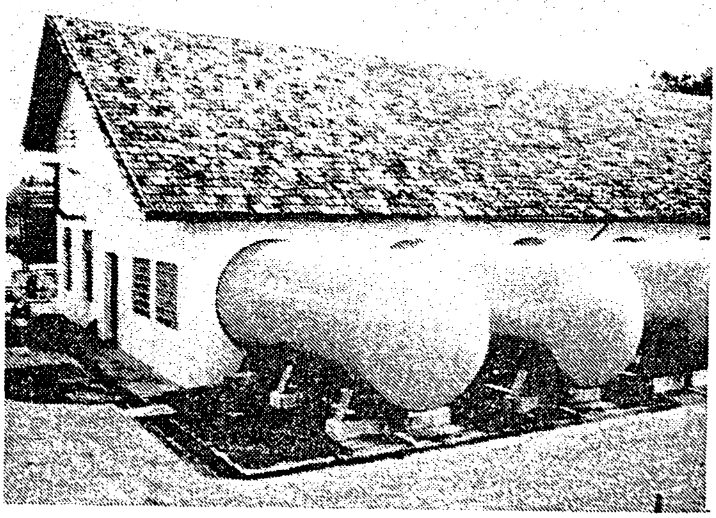


Fig. 5.15 Horizontal pressure filters in the open air.
(Alor Star, Malaysia)

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 major parts of the grains to the top and the coarse grains to the bottom. With the filtration coefficient λ_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

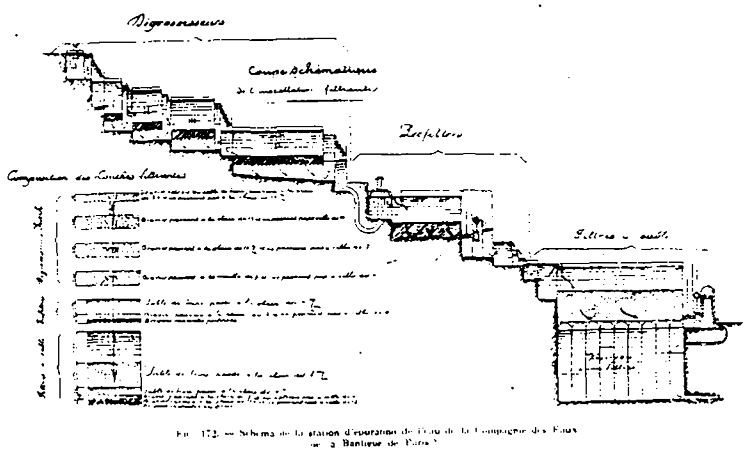


Fig. 6.1 Multi-stage rapid filtration as used by the Compagnie des Eaux de la Banlieuê de Paris.

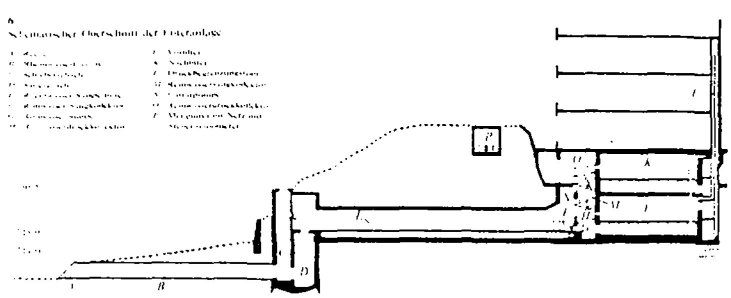


Fig. 6.2 Two-stage rapid filtration.

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 $c_a = 0.11 \text{ g/m}^3$ 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
ΣH =	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 (ρ_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 ρ_f = 1400-1700 kg/m^3 for the upper layer and baryta ($BaSO_4$) with ρ_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

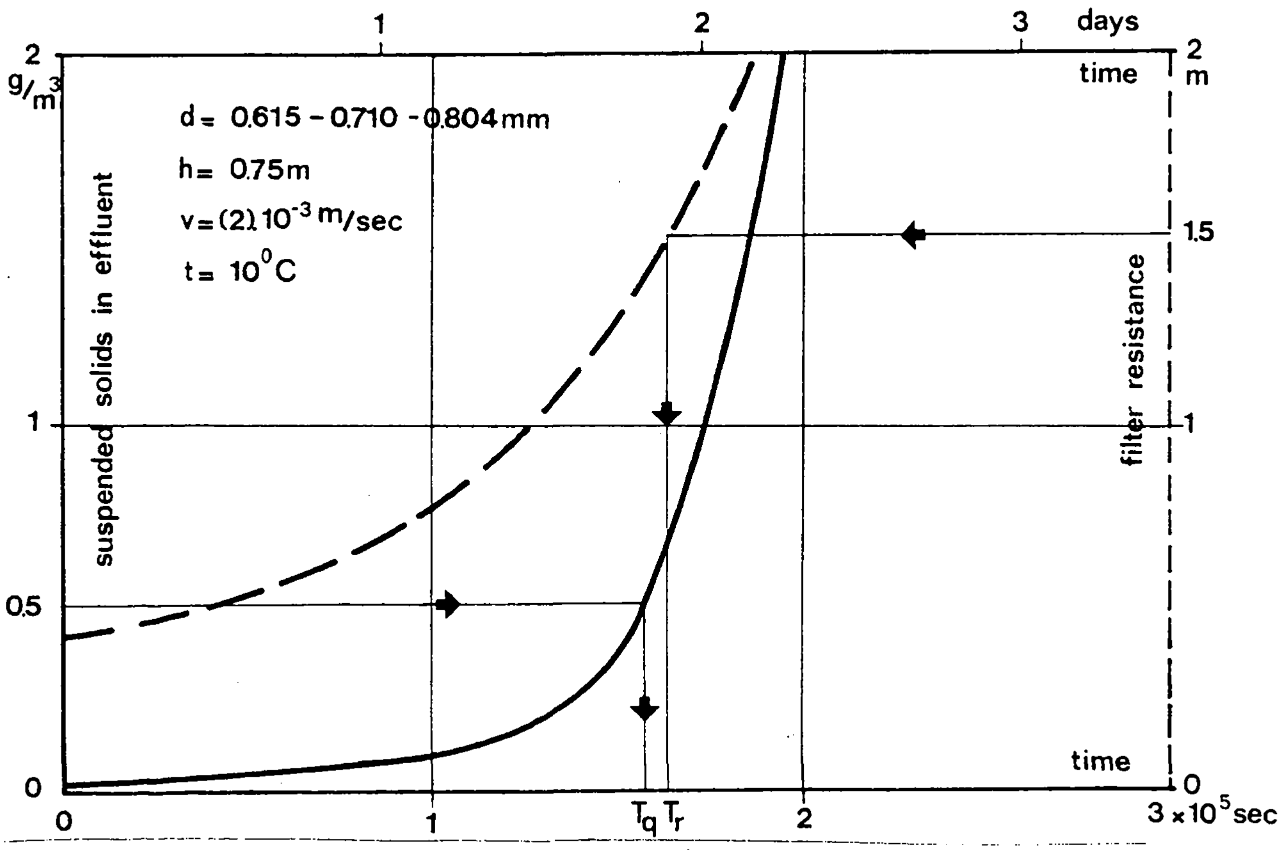


Fig. 6.3 Filtration results of fig. 2.21 recalculated for the reverse direction of flow.

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.

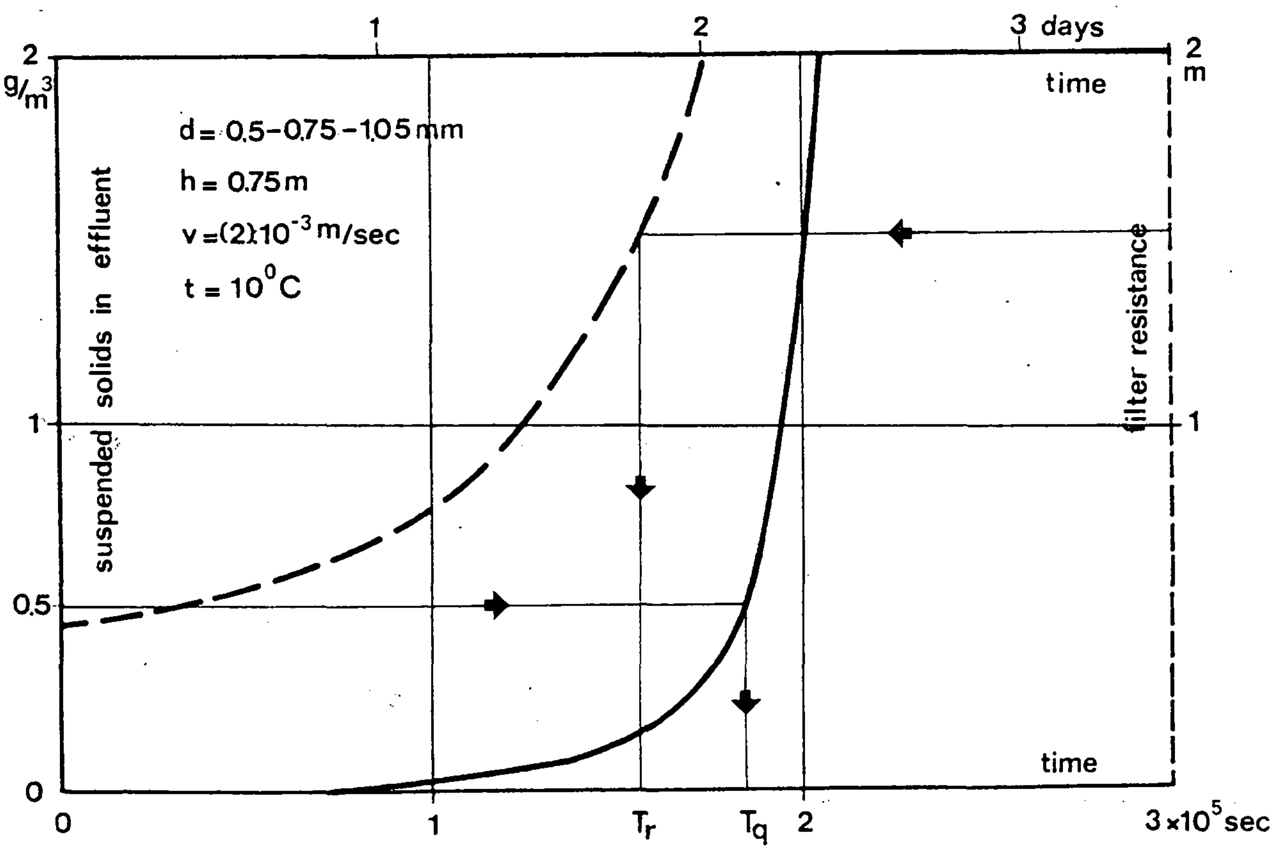


Fig. 6.4 Filtration results of fig. 6.3 recalculated for a less-uniform filtering material of the same hydraulic diameter.

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, ρ_f and ρ_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

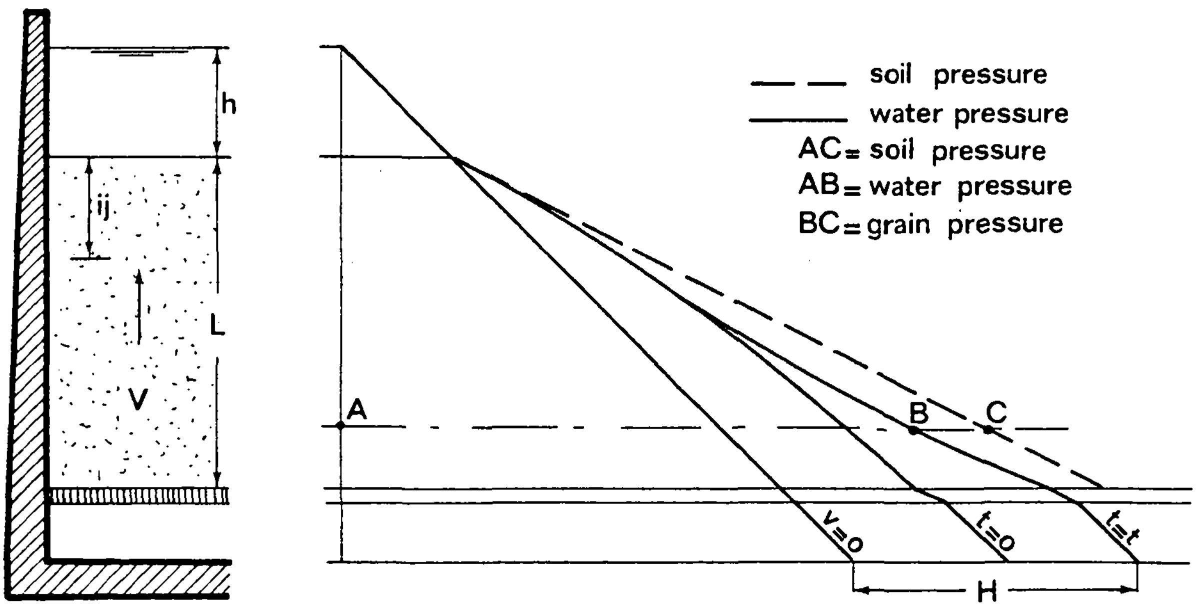


Fig. 6.5 Pressure distribution in the bed of an upflow filter.

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

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 σ_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

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

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-

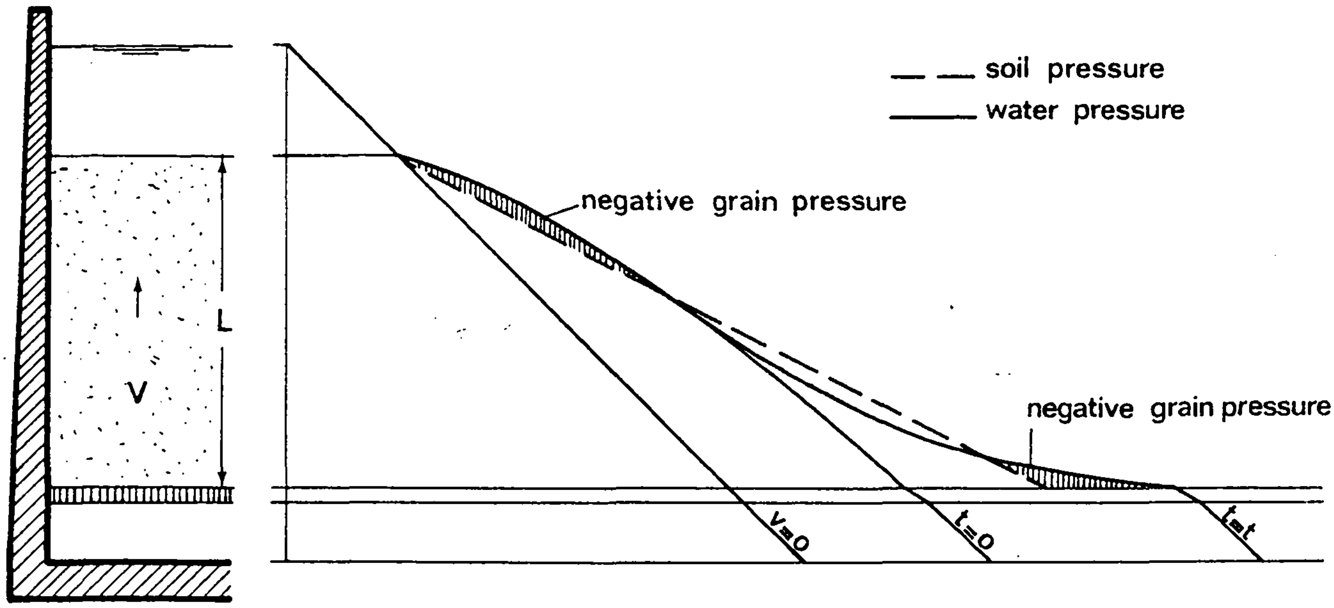


Fig. 6.6 Occurrence of negative grain pressure.

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

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

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

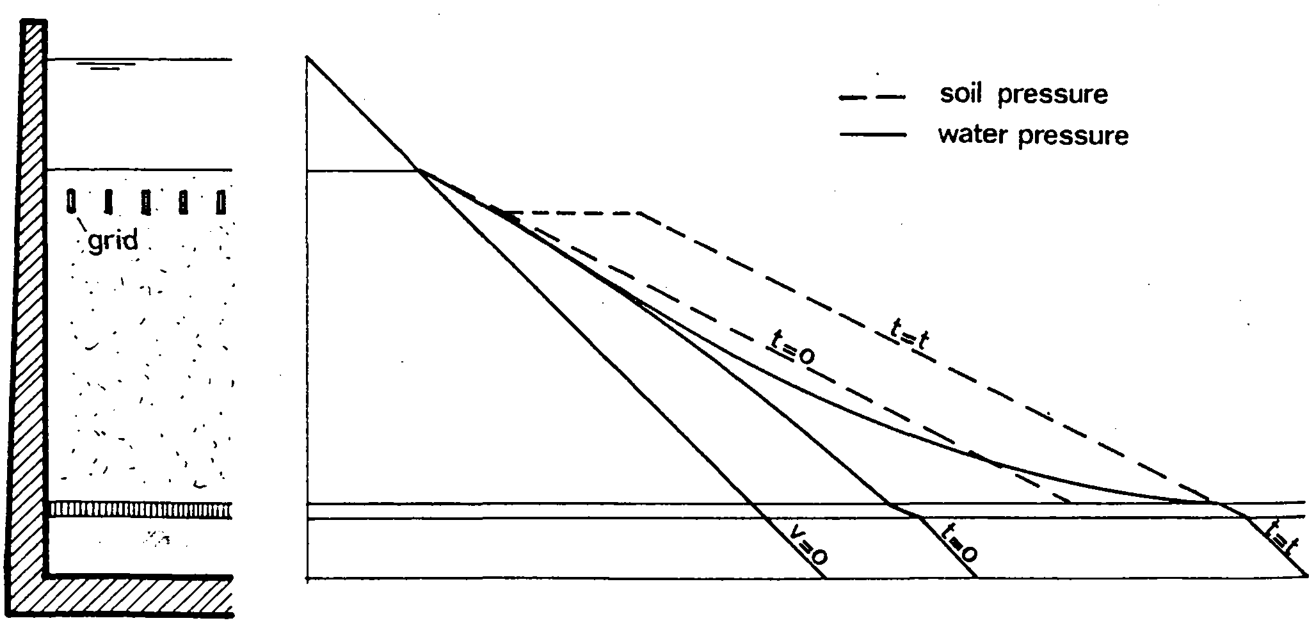


Fig. 6.7 Increase in soil pressure by the presence of a grid.

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.

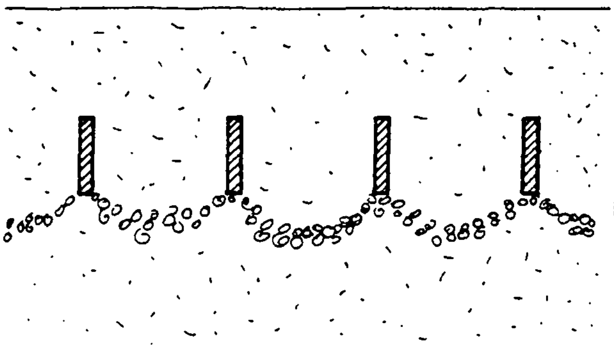


Fig. 6.8 Bridges of sandgrains preventing an upward movement of the filterbed.

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

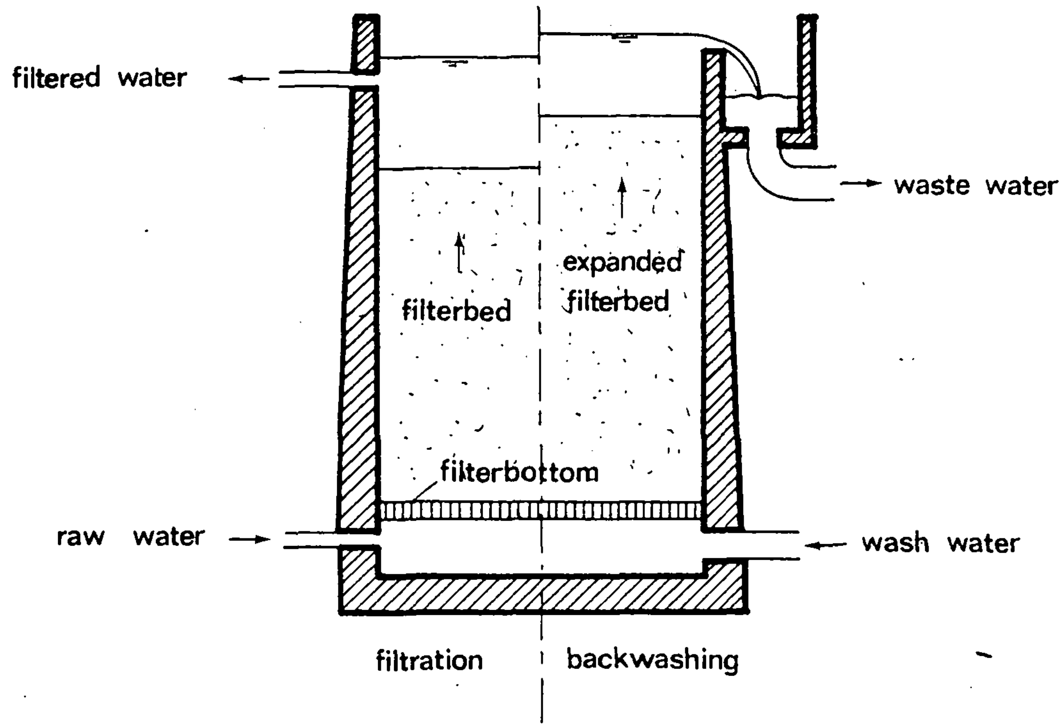


Fig. 6.9 Operation of an upflow filter.

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.

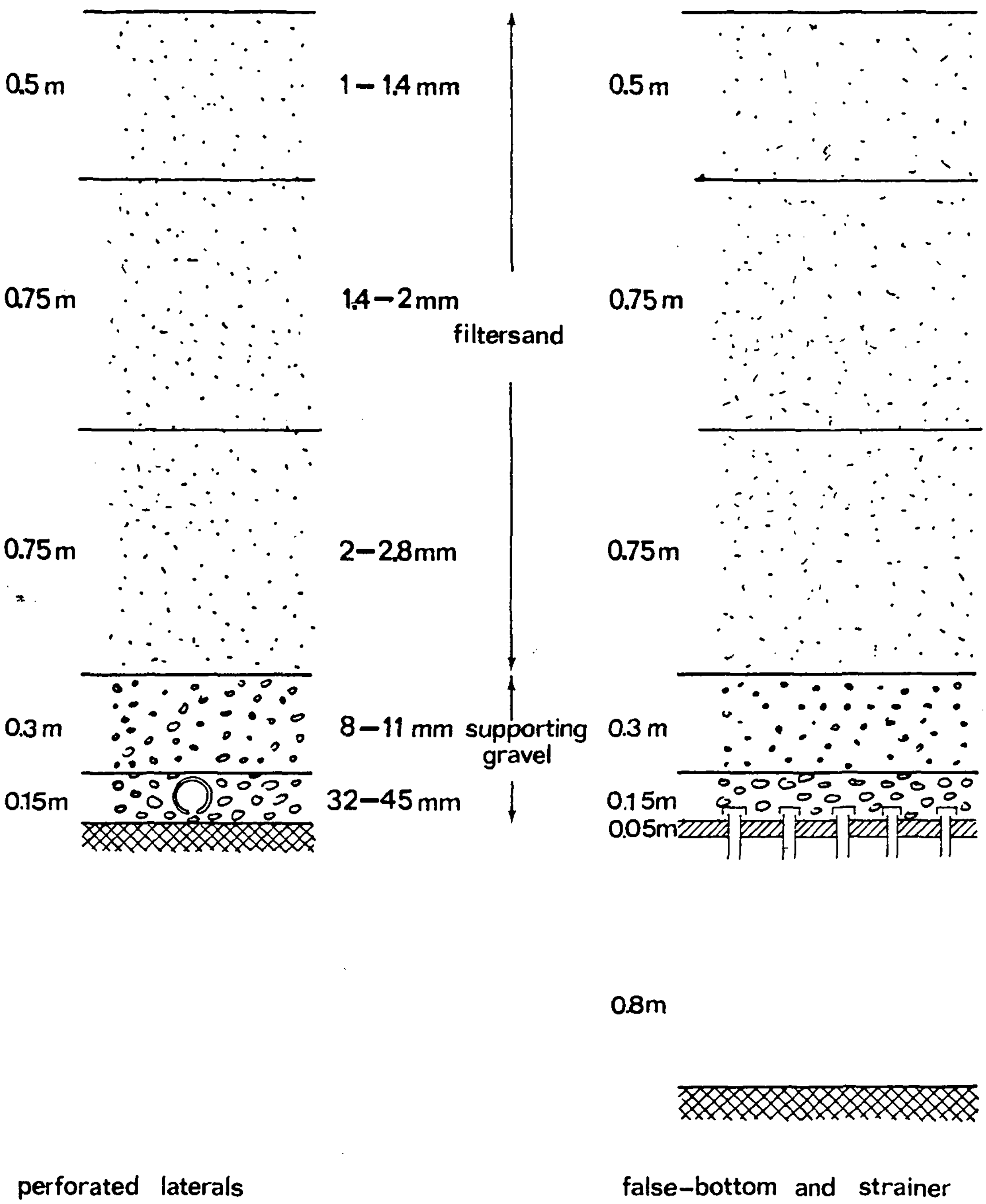


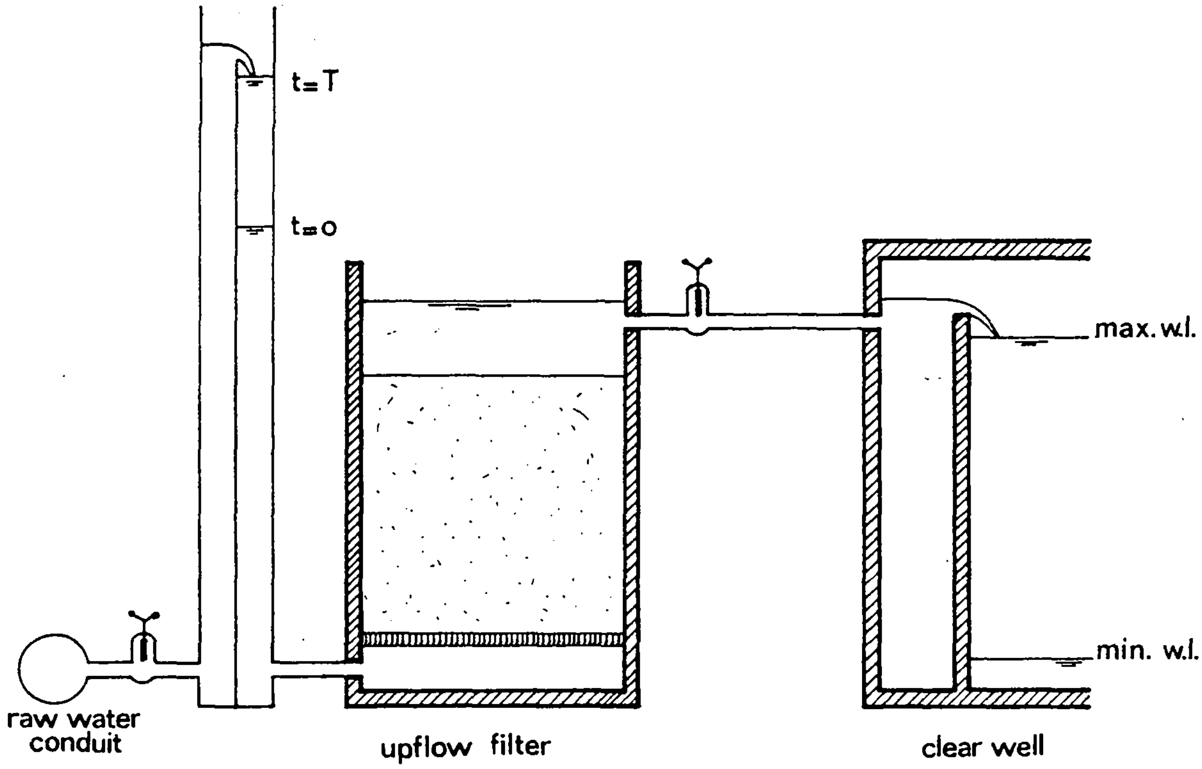
Fig. 6.10 Filterbottom construction for upflow filtration.

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.

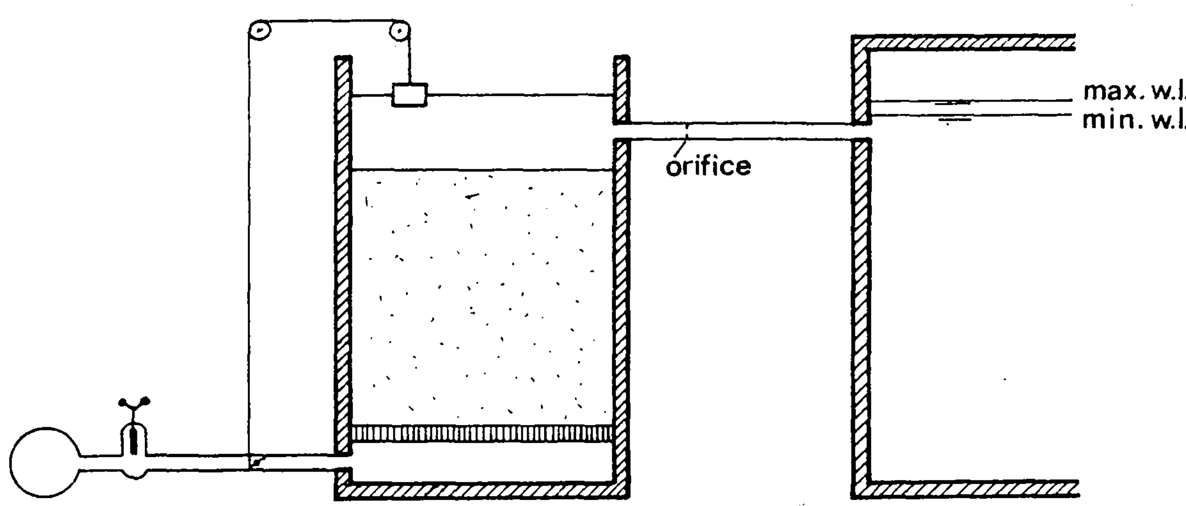
To aid in obtaining a stratified filterbed, it is usually built op of layers with upward decreasing grainsize, for instance as shown in fig 6.10. It goes without saying that the choice of the various grain sizes and the bed thicknesses requires careful thought and that for larger installations this choice 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.