Slow Sand Filtration

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Prof.dr.ir. L. Huisman

TU Delft
Technische Universiteit Delft

Faculteit der Civiele Techniek
Vakgroep Gezondheidstechniek & Waterbeheersing
Sectie Gezondheidstechniek

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Prof.dr.ir. L. Huisman

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CONTENTS page

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 is improved by part removal of suspended and colloidal matter, by a reduction in the number of bacteria and other organisms and by changes in its chemical constituents, in particular a destruction of organic matter by oxidation. In principle, the porous substance may be any stable material, as well as a granular bed of sand, crushed stones, anthracite, glass, cinders, etc, as a solid layer of porous concrete, stoneware, plastic and so on. In the field of drinking water purification, however, sand is almost used exclusively as filtering material because of its availability, relative low cost and the satisfactory experience that it has given.

The impurities removed from the water during the process of filtration, accumulate on the grains of the filterbed. In this way the effective pore space is reduced by which the resistance against the flow of water increases and the filtration efficiency drops. After some time this resistance has become 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 general types of filters may be distinguished, rapid filters and slow filters, which filters also differ greatly with respect to the filtration rate, that is the capacity per unit area of filterbed surface.

In rapid filters, the water flows through a bed of medium to coarse sand at high velocities. The effective grain size (compare section 2.3) varies from 0.5 to about 2 mm, while a common value of the filtration rate is between 1.5 and 3 mm/s (that is liters per second per m^2 of filterbed area). 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, impurities from the raw water penetrate the filter beds to great depths. Cleaning of a rapid filter is therefore only possible by back-washing, reversing the flow of water which expands and scours the grains and carries the accumulated impurities to waste (fig. 1.1).

In slow sand filtration on the other hand, 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 0.03 to 0.1 mm/s and is so small, that only after an extended period of service, a few weeks

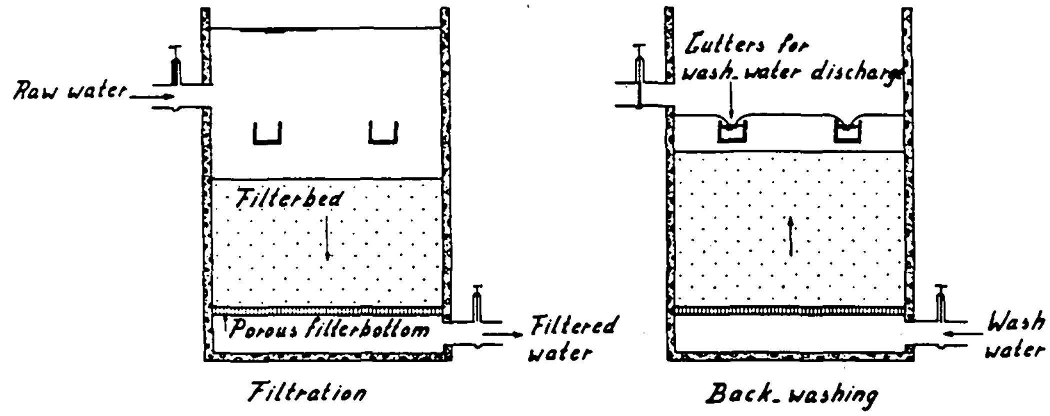

Fig. 1.1 Operation and cleaning of a rapid filter

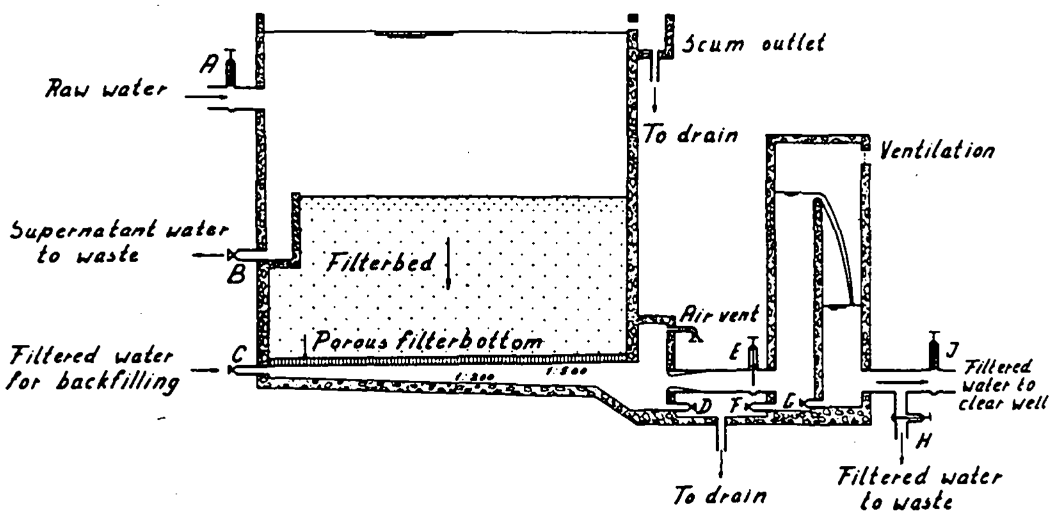

Fig. 1.2 Diagrammatic section of a slow filter

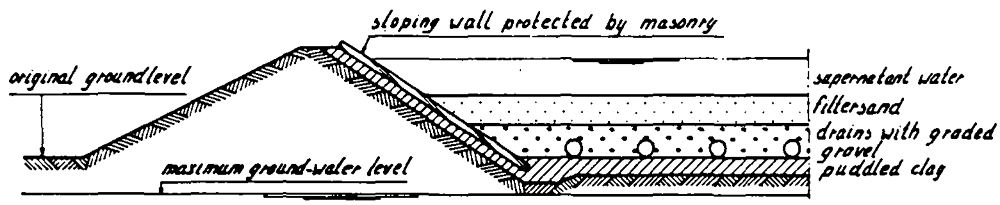

Fig. 1.3 Slow sand filter constructed of masonry on puddled clay

to a few months and more, cleaning is necessary. With the filterbed composed of fine grains - effective diameters between about 0.15 and 0.35 mm - suspended and colloidal matter from the raw water are retained in the very top of the filterbed. The cloggings 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 centimeters.

1.2 Elements of a slow sand filter

The traditional slow sand filter, as shown schematically in fig. 1.2, is an open basin, 2.5 to 4 m deep, usually rectangular in shape, built below finished ground level and varying in area from a few hundred to a few thousand m^2 and sometimes even larger. Essentially this filter consists of a box, formerly made of masonry on puddled clay with sloping sides (fig. 1.3), but nowadays constructed of mass-, reinforced- or pre-stressed concrete with vertical walls. This box is filled with a layer of sand, 0.7 to 1.3 m thick, on top of which the water to be treated is present to a depth of about 1.5 m. At the lower end this sandbed is supported by a system of drainage, the so-called filterbottom, which at the same time allows the passage of the filtered water. For convenience in drawing only, a porous filterbottom is chosen as underdrainage system of the filters shown in fig. 1.1 and 1.2, their use being by no means universal. In cold climates, slow filters must be covered to prevent freezing in winter time. In moderate and especially in hot climates covering is sometimes practized to prevent algal growth in the supernatant water and a subsequent rapid clogging of the filterbed (compare 2.2) and also to avoid bacteriological and other pollutions brought into the filter by the wind, by birds, etc. The slow sand filter finally is not complete without a number of influent and effluent lines, provided with valves and with controllers to keep the raw water level and the filtration rate constant. For clarity in presentation all these lines are drawn separately in fig. 1.2, although in real practice they are combined and concentrated as much as possible to reduce the cost of construction. During operation, the raw water to be treated enters the filter through valve A (fig. 1.2), flows down the sandbed and the underdrainage system and out through a venturimeter (or other measuring device) and the regulating valve E into a weir chamber. After overflowing the weir, the water is discharged through valve J - all other valves being closed - towards the clear well. With the regulating valve E any rate of filtration, from the minimum to the maximum allowable value may be obtained, while the weir on one hand prevents the filtered water level to drop below the top of the sand bed (compare 2.3) and on the other hand makes the filtration rate independent from uncontrollable variations of the water level in the clear well. At the same time this weir provides an appreciable amount of aeration, decreasing carbon dioxide and increasing oxygen contents. To keep partial oxygen and carbon dioxide pressures in this chamber as near as possible to those in the outside air, a good ventilation must be provided, which is also required for the filter proper, allowing air and other gases liberated in the underdrainage system to escape without hindering the movement of water. During the filtration process, a gradual clogging of the pores in the upper part of the filterbed occurs, increasing the resistance against vertical water movement. This increase is compensated by opening the regulating valve E every day a little bit, keeping the over-all loss of head the same and the filtration rate constant. When after an extended period of service the effluent valve is fully opened, a further increase in filter resistance would result in a lowering of the filtration rate and when this is not acceptable, the filter must be taken out of service for cleaning. If the composition of the raw water is such, that larger amounts of scum accumulate at the water surface, this scum must be removed regularly through scum outlets at the four corners of the filter, after the water level has been raised. This scum removal is especially important near the end of the filter run, not to render filter cleaning more difficult than strictly necessary.

Cleaning of a slow filter is commonly executed by scraping off the upper 1 or 2 cm of sand, asking, however, for a dry filterbed with a waterlevel about 0.2 m below the top. To drain the filter, the raw water inlet valve A is closed, allowing the filter to discharge over-night as much as possible in the normal way, through valves E and J to the clear well. The shallow layer of water remaining on the filterbed is removed through valve B, connected to a box of which one wall is built up of stoplogs (mostly made of concrete), so that the top of this discharge wall can be kept near the top of the sandbed, at whatever level this may be. The porewater in the upper 20 cm of the filterbed finally is taken out through valves D and F after which valves E and J are closed. After the cleaning operation has ended, the filter is slowly recharged with filtered water from below, through valve C, to a level of about 0.1 m above the top of the sandbed, allowing all air accumulated in the pores of this bed to escape. The remaining depth is filled with raw water from above, through valve A, taking care not to disturb the surface of the sandbed when this is protected by a shallow layer of water only. This may be accomplished in different ways, most simple by locating valve A in the same vertical line as the discharge box B. This box now acts as a stilling chamber and the water flows over the upper stop log with a small lateral velocity. When the raw water level in the filter has regained its normal value, valve H is opened, keeping valve J closed and the regulating valve E is turned just so far as to have the filter operate at about one quarter of its normal rate. In the next 12 or 24 hours the filtration rate is slowly increased to the required value, after which a water sample is taken. This sample is analysed in the laboratory and only when it satisfies the required standards may valve H be closed and valve J opened, bringing the filter back to normal operation. When the water quality is not satisfactory, however, valve J remains closed and the filtered water is carried to waste through valve H till further investigations prove the water quality to be acceptable. After a normal cleaning, this ripening period lasts for 1 or 2 days only, but when the filter has been drained for a longer period, for resanding, repairs, etc. many weeks may be required. After initial construction, this breaking-in period may even last for several months. To prevent deterioration of water quality during a longer interruption in normal service, all water must be removed from the filterbox and from the weir chamber through valves D, F and G. Back filling with filtered water from below must now be done with the utmost care, allowing air in the underdrainage system to escape through the air vent.

1.3 Application of slow sand filtration in drinking water practice

Since the dawn of human civilization, clarity of drinking water is the most important quality characteristic, to be obtained by sedimentation or filtration. Settling basins are easy to operate, but their effluent is not very clear. Filters give excellent results in this respect, but after some time clogging occurs, decreasing their capacity. Up till one and a half century ago, this problem could only be solved by removing the filtering material, washing it outside the filterbox and putting it back into place again, a rather laborious undertaking. A better way was found in England in the first decade of the 19th century, by the use of what is known today as a slow sand filter, combining two principles

a. the use of a fine filtering material, restricting the penetration of impurities to the upper mm's of the filterbed and allowing a restoration of filter capacity by scraping away the top layer of dirty sand to a depth of say one centimeter;

b. the use of low rates of filtration by which the interval between two successive scrapings could be enlarged to a few months.

The use of slow sand filtration to transform a turbid river water into a clear supply increased greatly after 1829 when James Simpson built such filters for the first time for a public water supply, the Chelsea Water Company in London.

The introduction of slow sand filters for public supplies coincided with the arrival of the Asiatic cholera in England in 1831. During the cholera epidemic of 1849, London was served by 8 drinking water companies of which some clarified the Thames river water by settling, others by slow sand filtration. Comparison of the cholera incidence in the various distribution areas showed beyond doubt that slow sand filtration was not only able to produce a clear water, but also to remove the pathogenic agents, to provide a water safe in hygienic respect. As a consequence, the Metropolis Water Act of 1852 required that all Thames derived water in an area of 5 miles around St Paul's Cathedral be slow sand filtered. Final and all too convincing proof of the slow sand filter's effectiveness in this respect was provided in 1892 by the experience gained in the neighbouring cities of Hamburg and Altona. Both cities took their drinking water from the river Elbe, but for Altona this water was subjected to slow sand filtration before going into supply, while in Hamburg the treatment consisted of settling only. When on August 12, Russian emigrants, encamped near the America pier, infected the stream, a cholera epidemic of majestic proportion spread with explosive rapidity through Hamburg. Of the 580.000 inhabitants in this city 19891 were infected with no less than 7582 deaths. In Altona with 143000 inhabitants on the other hand, morbidity and mortability amounted to 572 and 328 only, the majority of which cases could be attributed to contact infection. Also in later years the reliability of slow sand filtration in protection against contagious diseases was proved over and over again (for instance Cherbourg, 1909, typhoid; Egypt, 1947, cholera).

Since the middle of last century, slow sand filtration is especially applied to remove pathogenic organisms from the water, to obtain a drinking water safe in bacteriological respect. Slow sand filters are highly efficient in this accomplishment. When properly constructed and sensibly operated, they are able to reduce total bacteria counts by a factor 1000-10000 and E.coli contents by a factor 100-1000. This means that going out from a not too contaminated raw water, E.coli will mostly be absent in 100 cm^3, thus satisfying normal drinking water standards. This improvement in bacteriological quality in the meanwhile is not required for deep groundwater, which is safe in hygienic respect by virtue of its origin. Almost without exception, treatment by slow sand filtration is therefore limited to the purification of surface water. With a fairly clear water of low mineral content, from an impounding reservoir high up in the mountains for instance, slow sand filtration may be the sole treatment to which the water is subjected. Slow sand filters, however, are also very efficient in removing particulate suspended matter. By the application of fine filter grains, this material accumulates in the very top of the filterbed. With a more turbid water these deposits result in a rapid clogging and short filterruns. Although slow filters are able to cope with raw water turbidities of 100-200 g/m^3 for a few days, the turbidity should not rise above 50 g/m^3 for a longer period. Good results are only possible when the average turbidity is less as 10 g/m^3, while exellent results can first be expected when this value drops below 2 g/m^3 (measured as SiO_2). When raw water turbidities are higher than the figures mentioned above, a reduction may be obtained by pre-treatment. In order of increasing efficiency this pre-treatment may consist of:

- plain sedimentation;
- storage, if necessary with micro-straining for algal removal;
- rapid filtration;
- plain sedimentation and rapid filtration;
- storage and rapid filtration;
- plain sedimentation and coagulation supported multi-layered rapid filtration;
- coagulation, flocculation, settling and rapid filtration.

With regard to the danger of rapid clogging, it goes without saying that slow sand filtration is always the last treatment to which the water is subjected.

Chlorine as an agent for water sterilization was first used in Blankenberghe, Belgium in 1906 and gained wide application after the 1914-1918 war. With the introduction of chlorine in water works engineering, the sole mastery of slow sand filtration for the removal of pathogenic organisms was lost, while by the lower cost of chlorination the latter method gained rapidly. On the other hand, the inherent disadvantages of slow sand filtration became more pronounced as in our western society unskilled labor and space became scarce. The application of slow sand filtration is therefore declining and during the last decades only a few plants have been built. As most important disadvantages of slow sand filtration must be mentioned

a. the high cost of construction, per unit capacity about 3 times as large as for rapid filtration;
b. the large area of land required, for slow sand filters preceded by rapid filtration about 5 times as much as for rapid filters preceded by upward flow settling tanks;
c. the large force of unskilled labor required for the manual cleaning of slow sand filters, compared to nearly full automation for settling and rapid filtration. For the same plants as compared above and a capacity of 50 million m^3/year, this means labor forces of 10 and 2 man respectively.

For smaller installations of which numerous examples can be found with rural water supplies, the disadvantages mentioned above are less pronounced, while here slow sand filtration offers the enormous advantage of being safe and stable, simple and reliable. Slow sand filters have an enormous reserve capacity and are less likely to go wrong under unexperienced operation. They require a minimum of operational and maintenance skills, very important when expert supervision is not constantly available.

In developing countries the situation may just be the reverse of the one discussed above. Slow filters can be built of local materials and may now be cheaper as rapid filters, for which a large part of the equipment moreover must be bought abroad. Requirements to the filter sand are less strict and in many places suitable sand is readily secured. Land and labour are here in ample supply, while no foreign exchange is required for coagulating chemicals, lime, chlorine and so on. The most important aspect, however, is the reliability which can be maintained with simple means, with local skills and labor, while the complicated mechanical, electrical and electronical equipment of modern rapid filters asks for specialists, which here are commonly not available and otherwise must be paid very high wages.

When (polluted) river water contains larger amounts of organic matter, the systems mentioned above under b. give quite different results. With rapid and slow sand filtration, the organic matter is removed by bio-chemical oxidation, breaking down these compounds in mineral constituents such as water, carbon dioxide, nitrates, phosphates and so on. In this way the water going into supply will be low in bio-degradable organic matter. Bacteria can not thrive in such a water and as a consequence after-growth in the distribution system will be small. Even after longer detention periods deterioration of water quality will be limited and oxygen depletion is not to be feared. With settling and rapid filtration on the other hand, the removal of organic matter may now be insufficient and must be supported by prechlorination. This process is cheap, but it favours the breaking up of those compounds, which are easily attacked by chlorine. These substances are certainly not the same as the bio-degradable ones and as a consequence, chemically treated water will still contain appreciable amounts of assimilable organic matter. After adsorbtion on the surface of pipe walls, food concentrations are now sufficient for the growth of bacteria, which in their turn support higher organisms such as naîs communis and asellus aquaticus, to mention only the most popular ones. Indeed their presence does not affect the hygienic quality of the water, but it certainly lowers its aesthetic value, while the accompanying oxygen consumption may ask for frequent (and expensive) flushing of dead ends. This aftergrowth can be prevented by postchlorination in such amounts that chlorine residuals are maintained up to the tap, but in Europe the resulting chlorine taste is found very objectionable. When ammonia is present in the raw water, pre-chlorination must be carried out as break-point chlorination, asking for large doses, 8-10 gram of Cl_2 per gram of NH_4^+. This not only increases the Cl'-content of the finished water, e.g. with 10-50 g/m^3, but it also leads to the formation of trihalo methanes with cancerogenic properties. In the U.S.A. the total amounts of these compounds ($CHCl_3$, $CHCl_2Br$, $CHClBr_2$ and $CHBr_3$) is limited to 0.1 g/m^3, asking for a biological removal of organic matter and ammonia when present, in advance by storage in reservoirs or by artificial recharge (compare chapter 5) and as last step by slow sand filtration.

Summing up. for the situation of today slow sand filtration may be used to advantage for small plants and in developing countries where expert supervision is not constantly available and for polluted river water to prevent the formation of trihalo methanes. With a clear water from rivers or lakes, slow sand filtration may be the sole treatment (fig. 1.4). With a more turbid water pre-clarification is required (fig. 1.5), while for a contaminated water extensive pre-treatment may be necessary (fig. 1.6 and 1.7)

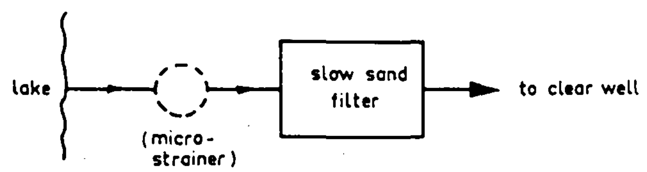

Fig. 1.4 Treatment of clear surface water

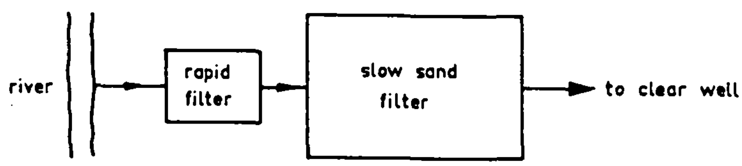

Fig. 1.5 Treatment of turbid river water

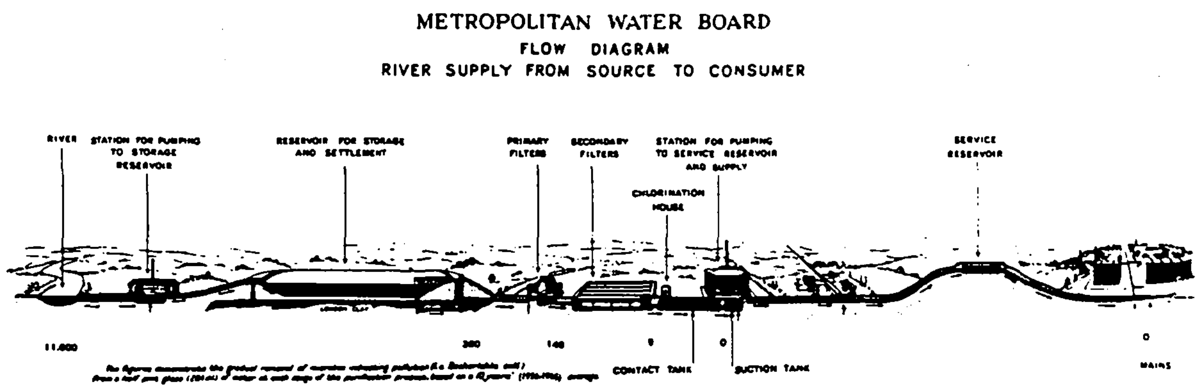

Fig. 1.6 Water supply of London

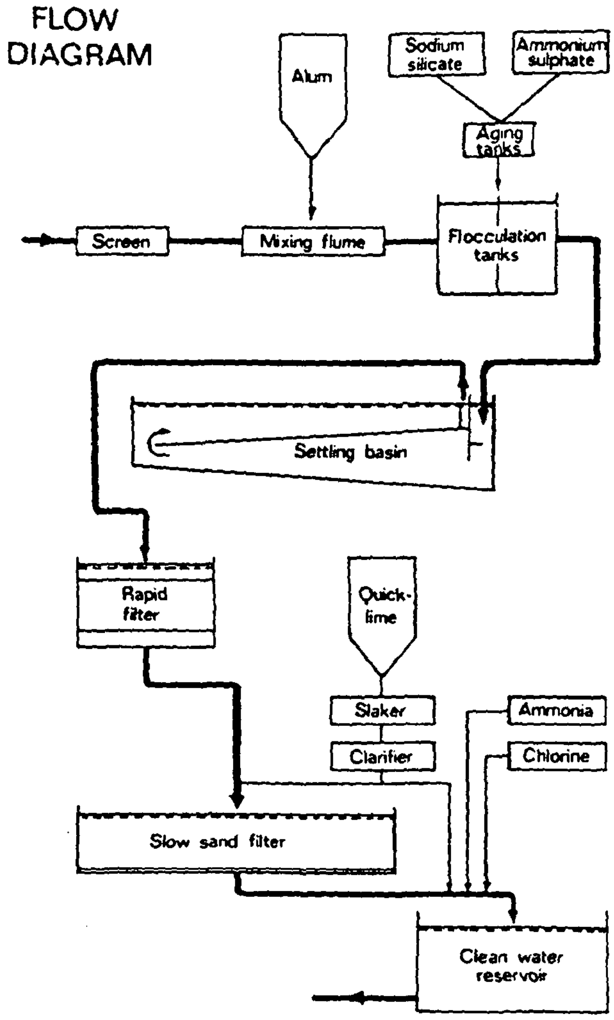

Fig. 1.7 Water supply of Stockholm

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) adsorbtion, (d) bio-chemical and (e) bacteriological activity. For clarity in presentation, these actions are described separately in the next pages. In nature no such partition is present, while the interaction of these processes together with others still only partly understood or even fully unknown, is of paramount importance. Slow sand filters are 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 pores of the filterbed. With sand grains of 0.15 mm diameter this means complete removal of suspended matter larger than 20 μm (fig. 2.1) and partial removal down to about 5 or 10 μ when by suspended matter deposited during the breaking-in period a filter skin has been formed with much smaller apertures than the minute openings between the filter grains. Colloidal matter (0.001-1 μ) and bacteria (1-10 μm), however, cannot be removed in this way. Mechanical straining takes place almost entirely at the surface of the filter, where it clogs the upper millimeters of the filterbed, increasing the resistance against the downward movement of water. Periodically this accumulation of material is removed by scraping, reducing the resistance to the original value of a new bed.

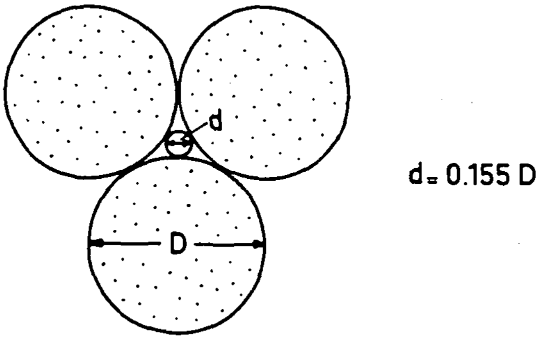

Fig. 2.1 Relation between grainsize and pore space

(b) Sedimentation removes particulate suspended matter of finer sizes than the pore openings by precipitation upon the sides 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 filtersand with a diameter d has a gross surface area of $\frac{6}{d}(1 - p)m^2$. For a normal porosity of 40% and a diameter of 0.2 mm, this gross area amounts to no less than 18000 m^2 per m^3 of filtering material. 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 with a thickness of 0.8 m will easily obtain a value of 1000 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 $(0.05)10^{-3}$ m/s not more than $(0.05)10^{-6}$ m/s. Sedimentation efficiency in the meanwhile is a function of the ratio between this surface loading and the settling velocity s of the suspended particles. For laminar settling Stokes gives:

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

in which d is the particle diameter, ρ and $\rho + \Delta\rho$ the specific mass density of water and suspended matter respectively, g the gravity constant (9.81 m/s^2) and ν the kinematic viscosity of the fluid. For water of 10^oC, $\nu = (1.31)10^{-6}$ m^2/s and $\rho = 999.7$ kg/m^3. Assuming moreover $\rho + \Delta\rho = 1010$ kg/m^3 simplifies Stokes law to

$$s = (4.29)10^3 d^2$$

With the surface loading of $(0.05)10^{-6}$ m/s as calculated above, all particles are retained with a diameter larger than

$$d^2 = \frac{(0.05)10^{-6}}{(4.29)10^3} \quad \text{or } d = 3.4\ \mu m$$

while finer particles or particles of smaller mass density are only partly removed, although flocculation accompanying the twisting movement of water through the pores of the filterbed increases sedimentation efficiency with depth.

(c) Without any doubt, adsorbtion is the most important purification process during filtration, retaining finely divided suspended matter next to colloidal and molecular dissolved impurities. Adsorbtion is effected in many ways, passively when a suspended particle has contacted a sand grain and is retained on the sticky gelatinous coating formed by previously deposited bacteria and organic matter and actively by the physical attraction between two particles of matter (London-van der Waals forces) and the electrostatical attraction between opposite electrical charges (Coulomb forces). The difference between passive and active adsorbtion in the meanwhile is small as the forces of attraction exert their influence over a small fraction of the pore space only. Again here adsorbtion is only possible after other forces have brought the particles in the immediate vicinity of the grains. Gravity has already been referred to as such a transport force, next to which may be mentioned inertia and hydrodynamic forces, moleculair and turbulent diffusion (fig. 2.2).

For active adsorbtion, the electrostatical attraction is most important, but this occurs only between opposite charges, while like charges even repel each other. By the nature of its crystalline structure, clean quartz sand has a negative charge and is thus able to adsorb positively charged particles such as (colloidal) flocs of carbonates, iron- and aluminium hydroxide next to kations of iron, manganese, aluminium and so on. Colloidal matter of organic origin, bacteria included, mostly have a negative charge, are consequently not attracted and indeed when a filter with clean sand is first taken into service, such impurities are not removed. During the

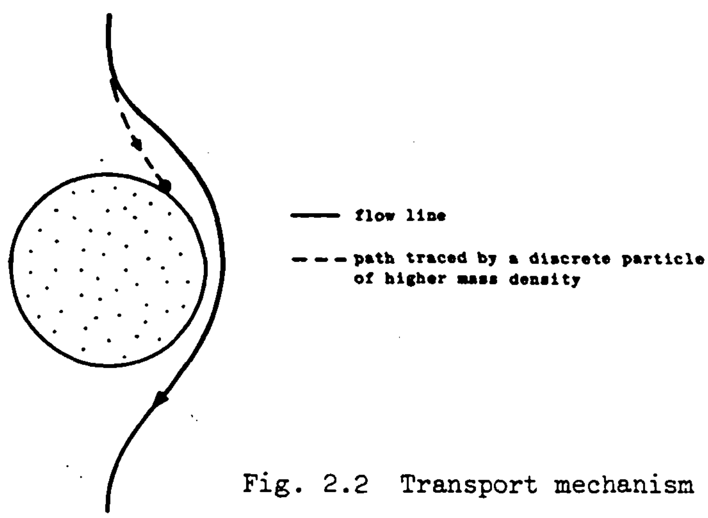

Fig. 2.2 Transport mechanism

ripening period of a slow sand filter only positively charged particles are adsorbed, locally, however, in such amounts that oversaturation occurs, by which the charge of the coating reverses and becomes positive. Negatively charged particles such as colloidal matter of animal or vegetable origin and anions of nitrate, phosphate, etc. may now also be attracted, till again the over-all charge reverses. As mentioned above, the combined surface area of the grains in the bed of a slow sand filter is enormous. After the initial breaking-in period this area shows a multi-varied and ever changing sequence of negatively and positively charged coatings, able to adsorb all impurities from the passing water.

(d) The matter accumulated on the sides of the sand grains does not stay here unchanged - with a clogging of the filterbed as ultimate result - but is oxidized by chemical and biological agents. Soluble ferrous and manganous compounds are transferred into unsoluble ferric and manganic oxide hydrates,

$$4Fe^{++} + O_2 + (2n - 4)H_2O + 8HCO_3^- \rightarrow 2Fe_2O_3.nH_2O \downarrow + 8CO_2$$

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

forming a thin coating around the sand grains. Organic matter is partly oxidised to provide the energy the bacteria need for their metabolosm

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

and partly converted into cell material for their growth. With the limited amount of organic matter provided by the inflowing raw water, however, only a restricted bacterial population can be maintained (fig. 2.3) and the growth mentioned above is therefore accompanied by an equivalent die-away, liberating organic matter. The primary and secondary dissimilation products are carried on by the water to be used again at greater depth by other bacteria. In this way the degradable organic matter present in the raw water is gradually broken down and finally converted into inorganic salts as water, carbon dioxide, nitrates, sulfates, phosphates and so on, to be discharged with the filter effluent. It should be stressed in the meanwhile that the bacterial action mentioned above needs time to establish itself, asking for a breaking-in period of adequate length during which bacteria from the raw water are adsorbed on the filter grains, where they multiply selectively using as food the organic matter present here. Degradation moreover takes place in many stages (for instance organic matter → amino-acids → ammonia → nitrite → nitrate), each one by a particular type of bacteria, present at a certain depth in the filterbed (fig. 2.4). Increasing the rate of filtration

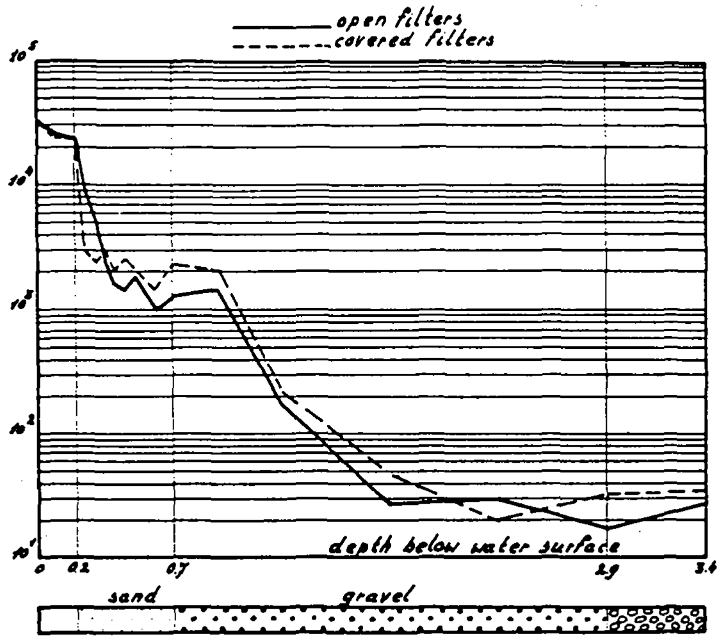

Fig. 2.3 Decrease in total bacteria count (22°C) with depth in a slow filter (filter rate 0.1 m/hour)

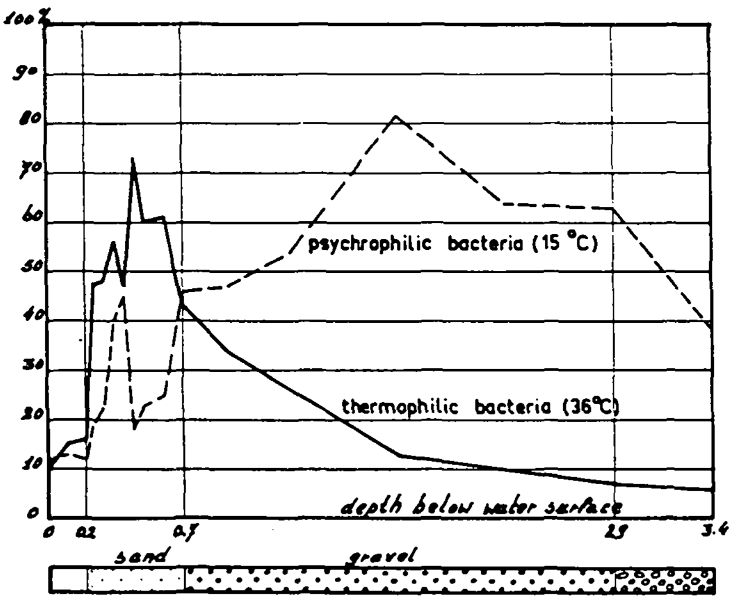

Fig. 2.4 Thermophile and psychrophile bacteria in percentage of total bacteria count at 22°C

shortens the detention time of the water in the filterbed, necessitating the bacteria to migrate to greater depth. This takes time and variations in filtration rate should therefore be effected slowly, over a period of a few hours. According to experience in temperate climates, the degradation process needs a depth of about 0.6 m to be completed, for which a minimum filterbed thickness of 0.7 m seems appropriate, to be increased by 0.3 - 0.5 m to allow a number of filter scrapings before resanding is necessary. In tropical climates and a much higher rate of bio-chemical processes these minimum and initial thicknesses could be decreased to 0.6 m and 1 m respectively.

It should be realized in the meanwhile that the bio-chemical degradation processes mentioned above need oxygen, 0.14 g per g of iron, 0.29 g per g of manganese and 2.0 g per g of organic matter. Ammonia present in the raw water will also be oxidised

$$NH_4^+ + 2O_2 + 2HCO_3^- \rightarrow NO_3^- + 3H_2O + 2CO_2$$

requiring no less than 3.6 g of oxygen per g of ammonia. When the potential oxygen consumption becomes larger than the oxygen content of the raw water, anaerobic conditions will set in, remobilizing iron and manganese and producing hydrogen sulfide next to other taste and odor producing substances. To prevent this from occurring, pre-treatment is necessary, now to lower the bio-chemical oxygen demand of the raw water.

Bio-chemical oxydation finally can only yield good results when sufficient time is available and the temperature is not too low. With slow sand filtration, time is usually in abundant supply. Going out from a filtration rate of $(0.05)10^{-3}$ m/s and a sandbed of 0.7 m thickness with 40% pore space, it takes the water more than 1.5 hours to pass the filterbed. When in winter time the temperature of the raw water decreases, the speed of biological as well as of chemical reactions goes down, roughly by a factor

$\eta = \frac{t + 10}{20}$ with t expressed in $^{\circ}$C. When the air temperature

stays below 2-5°C for any longer period, remedial measures are necessary, a covering of the slow filters to prevent heat losses as much as possible.

(e) The most important purifying action of a slow filter is the removal of bacteria, including E.coli and pathogens. By the mechanisms mentioned above, but also by their own movement, bacteria adhere to the surface of the filter grains where their food is concentrated. 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. In the upper part of the filterbed, moreover, several types of predatory organisms abound, feeding on bacteria. In the lower part of the filterbed bio-chemical oxidation has lowered the organic matter content so far as to starve the bacteria, which venture here. The micro-biological life in a slow sand filter finally produces various antagonistic actions, such as killing or at least weakening intestinal bacteria with chemical (antibiotics) or biological poisons (virusses). The over-all effect is marked decrease in the number of E.coli and as pathogens are less likely to survive, an even larger drop in their number. Starting with an average quality of raw water it is usual to find E.coli absent in 100 cm^3 samples of filtered water, thus satisfying normal drinking water quality standards. This reduction of intestinal bacteria will be larger as the proper flora and fauna of the slow sand filter is better developed, that is with sufficient food and not too low temperatures. As long as oxygen is available, the micro-biological activity is independent of the actual content, but at lower temperatures the activity of bacteria eating protozoa and nematodes drops sharply, while on the other hand the metabolism of intestinal bacteria slows down, again increasing their chance of survival when passing the sandbed. Below $2^{o}C$ the reduction in E.coli content may fall to a factor 2 only, making post-chlorination a necessity to obtain a water safe as a public supply.

2.2 Algal actions

During the long intervals between two successive filter cleanings, a large growth of algae - introduced with the influent - may occur in the supernatant water. This growth has many advantages and disadvantages, the balance between the two depending in particular on climatological conditions.

As autotrophic organisms algae are able to build up cell material from simple minerals such as water, carbon dioxide, nitrates, phosphates, etc., with the help of solar energy. As regards the carbon cycle, this process may be described schematically by the relation

$$n(CO_2) + n(H_2O) + \text{light} \rightarrow (CH_2O)_n + n(O_2) \qquad (1)$$

For their metabolism algae need energy, which they produce by oxidizing organic matter. The same reaction occurs when algae die, their cell material is liberated and consumed by bacteria in the filterbed

$$(CH_2O)_n + n(O_2) \rightarrow n(CO_2) + n(H_2O) + \text{energy} \qquad (2)$$

Reaction (2) is a continuous one, but reaction (1) is only possible when light is present. This means a strong diurnal variation in the oxygen content of the supernatant water and of the effluent (fig. 2.5). In severe cases anaerobic conditions may occur during the dark hours of the night, producing an effluent still containing ammonia, iron and manganese and of unpleasant taste and odor.

In tropical areas with only slight variations in daylight hours and temperature, growth is always larger than decay. The volume of algae increases continuously, blocking the filterbed surface, hindering the downward water movement and necessitating filter cleanings at shorter intervals.

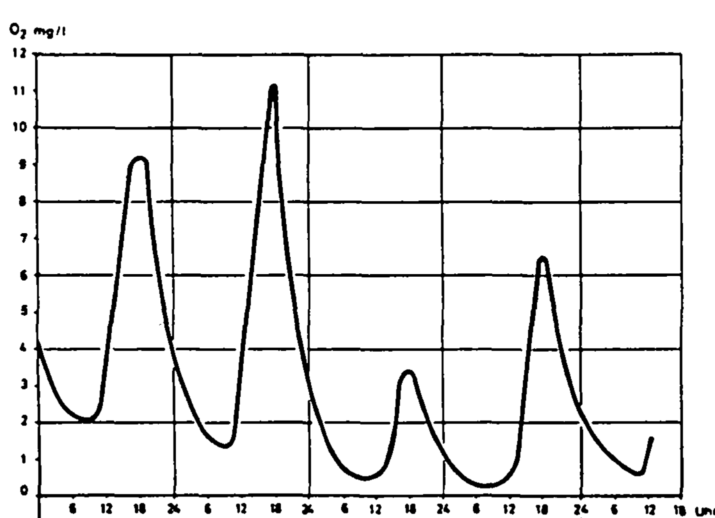

Fig. 2.5 Oxygen content of the effluent from a slow filter

With reaction (1) predominating, the carbon dioxide content of the supernatant water moreover goes down, shifting the Tilmans equilibrium

$$Ca(HCO_3)_2 \rightleftarrows CaCO_3 \downarrow + CO_2 + H_2O$$

to the right and again promoting filter clogging by deposition of insoluble carbonates.

During spring and summer in temperate climates, the situation is pretty much the same as described above for tropical ones. Due to the larger number of daylight hours, however, sudden outbursts of algal bloom may now occur, in particular with diatoms increasing the filter resistance and requiring additional filter cleanings. In autumn the temperature of the water will drop, living conditions for algae will become less favorable and they will die away more or less gradually. According to reaction (2) oxygen is consumed and carbon dioxide produced, lowering the pH and making the water more aggressive. Even when the oxygen content of the effluent is still satisfactory, part of the filter may now be covered with dead algae below which anaerobic conditions occur, deteriorating effluent quality as mentioned before. When the temperature drops suddenly - or other unfavorable conditions are produced - a massive die-away of algae will take place. So much degradable organic matter is now liberated that anaerobic decay sets in, liberating ethereal oils of abhorrent taste. Again the filter must be taken out of service to remove the dead organic matter. The unplanned additional filter cleanings not only make life for the resident engineer rather miserable, but they also endanger the safety of the supply when the remaining filters are not able to produce the required amount of water.

The presence of algae also has benificial effects, materially improving the quality of the effluent. As such an action may be mentioned in the first place, that algae also use organic matter from the raw water to build up their cell material. In quantitative terms the net result is again zero, but as regards the property of the organic matter there is a great improvement when stable compounds from the raw water are converted into material that is easily degradable the moment algae die and their cell

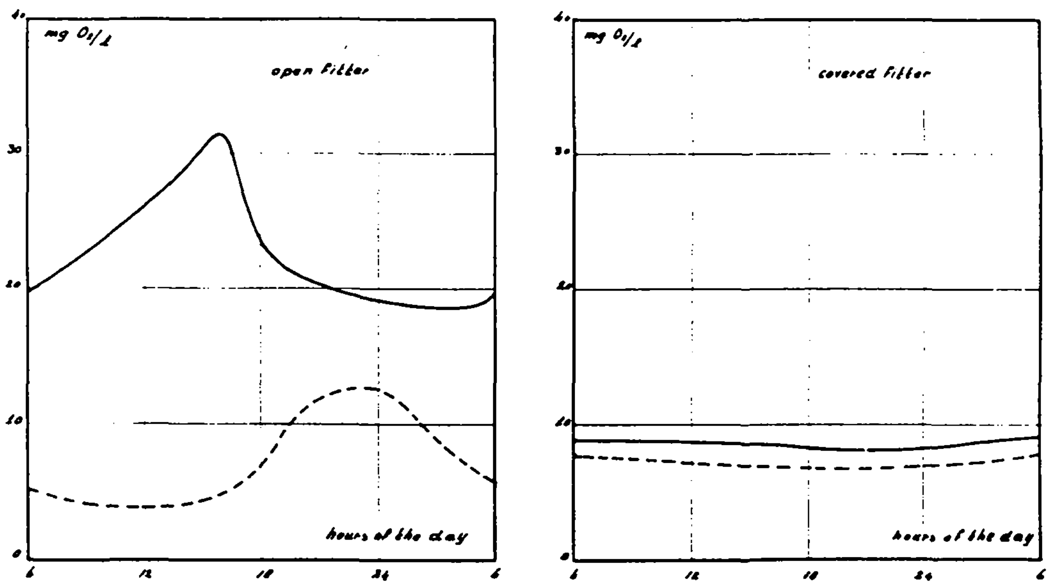

Fig. 2.6 Change of oxygen content in a slow sand filter during the day

material is liberated. According to fig. 2.6, the average oxygen content of the effluent from the covered filter is practically equal to that from the open filter, but in the open filter the oxygen consumption is about 15 g/m^3 compared with only 1.5 g/m^3 for the covered filter. This tenfold increase in the oxydation of organic matter is not only due to the carbon cycle as described by the equations (1) and (2) given above, but will partly also be produced by a conversion of unassailable into degradable organic material. With the rise in oxydative activity, the chance that harmfull organic substances (dead as well as living) are destroyed, is also materially increased. When algae are harvested periodically, the mineral content of the water moreover goes down, in particular the nitrates and phosphates as nutrients.

As other benificial factor of algae growth may be mentioned that filamentous species form on the surface of the filter a gelatinous mat, removing suspended matter and bacteria by straining and adsorbtion. Especially the true water bacteria will multiply here, creating a zooglea or Schmutzdecke of bacterial slime, an adhesive medium for plankton and diatoms. Straining and adsorbtion efficiences of the Schmutzdecke are thus enhanced and less suspended matter will reach the filter proper, thus prolonging filter runs. When under the influence of sunlight oxygen is produced, the bubbles of pure oxygen stick to the surface of the algae, increasing their buoyancy. The zoogleal mat together with part of the adhering filter skin is lifted and the filter resistance now even drops, with constant filtration rate sometimes over 0.1 or 0.2 m in a few hours. The Schmutzdecke also provides a favorable environment for protozoa and other higher organisms. Consequently they will flourish here, feeding on bacteria and thus materially reducing the number of E.coli and pathogens reaching the filterbed. According to some investigations, algae produce substances harmfull to bacterial life and thus actively reduce the chance of survival.

Algae growth with all the effects described above can be prevented by covering the filters, in this way excluding the light. Covering is a necessity in cold climates, to prevent freezing in winter time (fig. 2.7). In other climates it is a matter of choice, weighing advantages against disadvantages. The latter only occur when algae develop in larger numbers. This asks for sufficient light and nutrients, that is to say on one hand a clear water of low turbidity and on the other hand minimum amounts of mineral constituents such as carbon dioxide, nitrates, phosphates and so on. When these conditions prevail in temperate climates, additional and unexpected filter cleanings must be effected at short notice, in periods of higher temperature because of algal blooms and in periods of decreasing temperature because of a massive die-away of algae. This asks for a large force of unskilled labor which in industrialized countries is difficult

Fig. 2.7 Clearing away the ice at the Weesperkarspel plant for Amsterdam water supply

to get and expensive to pay. Covering the slow sand filters is now the appropriate solution, producing longer and more regular filter runs and allowing filter cleanings to be carried out at any time, also during the night or in periods of frost. Covering the slow filters at the dune water plant of Amsterdam Waterworks allowed the average filtration rate to be raised from $(0.03)10^{-3}$ to $(0.1)10^{-3}$ m/s, while the length of filter run even slightly increased and decidedly became more regular. Covering of slow filters has the added advantage of preventing pollution brought into the filter by wind, birds, etc. Indeed, the absence of a Schmutzdecke slightly reduces filter efficiency, especially with regard to the reduction of E.coli, but this can be solved by post-chlorination at low doses, 0.5-1 g/m^3.

In tropical climates, the temperature is more or less constant during the year. Algal blooms will be less pronounced, a massive die-away does not occur and there will be little necessity for filter cleaning at short intervals. Problems of ice formation will neither occur and here the balance seems to favour open filters. When with each filter cleaning algal material is removed, the reaction according to equation (1) predominates and the net result is an increase in oxygen content, allowing more organic matter to be degraded, and a decrease in carbon dioxyde content, rendering the water less corrosive. The purifying action of the Schmutzdecke moreover will improve filtration results, in particular bacteriological efficiency, very important when a safety chlorination cannot be provided.

2.3 Hydraulics of filtration

With slow sand filtration, the filtration rate v is so small that under all circumstances laminar flow conditions are present. The resistance H_o of a clean filterbed with depth L is consequently given by Darcy's law

$$H_o = \frac{v}{k} L$$

with k as so-called coefficient of permeability, expressed in m/s. Its value can best be determined in the laboratory by actually measuring the resistance of a representative sample of filtering material to be applied (fig. 2.8).

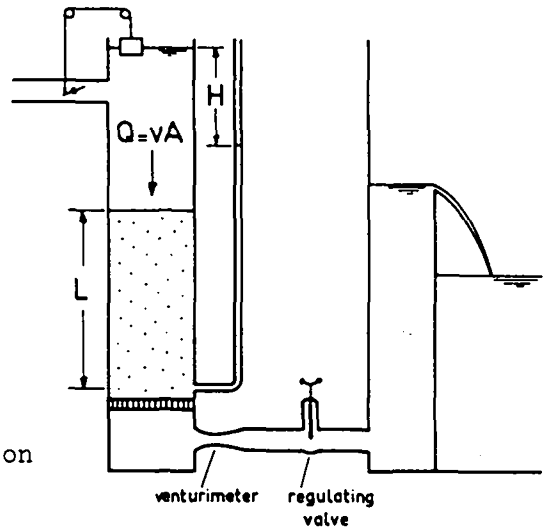

Fig. 2.8 Head loss during filtration

It can be approximated by the Carman-Kozeny equation for the flow through a bed of spherical grains with one and the same diameter d

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

with g as gravity constant, p as pore space and ν as kinematic viscosity depending on temperature

$$\nu = \frac{(1.31)10^{-6}}{0.72 + 0.028\ t} \quad \text{with t in } {}^{o}\text{C}.$$

In practise, the pore space p - about 0.38 - is mostly unknown, while the filter grains are never spherical and always vary greatly in size. With slow sand filtration, the filtering material moreover is characterised by the effective diameter d_e and the coefficient of uniformity U

$$d_e = d_{10} \quad , \quad U = \frac{d_{60}}{d_{10}}$$

in which the 10 and 60% diameters are determined by carrying out a sieve analysis (fig. 2.9), using square woven wire sieves. Taking all these complicatingfactors into account, the Carman-Kozeny equation may be transformed for rounded sand grains to

$$k = (30)10^3(0.72 + 0.028\ t)(0.18 + \log U)d_e^2\ \text{m/s}$$

For a temperature of 5^{o}C, U equal to 2.5 and effective sizes of 0.15 and 0.35 mm, this gives k values of 0.34 and 1.8 mm/s respectively. With a filtration rate of 0.1 mm/s and a bed thickness of 1.2 m, the initial resistances thus became 0.35 and 0.07 m.

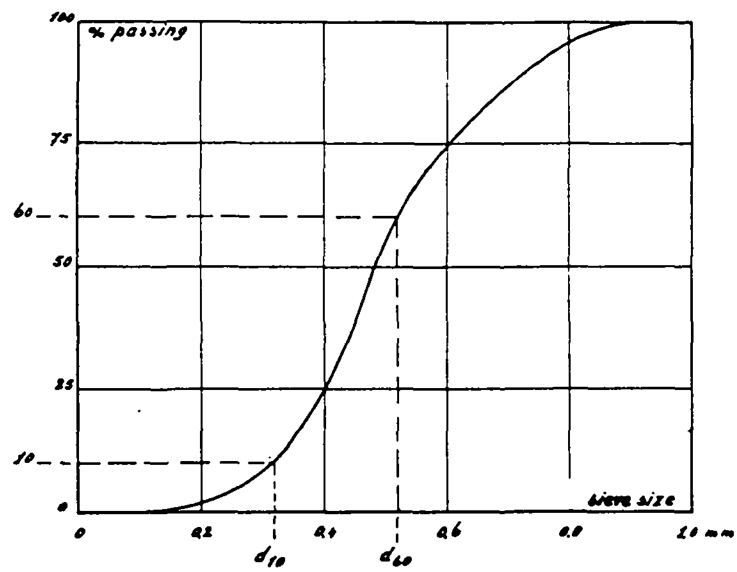

Fig. 2.9 Sieve analysis

During filtration, impurities from the raw water are deposited on top and in the first mm depth of the filterbed, producing a filterskin and reducing the pore space available for the flow of water. Resistance against downward water movement will thus increase, more rapidly as the raw water carries a higher suspended load, the filtration rate is larger and the filterbed is built up of finer grains. In contrast with deep bed filtration in rapid filters, a calculation of this increase in resistance is impossible and the necessary data should now be gathered from operating an experimental station (fig. 2.8). Generally speaking, the filter resistance first increases only slowly with time, but very sharply at the end of the filterrun. Except with fine grained filtering materials and a high initial loss of head, the length of filterrun cannot be augmented appreciably by increasing the maximum permissible filter resistance. Commonly this resistance is limited to 1 or 1.5 m and will seldom exceed 2 m.

With the small depth to which impurities from the raw water penetrate into the bed of a slow filter (surface filtration), the resistance against downward water movement will only increase in the very top of this sandbed, the bio-chemical reactions occuring at greater depth not attributing to the loss of head. This phenomenon has a marked influence on the pressure distribution in the filterbed. Without water movement, water pressures in the filterbed increase hydrostatically with depth. Resistance produced by the flow of water will lower these pressures, but with a clean filter the decrease is uniform with the maximum value equal to the initial loss of head H_o occuring at the bottom of the bed (fig. 2.10). The clogging accompanying filtration will result in an additional lowering of the pressure, but with surface filtration this lowering is constant over the full depth of the bed and as a consequence, the lowest pressures are found in the very top of the sandbed. When filtration goes on over extended periods of time, the pressure here may even become negative and this now offers grave dangers to the reliability of the slow sand filtration process. The point is that especially with open filters and an abundant algae growth, super-saturation of the supernatant water may occur. A pressure in the filterbed lower than atmospheric will now result in a release of the dissolved gases. This phenomenon is known as air binding because the released gas bubbles accumulate in the openings between the sand grains, hindering the downward water movement, increasing filter resistance and decreasing filter rates. When this air binding occurs over part of the filterbed only, the loss in capacity may hardly be noticeable but the remaining portion of the filter surface will be overloaded, resulting in a deterioration of effluent quality. The most dangerous situation, however, develops when by a drop in filtered water demand the filtration rate is lowered. Pressures in the filterbed will now rise (fig. 2.11) and when this takes place in a short period, there will be insufficient opportunity for the accumulated gas bubbles to dissolve. Instead they are forced out of the filterbed, upward through the filter skin, leaving openings through which the water may pass with imperfect

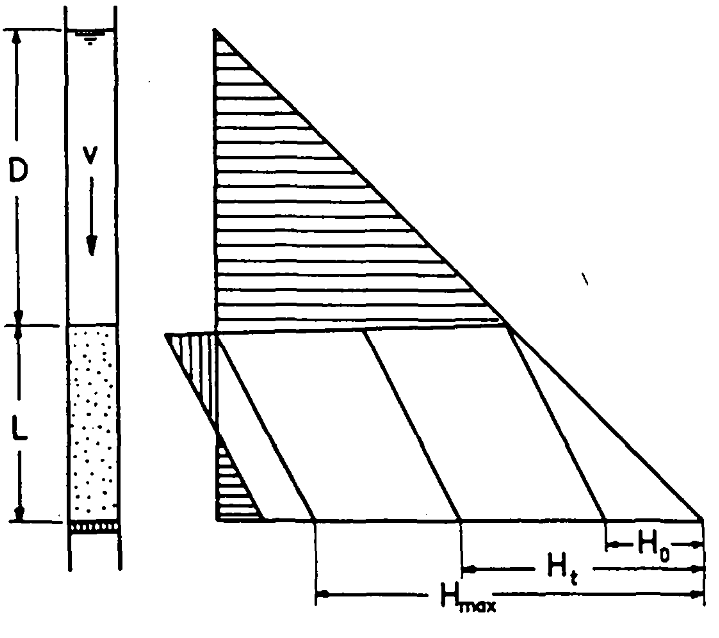

Fig. 2.10 Pressure distribution in the bed of a slow filter

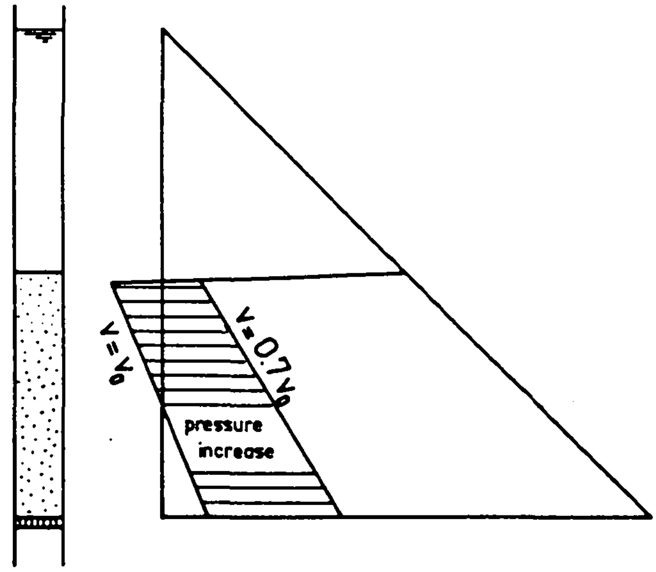

Fig. 2.11 Increase in pressure due to a reduction in filtration rate

treatment. This may have disastrious results when the slow filter is relied upon to produce a water safe as a public supply. With slow sand filtration negative pressures must therefore be prevented by all means and the maximum allowable resistance H_{max} has to be restricted to the combined values of the depth D of water on top of the filter and the (minimum) initial resistance H_o of the clean bed (fig. 2.10). In many filters a weir is constructed in the effluent line, making it physically impossible for the filtered water level to drop below the top of the sandbed (compare fig. 1.2).

2.4 Filter design

For the actual design of a slow sand filter, four dimensions have to be chosen in advance: the thickness of the filterbed, the grainsize distribution of the filtering material, the rate of filtration and the depth of supernatant water. Using a completely unknown source of which the quality is not comparable with raw water treated elsewhere, these choices should properly be based on the results obtained with a pilot plant, operating experimental filters for instance built from a section op pipe, 4 m long and a diameter of 0.5-1 m. With rapid filtration these tests are quite complicated as both the improvement in water quality as well as the length of filterrun depend on all the factors mentioned above. With slow sand filtration these tests are much simpler as experience gained in temperate climates (the Netherlands) as well as in tropical ones (India) have shown that for a given raw water source, the quality of the effluent only depends on the grainsize distribution of the filtering material. The rate of filtration has no influence on this quality, but together with the maximum allowable head loss it determines the length of filterrun. This gives a very simple and straight-forward design procedure

a. choose the minimum bed thickness at the standard value varying from 0.6 m in tropical areas to 0.7 m in temperate climates. For initial construction this thickness should be increased by 0.3-0.5 m to allow a number of scrapings before resanding is necessary;
b. going out from the results of the experimental plant, the grainsize distribution is chosen just so fine to obtain an effluent of acceptable quality;
c. on the basis of the same experimental results the rate of filtration is chosen so high that for a head loss of 1-1.5 m and normal operating conditions a length of filterrun of about 2 months is obtained;
d. in case the filtration rate as found above is rather small, improvement may sometimes be obtained by increasing the maximum allowable head loss. This asks, however, for a greater depth of supernatant water, increasing the depth of the filterbox and the cost of construction;
e. with regard to cleaning during which the filter is out of service for 2 or 3 days, small installations should be provided with one and large installations with two more filtering units than follows from the ratio between the required capacity and the chosen rate of filtration;
f. never forget that overloading a slow filter will not harm effluent quality, but it may result in a disproportionate increase in filter resistance, reducing the capacity;
g. in tropical climates and a more or less constant temperature, representative test results may be obtained at any time of the year. With temperate climates on the other hand results gained in winter time are decisive.

When the quality of the raw water asks for fine grains and the water moreover is turbid, only very low rates of filtration can be applied. Pre-clarification may now offer a more economical solution. Rapid filters as pre-treatment are attractive when they allow a rate of 0.02 mm/s to be increased by 15%, a rate of 0.04 mm/s by 35% or a rate of 0.06 mm/s by 65%.

3. CONSTRUCTION

3.1 Filterbox

The filterbed together with the underdrainage system below and the supernatant water above are encased in a box with a depth of 2.5 to 4 m, a surface area varying from a few hundred to some thousands of m^2 and constructed of masonry, brickwork or concrete. In some cases filters have been built to curved plans and random areas to conform to the contours of the site (fig. 3.1), while in extreme cases they were even laid out in complicated geometrical patterns to give the work the appearance of ornamental gardens. Nowadays, however, filters are built with rectangular plans of equal areas (fig. 3.2), while for very small installations also circular plans (and steel tanks) might be considered. To prevent any possible contamination of the effluent by groundwater entering the filter through cracks and fissures in walls and floor, the filter should always be constructed above the (highest) groundwater table, if necessary on built-up grounds. With regard to heat isulation and easy access for cleaning, filters are commonly constructed with the top of the wall only a short distance above finished ground level. On the other hand this distance should not be so small as to allow contamination by dust and debris blown in by the wind or by small animals (mice, rodents, etc.) falling into the filter.

With regard to the high cost of labour, masonry and brickwork are nowadays seldom applied. These materials moreover have the disadvantage of cracking under tensile stresses, resulting in a large loss of water when the underground is pervious. This loss of water may be reduced by constructing the filters on puddled clay with sloping sides (fig. 1.3), but this increases again the danger of fissuring by differences in settlement, while over the sloping walls the depth of water is small, allowing full penetration of the sunlight and thus resulting in an abundant aquatic growth. Sloping walls facilitate ice removal, but when mechanical means are available (fig. 2.7) this advantage is negligeable. At present filters are therefore mostly constructed of concrete with vertical walls. Either reinforced or mass concrete may be applied, depending on local circumstances such as availability and cost of steel bars for reinforcement, whether skilled steelworkers are at hand, etc. and above all on the

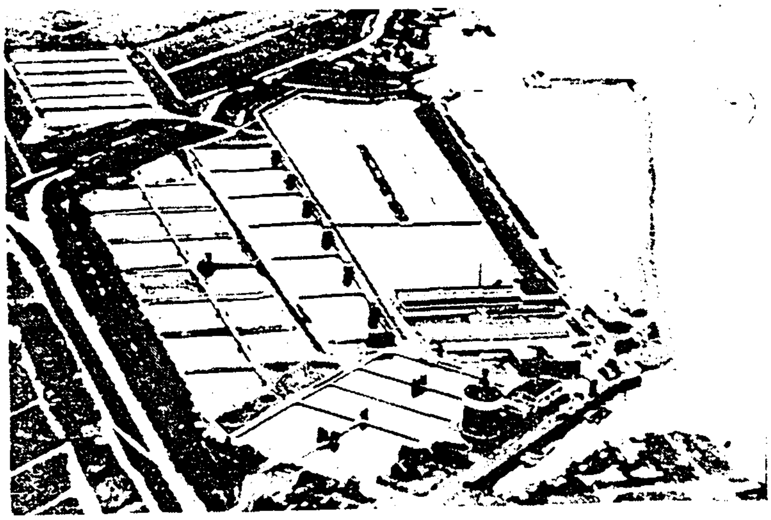

Fig. 3.1 Aerial view of the Honingerdijk plant for Rotterdam water supply

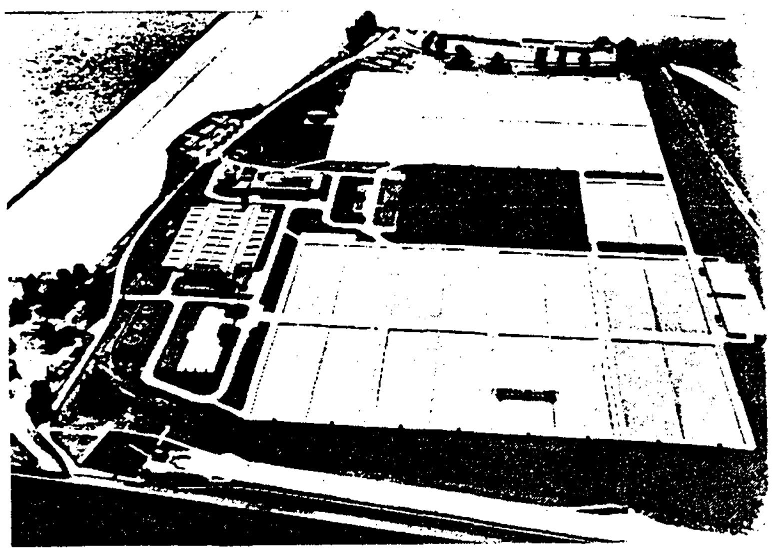

Fig. 3.2 Coppermills plant of the Metropolitan Water Board

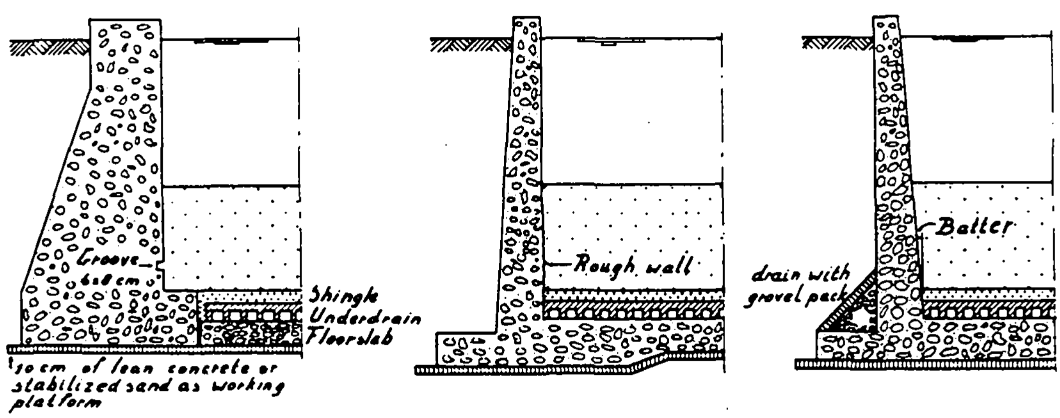

Fig. 3.3 Preventive measures against short-circuiting the filter bed along the vertical wall

experience and preference of the designing engineer. Short circuiting of the filterbed along the vertical walls must be prevented by all means. With mass concrete this is often obtained by providing the vertical wall with a groove as indicated in fig. 3.3 on the left, while with reinforced concrete the wall is roughened over the depth of the filterbed, for instance by constructing the shuttering here of unplaned boards with horizontal joints. The most effective measure, however, is giving the filter wall a slight outward batter (fig. 3.3 on the right). After settlement of the filtering material, the grains press with great force against the wall, making any cracks or holes here physically impossible. If so desired, the latter method may even be combined with a grooved or roughened wall as mentioned above. Formerly this short-circuiting was often made uneffective by keeping the underdrainage system some distance from walls, columns, etc, with a corresponding decrease in filtering area, however, as a result (fig. 3.4). To prevent fouling by aquatic growth as much as possible, the surfaces in contact with the water and the gravel of the underdrainage system should otherwise be as smooth as possible, for instance by applying wood shuttering covered with a plastic coating. The structural design of slow sand filters finally follows common rules, but particular attention should be paid to the special requirements of water retaining structures and to the effect of expansion and contraction, shrinkage of concrete, differences in settlement, etc., in order that a water-tight structure may result. This is a task more difficult to perform as the filterbed area is larger.

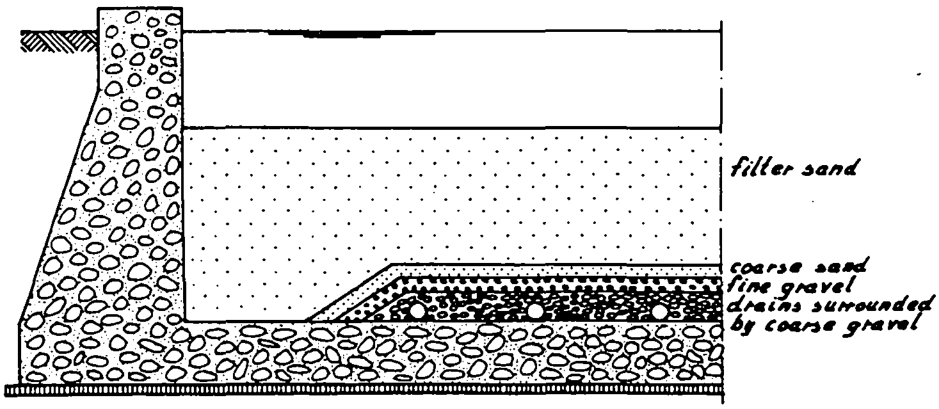

Fig. 3.4 Short-circuiting along the filter walls ineffective

3.2 Supernatant water

To prevent with certainty negative heads and air-binding as described in 2.3, the raw water depth on top of the filterbed should always be larger as the resistance due to clogging of the filterbed (fig. 2.10). In principle this allows a variable raw water level, going up as filtration proceeds and resistance against downward water movement is built up. A very simple filter control is now possible (fig. 3.5), but in practice constant raw water levels are always preferred. With a constant depth of water, the danger of disturbing the filter skin on top of the filterbed is negligeable, removal of floating impurities through the scum outlets is simple and a deep penetration of sunlight with a subsequent growth of rooted aquatic plants will not occur. Which depth must be applied, depends on the maximum allowable resistance (fig. 2.10) and is mostly between 1 and 1.5 m, while a value of 2 m is seldom surpassed. Above the normal water level a free-board is still necessary for which a value of 0.2 or 0.3 m will commonly suffice.

To remove all or part of the supernatant water a drain is commonly present (fig. 3.6), discharging into some reservoir from which the water is sometimes pumped to another filter or otherwise carried to waste. When this system of drainage is used every time a filter is cleaned, the time required for emptying may not be too large. This time interval can be calculated by determining for the discharge pipe the relation between

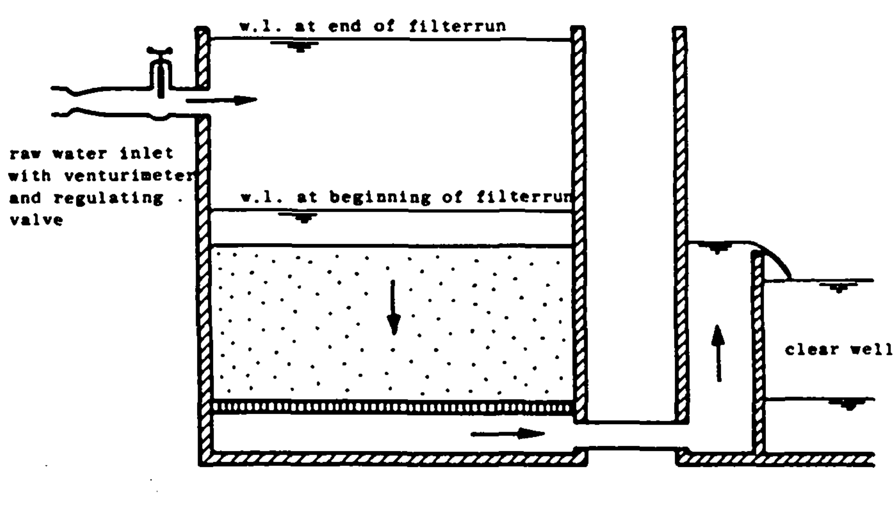

Fig. 3.5 Slow sand filter with raw water level rising as clogging of the filterbed goes on

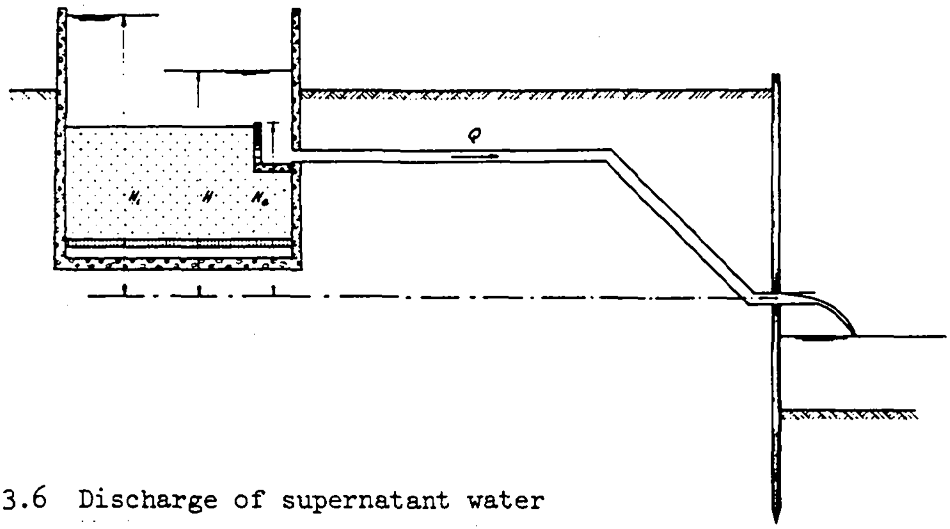

Fig. 3.6 Discharge of supernatant water

the capacity Q and the accompanying losses H due to friction and turbulence:

$$Q = \alpha\sqrt{H}$$

With A as filter area, a discharge Q during a time dt changes the raw water level by an amount dH equal to:

$$dH = - \frac{Q}{A}\,dt$$

Combining both equations gives:

$$\frac{dH}{\sqrt{H}} = - \frac{\alpha}{A}\,dt$$

Intergration between the limits $t = 0$, $H = H_i$ and $t = T$, $H = H_e$ gives as time required for emptying:

$$T = \frac{2A}{\alpha}\left(\sqrt{H_i} - \sqrt{H_e}\right)$$

With a pipe 0.2 m diameter, a wall roughness of 0.5 mm, 50 m long, α is about 0.048 giving with $A = 1000\ m^2$, $H_i = 2.0$ m and $H_e = 0.5$ m

$$T = \frac{2000}{0.048}\left(\sqrt{2.0} - \sqrt{0.5}\right) = 30000 \text{ s or a little over 8 hours.}$$

3.3 Sandbed

As filtering material for slow filters, sand is used exclusively. With regard to the large amounts required, ungraded materials are commonly applied. This is not a great disadvantage as the sand remains unstratified, in contrast with rapid filtration where during back-wash a hydraulic grading occurs. Still to obtain pores of uniform size with a larger porosity and permeability, the coefficient of uniformity should be below 3 and preferably below 2. A value less than 1.5 offers little advantage in terms of porosity and permeability and should therefore only be pursued when it entails no additional cost. Filtersand should be composed of hard and durable grains, preferably rounded, free of clay, loam, dirt and organic matter. If necessary the sand should be washed, by which also the finest grains are removed, lowering the coefficient of uniformity. To prevent cavities to develop in the filterbed when by rising carbon dioxyde content the water becomes agressive, the sand should not contain more than 2% of calcium and magnesium, computed as carbonates.

Preferably by means of tests carried out in an pilot plant, the effective diameter of the sand should be chosen just small enough to assure a good quality effluent and to prevent a deep penetration of the clogging matter from the raw water that can not be removed by scraping off the top layer of dirty sand. Commonly this effective grain size is found to vary from 0.15 to 0.35 mm, although finer as well as coarser materials have been applied in the past. When the desired grade of sand is not found in nature, it may be prepared by mixing two types of stock sand (fig. 3.7).

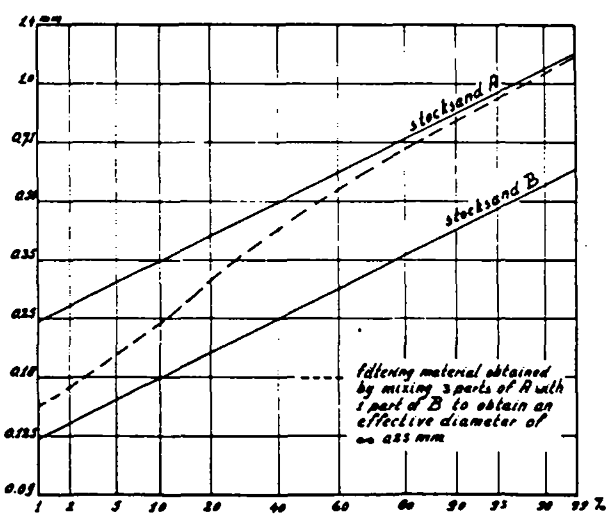

Fig. 3.7 Preparing filtering material by mixing two types of stock sand

This augments the coefficient of uniformity, while good care should be given to mix the material thoroughly, for instance using a concrete mixer. As last resort screening may be applied, removing the fine particles to increase the effective size and the coarse particles to obtain a more uniform sand.

The required thickness of the filterbed depends on the depth to which bacteria are active, degrading organic matter in many stages. As mentioned before, this depth is seldom larger than 0.5 m to 0.6 m depending on temperature (section 2.1) from which a minimum depth of the filterbed of 0.6 to 0.7 m would follow. With a raw water low in organic content, this depth may be slightly decreased, but value of 0.3 and 0.4 m as formerly applied do not utilize the full capacity of a slow filter and should consequently be avoided. When on the other hand the raw water is rich in organic substances, high rates and/or coarse grains are used, the depth of the filterbed may be increased, for instance to 0.8 m. These values in the meanwhile refer to the minimum thickness of the bed, while the initial thickness must be larger to allow a number of successive filter cleanings by skimming of the top 1 or 2 cm before resanding is necessary. Which additional depth should be provided depends on the depth of skim and the number of filter cleanings per year. With an average length of filterrun of 2 months and a depth of skim of 1.5 cm for instance, an initial thickness of 1.2 m allows a period of 5 years before the minimum value of 0.75 m is obtained.

Sometimes the filterbed is provided with a layer of activated carbon, about 0.1 m deep, to promote adsorbtion and to remove the last traces of taste and odour producing substances. Not to tax this carbon more than strictly necessary, it should be put near the bottom of the filterbed, but on the other hand at such a depth that it may easily be removed during resanding (compare 4.2) when saturation is suspected.

3.4 Filter bottom

On the floor of the filter box the underdrainage system is present, serving the twofold purpose of supporting the filtering material and providing an outlet for the water passing through the filter. It goes without saying that his support must be designed and constructed in such a way that no penetration of granular material can occur, which otherwise would clog the waterway below. Next to this the filtered water must be collected evenly over the entire area of the filter so as to have all portions of the filter bed perform as nearly as possible the same amount of work. This is not so much of importance with regard to water quality, but it will cause a more rapid clogging of the filterbed, shortening the length of filterrun. To prevent this as much as possible, the loss of head due to friction and turbulence, accompanying the flow of filtered water over the length and width of the filterbottom should be small, say less than 20% of the resistance of the clean filterbed. With d_1 = 0.2 mm, U = 1.5, t = 20°C, the filterbed has a coefficient of permeability k equal to $(0.55)10^{-3}$ m/s (section 2.3) and for a thickness of 0.7 m at a rate of $(0.06)10^{-3}$ m/s a resistance of 0.076 m when clean. The loss of head mentioned above should here be less than 15 mm water column, asking for wide passage ways to transport the water.

As shown in fig. 3.8 various filterbottom constructions are possible, ranging from stacked bricks to no-fines concrete poured in situ on retractable steel forms. For small filters perforated pipes may be more

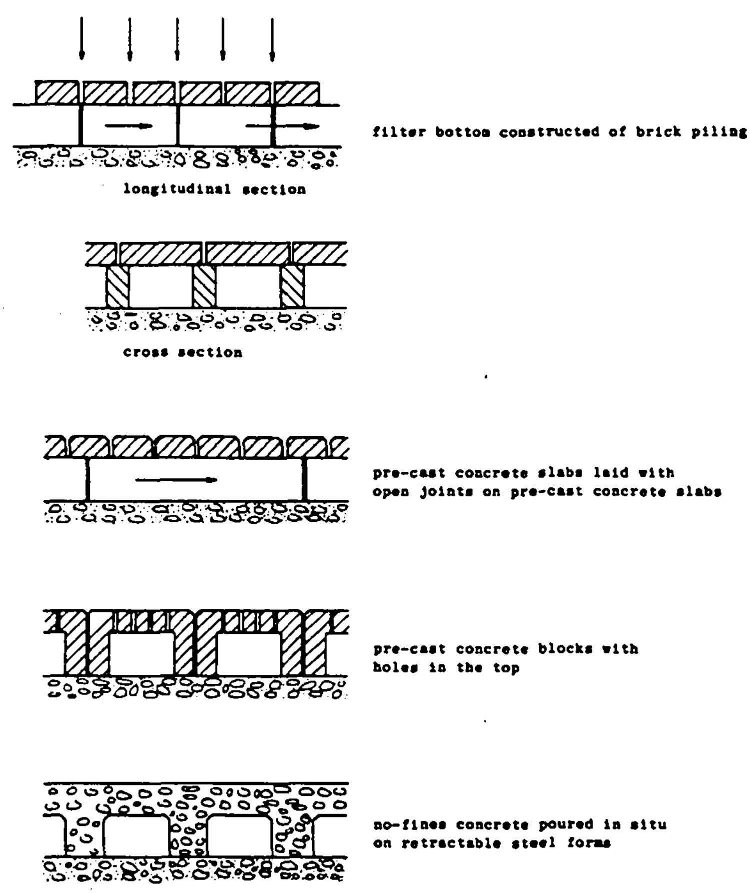

Fig. 3.8 Channelized underdrainage system for slow filters

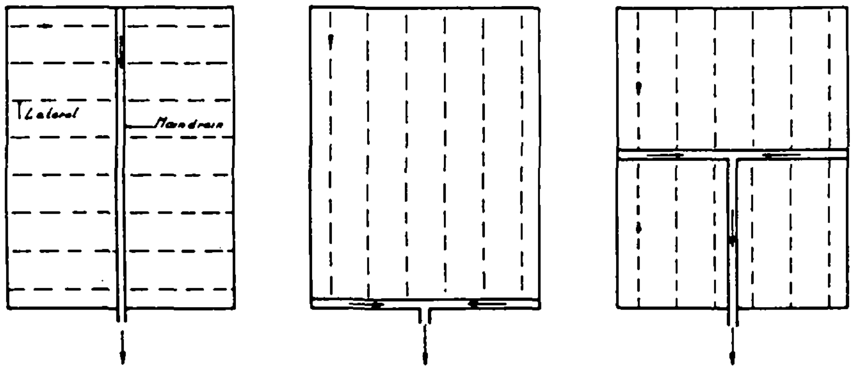

Fig. 3.9 Common arrangements for the main drain in a slow filter

attractive, connected to a main drain which leads the water out of the filter (fig. 3.9). The laterals can be made of many materials, porous concrete, vitrified clay or concrete pipes laid with open joints, but today asbestic cement and PVC are most popular, of the same type as used for the distribution system. Pipes with an inside diameter of say 0.08 m are set at intervals of about 1.5 m and are provided with holes 0.01 m diameter at the underside, 5 to 10 per running meter. The main drain is commonly not perforated and should have a cross-sectional area about equal to the combined cross-sectional areas of the laterals connected to it.

Between the underdrainage system described above and the filterbed proper, a system of gravel layers is required to prevent sand from entering the underdrains and to aid in a more uniform abstraction of the filtered water when it is recovered with a limited number of drains only. This supporting gravel system is built up of 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 more than a factor of about 1.5 apart. The gravel in the top layer should be fine enough to prevent filter sand from entering and clogging the openings between the gravel grains. Variation in grain size distribution of the filter sand should also be taken into account and therefore the lower limit of the first layer should be

smaller as 4 times the 85% diameter of the finest sand
larger as 4 times the 15% diameter of the coarsest sand

From layer to layer the gravel size should increase by a factor not exceeding 4, while with regard to clogging of the underdrainage system the lower limit of the bottom layer should be larger as 2 to 3 times the discharge openings here. As an example a filter is considered provided with sand having a

d_{15} between 0.14 and 0.18 mm
d_{85} between 0.3 and 0.4 mm

while as filterbottom the pre-cast concrete slabs of fig. 3.8 are chosen with joints 10 mm wide. The lower limit of the upper gravel layer should now be

smaller than (4)(0.3) = 1.2 mm
larger than (4)(0.18) = 0.7 mm

and the lower limit of the bottom gravel layer

larger than (2)(10) = 20 mm

These requirements may be satisfied by 4 layers with grain sizes of 1-1.5 mm, 4-6 mm, 15-25 mm and 30-50 mm (fig. 3.10). Grading gravel to limits not exceeding a ratio of 1:1.5 may be difficult and expensive and sometimes a cheaper solution may be had by increasing this ratio to 1:2. The intermediate layers should now increase in size not exceeding a factor 3, giving for the example quoted above again 4 layers with sizes of 0.8-1.5, 2.5-5, 7-15 and 20-40 mm.

Fig. 3.10 Underdrain system and supporting gravel layers in a slow filter

Gravel for slow filters should consist of hard stones, preferably rounded with a specific gravity no less than 2.5 and if necessary washed to remove all sand, clay, loam, dirt and organic impurities of any kind. Not more than 5% by weight should be lost after immersion for 24 hours in warm, concentrated hydrochloric acid. With regard to their function of separation, the thickness of the gravel layer should not be smaller as 3 times the upper limit. This would result in thickness varying of one to a few cm's only, which are not practical to apply. The minimum thickness varies commonly from 5 to 7 cm for the finer layers and from 8 to 12 cm for the coarser ones. With a larger number of layers this results in a sizable depth of gravel, increasing the depth of the filter box and its cost of construction and only slightly improving effluent quality. This is the reason that nowadays filter bottoms with fine openings are prefered. When the porous underdrains at the bottom of fig. 3.8 are made of gravel 5-10 mm, one layer of 1.2-2.4 mm is already sufficient if the effective size of the filter sand is larger than 0.3 mm. The grains of the gravel layers should be carefully packed, the larger ones around perforated drains even by hand, to prevent instability which could result in miniature landslides, disturbing the filter sand above and in severe cases even producing holes, through which the water passes with insufficient treatment.

With the large permeability of coarse grains, the resistance a gravel system offers to the downward movement of water is negligeable, not more than about 1 mm water column. The same holds true for the resistance in entering the filter bottom, next to which, however, the loss of head accompanying the horizontal movement must be considered.

3.5 Filter control

The flow of water through the filterbed of fig. 3.11 is characterized by 3 parameters: the raw water level H_1, the filtered water level H_2 and the rate of filtration v. According to Darcy's law, however, these 3 factors are interconnected by the relation

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

in which the proportionality coefficient α depends on the thickness and the grain size distribution of the filterbed and above all on the degree of clogging. This clogging increases as filtration goes on and thus the value of α will decrease with time. Still at any moment the equation above

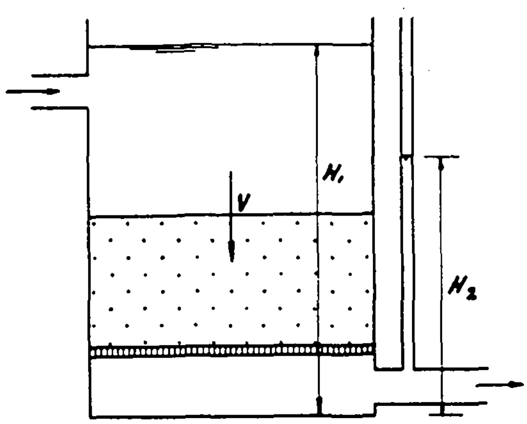

Fig. 3.11 Parameters of filtration

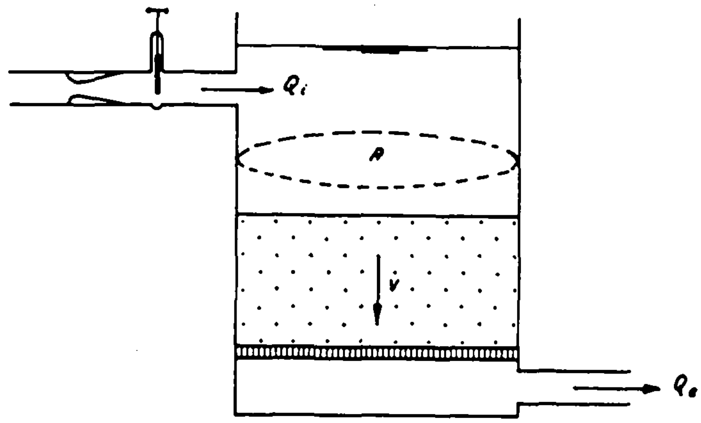

Fig. 3.12 Upstream rate control

gives a relation between the 3 parameters of flow and consequently only 2 of them may be chosen at will. In principle any combination of 2 parameters suffices, but as the amount of water produced by the filtration plant is of paramount importance, it is customary to choose as one of these parameters the rate of filtration. The other factor may be the raw water level or the filtered water level. In section 3.2 the importance of a constant raw water level has already been stressed and in slow sand filtration it is therefore the raw water level on top of the filter which next to the filtration rate is controlled. Both controls are effected by inserting an additional loss of head in the influent or in the effluent line and adjusting this loss of head in such a way as to obtain the desired results. When the upstream rate control of fig. 3.12 assures a constant raw water supply Q_i, the effluent rate Q_e is given by the relation

$$Q_e = Q_i + \beta A$$

in which A is the filter area and β the rate at which the raw water level falls in unit time. It is the outflow Q_e which determines the rate v at which the water percolates down the filter bed

$$v = \frac{Q_e}{A} = \frac{Q_i}{A} + \beta$$

The factor β in the equation above may easily obtain a value of $(0.03)10^{-3}$ m/s (0.11 m/hour). With Q_i/A equal to $(0.06)10^{-3}$ m/s, the effective filtration rates now change by 50% in one hour. In section 2.1, however, the importance of a constant filtration rate has already been emphasized and this is the reason that in slow sand filtration rate control must be effected in the effluent line. As it is impossible to put 2 controls behind each other in the same line, downstream rate control must therefore be accompanied by an upstream control of the raw water level.

Control of the raw water level on top of the filter may be accomplished automatically, for instance with a float controlled butterfly valve in the inlet line (fig. 3.13) or it may be done by hand, manipulating the raw water inlet valve or regulating the raw water supply pump. Manual control has the advantage that with a variation in raw water level of + and - 0.1 m for instance, the difference in capacity between the raw and filtered water pumps may be balanced and no separate storage reservoir is necessary for this purpose.

Control of filtration rate may again be effected automatically, using one of the many rate controllers manufactured for this purpose. With rapid filtration and a rapid increase in filter resistance, some form of automatic rate control is a necessity. With slow filters on the other hand, the increase in filter resistance and the resulting decrease in filtration rate proceeds so slowly, that manual control gives no difficulty whatsoever. In fig. 3.13 the gate valve in the effluent line is regulated by hand in such a way as to have the filter operate at the desired rate, which value can be read from the preceding venturimeter. When labour is scarce, this local control may be replaced by remote control, increasing initial costs and above all making daily supervision superfluous. This supervision will consequently not take place and defects may now occur for a long time before they are noticed. To prevent negative heads and air-binding with certainty, it is good practice to provide the effluent line with a weir (fig. 3.13), making it physically impossible for the filtered water level to drop too low.

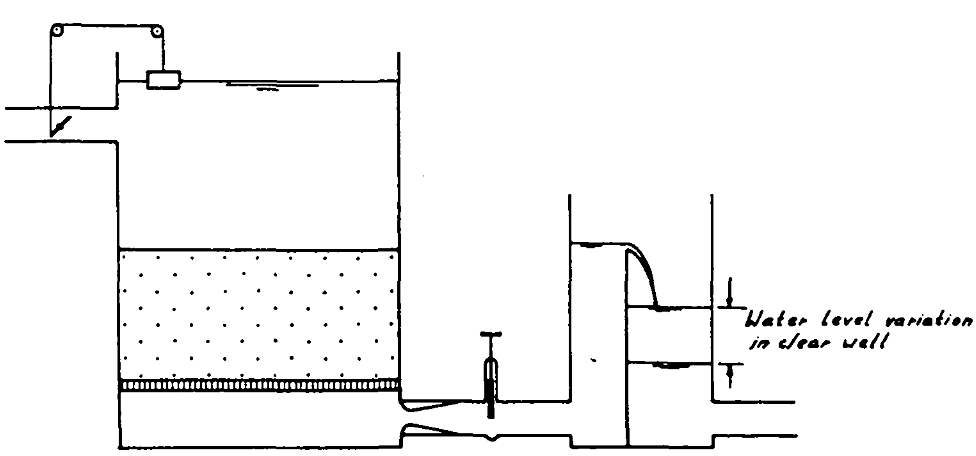

Fig. 3.13 Automatic control of raw water level, manual rate control with regulating valve

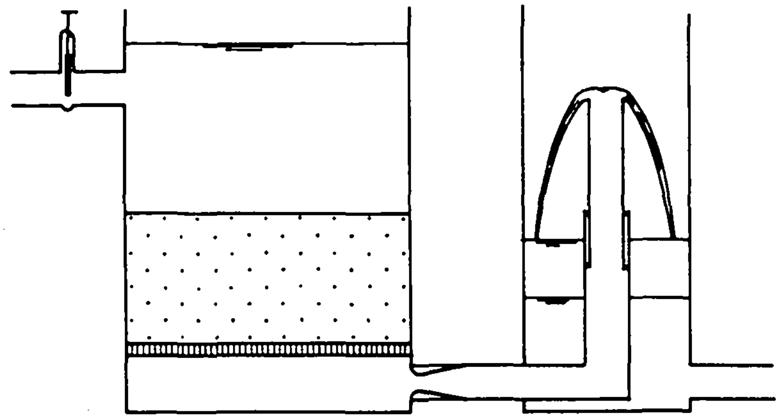

Fig. 3.14 Manual control of raw water level, manual rate control with adjustable overflow tube

In fig. 3.14, the weir and regulating valve of fig. 3.13 are combined in two concentric tubes of which the inner one can be raised and lowered at will, thus obtaining the desired rate of filtration.

When manual control of the raw water level is practised and some variation in this level is allowed, a corresponding change in filtration rate will occur. With a controlled raw water level, however, the maximum allowable filter resistance H_{max} is divided over the (laminar) resistance of the filterbed and the (turbulent) resistance of the rate control valve in the effluent line, thus

$$H_{max} = \alpha v + \beta v^2$$

A rise in raw water level by an amount Δ will now increas the combined resistance to $H_{max} + \Delta$, with constant values of α and β resulting in a higher rate of filtration. When the filter is clean, the term αv is small and may be neglected and a rise Δ in raw water level equal to 10% of H_{max} will now result in a 5% increase of filtration rate ($1.05^2 = 1.10$). Only at the end of the filtration run, when the control valve is fully opened ($\beta v^2 \approx 0$) will a 10% increase in available head be accompanied by a 10% increase in rate. Such a variation may be prevented by the construction shown in fig. 3.15. Compared with the designs of fig. 3.13 and fig. 3.14, the aeration of the filtered water is now much less vigorous.

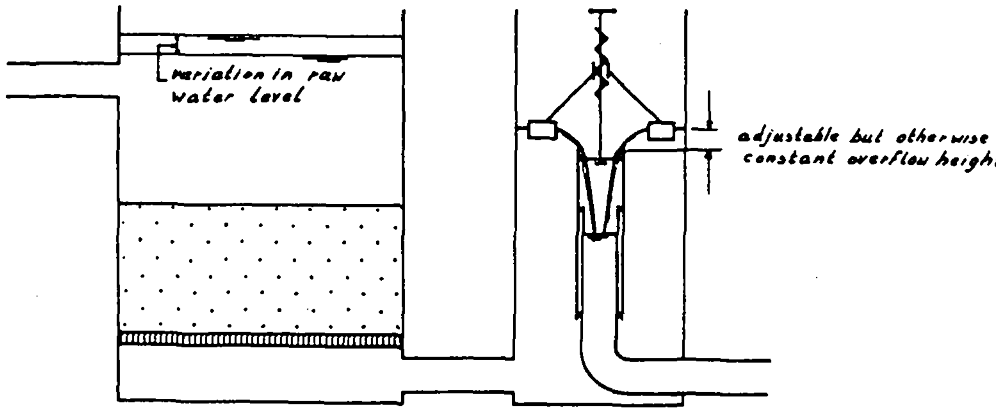

Fig. 3.15 Manual rate control, independent of variations in raw water level

The design of inlet and outlet lines together with all the appurtenances follows common rules, taking into account losses due to friction as well as turbulence and going out from the filter capacity as product of the filtering area and the maximum allowable filtration rate. Once the filter has been built, the filtering area is fixed, but the filtration rate is not, subject as it is to changes in raw water quality. By an improvement in raw water quality, either naturally or by some system of pre-treatment, this rate may even increase and it is therefore good practice to design the hydraulic structure on a capacity for instance 50% higher as anticipated in the original set-up. To increase the size of influent and effluent lines together with all the valves, controllers and so on afterwards is a very expensive undertaking.

3.6 Covering of slow filters

Covering of slow sand filters may be adopted for various reasons, as most important of which can be mentioned:

a. to prevent a deterioration of effluent quality in periods of low temperature. In this respect, covering is recommendable when the temperature of the water drops below 6°C during several months or below 2°C during one to a few months;
b. to prevent operational troubles in freezing weather as shown in fig. 2.7;
c. to prevent algae growth by excluding the sunlight, while next to this covering offers the advantage of preventing pollution by birds, wind-blown material and so on.

As already described in 2.2, prevention of algae growth may result in a slight change for the worse with respect to normal effluent quality, but it certainly makes the operation of slow filters less erratic and more stable, avoiding a serious deterioration of effluent quality during the (short) periods of a massive die-away of algae. With relative clear waters moreover, algae growth may constribute most to filter clogging and suppression of algae will now result in an appreciable increase in length of filter run, allowing on the other hand the application of much higher filtration rates. A sudden increase in filter resistance by algal blooms is neither to be feared and also in this respect operation is more regular. When exclusion of the sunlight is the task to perform, even the lightest construction will suffice. Nothwithstanding a forcible ventilation, a damp atmosphere may, however, result with condensation (of distilled and very agressive water) on the underside of the roof. To keep maintenance costs down, corrosion resistant materials must now be applied (fig. 3.16).

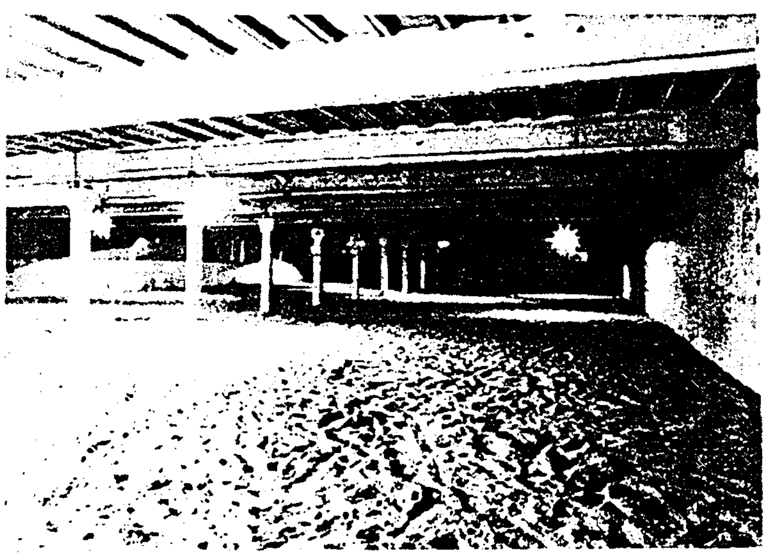

Fig. 3.16 Covering of slow filters with pre-cast and pre-stressed elements

The reasons for covering a filter as mentioned above under a. and b. are easy to understand. To prevent heat losses as much as possible, a good insulation is required. Formerly this was obtained by covering the filters with a layer of soil, certainly increasing heat capacity and thus propagating temperature changes with a large delay only. The insulation capacity, however, is limited, especially when the soil is wet and the reduction in

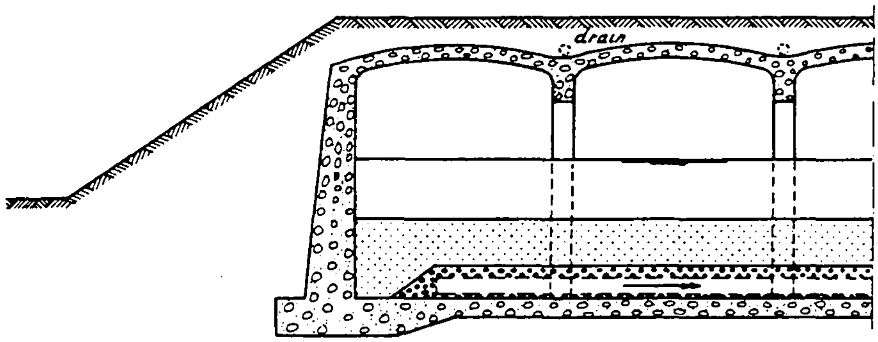

Fig. 3.17 Covering of slow filters with earth

temperature variation is consequently rather small. The weight of this soil layer moreover is tremendous, asking for a very heavy and expensive construction (fig. 3.17). Better results at a much lower price are nowadays obtained by using the modern materials for heat insulation, plastic foam for instance, with a very low weight and an excellent resistance against all types of attack. The same light-weight construction as mentioned for excluding the sunlight may now be used, provided that also the filter walls are equiped with this insulation.

When covering filters, good care must be given to provide ample access as otherwise transportation of filter sand during cleaning and re-sanding will be difficult and expensive.

3.7 Unit capacities and filter arrangements

The total area A of filter surface required is determined by the quotient between the amount Q of water to be treated and the allowable filtration rate v:

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

The values of Q and v in the equation above, however, vary during the year. In temperate climates maximum demand usually occurs in summer, but in cold climates this may happen in periods of subzero temperature when people leave their taps dripping to prevent freezing of the service pipes. The maximum allowable filtration rate also changes with time and in temperate and cold climates alike will be higher to much higher in summer as in winter. The value of A must therefore be determined as the maximum ratio between Q and v for various operating conditions.

The total area of filtering surface A is usually accommodated in a number of filters n, each with a filtering area F. To take care of unexpected situations as fighting a heavy fire for instance and especially to allow routine cleanings of filter beds also during periods of peak demand, additional units should moreover be provided. With slow filters this is an absolute necessity as over-loading is commonly not well possible. This would result in a rapid clogging and decreased rates, only to be restored by cleaning the filter, which takes at least two days. With the minimum of 1 filter as reserve, the total number is thus given by:

$$n \geqslant \frac{A}{F} + 1$$

while for larger installations at least 2 reserve units are required.

In designing a slow sand filtration plant, the relation between size and number of units must be carefully balanced, taking into account various and often contradictory considerations. As most important may be mentioned:

a. with regard to the danger of short-circuiting the filterbed along the walls of the filter box with a subsequent deterioration of effluent quality, the size of each filter should be at least 100 and preferably 200 m^2;
b. per m^2 of filtering area, larger units are lower in initial cost than smaller units;
c. to prevent cracks as a result of subsidence, temperature stresses and so on, the size of the individual filters should not be too large, not more than about 2000 m^2, depending on sub-soil and climatic conditions;
d. no plant handling water for domestic consumption should have less than 2 filters and the minimum number should preferably be increased to 4;
e. a larger number of filtering units provides a more flexible operation. With the smaller areas of the individual filters, cleaning of each filter can be done in shorter time or with a smaller labour force, reducing the cost of operation.

Only comparative designs, taking into account all the relevant local factors can give the final answer, but a first estimate of the number of units in a slow sand filtration plant may be had from the formula

$$n = 15\sqrt{Q}$$

in which Q is expressed in m^3/s.

When with open filters a rapid die-away of algae occurs in autumn, the filters must be cleaned to prevent a deterioration of filtered water quality. Enormous amounts of clogging matter have now to be removed from the filter, making access from all sides very desirable. Such filters are therefore built at some distance from each other, sometimes at random sizes and shapes, to adapt the plant as well as possible to the available site (fig. 3.1). With covered filters on the other hand, the amount of clogging matter is strongly reduced and already a few points of access to the filters suffice. With regard to the construction of the roof from pre-fabricated elements (fig. 3.16), equal sizes and rectangular shapes are now to be preferred, while sometimes they are even completely encased in a building (fig. 3.18), facilitating (mechanical) cleaning enormously. As already indicated in 1.2, operation of a slow sand filtration plant requires a large number of influent and effluent lines. With the separate units of fig. 3.2, they are best accomodated in a strip of land on both sides of which the filters are constructed.

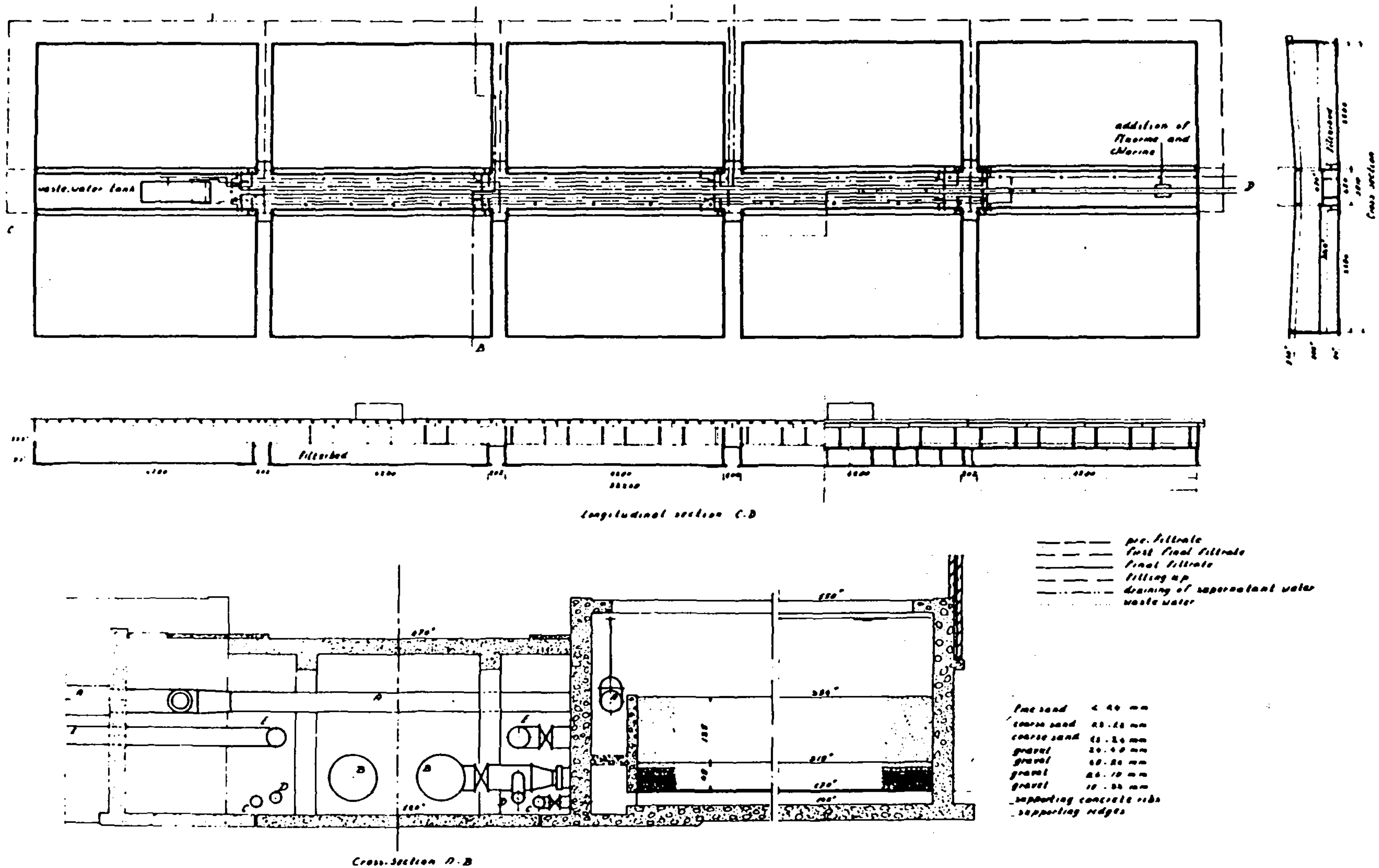

Fig. 3.18 Slow sand filtration plant for the Municipal Water Works of Amsterdam

4. OPERATION

4.1 Filtration

The operation of a slow filter in general has already been described in section 1.2, from which may be quoted that when after cleaning a filter is taken back into service, the effluent is first carried to waste, till examination has proved its quality to satisfy bacteriological requirements. This, however, is standard procedure when slow filtration is relied upon the produce a water safe as a public supply. With a larger number of filters, the somewhat lower effluent quality in physical and chemical respect of one filter during the ripening period, will not influence materially the quality of the mixed effluent of all the filters, while a safety chlorination might correct any insufficiencies with regard to the bacteriological quality. The breaking-in period may now be shortened and already after half a day the filter may be taken back into full service, before the results of a bacteriological examination are known.

As a biological treatment system, slow filters have to be handled with care. According to section 2.1, bio-chemical degradation of organic matter takes place in different steps, each at a well-defined depth below sand surface and by specific bacteria. A sudden increase in filtration rate might upset this balance and has to be avoided by the application of a large filtered water reservoir, smoothing out variations in filtered water demand. The same holds true for variations in the composition of the raw water. The bacteria in the bed of a slow filter are able to adjust themselves to suit changes in substrate, but again this asks for time. Variations in raw water quality may therefore only occur slowly, if necessary by the help of a preceding storage reservoir.

To keep well aware of the condition of a slow filter, good book-keeping is a necessity. The history of each filter should be established day for day, recording at least:

a. the date of last cleaning;
b. the date and the hour of taking back into full service (end of the ripening period);
c. raw and filtered water levels (measured each day at the same hour) and the daily loss of head;
d. the filtration rate and the hourly variations if present;

e. the quality of the raw water in physical (turbidity, colour) and bacteriological respect (total bacteria count, E.coli), determined on samples taken each day at the same hour;
f. the same quality aspects as mentioned above for the filtered water;
g. any incidents that occurred, such as plankton development, troubles with the Schmutzdecke, wind and rain, etc.

With modern equipment, continuous recording of raw and filtered water levels and filtration rate is cheaper and provides better and more complete information.

As mentioned already several times, the operation of open filters may suffer from algae growth. Algae supplied by the raw water may easily and at low cost be removed by micro-strainers, which nowadays can be made with very small openings indeed, down to 10 µm. Often, however, the real trouble originates from algae growing in the supernatant water itself and improvement is now only possible by excluding the sunlight. Most effectively this is obtained by covering the filters, but when these algae troubles occur only occasionaly, a cheaper solution may be had by increasing the turbidity of the raw water, thus reducing the penetration of sunlight. With a turbid river water and rapid filters as pre-treatment, this may easily be obtained by by-passing the rapid filters, carrying the raw river water directly to the slow filters. Some water engineers reduce occasional outburst of algae growth as mentioned above with pre-chlorination (0.2-1 mg/l) or with copper sulfate (0.15 mg/l), applied directly to the filter influent. Such a remedy in the meanwhile is not without danger as overdosing will harm the biological action of the slow filters, perhaps more seriously lowering effluent quality. Swabbing of the filter walls after cleaning the sandbed is otherwise an effective method to prevent aquatic growth here, thus preserving good appearances.

Slow filters are not fit to treat water low in dissolved oxygen content and high in potential oxygen demand as this would result in anaerobic conditions and an unacceptable quality of the effluent. When such conditions happen only occasionally and to a limited extend, however, pre-treatment to reduce the amount of oxygen consuming organic compounds and to increase the dissolved oxygen content, may now be replaced by a much cheaper recirculation of part of the effluent as shown in fig. 4.1.

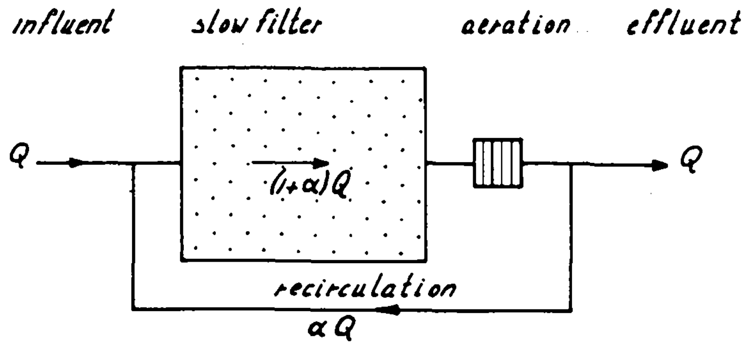

Fig. 4.1 Recirculation of effluent

The results to be obtained may best be explained by an example, assuming for instance for the raw water in an amount Q an oxygen content of 6 g/m^3 and a potential oxygen consumption of 8 g/m^3. Without recirculation, the effluent would have an oxygen content of 0 and an oxygen demand of 2 g/m^3. With recirculation to an amount αQ and supposing that aeration increases the oxygen content of the effluent from c_e to 9 g/m^3, the oxygen balance reads:

$$6Q + 9\alpha Q - 8Q = c_e(1 + \alpha)Q \qquad \text{or}$$

$$c_e = \frac{9\alpha - 2}{\alpha + 1}$$

To prevent anaerobic conditions in any part of the filterbed with certainty, c_e should not drop below 3 g/m^3. Substitution of this value gives $\alpha = 5/6$, nearly doubling the filtration rate. This will certainly increase the head loss and may thus end the filter run prematurely, but it will not increase the rate of clogging as the amount of impurities brought to the slow filter by the raw water remains practically unchanged.

4.2 Manual cleaning

Cleaning of a slow filter is necessary when the maximum allowable head loss during operation has been obtained or when by a massive die-away of algae effluent quality becomes unacceptable. The time-honored method of cleaning a slow filter is shown in fig. 4.2. It consists of skimming

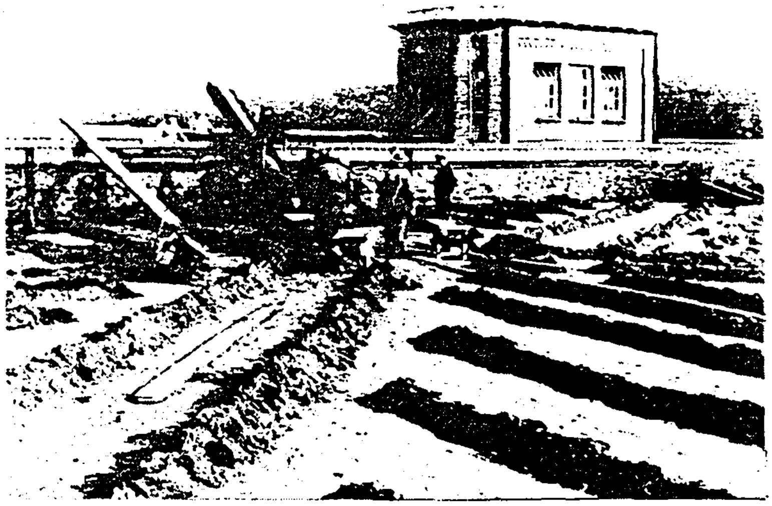

Fig. 4.2 Manual cleaning of a slow filter

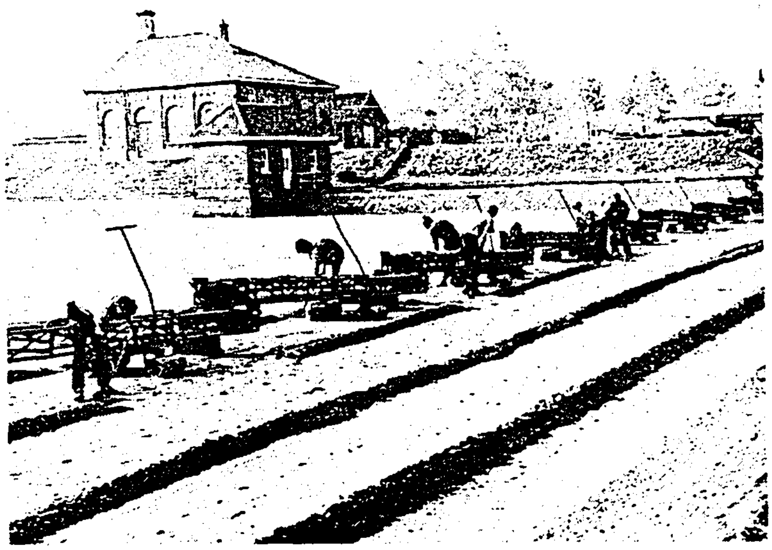

Fig. 4.3 Transportation of sand with a chain of portable conveyor-belts

the sand surface with hand shovels, removing the top layer to a depth varying from less than 1 cm to over 2 cm, depending on the amount of penetration, that is on the quality of the raw water and especially on the grain size distribution of the filterbed. The scraped-off mixture of sand and silt is piled in ridges or on heaps from where it is carried outside the filter in barrows or hand-carts wheeled over wooden planks laid on the sand. For larger installations transportation by means of a chain of portable belt conveyors as shown in fig. 4.3 will prove more economical.

When by outbursts of algae growth in spring and summer a large number of filters must be cleaned simultaneously, some plants only rake the surface of the sand bed, breaking up the Schmutzdecke and the filter skin and thus lowering the resistance against downward water movement. When slow filters are applied to produce a water safe as a public supply, this is a rather dangerous undertaking as an important barrier against the passage of pathogenic bacteria is also destroyed in this way.

Before the scraping described above and in the next section, can be carried out, draining the filterbed to a depth of about 0.2 m below sand surface is necessary (compare section 1.2). Not to loose any more time as strictly necessary, this draining is usually done overnight. A dry sand bed, however, attracts scavenging birds, who might seriously pollute the filter. Good care must therefore be taken that as soon as the sandbed is exposed, the filter cleaners arrive. Birdscaring devices usually are little efficient and for a short time only.

When after a number of scrapings the depth of the filtering material has reached the minimum allowable value (0.5 to 0.8 m, larger as the grain size distribution is coarser), re-sanding of the filter is necessary. In the course of the preceding years, however, impurities from the raw water as well as bio-chemical degradation products will have accumulated to much greater depths than the top layer mentioned above. The upper 0.3 to 0.5 m (larger as the grain size distribution is coarser) of the minimum thickness will consequently show a certain amount of fouling and an increased resistance against filtration. To prevent a persistent and inaccessible clogging to take place here, this depth of sand should therefore first be removed, to be placed on top of the new sand after this has been brought in (fig. 4.4). This is not only done to save on the amount

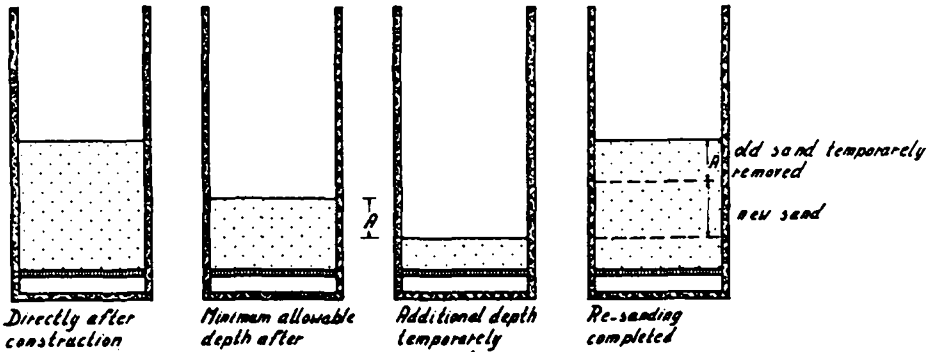

Fig. 4.4 Re-sanding of a slow filter

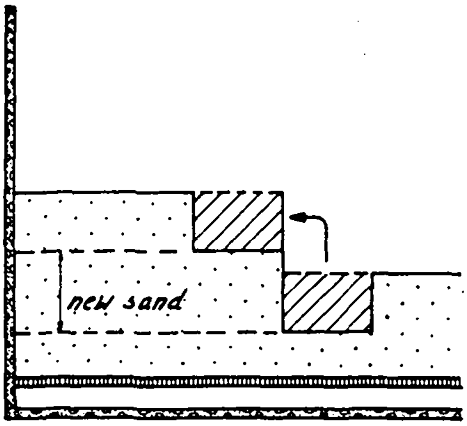

Fig. 4.5 Throwing-over of residual sand

of new sand required, but especially to ensure that the biological work of the filter will re-start with a minimum delay when it is again put into use. The "throwing over" of the residual sand in the meanwhile may also be carried out strip for strip, thus effecting some saving in the cost of transportation (fig. 4.5).

In regions where sand is expensive, the sand taken out of the filter during the various scrapings may be cleaned by washing, stored and used for re-sanding when necessary. Good care should be given that the sand is washed directly after removal from the filter as during storage of dirty sand the organic materials surrounding the grains continue to consume oxygen. This will result in anaerobic conditions, in putrescence and in the creation of taste and odour producing substances which are difficult to remove afterwards. Washing of the sand may be effected in different ways of which fig. 4.6 and 4.7 show a few examples. A completely clean sand, however, is difficult to obtain and when after drying the strongly adhering organic coating is exposed to air, oxydation sets in, making this material soluble and available as a nutrient for bacterial growth. Under favourable conditions in summer, bacteria in the sand will multiply, giving high bacterial counts when this sand is used for resanding. Preferably this re-sanding should therefore be executed in winter, although working conditions are then less favourably. Washing of filter sand has also the disadvantage of removing the finer grained particles, thus increasing the effective diameter and resulting in a deeper penetration of the impurities from the raw water.

With washing of the sand required directly after scraping, a logical idea would be to return this sand to the filter immediately. When the washer is placed in or next to the filter and the sand is deposited on a

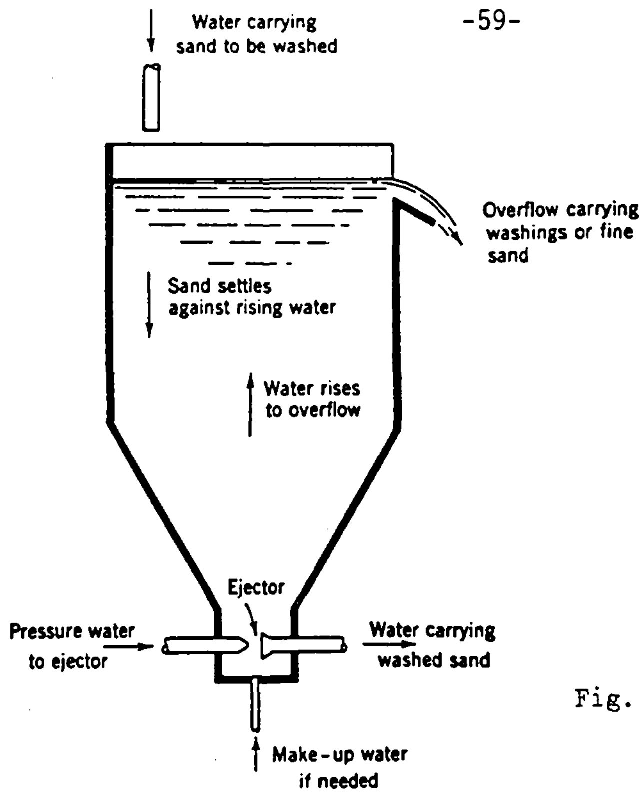

Fig. 4.6 Sand washing machine, ejector type

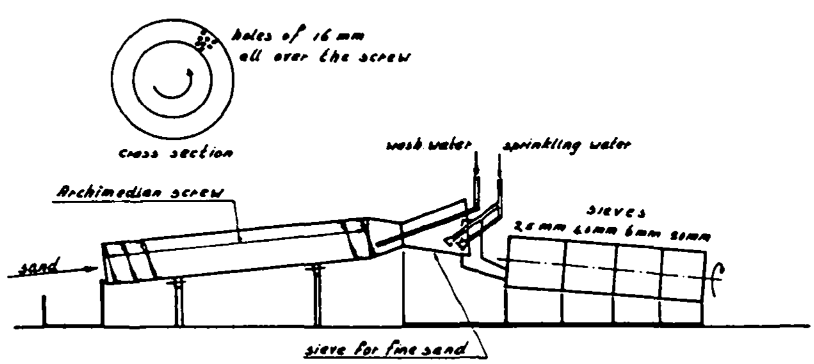

Fig. 4.7 Sand washing machine, cylinder type

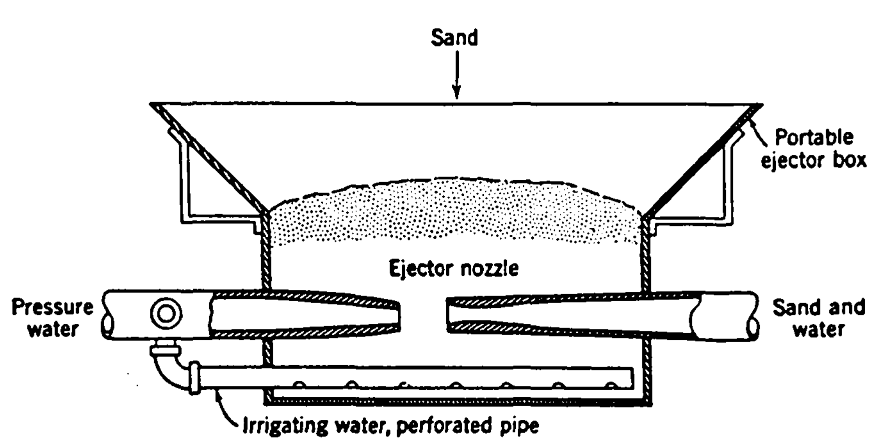

Fig. 4.8 Hydraulic sand ejector

small square or strip of the bed differing on each occasion, an appreciable saving in the cost of transportation could be obtained. In this way the depth of filtering material moreover remains more or less constant, allowing the use of shallower depths directly after construction with a subsequent saving in initial cost. In fact this method was formerly used on a large scale, but it was abandoned early in the present century for the sand reservation method described at the beginning of this section, the disadvantages being that the filterbed never had a uniform thickness, while only the top 0.05-0.1 m was ever washed in a complete cycle. Beneath this top layer the sand remained in place indefinitely, resulting in a persistent clogging.

The manual method of slow filterbed cleaning offers the advantage that no special tools neither skills are required, but it is costly in terms of time and labor involved. In regions where labor is scarce and expensive, some saving in costs could be obtained by a mechanization of the cleaning process. Already for a long time, this mechanization has been applied with regard to transportation, using a chain of portable belt conveyors to move the sand out of the filters (fig. 4.3) and dumpers or hydraulic sand ejectors (fig. 4.8) to convey it to the central washing or disposal site. Full mechanization, however, only arose during the last decades. This will be described in the subsequent sections, to be concluded with an analysis of the savings in time and labor thus to be obtained.

4.3 Mechanical cleaning

The first fully mechanized system for cleaning slow sand filters was developed by the Metropolitan Water Board in London, where filterbed areas are very large indeed, up to 4000 m^2, making scraping by hand a laborious and time consuming operation. In the system worked out there, the various phases of slow filterbed cleaning are mechanized separately, using modified light agricultural track-laying tractors, moving over the filterbed itself. Special constructions of the filterbox are thus avoided and this system may therefore be applied as well with new filters as with existing ones. To prevent compaction of the filterbed and subsequent higher filter resistance (measured each time directly after cleaning), the soil pressure must be low. By the application of tracked vehicles. the pressure is less than 3 N/cm^2 and difficulties will not arise.

The most important feature of the London system is the use of skimming machines which scrape off the required amount of sand in strips 1.5 m wide (fig. 4.9). The depth of skim can be pre-set between 1 and 3 cm (fig. 4.10), while 2 skimming speeds are available, 7.5 and 15 m per minute (0.45 and 0.9 km/hour), the higher speed only to be used when no more than a thin layer of sand has to be removed and the sand is reasonably dry. The skimming itself is carried out by means of blades from which the sand is transported by a screw conveyer and a belt-loader, discharging backward into another tractor fitted with a dumper body (fig. 4.11).

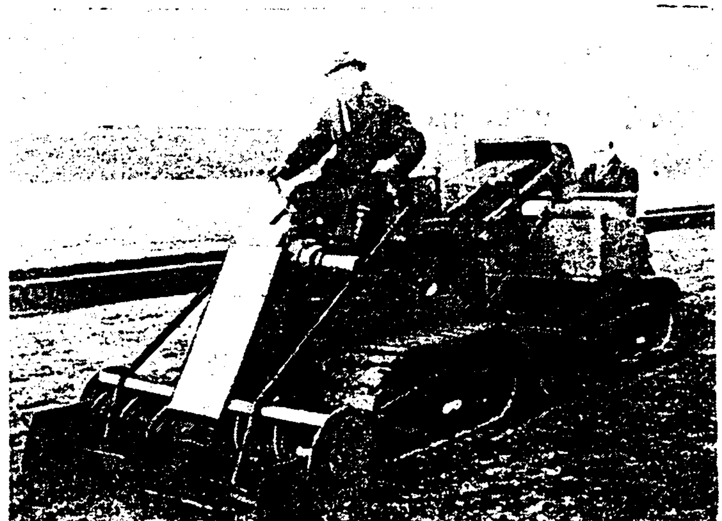

Fig. 4.9 Skimming tractor with following dumper

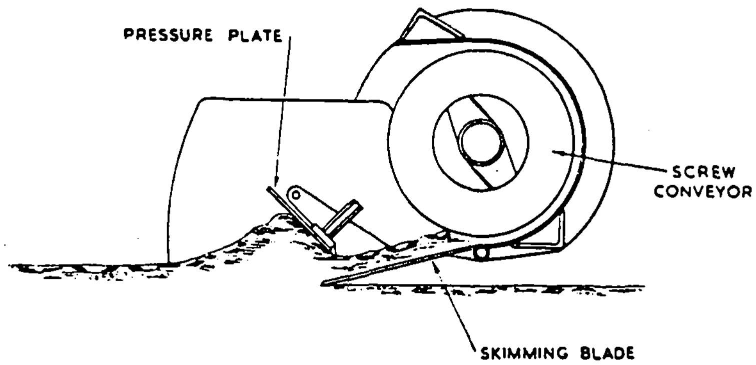

Fig. 4.10 Action of pressure plate in controlling depth of skim

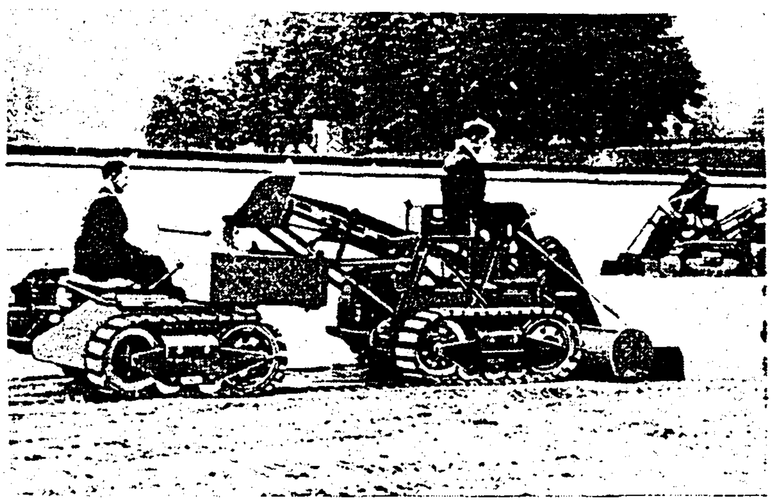

Fig. 4.11 Skimming tractor with dumper tractor behind

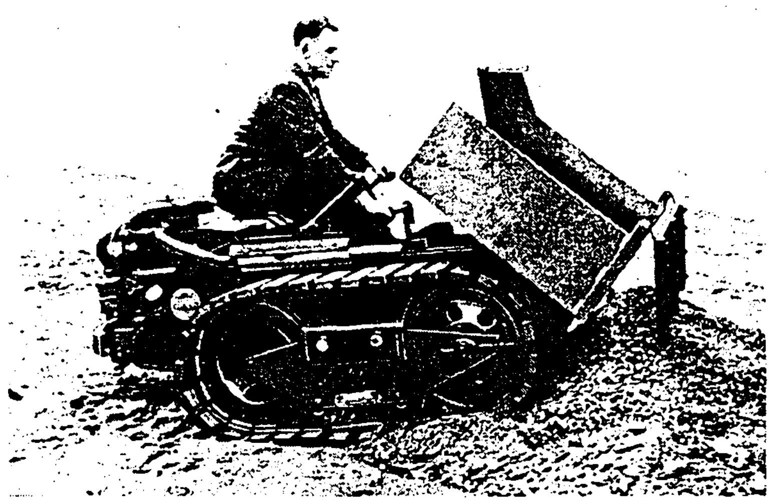

Fig. 4.12 Dumper tractor, discharging its load of dirty sand

Two dumpers serve one tractor and alternately discharge their loads at the filter side (fig. 4.12), where the sand is lifted out by a mobile crane and brought to the central washing plant in wheeled dumpers. To obtain a smooth finish of the cleaned filterbed surface, special rake attachments to the tracked dumpers may be applied (fig. 4.13), while for temporary removal of an additional depth of sand prior to resanding trenching machines may finally be utilized (fig. 4.14).

With the London system of mechanical filterbed cleaning, an appreciable saving in time is indeed obtained, while labor requirements are about cut in half and moreover the unskilled labor of the manual cleaning is transformed in semi-skilled labor, improving working conditions. Taking into account the capital investment for this system, the saving in cost is not impressive while the presence of motorized vehicles on the bed of a slow filter always brings with it the danger of pollution by oil. During the last years, demand for light-weight agricultural tractors has decreased so much, that the firm which provided the Metropolitan Water Board with the equipment described above, has stopped this production!

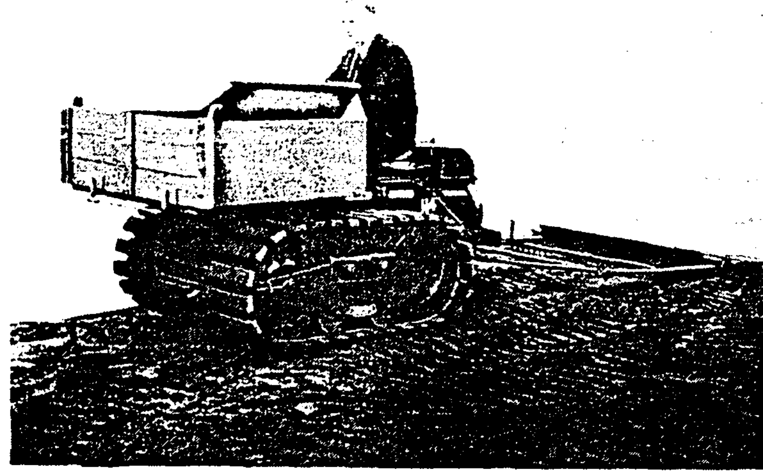

Fig. 4.13 Special rake attachment to obtain a smooth finish of the cleaned filter bed surface

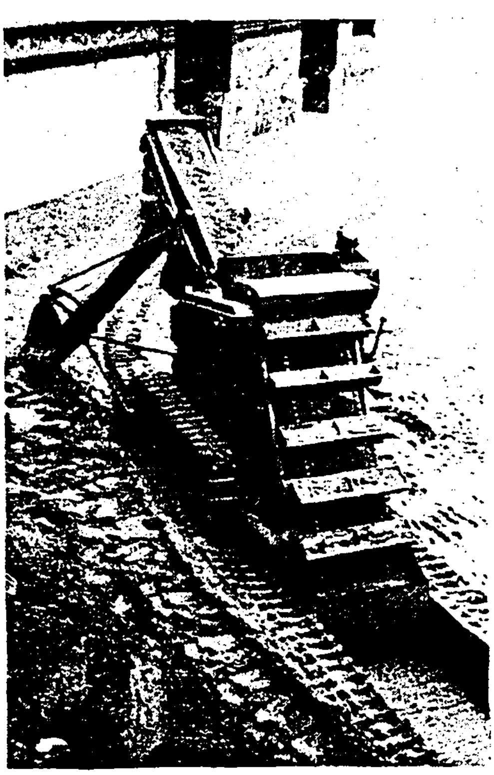

Fig. 4.14 Trenching machine for re-sanding operations

With regard to dwindling ground-water supplies, the Berlin Waterworks started artificial recharge. The spreading basins are covered with 0.4 m of coarse sand (0.5-2 mm), while the raw surface water percolates downward at a rate of $(0.01)10^{-3}$ m/s. Notwithstanding the use of coarse sand and the applications of low filtration rates, clogging proceeds rapidly and cleaning is necessary every 3 weeks. This cleaning is effected by scraping off the top layer to a depth of about 1.5 cm with the help of mechanical equipment, which without any change could be used for slow filters as well. This Berlin system of mechanical slow sand filterbed cleaning operates along the same general lines as the London system described above. Again the skimmer moves on tracks (soil pressure < 3 N/cm^2) at a velocity of 0.44 or 0.73 km/hour (for transportation 2.94 km/hour), but the 1.2 m wide blade has now an additional support by 6 pneumatic tires, preserving a constant depth of skim (up to 5 cm) even with a wavy surface (fig. 4.15). In stead of small tracked dumpers, large wheeled dumpers are used, supplied with oversized low pressure pneumatic tires (fig. 4.16). Their speed in bringing the dirty sand to the central washing or disposal site is so large, that now one dumper for each skimmer suffices and this dumper may be applied for many other duties as well. The most important difference in the meanwhile is the circumstances that equipment for the Berlin system is still available.

The new slow sand filters for the dune water treatment plant of Amsterdam are constructed inside a building (fig. 3.18), at both sides of a two-level corridor. The filters have dimensions of 25 x 40 m^2 and the width is spanned by a travelling bridge (speed 0.2 m/s) from which the scraper mechanism is suspended (fig. 4.17). This assures a sand surface which remains completely flat. Together with the slight depth to which impurities penetrate the fine filter sand ($d_{10} = 0.12$ mm, $d_{60} = 0.2$ mm), this allows a small depth of skim with a maximum value of 1 cm. The blade has a width of 2.5 m and moves at a speed of 0.15 m/s in forward and of 0.30 m/s in backward direction. A screw of 0.6 m diameter with 32 revolutions per minute conveys the sand to a 0.3 m wide bucket elevator, which transports the material with a speed of 1 m/s to a bunker, again suspended from the bridge. The bunker has a capacity of 1 m^3, sufficient for scraping 2 strips of 2.5 x 25 m to a depth of 0.8 cm. When full, the bunker is emplied into a steel tank, placed alongside the filter in the corridor, which in its turn is transported by a fork lift truck to the disposal site at a short distance outside the filter building. With this system labor requirements are very small, only 2 men who can clean a filter of 1000 m^2 in less than 1.5 hour.

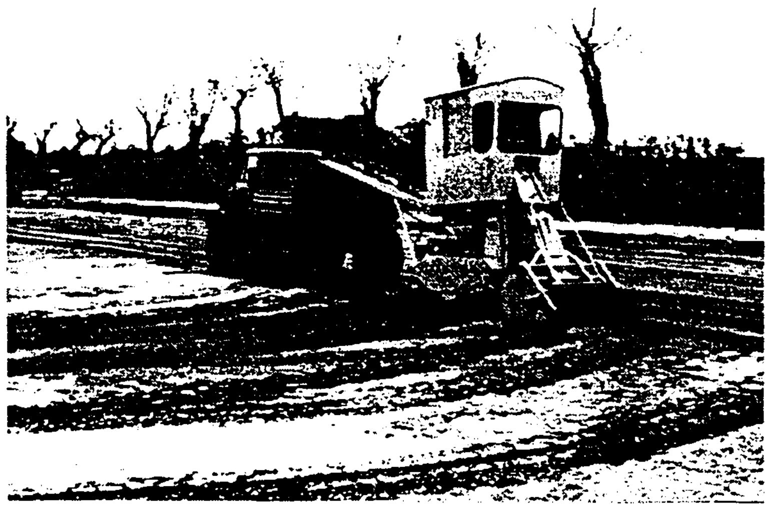

Fig. 4.15 Mechanical cleaning at the Berlin Water Works

Fig. 4.16 Wheeled dumper with oversized low pressure pneumatic tires

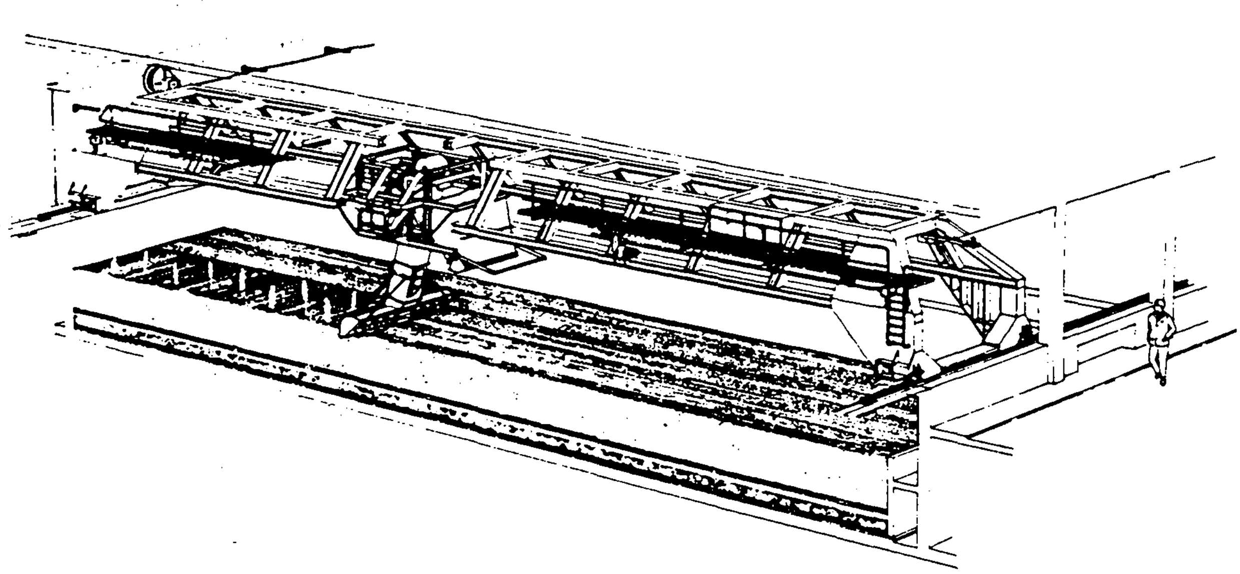

Fig. 4.17 Mechanical cleaning at Amsterdam Water Works

4.4 Hydraulic cleaning

Rapid filters are customarily cleaned by back-washing, but with slow filters and large filterbed areas, this would ask for immense quantities of water which cannot be supplied economically. Washwater requirements, however, could be reduced by a subdivision of the filterbed, washing it section for section in succession. This idea was first realised in 1933 by Sivade for the water supply of Paris by the Compagnie Générale des Eaux. With slight modifications the same system has been used in London since 1958 in Antwerp since 1967 and in various other places as for instance Istanbul.

The Sivade system of back-washing slow sand filters in situ essentially consists of a serie of long, narrow, two-storied, bottomless boxes, 0.3 m wide and having a combined length equal to the width of the filter. The boxes are carried by a gantry spanning the bed (fig. 4.18) and the cleaning operation starts by lowering the boxes (fig. 4.19), allowing their edges to penetrate the sand by about 5 cm, thus isolating part of the filterbed. A system of injection lances at 0.3 m centers, passing through the boxes in guide tubes, is subsequently lowered 15 to 30 cm into the filterbed (fig. 4.20), to a depth well below the deepest penetration of the impurities from the raw water. At the top the hollow lances are connected to a header, receiving pressure water from a hydrant with a flexible hose. At the lower end the lances are equiped with heads having radially disposed orifices through which the wash water is admitted to the filterbed. To assure an equal supply of wash water to each lance, the resistance of the orifices mentioned above, must be large compared with the losses due to friction and turbulence in the supply system. This asks for fairly high pressures, usually between 10 and 30 m water column. When back-washing slow filters, real scouring is not required and a small amount of sand bed expansion will suffice, asking for (average) rates of $(3\text{-}10)10^{-3}$ m/s, depending on the grain size distribution of the filtering material. Even with large filters, the width will seldom surpass 50 m and the area enclosed by the caissons is consequently small, no more than 15 m^2 for which low amounts of back-wash water are already sufficient. After passing the top layer of the filterbed, the wash-water together with impurities floated to above accumulate in the lower story of the serie of boxes. Through apertures in the ceiling it flows next to the upper story from which it is removed by a suction pump and carried to a drain running alongside the filter. To assure an even ab-

Fig. 4.18 In situ washing plant at Oelegem, Antwerp Water Works

Fig. 4.19 In situ washing machinery at Oelegem, Antwerp Water Works

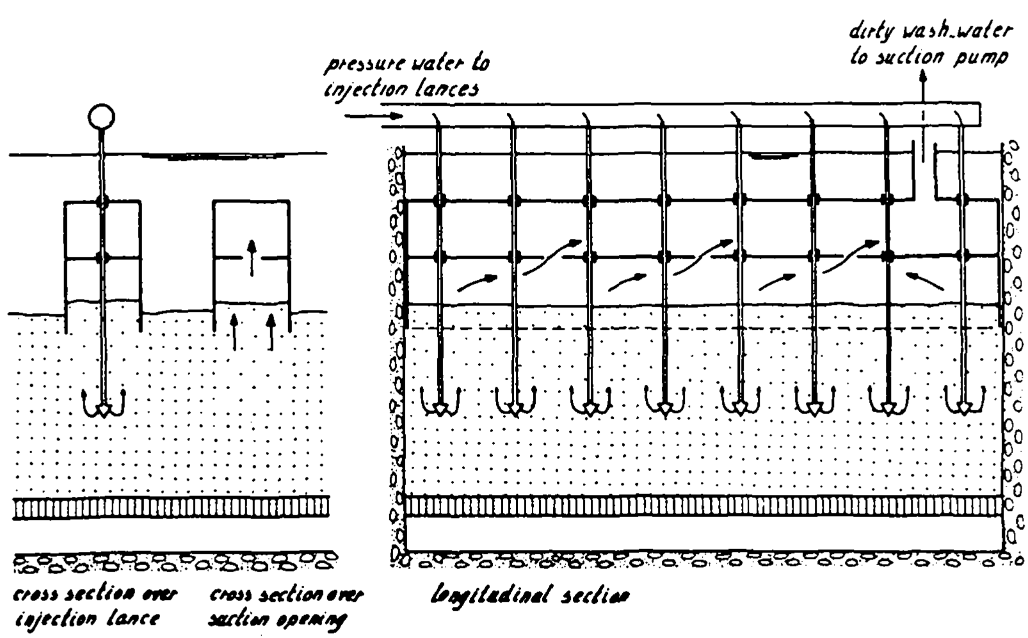

Fig. 4.20 Selecteur sivade

straction of the wash-water over the full width of the filter, the apertures mentioned above are adjustable, allowing uniform suction conditions to be obtained. The capacity of the suction pump moreover is equal (or slightly larger) than the rate at which the pressure water is supplied. The back-washing process lasts for about 1 minute, after which the lances are withdrawn into the lower chamber of the boxes and both lances and boxes are hoisted together clear of the sand. The supporting bridge is then moved over a distance of 0.3 m along the length of the bed, the caisson and the lances are lowered and the washing process is repeated (fig. 4.21). All these operations are controlled electrically and sequenced in such a way that the plant works automatically. When a tough layer of filamentous sessile algae have accumulated on top of the filterbed, the Schmutzdecke has to be removed prior to back-washing. This can be done mechanically by hydraulically operated rakes, fastened at the side of the supporting bridge remote from the caissons (fig. 4.22).

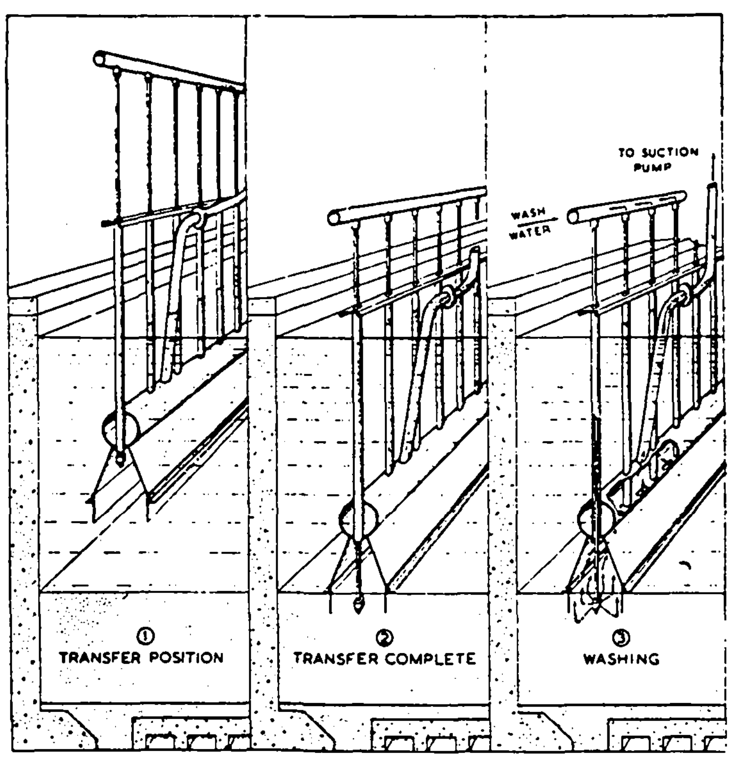

Fig. 4.21 Mode of operation for in situ sand washing

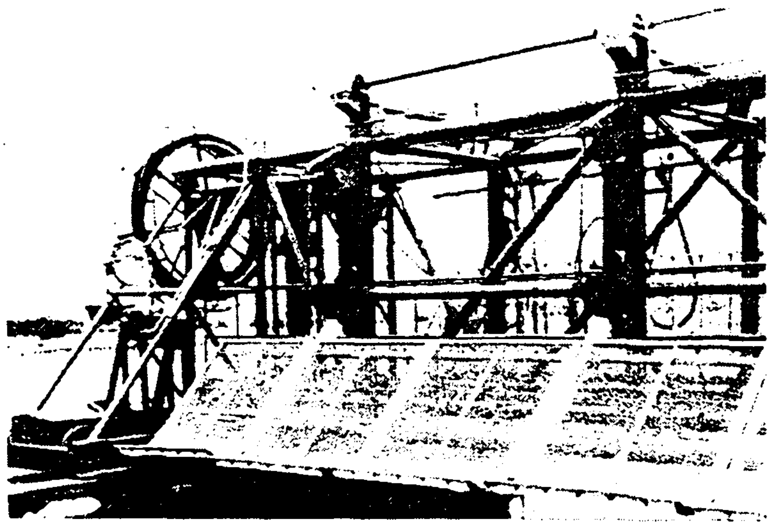

Fig. 4.22 Rakes to remove aquatic weeds prior to washing (Ashford Commons, Metropolitan Water Board)

Hydraulic cleaning of slow sand filters offers the advantage of an enormous saving in time and labor. Especially because no time is lost in either emptying the filter or recharging it with water, even the largest unit may be cleaned by 1 man in an 9 hour day, while for smaller filters an afternoon will suffice. With no reduction in sand bed thickness by successive scrapings, a smaller initial depth will suffice. Assuming that for the undisturbed sand below the lances a thickness of 50 to 60 cm is necessary (according to 3.3 larger as the grain size distribution is coarser) and the maximum penetration of these lances amounts to 20 cm, the total sand bed thickness may be reduced to 70 or 80 cm only. As disadvantages of hydraulic back-washing must be mentioned in the first place that capital investment is high. Because the installation is suspended from a travelling bridge, it may only be applied with new plants where the filter can be built so narrow that spanning its width is economically feasible. The construction of the filter walls moreover should allow the placement of rails over which the bridge can move along the length of the filter. Back-washing will also result in an hydraulic classification of the filtering material, bringing the finest grains to above, promoting filter clogging and resulting in shortened filter runs. Properly speaking graded sand should be used, but with regard to the huge amounts required, the cost of such material is prohibitive. Since the filter is not emptied during cleaning, the environment never becomes unfavourable to algal growth. On contrary, the algae remain in an active state of division and as they are only partly removed, increased clogging and reduced lengths of filter run will again be the result. The most serious draw-back of hydraulic cleaning, however, is the circumstance that by back-washing impurities from the surface may be carried to greater depths. For a large part this is due to an uneven distribution of the wash water with only one point of application per area of 0.3 x 0.3 m^2. When the filter is now taken back into service, these contaminations escape the purifying powers of the top layer of the filter bed, resulting in a deterioration of effluent quality. This will be especially serious when occasionally the lances are lowered to the bottom of the filterbed, so as to remove minute amounts of impurities penetrated to greater depth. To obtain a water free from bacteriological contamination, ripening periods of long duration are thus necessary. Technically this is often not feasible and a post-chlorination is now applied to compensate the defects in the slow sand filtration process. An important barrier in assuring a safe and potable water is thus lost.

Properly speaking, the local application of wash-water, without the equalizing effect of supporting gravel layers as with rapid filters, must result in an uneven distribution of wash-water. This will already be evident when looking at the cleaned filterbed surface, where the position each lance has occupied is indicated by a crater. Due to return flows, dislodged organic impurities accumulate below these craters and often their oxygen demand is so large that local anaerobic conditions result, as shown by black spots from iron sulfides (fig. 4.23)

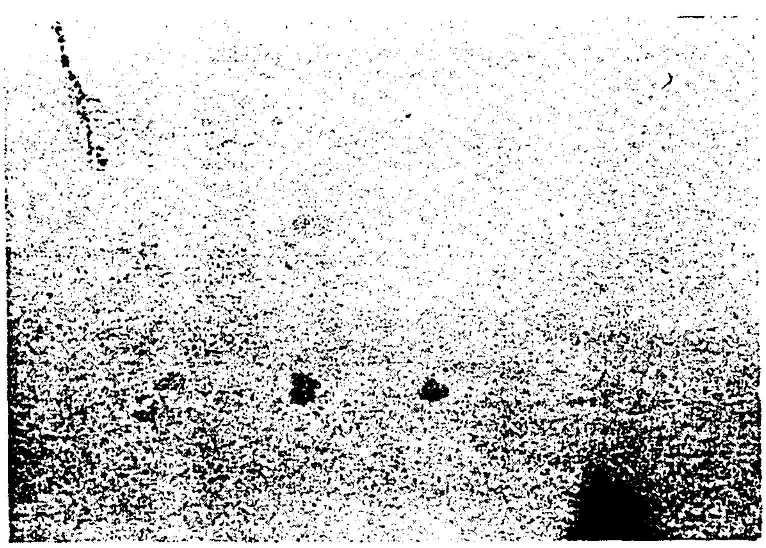

Fig. 4.23 Black spots on filterbed surface

4.5 Cost of cleaning in terms of time and labor

From the fore-going description it will be clear, that the initial cost of the equipment for manual cleaning is small to negligeable, for the mechanical cleaning as developed in London and Berlin fairly high, while it is very high for the systems suspended from a travelling bridge as is the case with the mechanical system in Amsterdam and the hydraulic systems applied in Paris, London and Antwerp. From a purely economical point of view, these expenses are only warranted when a equal or larger reduction in labor cost can be obtained. Next to this some importance must be given to a saving in time, while an improvement in working conditions may also be a factor to take into account.

Going out from a filter of 2000 m^2 bed area, and neglecting the time required for draining the filter overnight, the cost of cleaning in terms of time and labor involved is tabulated below for the various systems. For

cleaning method	manual	mechanical			hydraulic
		London	Berlin	Amsterdam	
time in hours required for:					
draining	2	2	2	2	0
cleaning	9	4	5	3	6
back-filling	5	5	5	5	0
ripening	24	24	24	4	4
total	40	35	36	14	10
number of men required	8	4	2	2	1
number of man hours	75	20	15	10	10

smaller filters the saving compared with manual cleaning will be somewhat less, for larger filters on the other hand a little higher. As a result of the additional operations to be carried out, a saving in time worth mentioning is only obtained with the systems suspended from a travelling bridge. This is mainly due to the circumstance that no contamination of the sandbed by human contact is possible, allowing a much shorter ripening period, while with hydraulic cleaning also no time is lost in draining and back-filling the filter. Assuming an average of 5 filter cleanings per year, the saving in labor compared with manual cleaning amounts to roughly 300 man hours per filter a year. With a plant consisting of 10 filters for a capacity of 50 million m^3/year, the saving in money will be about ƒ 60000 /year, corresponding with a capital outlay of ƒ 450000 (interest at 10%, writing-off period 15 years). This will certainly be sufficient for the mechanical systems applied in London and Berlin, but it is by far inadequate when a travelling bridge is necessary. The calculations given above in the meanwhile, only hold true as long as work of full economic value can be found for the laborers when they are not engaged with manual filter cleaning. The 8 people required for this job work altogether 14000 hours/ year of which in the example given above only 3000 hours/year are spent in cleaning. When such work is not available, the saving in labor cost by mechanization rises to about ƒ 280.000/year, equal to a capital outlay of ƒ 2.100.000. Even for the mechanical system used in Amsterdam as well as for the various hydraulic systems, this amount will be sufficient.

5. ARTIFICIAL RECHARGE AS A SLOW SAND FILTRATION PROCESS

5.1 Introduction

When in the middle of last century the importance of a safe drinking water in the prevention of contagious diseases such as typhus and cholera became evident, one city after another constructed a piped supply. With desinfection by chemical agents still unknown, such a reliable water free of pathogenic bacteria, could at that time only be obtained in two ways:
a. by slow sand filtration of surface water from (clean) streams and lakes;
b. by the use of groundwater which is safe by virtue of its origin.
Slow sand filtration has been described in detail in the preceding chapters from which it will be clear that in principle it is an imitation of natural self-purification. The recharge of groundwater by downward percolating rain water and by a natural infiltration from streams and lakes, is subject to the same purifying powers, even to a greater extent as now infiltration rates are so much smaller and detention times so much larger. Indeed, epidemiological evidence over centuries has proved that groundwater is the most reliable source for human consumption and this is the reason that the waterworks mentioned above were based as much as possible on groundwater abstraction.

Most groundwater sources, however, are rather limited in their yield, while in subsequent years water consumption rose continuously. About the turn of the century, many cities faced a shortage of drinking water and additional supplies had to be found. Past experience with groundwater in general was excellent, but with surface water supplies and a growing pollution of the raw water a number of difficulties had been encountered. This additional load of pollutants and the subsequent difficulties could not be overcome by the use of a better purification system, as this was not yet available. As a result, an outspoken preference for groundwater was brought about, which led to the application of artificial recharge such as first practised by Richert in 1897 for the water supply of Gothenburg in Sweden. By augmenting the amount of water entering the aquifer with artificial means, a corresponding increase in the amount of groundwater abstracted was possible, while with a judicious design of recharge and recovery works the bacteriological quality of the water abstracted remained impeccable.

5.2 Methods of artificial replenishment

The amount of (surface) water entering an aquifer may be increased in two different ways

a. directly by spreading surface water over pervious soils in basins, ponds or ditches, by flooding less-pervious areas or by injecting the water with the help of wells, shafts or pits;
b. indirectly by locating the means for groundwater abstraction close to surface streams with pervious banks. The lowering of the groundwater table accompanying abstraction will then result in an increased inflow of water from these streams.

The direct methods of artificial replenishment are commonly called artificial recharge and the indirect methods induced recharge.

Unintentional induced recharge occurs where a catchment is crossed or bounded by a stream with pervious banks and groundwater abstraction has lowered the groundwater table below the water level in the stream. For intentional induced recharge, however, the collectors are purposely set near and about parallel to such a stream to augment the capacity of the catchment with the amount of infiltrating surface water (fig. 5.1). The distance between the collectors and the bank of the stream deserves carefull consideration. On one hand this distance must be so small that the lowering of the groundwater table more inland does not harm other interests such as agriculture for instance, while on the other hand this distance must be so large that during the underground travel the surface water from the stream is converted into a reliable groundwater, safe as a public supply. The last mentioned consideration asks for a minium length of flow, a minimum flowing-through period and above all for the prevention of short-circuiting.

In artificial recharge, water from another source, usually surface water from a river perhaps a considerable distance away, is conveyed to points from which it percolates into a body of groundwater (fig. 5.2). Not only the means for groundwater recovery, but also those for supplying the aquifer with surface water, may now be constructed in various ways, depending on sub-soil conditions. When the aquifer is homogeneous up to the surface, recharge is commonly effected with the help of spreading basins, constructed either as ditches parallel to the collectors in case the coefficient of transmissibility of the aquifer is low (fig. 5.3) or as ponds at greater distances from the collectors when this transmissibility

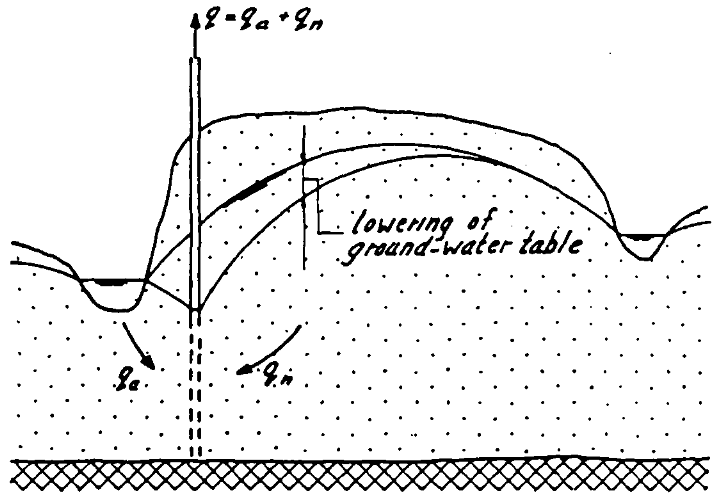

Fig. 5.1 Induced recharge with a line of wells parallel to a stream

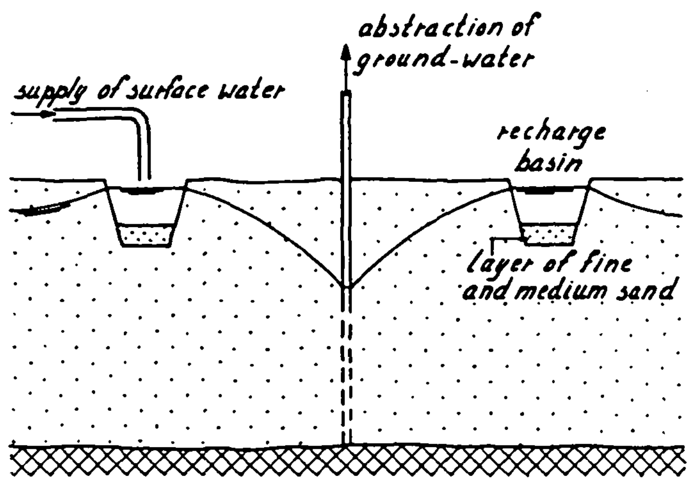

Fig. 5.2 Artificial recharge

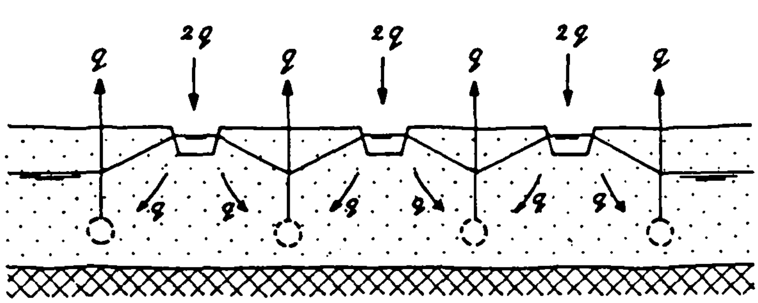

Fig. 5.3 Artificial recharge with ditches

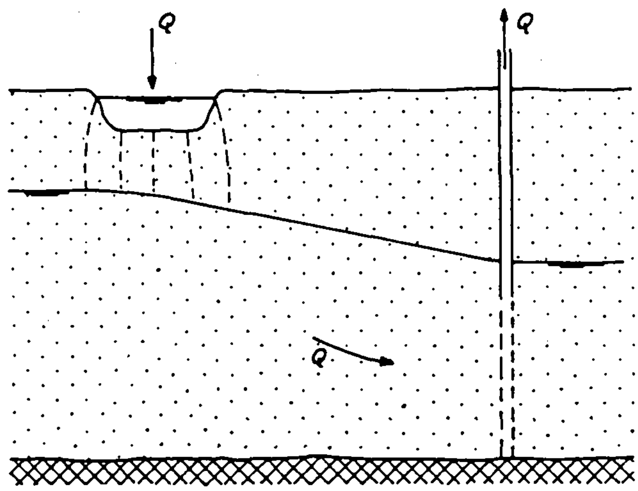

Fig. 5.4 Artificial recharge with ponds

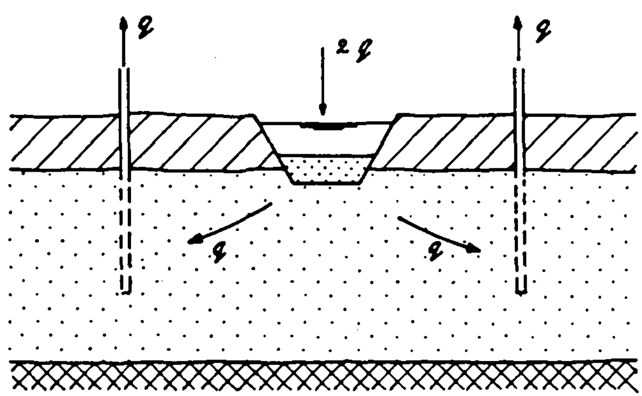

Fig. 5.5 Artificial recharge with spreading basins

is high (fig. 5.4). The same spreading basins may be used with confined aquifer if only the thickness of the overlying less-pervious layer is small and full penetration economically feasible (fig. 5.5). In the opposite case recharge must be carried out with wells, shafts or pits (fig. 5.6). Whatever means for introducing the surface water are applied, again the distance between recharge works and collectors, must be chosen large enough to guarantee under all conditions a satisfactory quality of the artificial groundwater recovered. With a judicious design of the recharge schema, however, any influence on the groundwater table outside the catchment area can be prevented and adverse effects do not need to be feared. Also because a direct link with the source of supply is absent, a wider choice of recharge sites is now available.

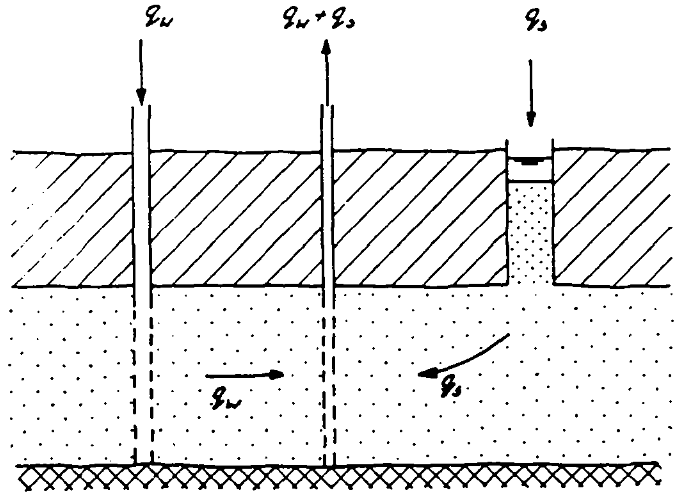

Fig. 5.6 Artificial recharge with wells and shafts

From the very beginning, induced recharge suffered from a decline in capacity when by deposition of suspended matter a clogging of banks and bottom of the influent stream occurs. Under natural conditions, some of these cloggings are periodically scoured away during flood flow, but this ceases altogether when dams are constructed to regulate the river flow, storing peak discharges in impounding reservoirs. In case the river water is polluted by discharges of domestic or industrial wastes, induced recharge becomes even less attractive as together with the water these impurities are drawn into the aquifer. With artificial recharge such a contamination may be prevented by pre-treating the water, removing for instance oxygen consuming organic compounds by aeration and rapid filtration prior to recharge and thus maintaining aerobic conditions over the full length and width of the aquifer, up to the recovery works. When the river is charged with industrial wastes, the possibility of a poisonous effluent can never be excluded and properly speaking induced recharge is no longer acceptable. With regard to the continuity in supplying drinking water to the consumers, the abstraction of artificial groundwater cannot be stopped with as result that poisonous river water is drawn into the aquifer, subsequently endangering the quality of the water collected. Induced recharge is only feasible when the stream carries a good quality water under all conditions. In densely populated and highly industrialized regions, this is no longer the case and here induced recharge must be replaced by artificial recharge, for instance as shown in fig. 5.7.

5.3 Slow sand filtration in artificial recharge

The bottoms of the spreading basins for artificial recharge, are usually covered with a layer of sand, 0.5 m deep and with a grain size distribution equal to or slightly coarser as that applied in slow sand filtration. In fine grained aquifers and an extremely small penetration of impurities from the raw water, this sand cover is necessary to prolong filter runs, while in coarse grained aquifers it is required to prevent a deep penetration of impurities, which would be difficult and expensive to remove. With this sand cover, spreading basins are essentially slow sand filters, even operating at a higher efficiency as filtration rates are lower, $(3\text{-}10)10^{-6}$ m/s, and the water is forced to travel through the underground over much larger distances (commonly more than 50 m) with extended detention times of a few weeks to a few months instead of a few hours. Of course a good quality effluent is only possible when during the underground travel no pollution occurs. With unconfined aquifers pervious to ground level this is difficult to prevent, but it is very simple with confined aquifers protected at the top by an impervious layer of clay or loam (fig. 5.5 and 5.7). This profitable situation is found in the Ruhr district of Germany (fig. 5.8), where formerly induced recharge was practised with the help of galleries at a distance of 50 m parallel to the bank of the stream. By a gradual clogging of the river bottom following the construction of impounding reservoirs in the tributaries, the capacity of this induced recharge is now negligeable and replenishment is carried out by twin spreading basins, constructed at a distance of 50 m on the other side of the gallery. These basins, however, are provided with a sand cover, preventing impurities from the raw water to enter the water bearing gravel layer. At the beginning of the filtration run, the water level in the basins is some distance below the water level in the river, which is kept at a more or less constant height with the help of weirs. By clogging, the resistance against downward water movement increases and with the same infiltration rate, the water level in the basins goes up till finally no more river water flows in by gravity. The influent pipe is now closed and the water level falls to below the top of the sand bed, which subsequently is cleaned by scraping in exactly the same way as with ordinary slow filters (compare section 4.2 and 4.3). The other half of the twin basins remains in service, providing the required rate of recharge and thus assuring the continuity of the supply.

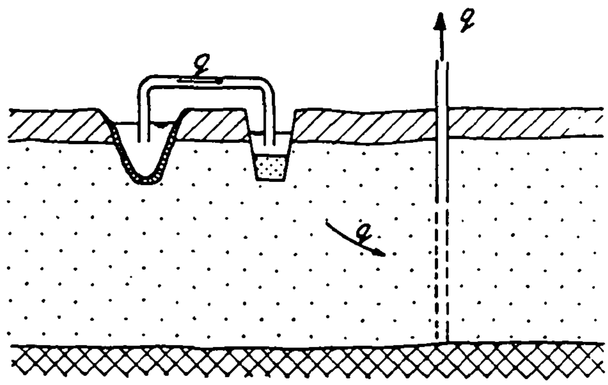

Fig. 5.7 Artificial recharge replacing induced recharge

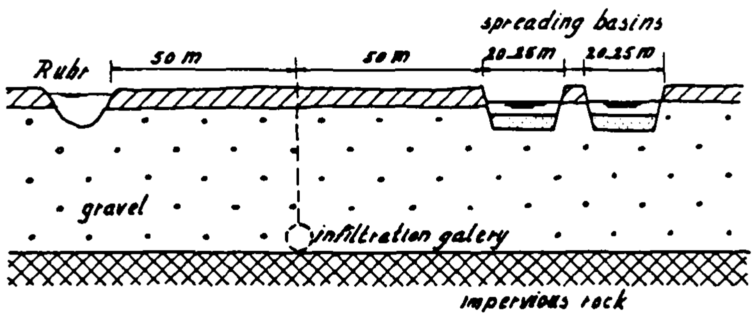

Fig. 5.8 Artificial recharge in the Ruhr District

Notwithstanding all the care that is given to a protection of the Ruhr river against contaminations, a deterioration of river water quality during the last decade could not be prevented. Especially the amounts of oxygen consuming organic compounds increased, lowering the oxygen content of the effluent and bringing with it the danger of anaerobic conditions. This, however, would dissolve iron and manganese in the sub-soil, asking for a post-treatment by aeration and again filtration and thus defeating the purpose of artificial recharge as a slow sand filtration process. An increase in the oxygen content of the effluent is consequently a necessity and this may be obtained by a pre-treatment, increasing the oxygen content of the raw water by aeration and decreasing its oxygen demand by pre-filtration, removing part of the oxygen consuming organic substances. In the Ruhr district, this pre-filter is again constructed as a spreading basin, with the water level even at a considerable distance below the top of the coarse sand bed, so allowing aeration to accompany the downward water movement. Schematically this system is shown in fig. 5.9, while the results obtained are presented in fig. 5.10.

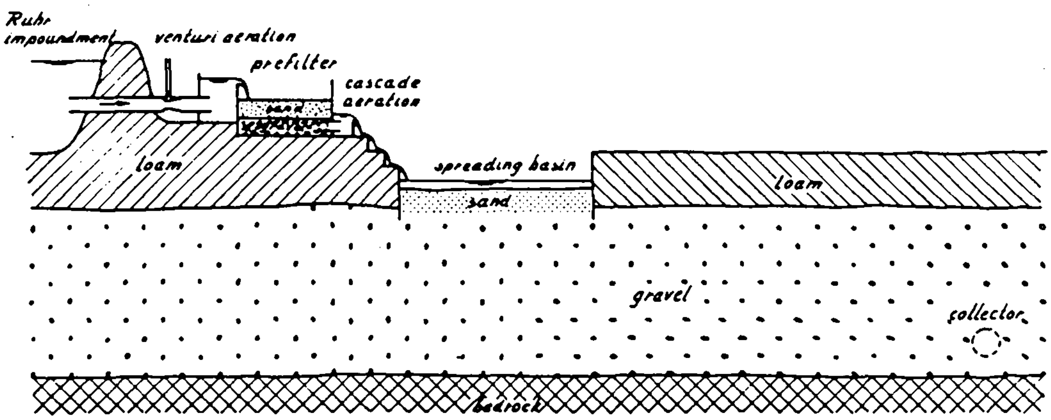

Fig. 5.9 Artificial recharge with pre-filters

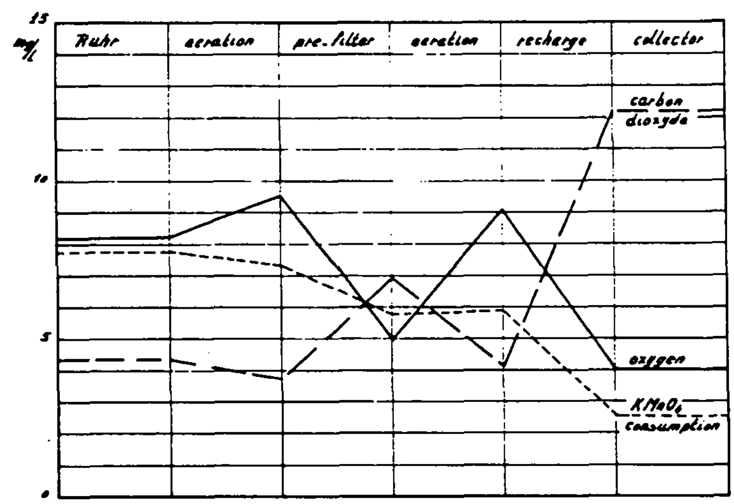

Fig. 5.10 Changes in the composition of the water when treated according to fig. 5.9

5.4 Reduction of maximum salt contents

Inorganic salts are commonly not affected by filtration, neither by artificial recharge. On the other hand the amounts of inorganic wastes discharged into public waterways increases continuously, augmenting the salt content of these streams. One of the most troubling cases in this respect is the disposal of kitchen salt (NaCl), by-product of many industrial activities into the river Rhine. As average over the period 1975-1980 this salt load amounted to no less than 380 kg Cl'/s, for an average flow of the river Rhine at 2000 m^3/s corresponding with an average Cl' content of 190 g/m^3. The flow of the river Rhine, however, is not constant but varies over the year with high discharges in winter and spring and low discharges in summer and autumn. Together with a more or less constant chloride load, this produces a strong variation in chloride contents. Going out from schematical flow patterns, fig. 5.11 shows the fluctuation of the chloride content in a dry year and in a normal year. According to this figure the chloride content obtains maximum values of 430 and 310 g/m^3 respectively, which are too high for a public water supply of good reputation. These contents could be reduced by desalination, but even with reverse osmosis and a limited reduction in salt content, the cost will be high. A more economical solution can be obtained by mixing water of different ages, thus smoothing out variations in river water quality. With artificial recharge this may be effected by varying the distance between recharge and recovery works, as shown in fig. 5.12. Any frequency distribution of detention times can now be obtained. Going out from the linear variation in detention times of fig. 5.13, the fluctuation in chloride content before and after recharge is presented in fig. 5.14. With a difference ΔT between maximum and minimum detention times of 1 and 2 years, the highest Cl' content of the artificial groundwater is now reduced to 264 and 228 g/m^3 respectively, a fair result indeed.

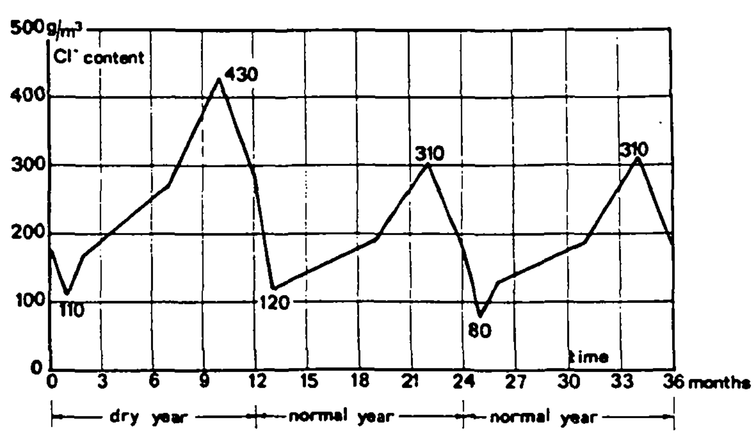

Fig. 5.11 Variation in the Cl'-content of water from the river Rhine in a dry year and a normal year

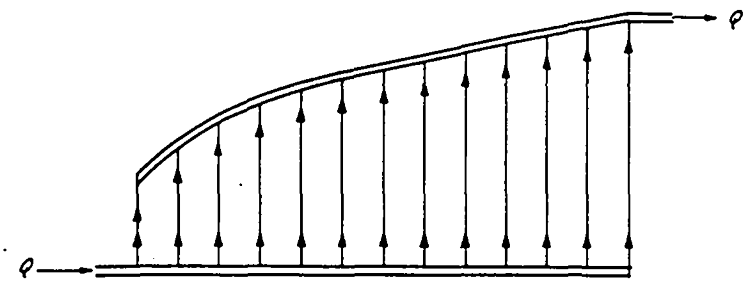

Fig. 5.12 Plan of a recharge scheme with varying detention times

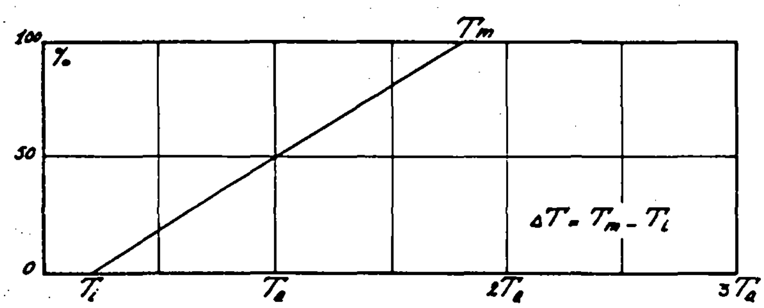

Fig. 5.13 Cumulative frequency distribution of detention times

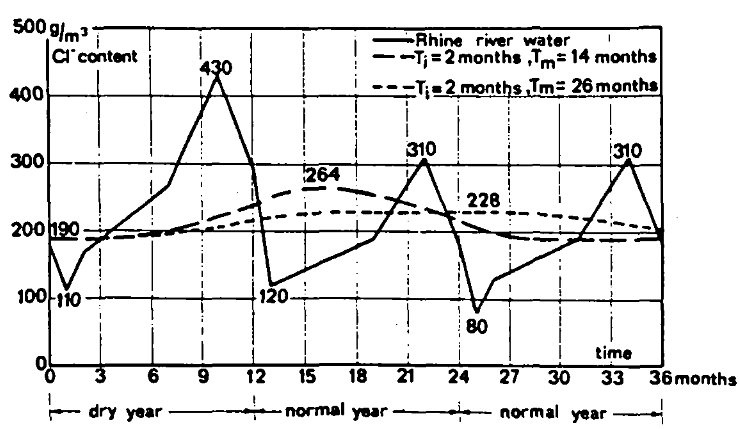

Fig. 5.14 Variation in the Cl'-content of water from the river Rhine after recharge according to fig. 5.13

Increasing the detention times of the recharge water in the sub-soil from a few weeks as shown in fig. 5.8 to several months, does not need to be expensive when a groundwater catchment already exists. With the amount of available rainfall, that is the difference between recharge by precipitation and losses due to evapo-transpiration, equal to a quite normal value of 0.3 m/year, the capacity of such a catchment is limited to 0.3 million m^3/year per km^2 of surface area. With an aquifer depth of 50 m and a pore space of 40% for instance, the amount of groundwater present in the pores of the water bearing formation amounts to 20 million m^3 per km^2 of surface area allowing a recharge capacity of 30 million m^3/year or no less than 100 times the safe yield originating from rainfall, when an average detention time of about 8 months is required. For densely populated, heavily industrialized areas with a strong pollution of public waterways, this type of artificial recharge will give the best results possible, providing a water safe in bacteriological respect and of acceptable chemical composition. Although not strictly necessary, it may be recommended for other areas as well.

6. SLOW SAND FILTRATION IN DEVELOPING COUNTRIES

6.1 Introduction

It is sad to note that from every 10 water supplies constructed in rural areas of developing countries, 7 no longer operate after 3 years. Many reasons can be given for this failure, as most important of which must be mentioned

a. the community was not involved in the planning process;
b. the cost of operating the piped supply was too high;
c. operation was beyond the technical skills of the locally available personnel.

Today, the problems mentioned above under (a) are sought to be solved by community participation, by a bottom-up approach keeping the villagers constantly informed and by consulting their representatives from the beginning to the end of the design and construction period of the new supply. This, however, is beyond the scope of these lecture notes.

For a large part, the operational difficulties (b) and (c) above can be alleviated by the use of groundwater, which by virtue of its origin is safe in hygienic respect, while after constructing the means for abstraction, little supervision or maintenance is required. Unfortunately, however, groundwater is not always available. It may be situated at a great depth below ground surface, making abstraction expensive or it may contain toxic substances such as heavy metals (fluorides in Tanzania), endangering the health of the consumer. Surface water from rivers and lakes must now be used, but this is commonly polluted by suspended and organic matter and may always be contaminated with pathogenic micro-organisms. Before going into supply, this water must be purified for which in industrialized countries many systems are available. Treatments requiring a steady supply of coagulants, flocculants, desinfecting agents, etc., are bound to fail sooner or later in rural areas of developing countries. This leaves as only possibility the use of slow sand filters, with as main advantages

a. design and construction is simple;
b. they can be built from locally available material by local craftsmen;
c. cost of operation and maintenance are low;
d. after some training, operation and maintenance may be taken care of by a member of the community,

while the disadvantages present in industrialized countries are now of much less importance:

e. in tropical areas there are no periods of low temperature with a subsequent drop in efficiency;

f. in rural areas of developing countries large areas of land are readily available;

g. with many people out of work, unskilled labor for manual cleaning is in abundant supply.

With a relatively clear water from lakes, slow sand filtration may be the sole treatment to which the water is subjected. With a turbid water as found in rivers, preceding clarification by settling (in dug basins) may be necessary.

The disadvantages mentioned above under (f) and (g) are even less important as water consumption in the villages of the developing world is extremely small. Population numbers vary from a few hundred tot several thousands (average over 580000 villages in rural India 900 people), while water use is also low as the large majority of the population have to carry the water from public standposts to their homes. The number of yard connections (a single tap outside the house) is limited, while multiple tap house connections are an exception. As calculated below, this gives an average consumption of about 40 liters per capita per day

type of supply	consumption in l/c,d	% of population	average consumption
standpost	20	70	14
yard connection	60	25	15
house connection	150	5	7,5
			36,5
leakages, 10%			3,5
		average consumption	40 l/c,d

by population growth and a shift to a better supply increasing to perhaps 50 l/c,d in future.

In the areas under consideration, water consumption is concentrated at the beginning and end of the day, for instance as shown in fig. 6.1. To allow the slow filters to operate at a constant rate, a large clear water storage is necessary, according to fig. 6.2 equal to 45% of the daily demand. To allow maintenance and small repairs of pumping equipment, actual capacities must even be larger, say 100% of daily consumption.

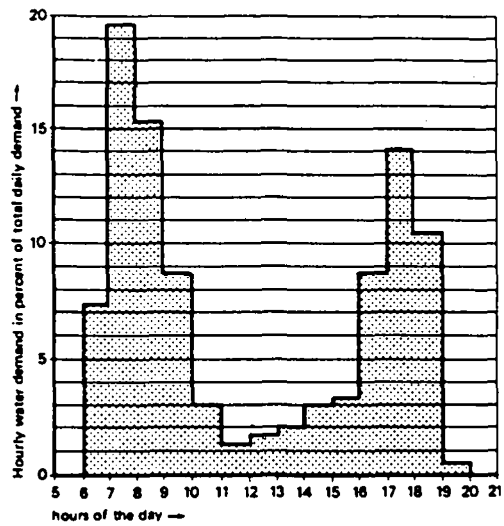

Fig. 6.1 Fluctuation in water demand over the day (rural community in East-Africa)

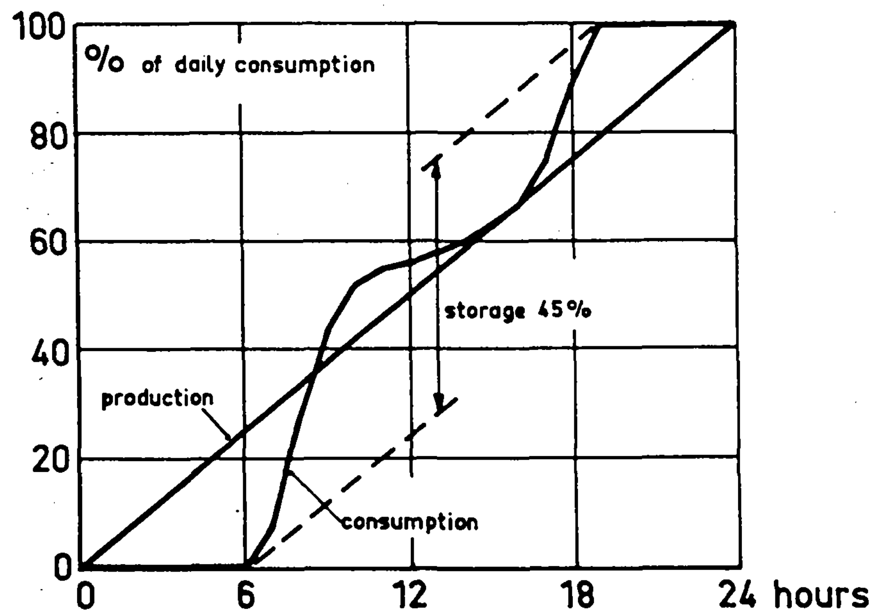

Fig. 6.2 Cumulative water consumption of fig. 6.1 and production during 24 hours

6.2 Design and operation

In rural areas of developing countries, slow sand filters are the sole means to provide a water safe in hygienic respect, continuous post-chlorination being not feasible due to lack of money and management skills. This poses additional requirements

a. with regard to the purifying action of the Schmutz-decke, the filters should always be built open. To prevent contamination by wind blown dust and animals or children falling into the filter, the top of the wall should be at least 0.6 m above ground surface;
b. the minimum depth of sand in the filter bed should be 0.7 m with an additional depth of at least 0.3 m to provide for periodical cleaning;
c. the filters should be designed for a consumption expected after 10-15 years of operation. The lay-out of the plant should allow for future expansion;
d. for the demand mentioned above and all filters in operation, the filtration rate should be less than $(0.03)10^{-3}$ m/s. When filtering units are out of service for cleaning or maintenance, the filtration rate should not exceed a value of $(0.06)10^{-3}$ m/s and preferably be smaller;
e. to ensure uninterrupted supply, the plant should contain at least 2 filtering units, with regard to short-circuiting along the walls with an area not less than 15 m^2. With continuous operation, this means a capacity of $(0.9)10^{-3}$ m^3/s or 78000 l/day, sufficient for 1500 people. When demands are much smaller, pre-fabricated units for instance made of ferro cement or fibre reinforced plastic could be considered;
f. intermittent operation of a slow filter is not allowed as this may result in a break-through of faecal pollution, 4-5 hours after restarting the filter. Declining rate filtration as described in next section is an acceptable alternative;
g. after cleaning of a filter the effluent should be run to waste for at least 24 hours. After this time it may be considered bacteriologically safe and can be put into the supply;
h. after construction and after re-sanding, the breaking-in period mentioned above should be augmented to 6 and 2 weeks respectively. When for one reason or the other the effluent has to be used as drinking water, post-chlorination must be applied or the population advised to boil the water for at least 20 minutes;
i. during construction, a local man or woman should be trained as operator;
j. whenever feasible, gravity flow should be adapted. For the situation of fig. 6.3, the filtered water pumps can be omitted by building the installation on a low hill, say 10-15 m above village level. Doing away with the raw water pumps, however, asks that the intake is shifted a (considerable) distance upstreams, requiring an expensive raw water supply pipe.

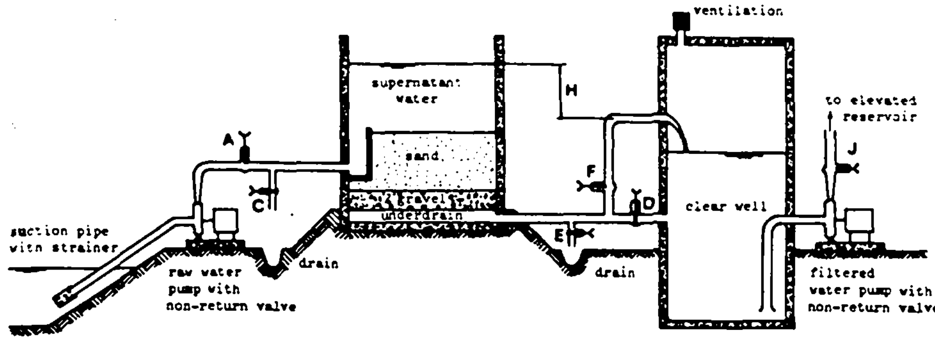

Fig. 6.3 Simplified operations of slow sand filter

During operation of the filter shown in fig. 6.3, the pumps are running, valve A and J are fully open, valve F partially and all other valves fully closed. Valve F is opened, so far as to keep the level of the supernatant water in the filter at the maximum level. When this results in a too high level of the water in the clear well, the operation of the pump is interrupted for a short period. Valves C and E are used for cleaning, to drain the supernatant water and the pore water in the filterbed respectively. Valve D finally is used to back-fill the filter with water from the clear well, that is with water from other filtering units. When this results in an inadequate supply, the population should be warned well in advance.

6.3 Declining rate filtration

Operating the plant of fig. 6.3 asks for continuous supervision, in particular for the internal combustion engines driving the pumps. Operation during 24 hours thus asks for three shifts. This is expensive in terms of labor, often not possible due to lack of skilled personnel, while in other places people are loath to leave their homes after dark. These are the reasons that in many cases operation of the plant is limited to one shift, say for 10 hours, from 7 to 17 hours. According to fig. 6.4 the required amount of storage is now only slightly higher, but the required filterbed area increases theoretically by a factor 2.4, augmenting the cost of construction. To prevent large and sudden variations in filtration rate, valves A and F are not closed when the pumps stops, but remain in the same position as during operation, the non-return valves preventing the water from flowing back. In this way filtration continues at a gradually lower rate, depleting the supernatant water on top of the filterbed. This store of

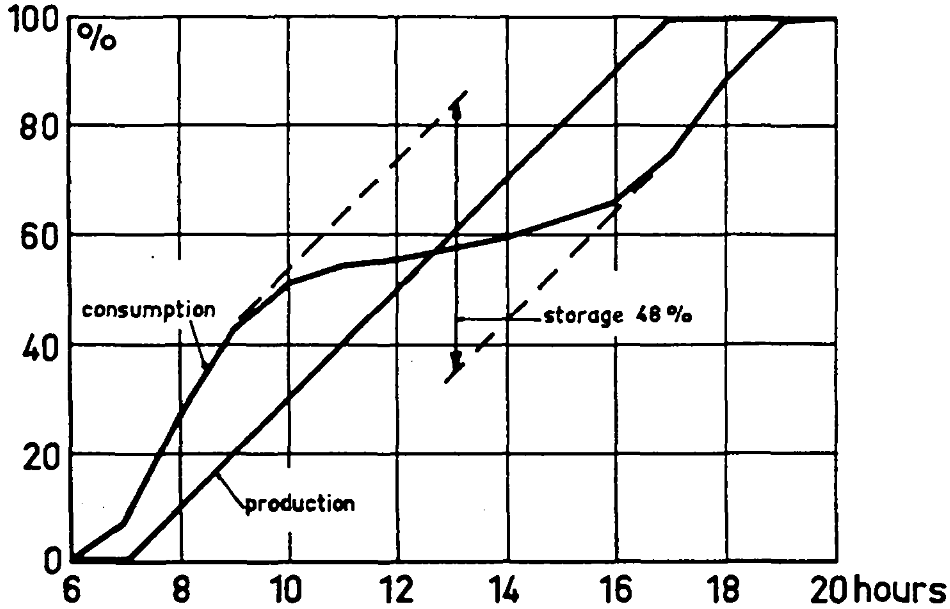

Fig. 6.4 Cumulative water consumption of fig. 6.1 and production during 10 hours

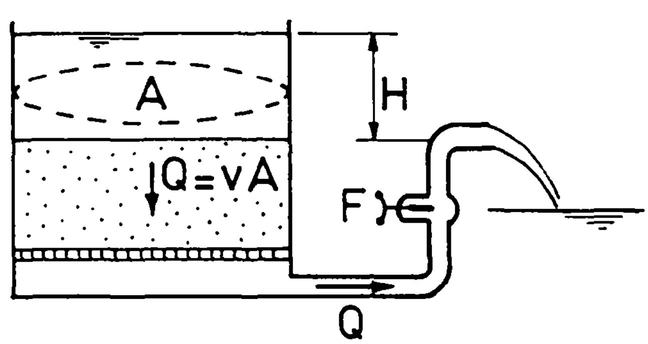

Fig. 6.5 Declining rate filtration

water is replenished after starting the pump, causing the filtration rate to rise gradually. The situation after stopping the pump is shown schematically in fig. 6.5, where the resistance H is made up of two parts, the laminar resistance of the partly clogged filterbed and the turbulent resistance of the partly closed regulating valve F with accessory piping. In formula

$$H = \alpha v + \beta v^2 \quad \text{or} \quad dH = \alpha dv + 2\beta v dv$$

with α and β as constants varying with time and v as filtration rate. This filtration lowers the water level in the filter

$dH = -vdt$ Combining both equations gives

$dt = -\alpha \frac{dv}{v} - 2\beta dv$ Integrated

$t = -\alpha \ln v - 2\beta v + C$

The boundary condition $t = 0$, $v = v_1$, gives

$0 = -\alpha \ln v_1 - 2\beta v_1 + C$, combined

$$t = \alpha \ln \frac{v_1}{v} + 2\beta(v_1 - v)$$

At the beginning of the pumping period, at $t = t_2$, the filtration rate has dropped to v_2, while the pump delivers an amount $Q_o = v_o A$. The formula for the resistance remains the same, but the continuity equation changes to

$dH = (v_o - v)dt$ Combination of both formulas now gives

$dt = \alpha \frac{dv}{v_o - v} + 2\beta \frac{vdv}{v_o - v}$ Integrated

$t = -(\alpha + 2\beta v_o)\ln(v_o - v) - 2\beta v + C$

Substitution of the boundary condition gives

$t_2 = -(\alpha + 2\beta v_o)\ln(v_o - v_2) - 2\beta v_2 + C$, combined

$$t - t_2 = (\alpha + 2\beta v_o)\ln \frac{v_o - v_2}{v_o - v} + 2\beta(v_2 - v)$$

At the end of the pumping period, at $t = t_3$ the filtration rate has increased to v_3 which must be equal to the value of v_1 mentioned above. As an example, the situation half-way the filterrun will be considered when the resistance is about equally divided between the filterbed and the regulating valve

$$\alpha v_o = \frac{H_o}{2} \quad , \quad \beta v_o^2 = \frac{H_o}{2}$$

Assuming moreover $H_o = 1.2$ m, $v_o = (0.04)10^{-3}$ m/s, $t_2 = 14$ hours = 50400 s, $t_3 = 24$ hours = 87600 s, gives

$$0 < t < t_2 \qquad t = (15)10^3 \ln \frac{v_1}{v} + (30)10^3 \frac{v_1 - v}{v_o} \qquad (1)$$

$$t_2 < t < t_3 \qquad t = t_2 + (45)10^3 \ln \frac{1 - v_2/v_o}{1 - v/v_o} + (30)10^3 \frac{v_2 - v}{v_o} \qquad (2)$$

in which the value of v_1 must be chosen such that it equals the value of v at t_3 from equation (2). The results thus obtained are presented in fig. 6.6 and show a gradual variation in filtration rate between the limits of $(0.004)10^{-3}$ m/s and $(0.029)10^{-3}$ m/s.

When at the end of the design period a population of 1500 people is expected with an average consumption of 50 l/c,d, the calculation given above yields

pumping rate	Q_o = 2.08 l/s
filterbed area	A = 52.1 m^2

asking for 2 raw water pumps each with a capacity between 2 and 2.5 l/s (depending on availability) and 3 filters each with an area of 18 m^2 (fig. 6.7). When water consumption rises beyond the figures mentioned above, the simplest solution is a gradual increase in the period of operation. With continuous operation the capacity of the plant rises to 140 m^3/day, adequate for 2000 - 2500 people depending on per capita per day consumption.

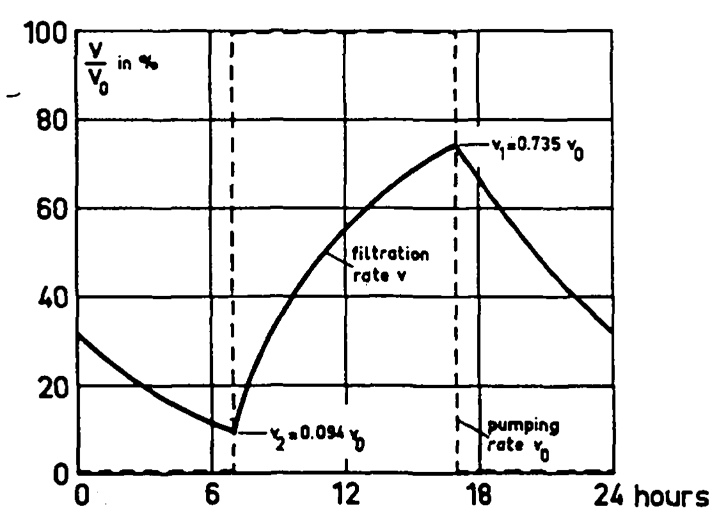

Fig. 6.6 Variation of filtration rate with time

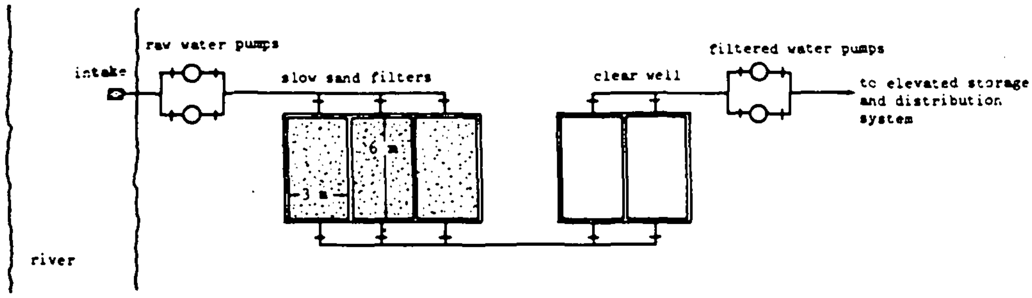

Fig. 6.7 Plan of slow sand filtration plant, serving 1500-2500 people

6.4 Appropriate technology

In rural areas of developing countries, the materials necessary to construct the treatment plant of fig. 6.3 and 6.7 are commonly not available. In particular this holds true for

a. pumps and motors;
b. pipes with valves and fittings;
c. steel bars for reinforcing concrete or masonry work,

while cement is used so widely that it will not be too difficult to obtain. Sand for filters, concrete and cement can often be found locally. The same holds true for gravel in filterbottoms and concrete for which also broken material from solid rock or well-fired bricks may be used.

With circular filters of small diameter the tangential stresses are so low that the steel bars in reinforced concrete may be replaced by chicken wire in ferro-cement. Larger filters with vertical walls can be built from mass concrete or masonry work, but a much cheaper solution is building the filter in an excavation with sloping walls as shown in fig. 6.8.

Pipes with fittings and other appurtenances are always necessary, but the amount required can greatly be reduced by omitting a distribution system, providing water with stand-posts at the treatment plant only, as shown in fig. 6.9. To take the water home now requires a greater walking distance, but with small villages and the houses built close together this is not too objectionable.

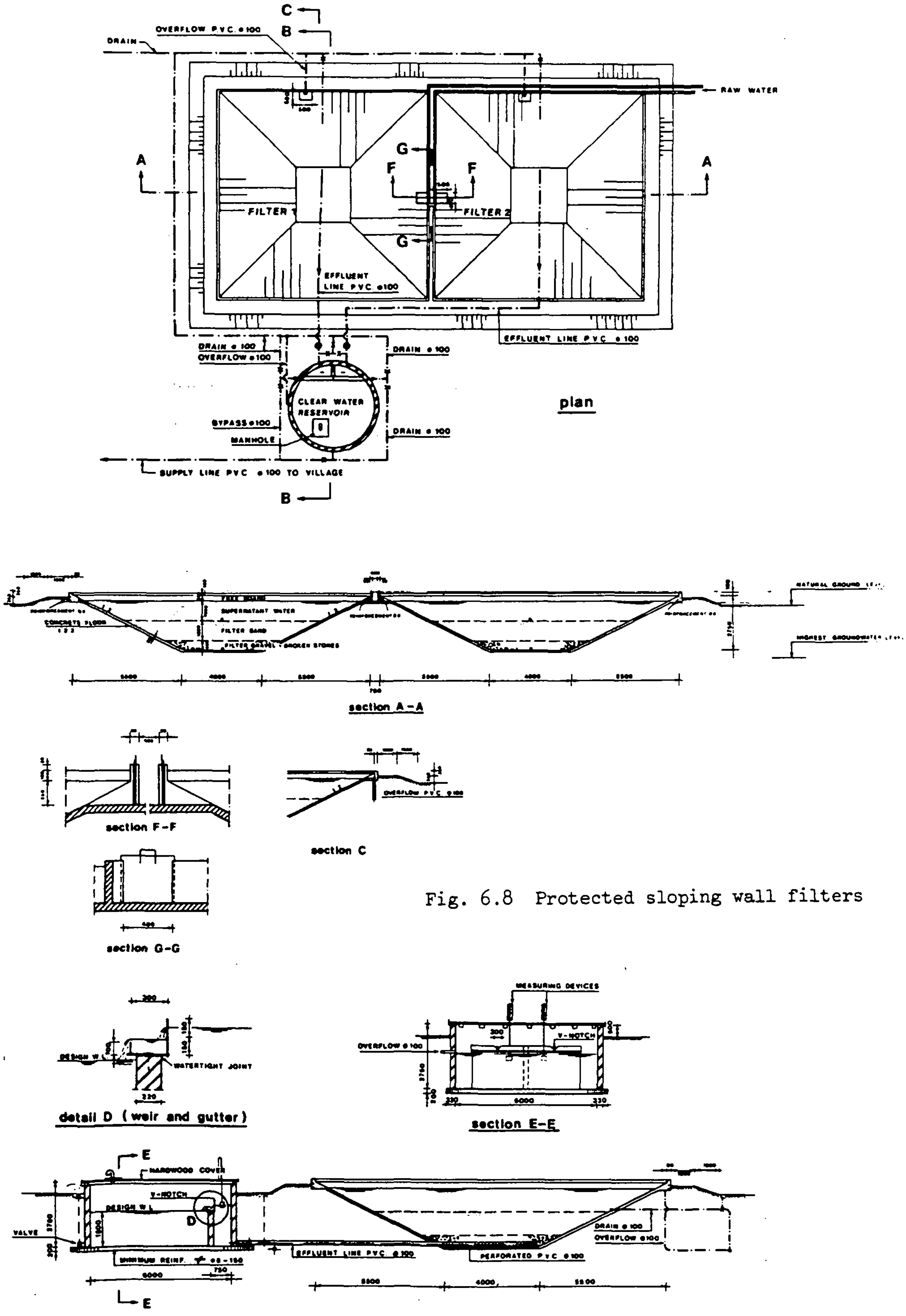

Fig. 6.8 Protected sloping wall filters

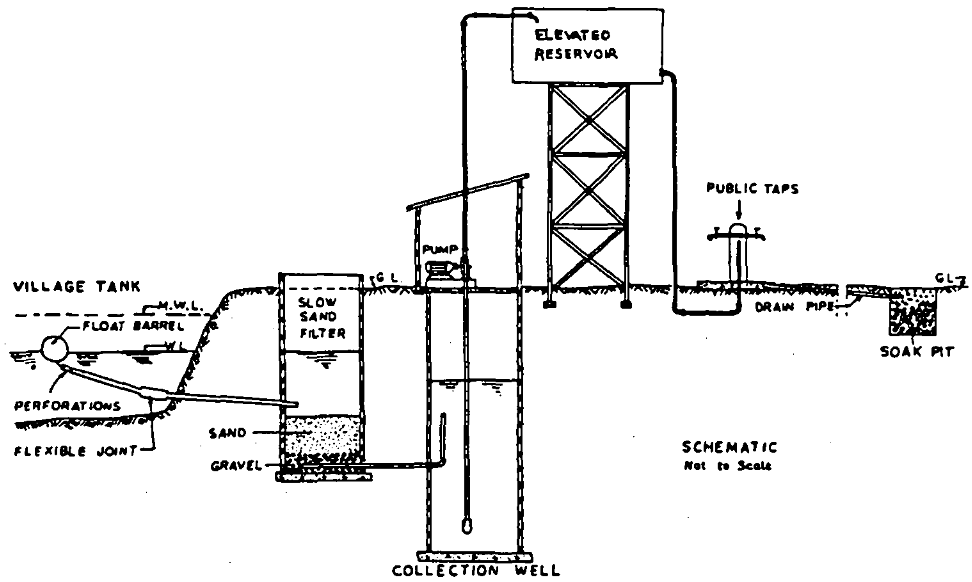

Fig. 6.9 Village water supply without distribution system

For small villages and a low consumption, the motor driven pump of fig. 6.9 may be replaced by a handpump. Assuming a population of 500, a consumption of 20 l/c,d and a lift of 10 m, the work to be done is theoretically equal to 10^6 Newtonmeter or Wattseconds and in practice a factor 2 higher. A full-grown man can easily produce 100 Watt continuously, meaning that he has to operate the handpump for

$$T = \frac{(2)10^6}{100} = 20000 \text{ s} = 5.5 \text{ hours}$$

an unpleasant task, but no too cumbersome.

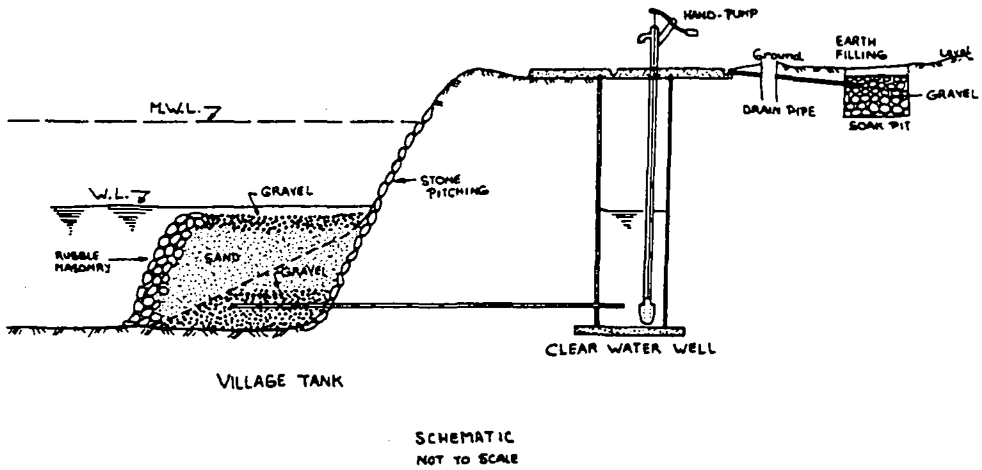

Fig. 6.10 Village water supply with induced recharge

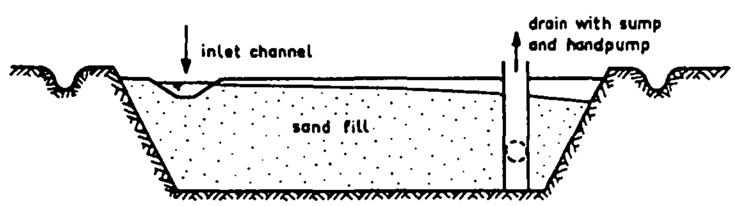

Fig. 6.11 Village water supply with artificial recharge

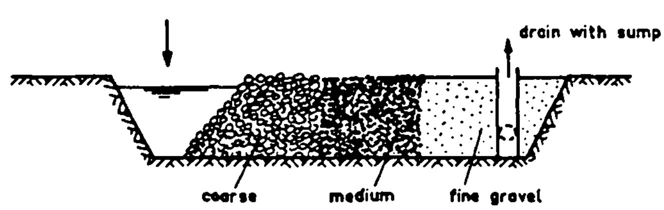

Fig. 6.12 Horizontal roughing filter

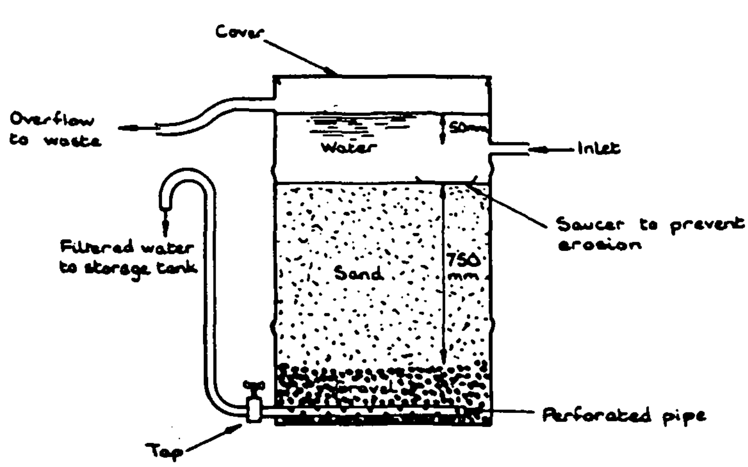

Fig. 6.13 Household sand filter, capacity 1 l/s

Appropriate solutions can also be obtained by using the techniques of artificial recharge described in chapter 5. An example of induced recharge is shown in fig. 6.10, but it is doubtful whether this construction will work on the long run. After passing the rubble masonry, the suspended matter in the water from the village tank will clog the sand fill, reducing the entrance rate. A better solution in this respect is shown in fig. 6.11. Pre-treatment is now possible by settling or horizontal roughing filtration (fig. 6.12), while after clogging the inlet channel can be cleaned without difficulty. For the consumption mentioned above, 10 m^3/day = $(0.12)10^{-3}$ m^3/s, an infiltration rate of $(0.02)10^{-3}$ m/s requires an inlet channel of 6 m^2 area at the water line, say 1.5 m wide and a length of 4 m perpendicular to the plane of drawing. For a detention time of one week, a pore volume of 70 m^3 and a total volume of the sand fill of 200 m^3 is required, for instance 2.5 m deep and a surface are of 5 by 16 m^2.

For a single family, quite small sand filters will suffice, for the capacity of 1 l/minute mentioned in fig. 6.13 a diameter of 0.8 m only, for which old oil drums or ferro cement containers can be used.

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