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SEDIMENTATION AND FLOTATION

MECHANICAL FILTRATION

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L.S.

Bij de samenstelling van elk diktaat wordt er uiteraard naar gestreefd om fouten te voorkomen en de inhoud zo overzichtelijk mogelijk aan te bieden.
Niettegenstaande dat kunnen toch onduidelijkheden voorkomen en kunnen fouten zijn ingeslopen.
Indien U dan ook bij de bestudering van dit diktaat:

- onjuistheden ontdekt
- op onduidelijkheden stuit
- of gedeelten ontmoet, die naar Uw mening nadere uitwerking behoeven, verzoeken de samenstellers U dringend hen daarvan mededeling te doen.

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

contents, continued

Sedimentation and flotation

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1. Introduction

1.1. Definitions and terms

Sedimentation and flotation are purification processes, whereby the water to be treated is kept in a tank for a considerable period of time. With a large cross-sectional area of this tank, the velocities of flow will be small, creating a state of virtual quiescence. Under influence of gravity (fig. 1.1), particles with a mass density higher than that of the surrounding fluid will now move downward (sedimentation), while particles with a lower mass density will move upward (flotation). In this way, the suspended particles present in the raw water are retained, either in the scum layer at the water surface or in the sludge layer at the tank bottom, allowing the water to leave the tank in a clarified condition. As will be shown in the next chapter, the rate of vertical rise or fall of the suspended particles will be higher and the time needed for adequate clarification will be smaller as the particles are larger and their mass density differs more from that of the liquid in which they are suspended. Artificially the size of the particles may be increased by flocculation (fig. 1.2), creating velocity gradients which bring the

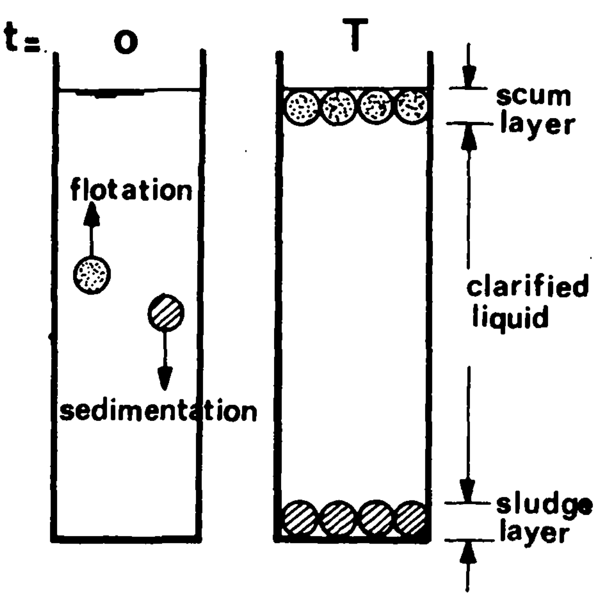


Fig. 1-1 Sedimentation and flotation.

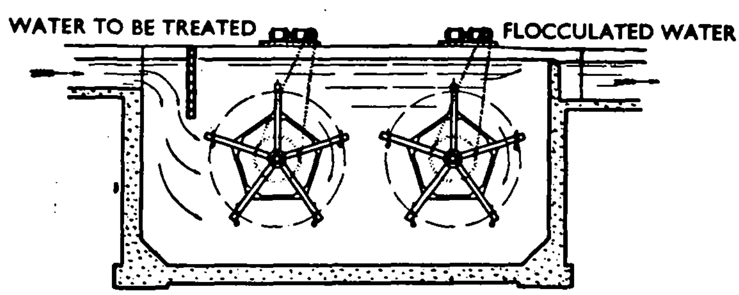


Fig. 1-2 Flocculation.

particles into contact with one another. In case the particles carry a like electric charge, they will repell each other and aggregation by flocculation is only possible after the opposing electric forces have been neutralized by the addition of coagulants such as alumn and iron (chemical coagulation). The mass density of the suspended flow may be altered by adding heavier or lighter substances to the water to be treated. Sedimentation can be advanced by mixing clay, bentonite powdered stone, etc, to the incoming raw water, while flotation can be promoted by bubbling in air, chlorine gas, etc, at the tank bottom. The last mentioned process is so simple and effective that it is also used for separation of suspended particles that are slightly heavier than the surrounding liquid (e.g. algae). Dissolved impurities finally can also be removed by sedimentation when chemicals are added to throw them out of solution (chemical precipitation).

In the field of water and waste water engineering, many settling tanks operate by natural forces alone, by gravitation and by natural aggregation of suspended particles. This process is called plain sedimentation and may further be subdivided in discrete settling and in flocculent settling. Discrete settling occurs when the amount of natural aggregation is negligeable as for instance is the case with sand grains. During the whole settling period the suspended particles maintain their identity and consequently move down at a constant rate (fig. 1.3 left). With flocculent settling on the other hand, particles overtaking one another will coalesce and will henceforth go down at the higher rate of the aggregate (fig. 1.3 right). Flocculent settling is predominant with organic impurities as abound in municipal sewage and many industrial wastes

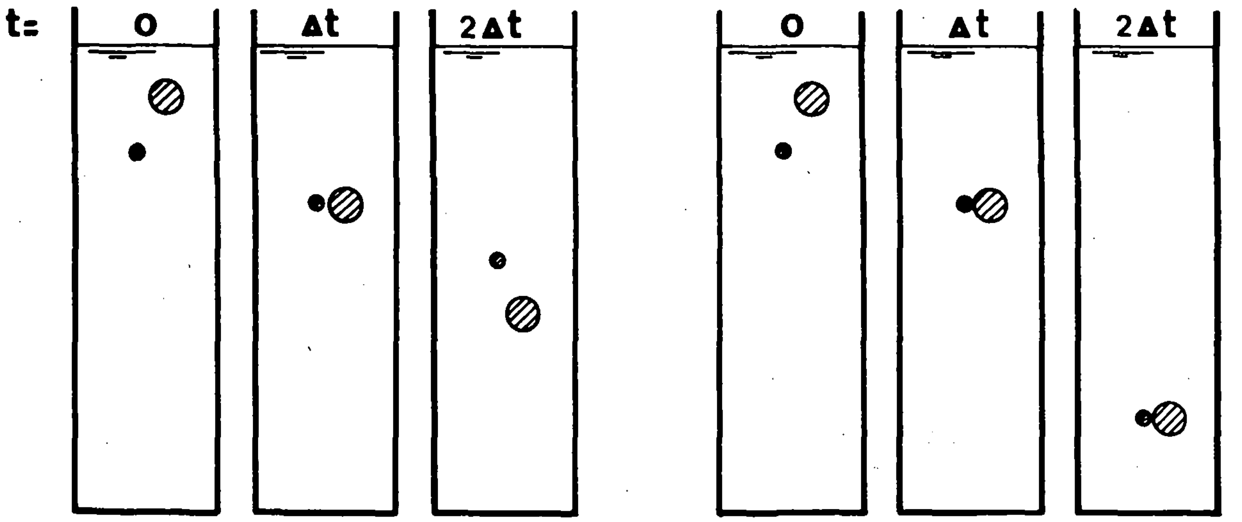


Fig. 1-3 Settling of discrete and of flocculent particles.

1.2. Types of settling tanks

Settling tanks are constructed in great variety, being rectangular, square or circular in plan, with the water at rest or in continuous flow, either in horizontal or in vertical direction.

Quiescent settling in fill and draw tanks (fig. 1.4) is nowadays largely confined to the preparation of boiler feed water and to the purification of industrial wastes that are discharged in batches. Due to the sequential operation of filling, standing and drawing-off, they are less suited for public water supplies or municipal waste water treatment plants where a continuous flow must be maintained.

For the removal of discrete particles, horizontal flow basins as shown in fig. 1.5 give the best results. With the same capacity

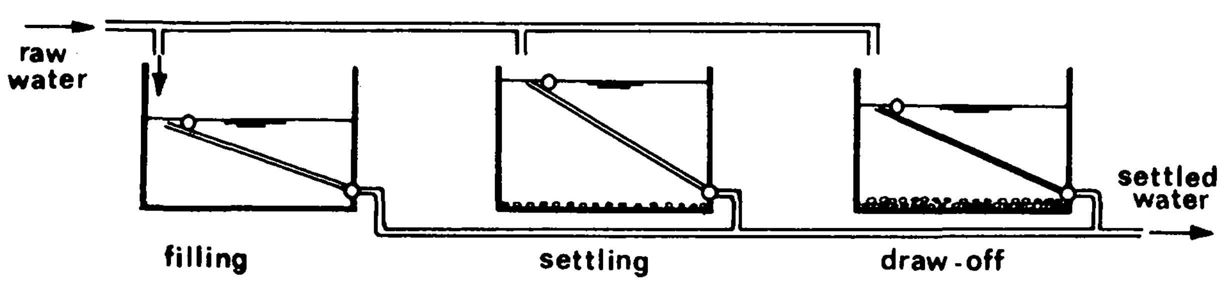


Fig. 1-4 Quiescent settling in draw-and-fill tanks.

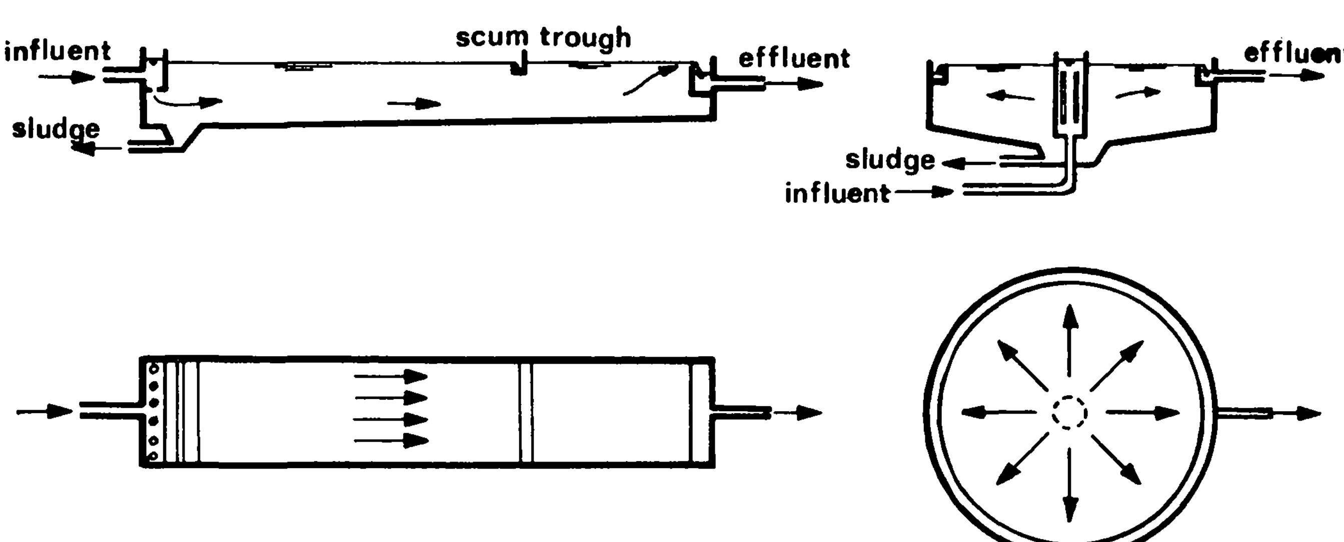


Fig. 1-5 Horizontal flow basin, rectangular and circular.

and tank volume, long, narrow and shallow rectangular basins have the highest efficiency. Depending on local circumstances, however, square or circular basins may still be preferred because they make a better use of the grounds available, offer a saving in the amount of building material required, allow the use of pre-stressed concrete, in stead of ordinary reinforced concrete, etc. With the same volume and efficiency, a larger capacity can be obtained by increasing the area upon which the settled-out material may accumulate. This can be accomplished in different ways of which fig. 1.6 shows the use of horizontal trays and of steeply inclined plates or tubes. In this way, a tremendous capacity for clarification can be packed in a small volume, but this is bought with a highly increased cost of construction per unit tank volume and again local circumstances must decide whether this proposition is attractive or not.

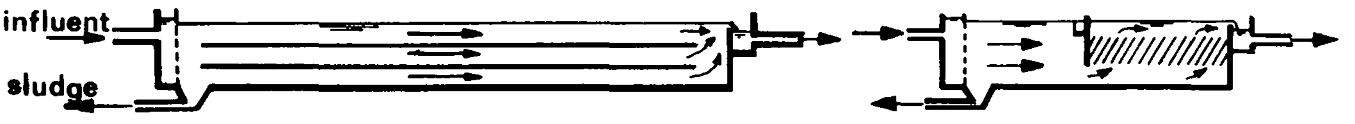


Fig. 1-6 Tray settling tank and tilted plate separator.

For the removal of flocculent particles, the same horizontal flow basins as described above may be applied, but in many cases better results can be obtained with vertical flow basins of larger depth and more elaborate inlet constructions to divide the incoming water as equally as possible over the entire floor area. A circular plan has no disadvantages in hydro-dynamic respect and when a rectangular plan is preferred, it will deviate only little from a square. In fig. 1.7 to the right, a cone-shaped tank of great depth is shown in which the velocity of upward flow gradually decreases. Near the top of the cone, this displacement velocity will equal the settling velocity for a major part of the suspended flocs and here a stationary sludge blanket will arise. In this blanket, the concentration of flocs is very high, promoting coalescence by which even finely divided suspended matter can be "filtered out". On the other hand, the large depth increases the cost of construction and again the most attractive solution can only be found by a cost-benefit analysis, taking all relevant factors into account.

From the tank designs shown in fig. 1.5 to 1.7 inclusive, it will be clear that in a continuous flow settling basin many elements can be distinguished. As most important may be mentioned

a. the inlet construction to disperse the influent flow with the suspended matter uniformly over the cross-section of the basin;
b. the settling zone in which the suspended particles subside through the flowing water with a minimum of disturbance caused by the fluid displacement;
c. the outlet construction to collect the clarified liquid uniformly over the cross-section of the basis;
d. the sludge zone in which the removed solids accumulate and from which they are withdrawn for disposal.

As minor elements may be mentioned the equipment for bringing the sludge to a sump near the inlet end and for subsequent removal from the tank, the scum baffle to prevent floating matter from escaping with the effluent and the scum trough for (periodic) removal of the impurities accumulated at the water surface, vertical baffles for guiding the flow of water in this way preventing adverse effects on the settling process, etc. All these elements present special problems of hydraulic and process design, which must be solved adequately to prevent a reduction in basin efficiency.

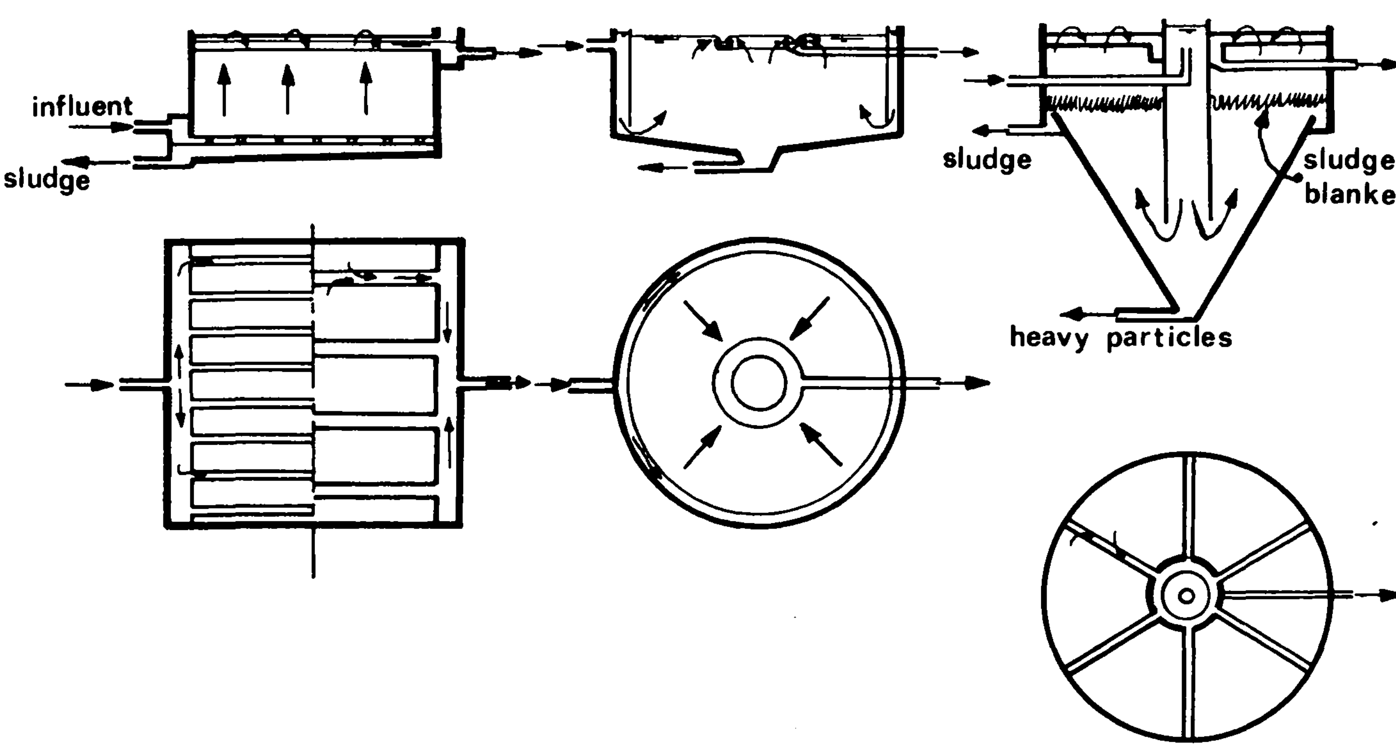


Fig. 1-7 Vertical flow settling tanks.

1.3. Application of sedimentation and flotation in water and waste water treatment

With public and industrial water supplies, filtration is the main treatment to which the water is subjected. Rapid and slow sand filters, however, can only operate to satisfaction when the suspended matter content of the influent is not too high, not more than $(2\text{-}5)10^{-3}$ kg/m^3 for rapid and slow filters when they constitute the final treatment and not more than $(10\text{-}20)10^{-3}$ kg/m^3, in exceptional cases $(50)10^{-3}$ kg/m^3 for the preliminary treatment in a two stage filtration process. Many rivers carry a higher suspended load and sedimentation is now an attractive proposition to obtain the desired amount of clarification. In many cases plain sedimentation will satisfy all requirements, but when the colloidal content is high, better results at a lower price may be obtained by chemical coagulation (fig. 1.8 upper half). During drought periods, the quantity or quality of the water in some rivers is too low and storage reservoirs are now necessary to assure an uninterrupted supply. In such reservoirs the water is often kept for months on end and loses nearly its full silt content. The effluent, however, may still be very turbid when nutrients abound and a heavy growth of algae develops. With a mass density nearly equal to that of the surrounding water, these algae are difficult to remove by sedimentation alone.

As already mentioned before, sedimentation may also be used to remove dissolved impurities, after chemicals have been added to bring them out of solution. Anaerobic ground waters may have a high content of dissolved ferrous iron. By aeration this iron will be oxydised and converted into insoluble ferric hydroxyde, which may subsequently be removed for a large part by settling, by plain sedimentation (fig. 1.8) or better by chemical coagulation. A major application concerns the removal of hardness by lime-soda softening, converting the dissolved calcium and magnesium salts into unsoluble calcium carbonate and magnesium hydroxyde. With rather hard waters, the amount of sludge retained in the settling basin is enormous, $(100\text{-}500)10^{-3}$ kg/m^3, creating a serious disposal problem. Re-use after calcination is now a better solution, but this is only possible when a clean sludge is obtained, with turbid river waters asking for a two-stage chemical coagulation process where silt and other

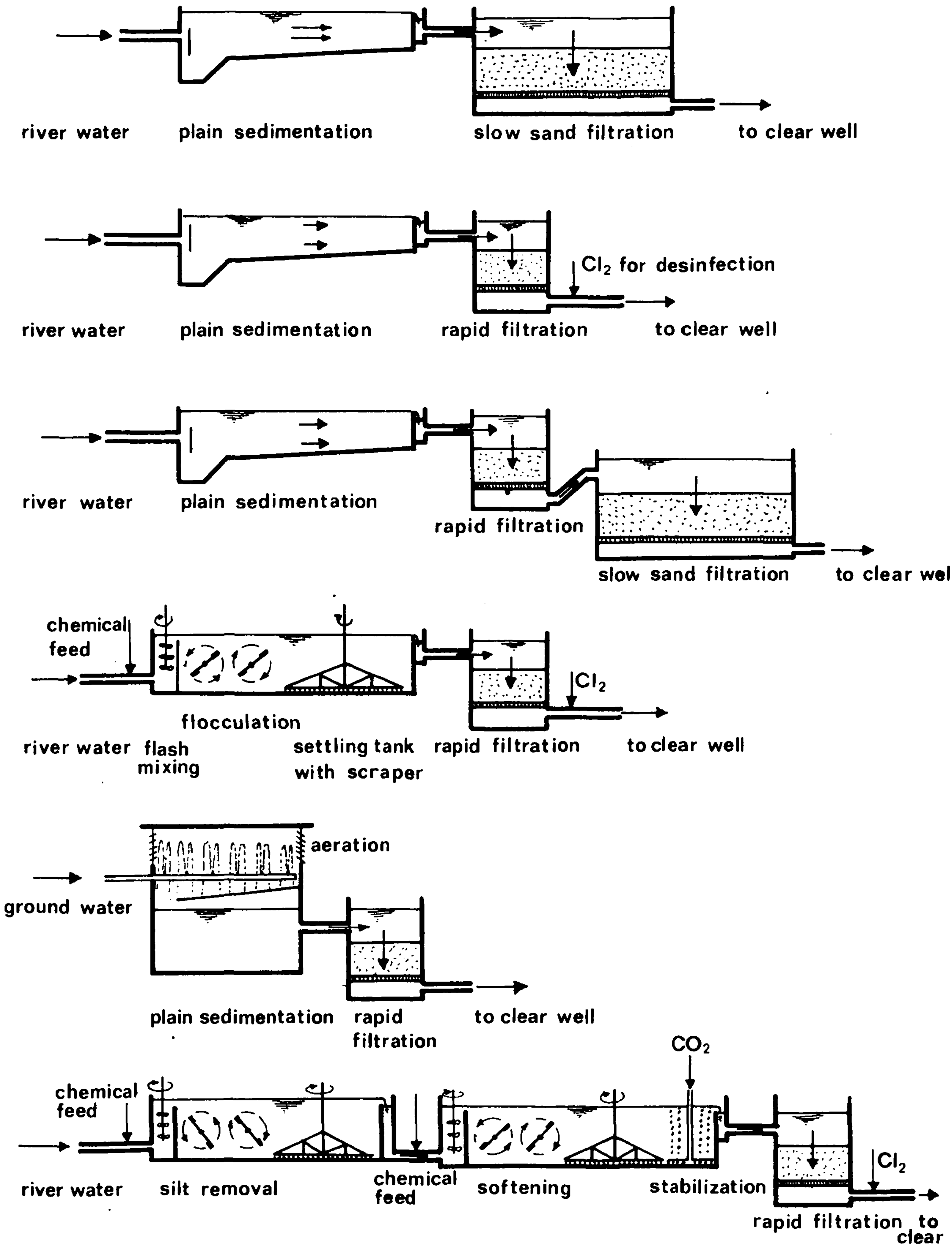


Fig. 1-8 Sedimentation in water supply practice.

impurities are retained in the first sedimentation basin (fig. 1.8).

In the treatment of domestic and industrial waste-waters, sedimentation is of paramount importance, removing the major part of the impurities. With mechanical treatment (fig. 1.9a) it is even the sole process to which the water is subjected, recovering 60 to 80% of its suspended matter content. Due to the presence of colloidal and molecular dissolved organic impurities, the reduction in biochemical oxygen demand is much lower, 30 to 50%. Mechanical treatment is thus only acceptable when the effluent is discharged into receiving waters with a high capacity for self-purification. Some improvement may be obtained by preceding the settling process with chemical coagulation and flocculation, but the classical waste water treatment incorporates aerobic micro-biological processes in trickling filters or in activated sludge aeration tanks. To prevent a rapid clogging of these installations, primary settling tanks are again necessary to remove the major part of the suspended load, after which the remaining particulate and dissolved impurities can be retained by absorbtion on the bacterial slimes present in the aerobic contact processes mentioned above. By the action of micro-organisms, this material is further degraded, partly to provide the energy they need for their living processes (metabolism) and partly to built up new cell material. In the last case, however, suspended matter is formed that may be removed in the subsequent secondary settling tanks (fig. 1.9b).

The primary and secondary settling tanks mentioned above are often supplemented with other sedimentation and flotation processes. In particular with a combined sewerage system, the sewage may contain larger amounts of sand and other heavy and inert material. This grit as it is called, has a high settling rate and will completely be retained in the primary settling tanks. In many cases, however, a preceding removal by sedimentation in grit chambers is more attractive, among other things because this material is non-putrescible, enormously facilitating the disposal problem. Next to suspended particle heavier than water, every sewage will contain substances with a lower mass density. With domestic sewage this concerns oil and grease, but with municipal waste waters larger amouts of other substances may also be present. In the settling tanks mentioned

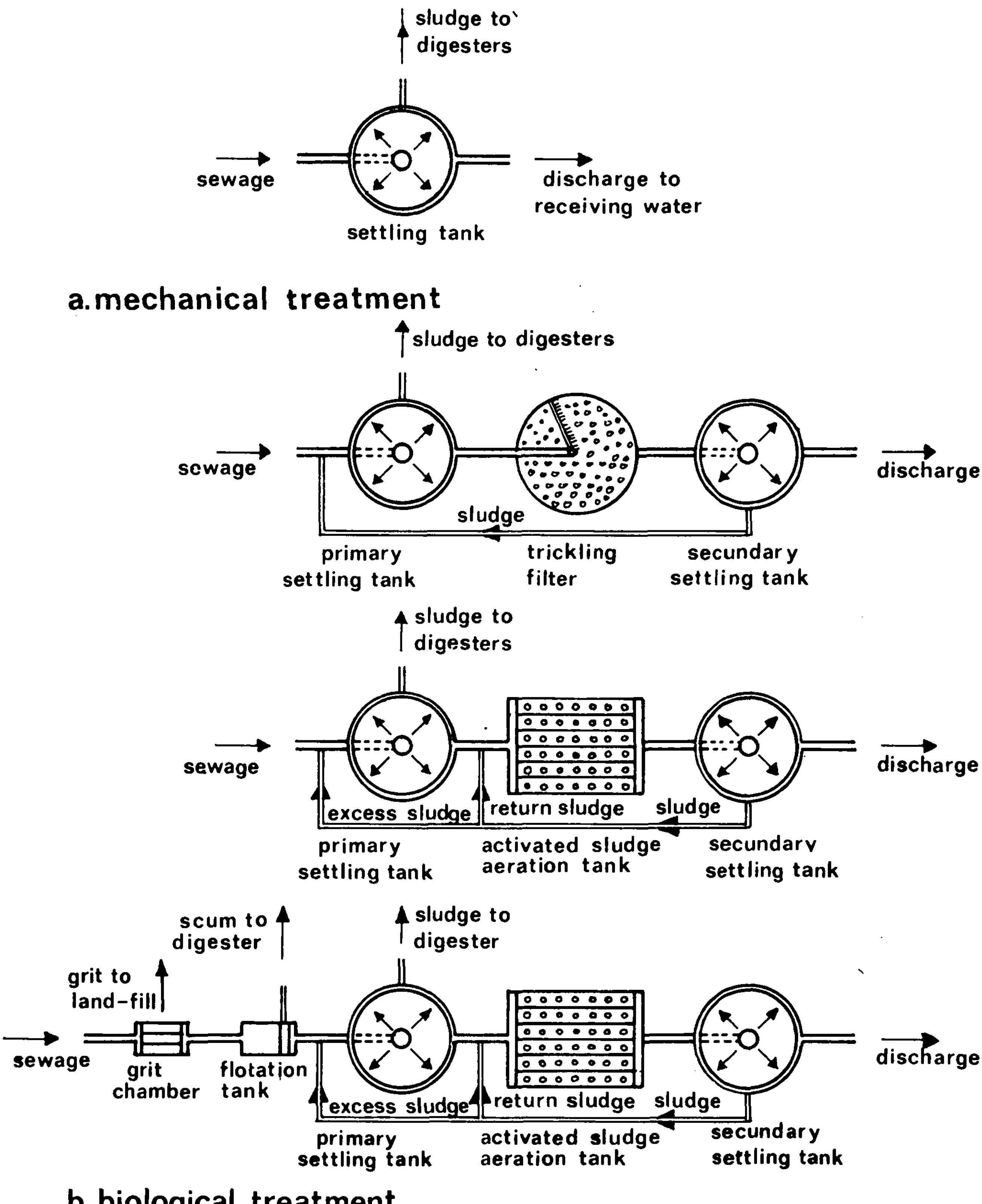


Fig. 1-9 Sedimentation and flotation in the treatment of sewage and industial wastes.

above, this material is retained by flotation, accumulates at the water surface and is removed by skimming devices. With some industrial effluents, the amount of floating matter is high and again here a separate removal in flotation tanks ahead of the the settling tanks proper may be advantageous (fig. 1-9 bottom.)

2. Principles of discrete settling

2.1. Settling of a single particle

Discrete settling occurs when during the whole process the suspended particle does not change its size, shape or weight. When such a particle is released in a still fluid (fig. 2.1), it will move vertically downward when its density is larger than that of the surrounding liquid. The particle will accelerate untill the frictional drag of the fluid equals the value of the impelling force, after which the vertical velocity of the particle with respect to the suspending

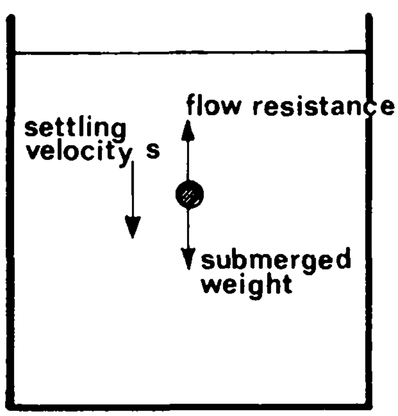


Fig. 2-1 Settling of a single particle in quiescent water.

liquid will be constant. The impelling force equals the submerged weight of the particle

$$F_i = (\rho_s - \rho_w)gV$$

in which ρ_s and ρ_w are the mass densities of particle and water respectively, g is the gravity constant (9.81 m/sec^2) and V the volume of the particle. According to Newton the frictional drag equals

$$F_d = c_D \frac{\rho_w}{2} s^2 A$$

with the drag coefficient c_D a dimensionless number, s the terminal or settling velocity of the particle and A its projected area in the direction of motion. Equality of both forces with uniform movement gives as settling velocity

$$s = \sqrt{\frac{2}{c_D} \frac{\rho_s - \rho_w}{\rho_w} g \frac{V}{A}}$$

For a sphere of diameter d

$$A = \frac{\pi}{4} d^2 \ , \quad V = \frac{\pi}{6} d^3 \ , \quad \text{substituted}$$

$$s = \sqrt{\frac{4}{3c_D} \frac{\rho_s - \rho_w}{\rho_w} gd}$$

The value of c_D in the meanwhile is not constant as Newton supposed, but depends on the magnitude of the Reynolds number for settling

$$Re = \frac{sd}{\nu}$$

with ν as kinematic viscosity of the surrounding liquid. With pure water

$t =$ 0	5	10	15	20	25	30	35	40		^0C
$\nu =$ 1.79	1.52	1.31	1.15	1.01	0.90	0.80	0.73	0.66	$\times 10^{-6}$	m^2/sec

The observed relation between c_D and Re for particles with various shapes is shown in fig. 2.2. For a sphere this relationship may be schematized as follows

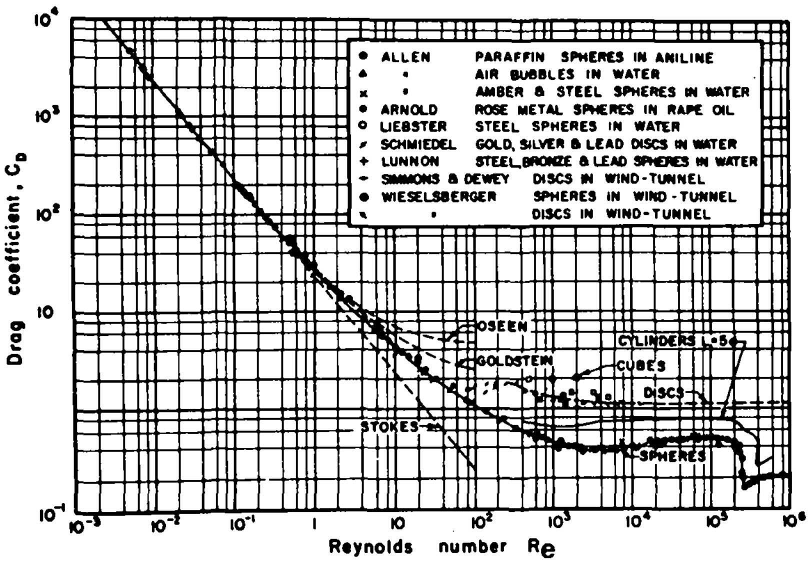


Fig. 2-2 Drag coefficient as function of the Reynolds number.

Re < 1, the upward flow of water along the downward moving particle occurs under streamline conditions, the frictional resistance is only due to viscous forces and c_D varies inverse proportional to Re

$$c_D = \frac{24}{Re}$$

Re > 2000, the flow of water along the settling particle takes place under fully developed turbulent conditions. Compared with the eddying resistance the viscous forces are negligeable and c_D is virtually constant. For spheres and up to Reynolds numbers of 10^5

$$c_D = 0.40$$

1<Re<2000, a transition region in which the viscous and eddying resistance are of equal importance. An exact formula for c_D cannot be given, but for Reynolds numbers below 10^4 (including the laminar flow region) a good approximation (fig. 2.3 dotted line) may be had with

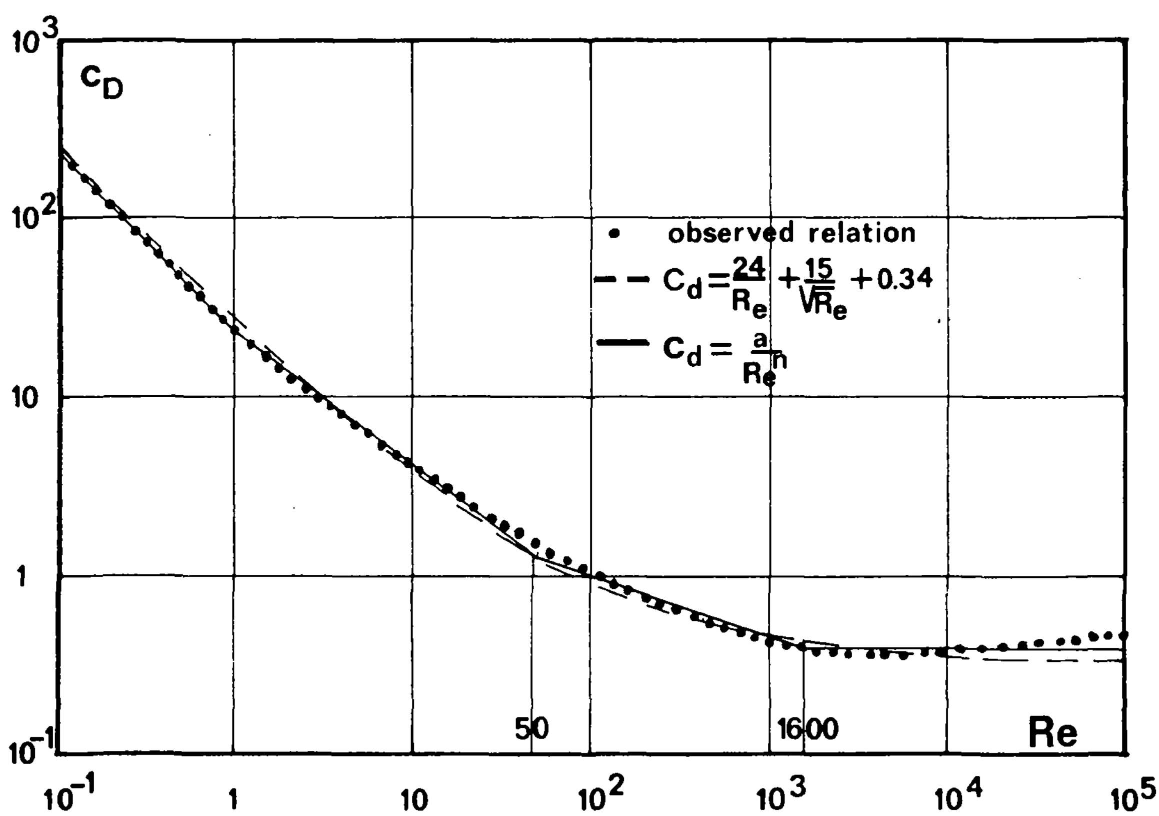


Fig. 2-3 Approximation for the observed relation between c_d and Re for spherical particles.

$$c_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$$

A better approximation (fig. 2.3 full lines) and more workable formulae may be obtained by subdividing the transition region,for instance

$1<Re< 50$ $\quad c_D = \dfrac{24}{Re^{3/4}}$

$50<Re<1600$ $\quad c_D = \dfrac{4.7}{Re^{1/3}}$

$1600 < Re$ $\quad c_D = 0.40$

Substitution of these values in the formula for the settling velocity gives

$Re< 1$ $$s = \frac{1}{18}\,\frac{g}{\nu}\,\frac{\rho_s - \rho_w}{\rho_w}\,d^2$$

$1 < Re< 50$ $$s = \frac{1}{10}\,\frac{g^{0.8}}{\nu^{0.6}}\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{0.8} d^{1.4}$$

$50< Re<1600$ $$s = \frac{1}{2.13}\,\frac{g^{0.6}}{\nu^{0.2}}\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{0.6} d^{0.8}$$

$1600 < Re$ $$s = 1.83\, g^{0.5}\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{0.5} d^{0.5}$$

while fig. 2.4 shows the settling velocities of discrete spherical particles as function of their size and mass density at a temperature of 10 ^{0}C. In nature, however, spherical particles are an exception and generally their shape is quite irregular. With the same volume and weight this means a larger projected area in the direction of motion and a higher value of the drag coefficient c_D under turbulent flow conditions. By both phenomena, the settling rate will be smaller to much smaller than follows from the formulae given above.

In drinking water practice, intakes for surface water are constructed in such a way that only a minimum amount of suspended matter is entrained. The turbidity still present in the raw water is caused by small particles of sand or silt and by somewhat larger flocs of a low mass density, which even after chemical coagulation or precipitation settle well within the laminar region (fig. 2.4). According to the formula concerned, the settling rate is now pro-

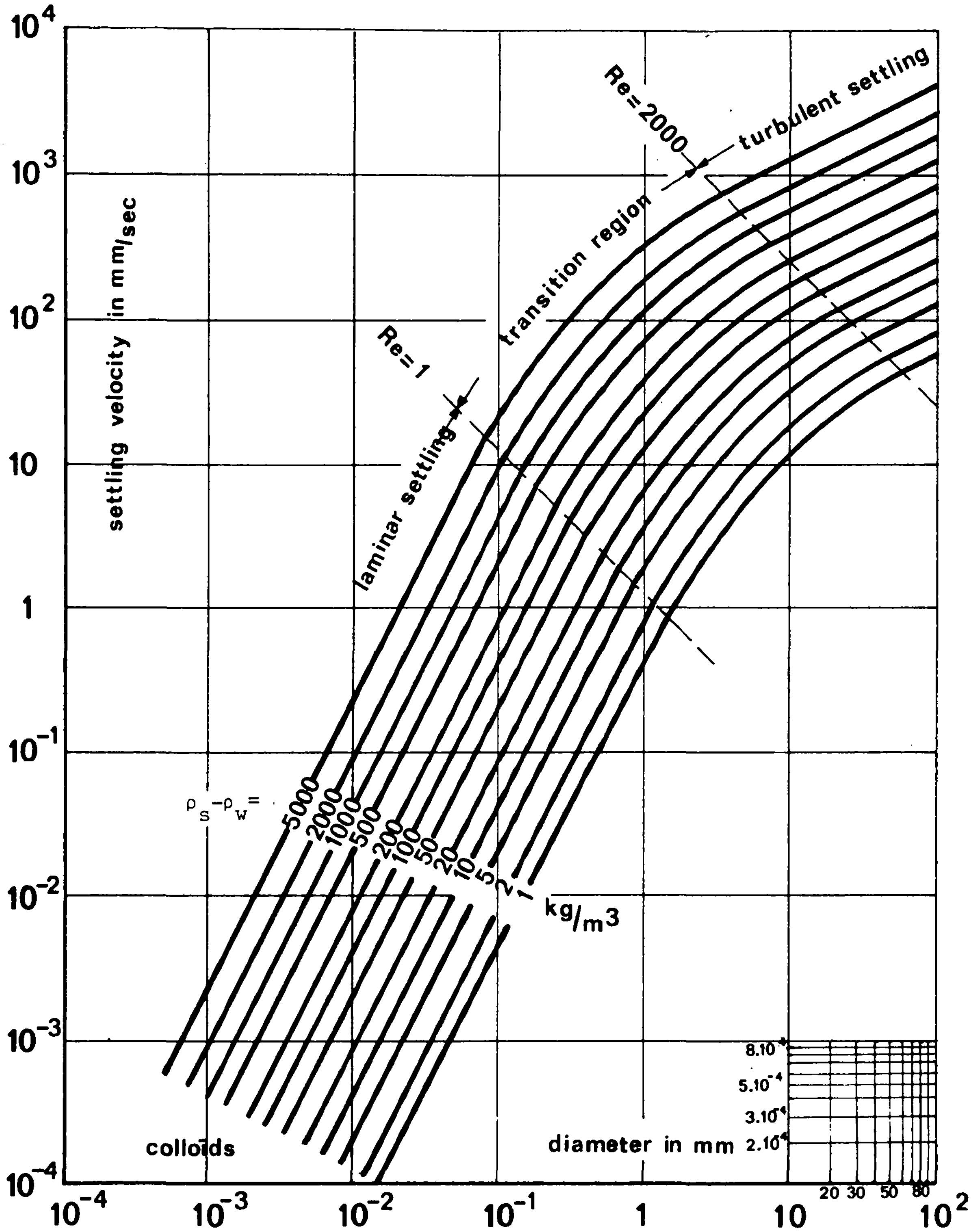


Fig. 2-4 Settling velocities of discrete spherical particles in quiescent water at 10°C.

portional to the square of the diameter. When by chemical coagulation the particle size is increased by a factor 10, the rate of subsidence will grow by a factor 100, greatly enhancing clarification efficiency. According to the same formula, the settling rate depends on the viscosity, that is on the temperature of the water (fig. 2.5). When from summer to winter this temperature drops from 25 to 0 ^{0}C, the kinematic viscosity will increase by a factor 2, reducing the settling rate in winter to only 50% of that in summer with a corresponding decrease in clarification efficiency.

In waste water treatment plants, all substances must be handled which the public and industry think fit for disposal into the sewerage system. This not only means a much higher suspended load, but also the presence of larger or heavier particles. In grit chambers these properties combine, resulting in turbulent settling. The downward velocity of the particle is now independent of the viscosity, that is of the temperature of the water and increases with the root of the diameter only. When by chemical coagulation the particle size is again increased by a factor 10, the rate of subsidence will grow by a factor 3, giving only a limited increase in sedimentation efficiency. In primary tanks, settling occurs in the laminar as well as in the transition region, the influence of viscosity and particle size being larger as the Reynolds number is smaller. In secondary tanks finally the flocs may have a fairly large size, but their mass density differs only little from that of the surrounding fluid and settling is therefore almost entirely laminar.

At the beginning of the settling process, the particle is at rest. In a still fluid it will then start to move down, gradually increasing its velocity untill the terminal values calculated above for steady movement are attained. During this process of acceleration, the settling rates are consequently smaller and this might reduce basin efficiency when the unsteady period is of any greater duration. During this period the movement of the particle is governed by

force = mass × acceleration

$$F_i - F_d = \rho_s V \frac{ds'}{dt}$$

with s' as rate of subsidence. For laminar settling of spheres

$$F_i = (\rho_s - \rho_w) g \frac{\pi}{6} d^3$$

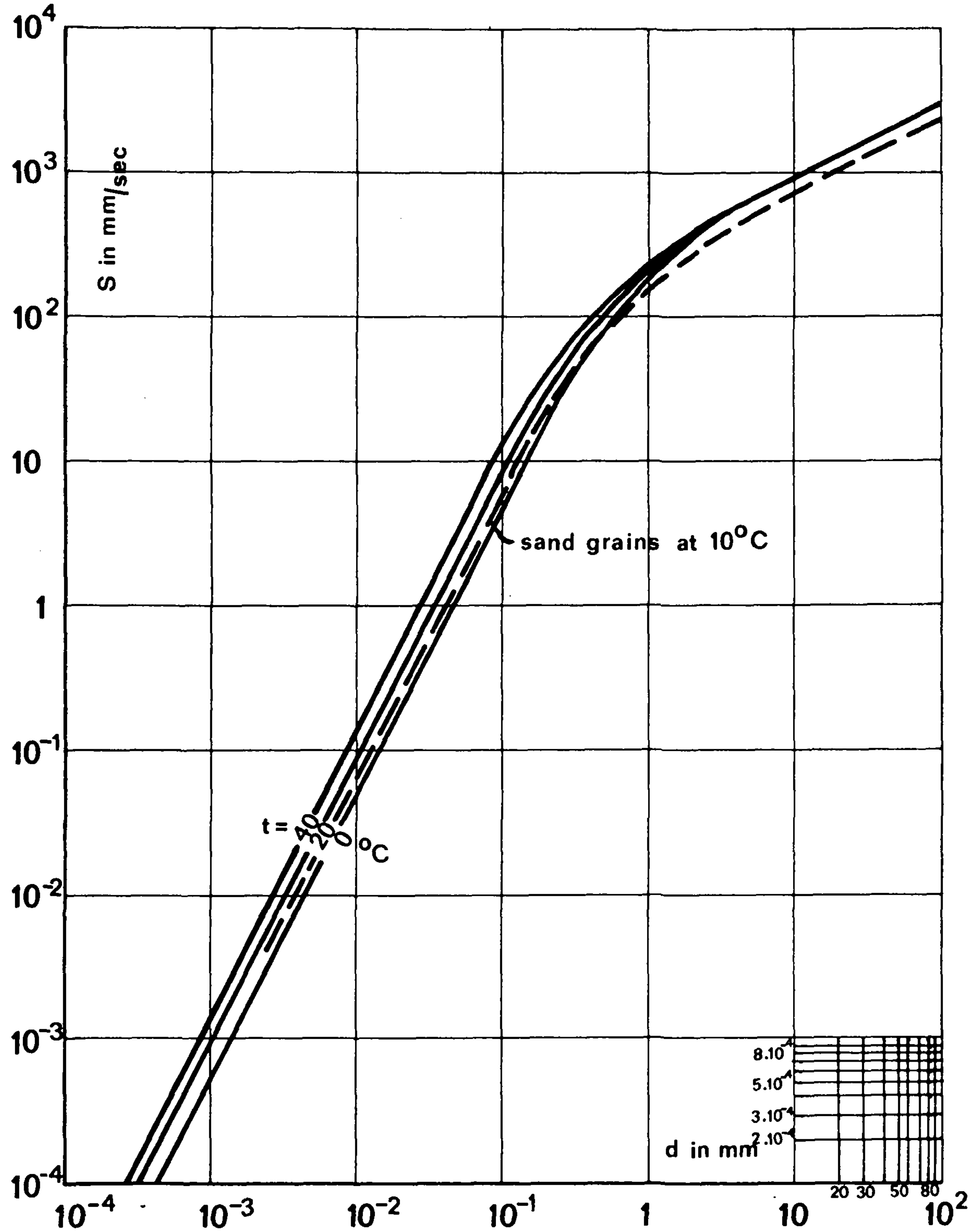


Fig. 2-5 Settling velocities of discrete spherical sand grains ($\rho_s - \rho_w = 1650$ kg/m^3) in quiescent water at different temperature.

$$F_d = 3\pi\nu\rho_w s'd$$

$$V = \frac{\pi}{6} d^3 \qquad \text{, substituted}$$

$$\frac{ds'}{dt} = \frac{\rho_s - \rho_w}{\rho_s} g - 18 \frac{\rho_w}{\rho_s} \frac{\nu}{d^2} s'$$

With the terminal velocity for laminar settling equal to

$$s = \frac{1}{18} \frac{g}{\nu} \frac{\rho_s - \rho_w}{\rho_w} d^2$$

this may be simplified to

$$\frac{ds'}{s - s'} = 18 \frac{\rho_w}{\rho_s} \frac{\nu}{d^2} dt = \alpha dt$$

Integration with the boundary condition

$$t = 0 \ , \quad s' = 0 \quad \text{finally gives}$$

$$s' = s(1 - e^{-\alpha t})$$

According to this formula, the particle will never obtain the full terminal velocity s. A value of 0.999 s, however, is reached at a time t determined by

$$e^{-\alpha t} = 0.001 = e^{-6.9}$$

$$t = \frac{6.9}{\alpha} = \frac{6.9}{18} \frac{\rho_s}{\rho_w} \frac{d^2}{\nu}$$

With flocculated mud particles of 1 mm diameter and a mass density of 1010 kg/m^3 in water at 10 ^0C

$$t = \frac{6.9}{18} \frac{1010}{1000} \frac{10^{-6}}{(1.31)10^{-6}} = 0.3 \text{ sec}$$

With normal settling times varying from minutes to hours, this delay is completely negligeable.

2.2. Hindered settling

Without saying so expressly, it has been assumed in the preceding section that the particle settles in a fluid of infinite extend. In reality, however, the cross-sectional area of the settling tank is always limited, meaning that the water displaced by the particle must flow back, upward along the downward moving particle. Compared to the water, the settling velocity remains at the value s calculated above. Compared to the stationary walls of the container, however, the settling rate is reduced to

$$s' = s - v_d$$

with v_d as displacement velocity of the fluid. In fig. 2.6 it is assumed that a single spherical particle of diameter d settles in the cylindrical container of diameter D, giving as continuity equation

$$s'\frac{\pi}{4}d^2 = v_d\left(\frac{\pi}{4}D^2 - \frac{\pi}{4}d^2\right) \quad \text{or}$$

$$v_d = s'\frac{d^2}{D^2 - d^2} \quad \text{and}$$

$$\frac{s'}{s} = 1 - \left(\frac{d}{D}\right)^2$$

According to this formula, the reduction in settling rate is less than 1% in case

$$D > 10d$$

The velocity distribution sketched in fig. 2.6 in the meanwhile may be an acceptable approximation under highly turbulent conditions. When the upward flow of water along the downward moving particle is laminar, however, the velocity field will be more as shown in fig. 2.7 where the reduction in settling rate is much higher. According to calculations and experiments, this reduction is now less than 1% in case

$$D > 100d,$$

a condition which is always fullfilled, even in laboratory tests using cylindrical containers of 0.2 m diameter only.

In case a fluid contains a number of particles, in a volumetric

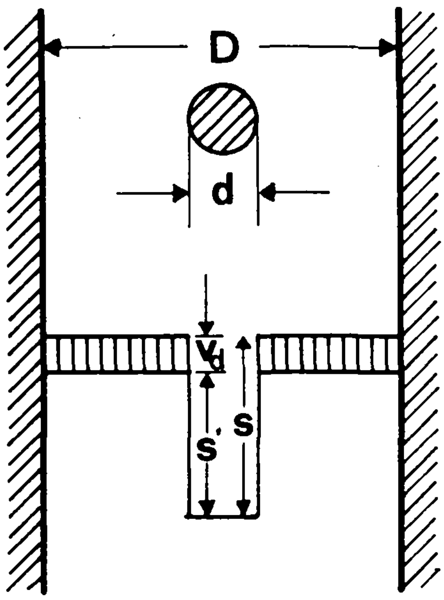


Fig. 2-6 Displacement velocity field under highly turbulent conditions.

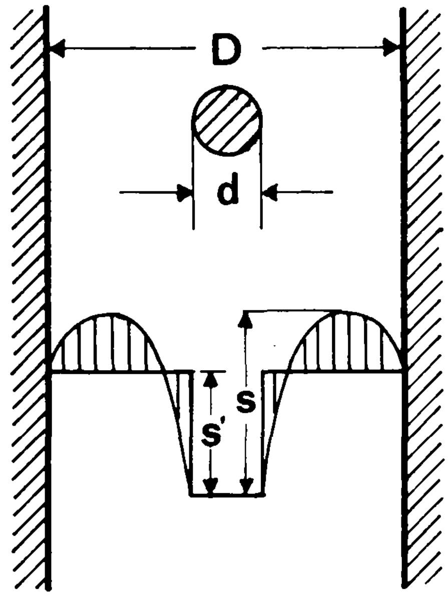


Fig. 2-7 Displacement velocity field under laminar flow conditions.

concentration c_v, a further reduction in effective settling rate will take place. Going out from the velocity field of fig. 2.6, the continuity equation gives

$$s'c_v = v_d(1 - c_v)$$

With $s' = s - v_d = s - s' \frac{c_v}{1 - c_v}$

the reduction in settling rate becomes

$$\frac{s'}{s} = 1 - c_v$$

With the velocity field of fig. 2-7 and an entrainment of water the subsiding particle, this reduction will be much larger. According to the experimental results shown in fig. 2-8.

$$\frac{s'}{s} = 1 - fc_v^{2/3} \quad \text{with}$$

turbulent settling spheres	f = 1.2
turbulent settling sand grains	1.4
laminar settling spheres	2.0
laminar settling red blood cells	2.5
laminar settling lime and alumn floc	2.8

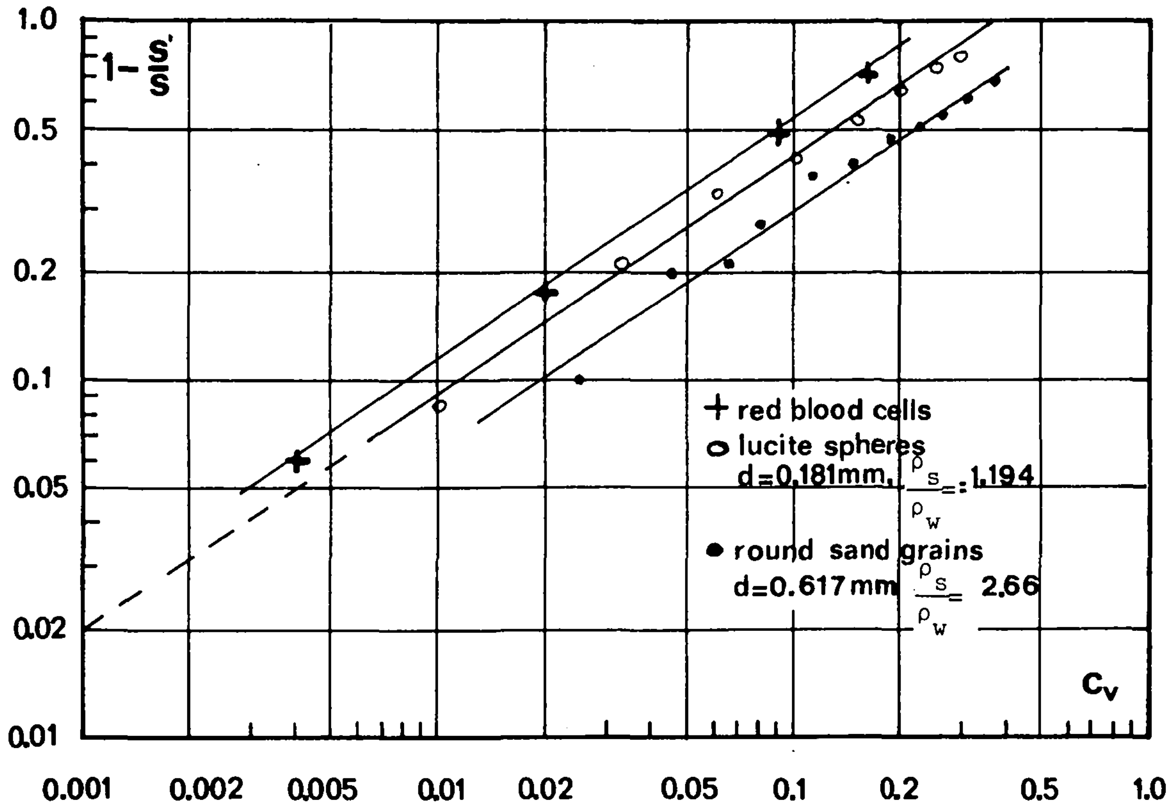


Fig. 2-8 **Reduction** in settling velocity.

With the average value of f at 2.0, a 5% reduction in settling velocity must be expected when the volumetric concentration of the suspended particles equals 0.004 or 4000 ppm corresponding with a gravimetric concentration somewhere between 200 ppm for silt containing 95% water and 1500 ppm for sand grains. In drinking water practice such values are seldom surpassed, but in settling tanks following the activated sludge process volumetric concentrations up to 20000 ppm occur, reducing the settling rate by 15 to 25%.

2.3. Frequency distribution of settling velocities

In a natural water, particles of various sizes, shapes and mass densities will be present, all having a different settling velocity. With the formulae of the preceding sections, the frequency distributions of these settling velocities could be calculated when the basic factors of volume, weight and shape of the suspended particles are known. Mostly, however, these factors are unknown, while their determination would prove to be very difficult or even impossible. This is the reason that the frequency distribution of the velocities at which the various particles move down is measured directly in the laboratory, disregarding completely the mathematical theory of settling. The apparatus used is shown in fig. 2.9 and consists of a cylindrical container, nowadays made of clear plastic, with a diameter of 0.15-0.3 m and for discrete settling with a depth of 1-2 m. By means of a water bath, the temperature of the container is kept constant, while at various depths water samples can be taken. The container is filled with a representative sample of the suspension to be tested and stirred gently to distribute the particles evenly over the full depth. The test starts when the water has come to rest. At this moment, and at various time intervals thereafter,

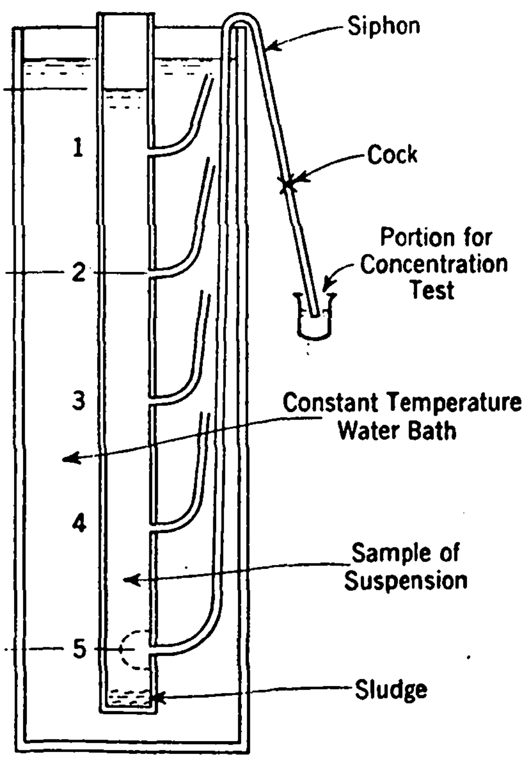


Fig. 2-9 Apparatus for quiescent settling analysis.

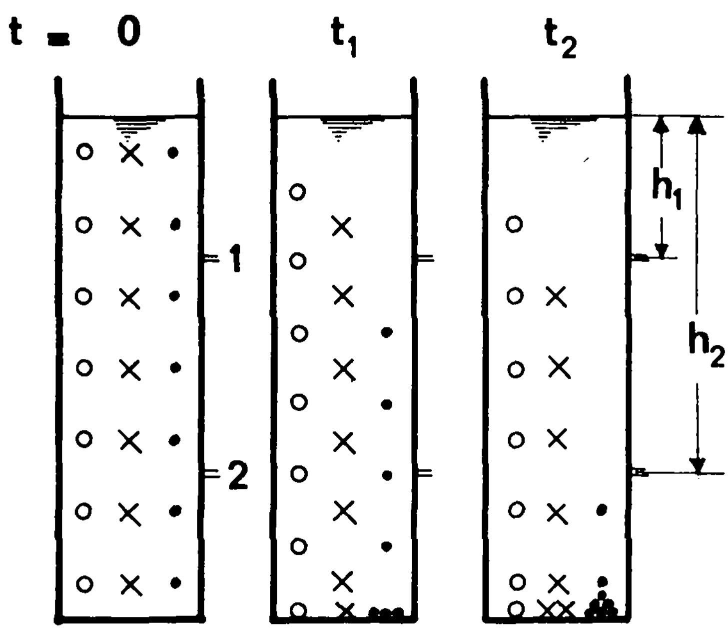


Fig. 2-10 Quiescent settling test.

water samples are taken at different depths and analysed for suspended solids, turbidity, color, iron, alumina, hardness, chemical and bio-chemical oxygen demand or any other index that is reduced by settling. The principle of this experiment is shown in fig. 2.10, where 3 types of particles with 3 different settling velocities have been assumed. With each settling velocity at a constant value during the whole test, the downward movement of the particles is uniform, meaning that a sample for instance taken at time t and depth h_1 does not contain any particles, but the concentration of x and 0 particles is exactly the same as in the original suspension. As regards these concentrations, the results of the settling test shown in fig. 2.10 may be summarized as follows

time	depth	particles present
t_1	h_2	0 × ●
t_1	h_1	0 ×
t_2	h_2	0 ×
t_2	h_1	0

from which the individual concentrations may be found by subtraction. As regards the settling velocities of the various particles, the results are less exact

● particles, $\frac{h_1}{t_1}$, $\frac{h_2}{t_2} < s < \frac{h_2}{t_1}$

× particles, $\frac{h_1}{t_2} < s < \frac{h_1}{t_1}$

0 particles, $s < \frac{h_1}{t_2}$

In a natural water, however, a large variety of particles will occur and the frequency distribution of their settling velocities is consequently a smooth curve. When again a sample is taken at depth h and time t, no particles with a settling velocity in excess of h/t will be found, while all particles with a settling velocity less then h/t are present in their original concentration. The observation thus reads
amount at depth h, time t-s h/t.
Provided that an adequate number of samples is taken and analyzed, an easy and accurate determination of the settling velocities can be obtained in this way. For the test results of table 2.1, the frequency distribution of the settling velocities is shown in fig. 2.11.

Table 2.1 Settling analysis for a suspension of discrete particles.

t	0	900	1800	2700	3600	5400	7200	sec
h = 0.5 m								
h/t		0.556	0.278	0.185	0.139	0.093	0.069	10^{-3} m/sec
C	86 = C_o	57	25	8	3	1	0	ppm
100 C/C_o	100	66	29	9	4	1	0	%
h = 1.25 m								
h/t		1.389	0.695	0.463	0.347	0.232	0.174	10^{-3} m/sec
C	86	83	63	49	37	16	6	ppm
100 C/C_o	100	96	73	57	42	19	7	%

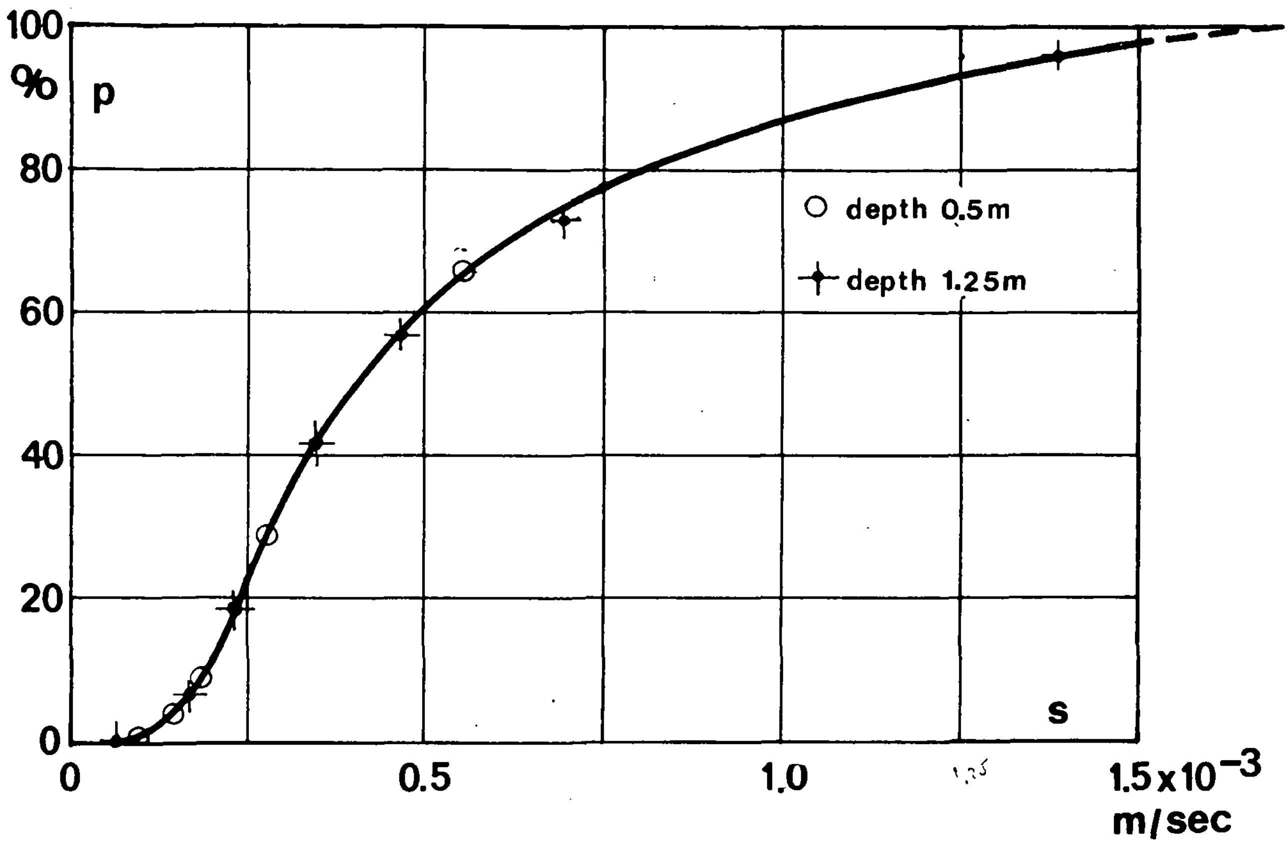


Fig. 2-11 Cumulative frequency distribution of settling velocities calculated from the data of table 2.1.

2.4. Quiescent settling

When water is left standing for a time T_o in a tank of depth H, all particles with a settling velocity larger than

$$s_o = \frac{H}{T_o}$$ have completely disappeared.

Particles with a settling velocity $s < s_o$ are only partly removed, only from the amount of water which at the beginning of the settling process is within a distance $h' = sT_o$ from the bottom. The removal ratio for these particles consequently equals

$$\frac{h'}{H} = \frac{s\,T_o}{s_o T_o} = \frac{s}{s_o}$$

With the notations of fig. 2.12 to the left, the over-all removal thus becomes

$$r = (1 - p_o) + \int_0^{p_o} \frac{s}{s_o}\, dp \qquad \text{or}$$

$$r = (1 - p_o) + \frac{1}{s_o} \int_o^{p_o} s\,dp$$

in which the integral represents the shaded area. As shown in fig. 2.12 to the right, this removal ratio can easily be found graphically by drawing a horizontal line in such a way that the two shaded areas are equal. For the particles of fig. 2.11, the removal ratio r as function

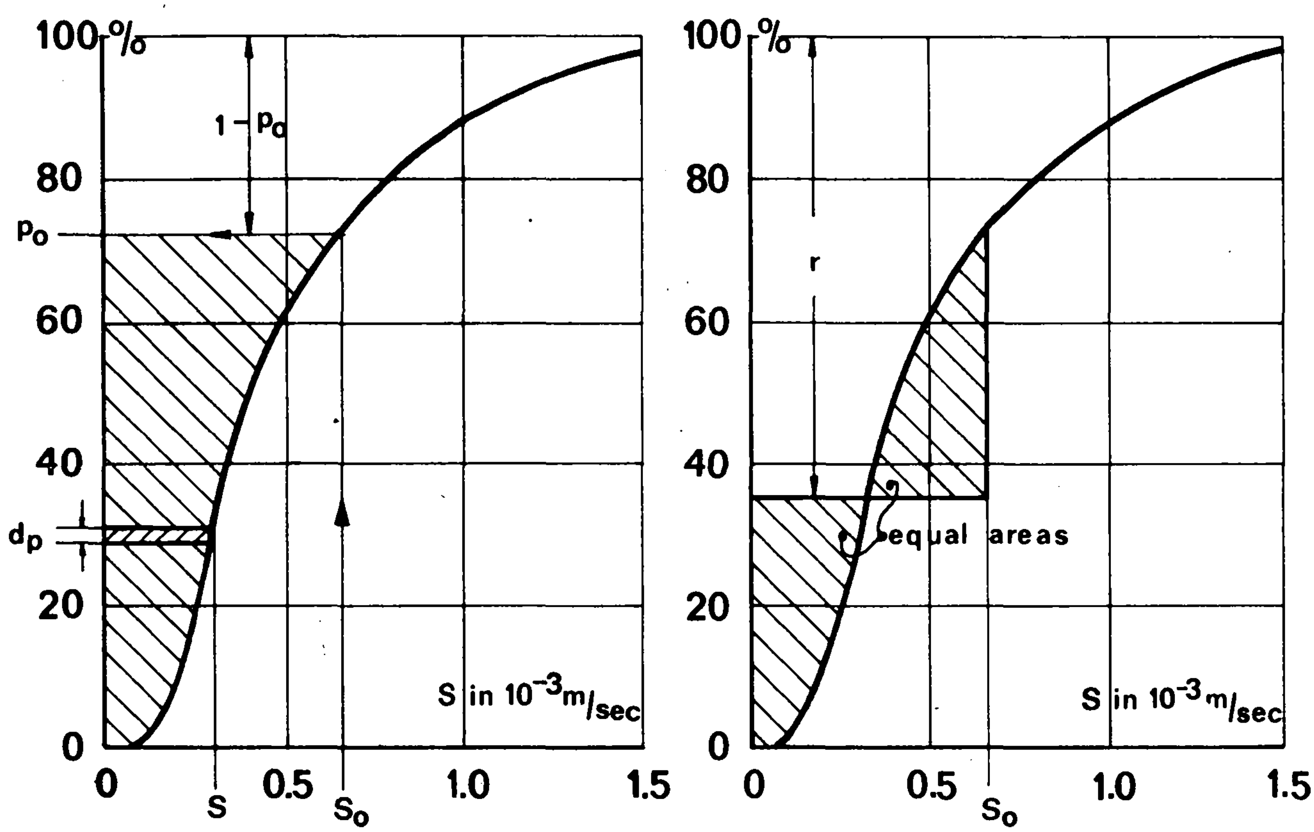


Fig. 2-12 Cumulative frequency distribution of settling velocities.

of $s_o = H/T_o$ is shown in fig. 2.13.

When an amount of Q m^3/sec must be clarified by settling in the fill-and-draw tanks of fig. 1.4, a detention time T_o asks for a volume

$$V = QT_o = AH$$

with A as surface area and H as depth of the basin. According to the calculations given above, the removal ratio is a function of the factor

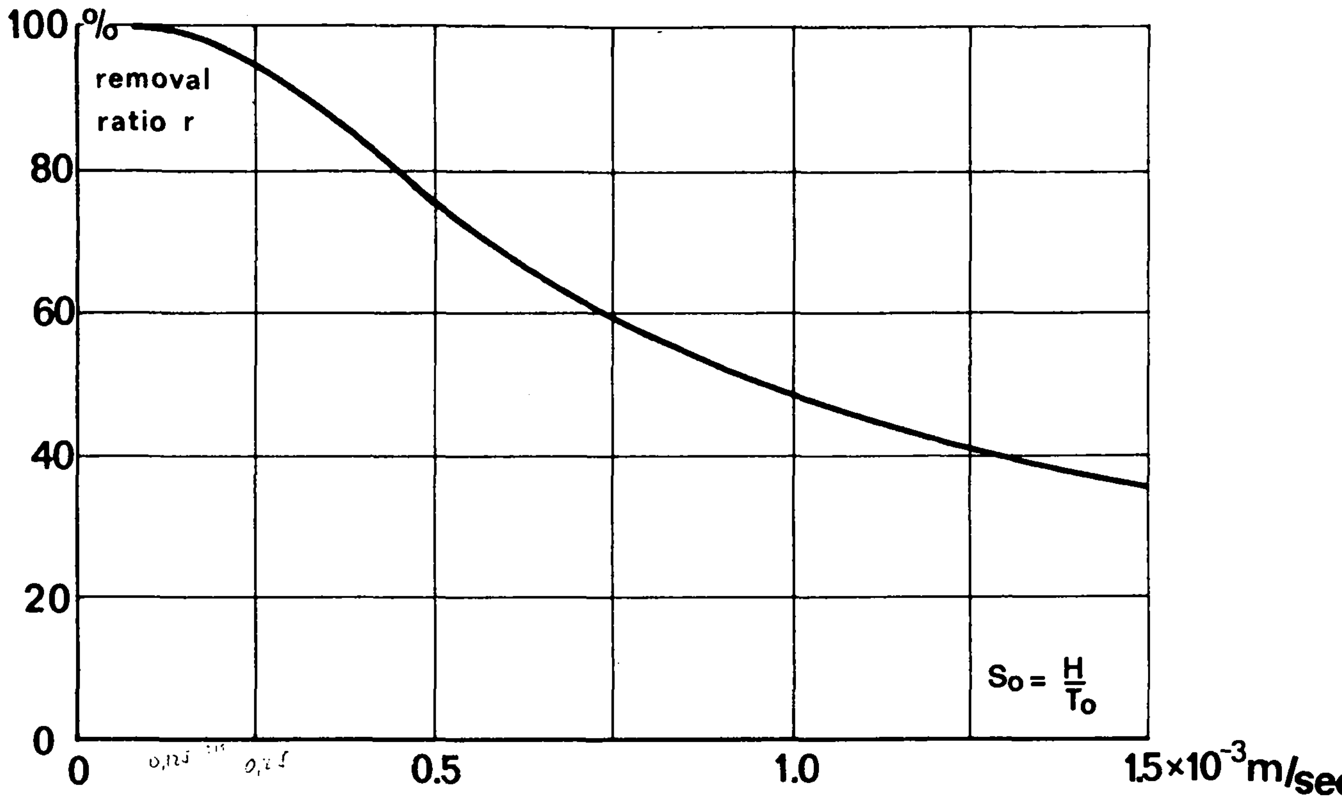


Fig. 2-13 Removal ratio for the particles shown in fig. 2-11.

$$s_o = \frac{H}{T_o} = \frac{Q}{AH} H = \frac{Q}{A}$$

With a fixed capacity Q, the removal ratio consequently only depends on the surface area A and is independent of the tank depth H. For a capacity Q = 0.5 m^3/sec and settling properties of the suspended particles as shown in fig. 2.11, the influence of tank surface on removal ratio is shown in fig. 2.14. When the removal ratio is low, already a slight increase in tank area gives an appreciable improvement. For a further augmentation of high efficiencies, however, excessive increases in tank area are needed.

The amount of suspended matter remaining in the tank decreases with time and will be larger at a greater depth below the surface. At a time t water at a depth h below this surface cannot contain particles with a settling velocity in excess of s = h/t, while the concentration of particles with a lower settling velocity has remained unchanged. This concentration may consequently be read as p = f(s) from fig. 2.11.

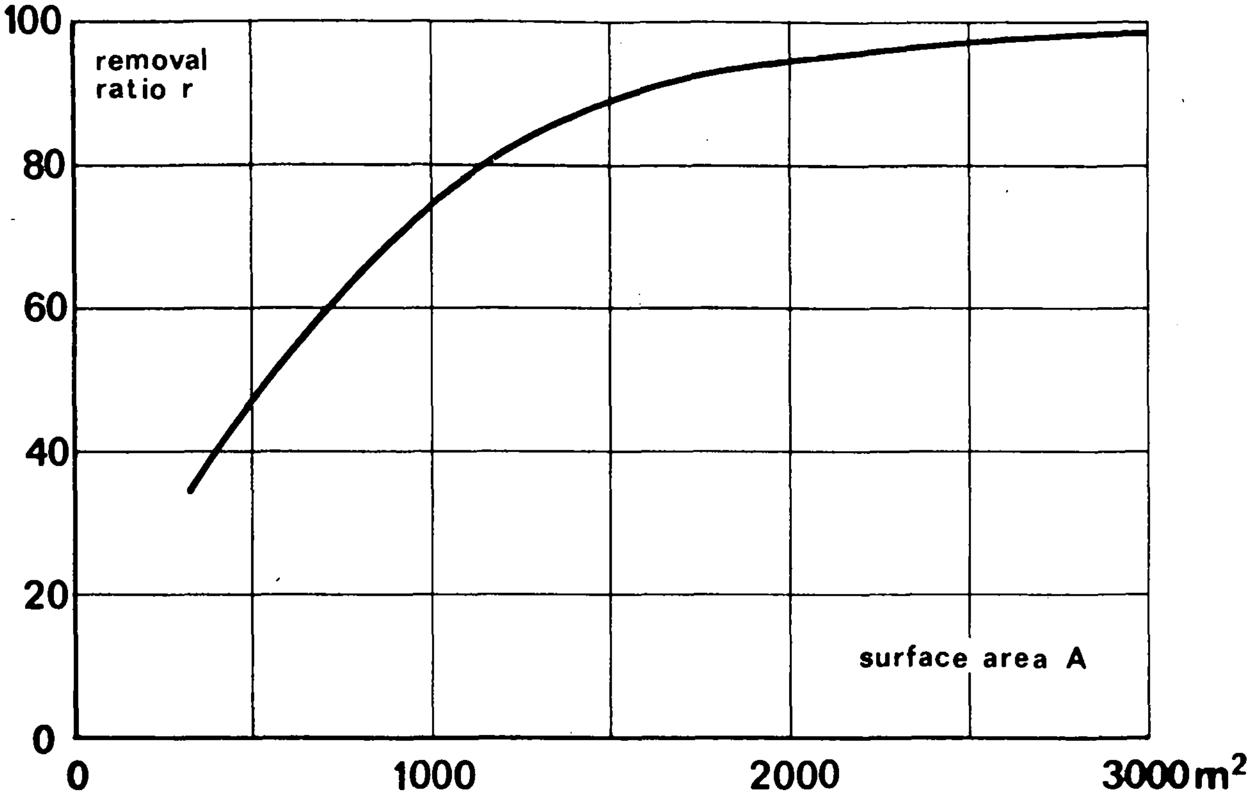


Fig. 2-14 Removal ratio as function of the surface area for the treatment of the suspension shown in fig. 2-11 in an amount of 0.5 m^3/sec.

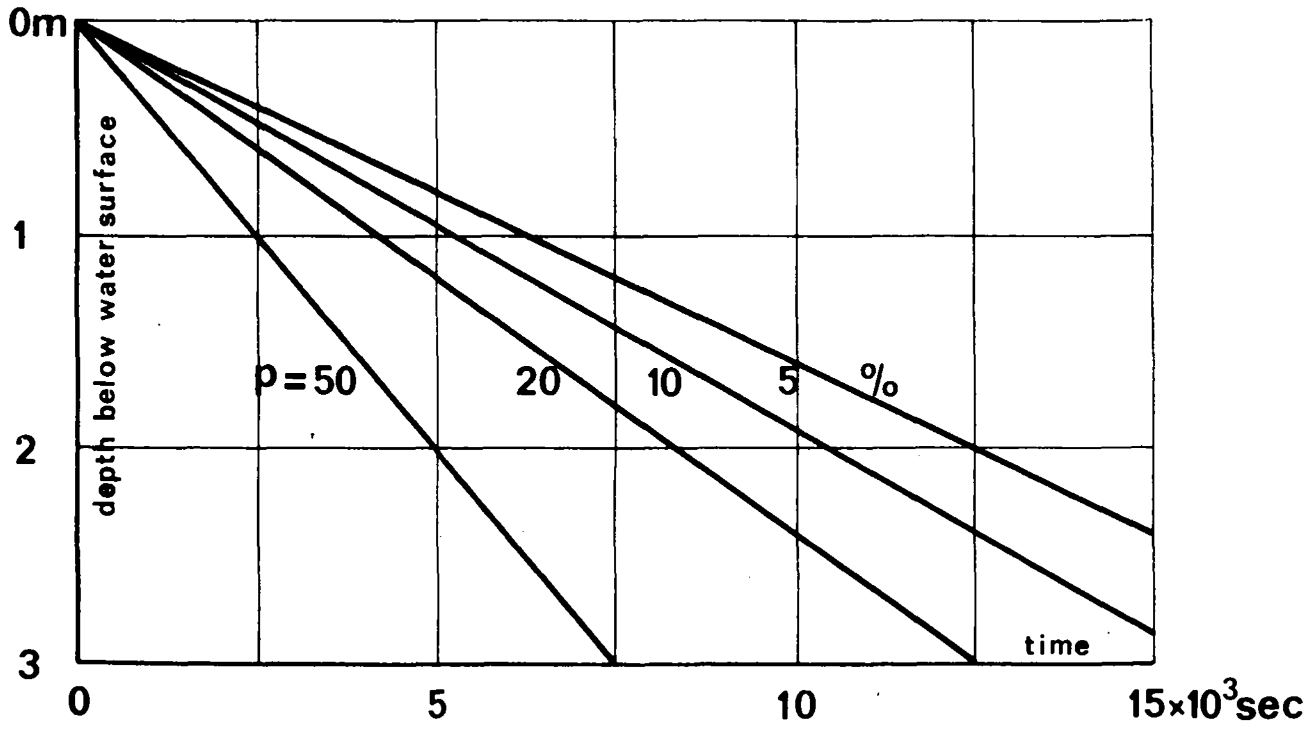


Fig. 2-15 Clarification of the suspension from fig. 2-11 as function of time and depth.

The lines of equal concentration as function of time and depth are shown in fig. 2.15, while fig. 2.16 pictures the cumulative frequency distribution of settling velocities in the tank effluent after 90% of the material from fig. 2.11 has been removed.

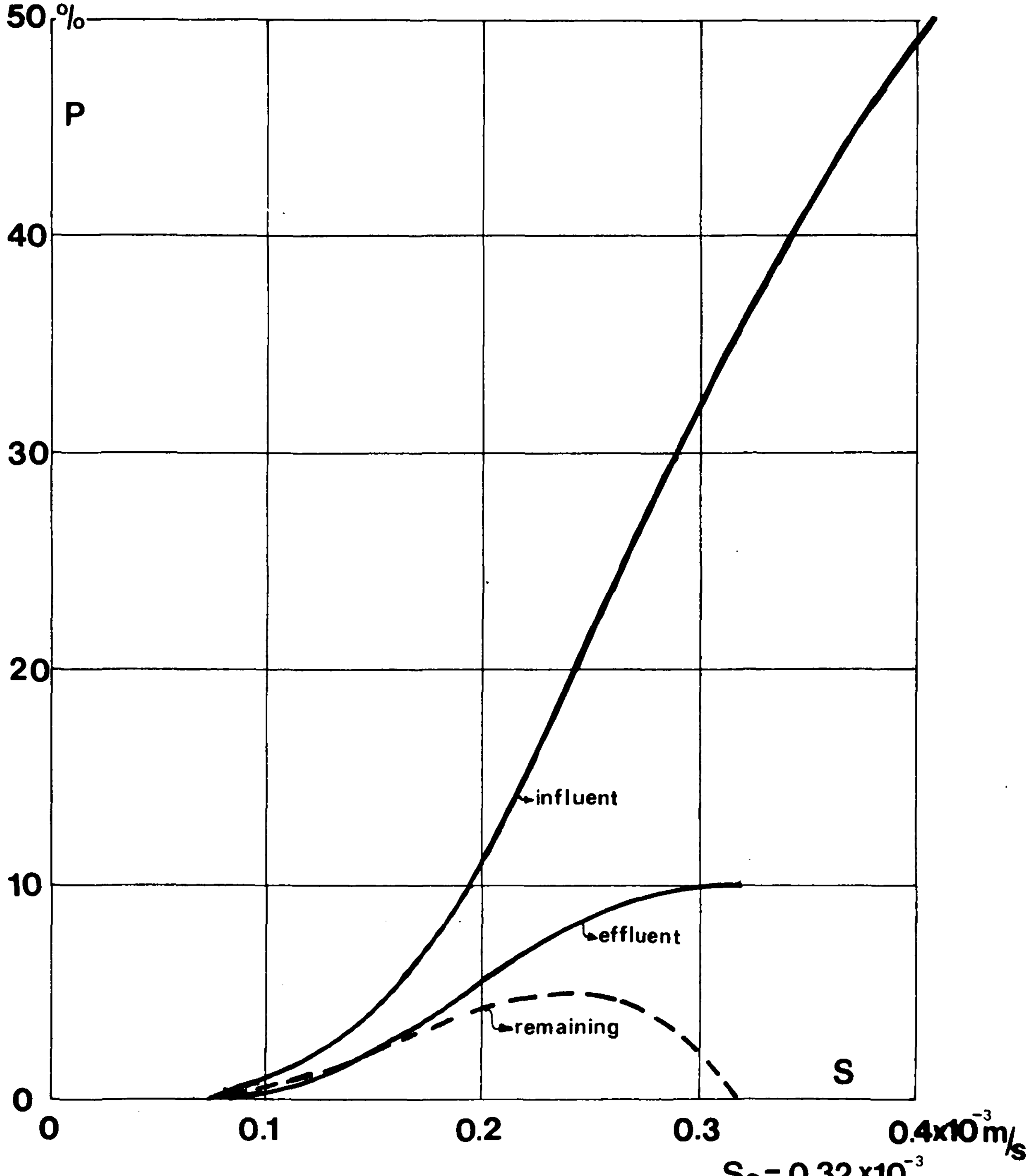


Fig. 2-16 Cumulative frequency distribution in effluent after removing 90% of the suspended matter shown in fig. 2-11.

2.5. Settling tests in a cone-shaped vessel

The amount of settleable material present in a suspension can be determined with a simple test, using a container with a volume V of usually 1 liter and a height H of commonly 0.4 m, in which a representative sample is left standing for a time T_o, varying from one case to another between $\frac{1}{4}$ and 2 hours. With a cylindrical container (fig. 2.17, left), the amount of sludge accumulated at the bottom equals rV, in which the removal ratio r is a function of $s_o = H/T_o$. For the suspension of fig. 2.11, this relation $r = f(s_o)$ is shown in fig. 2.13. To facilitate the reading of the sludge volume, however, commonly cone-shaped vessels are applied (fig. 2.17, right). Here the depth varies from 0 to H by which the removal ratio will be higher and the sludge accumulation S larger than with a cylindrical vessel of the same volume and height. With the notations of fig. 2.18

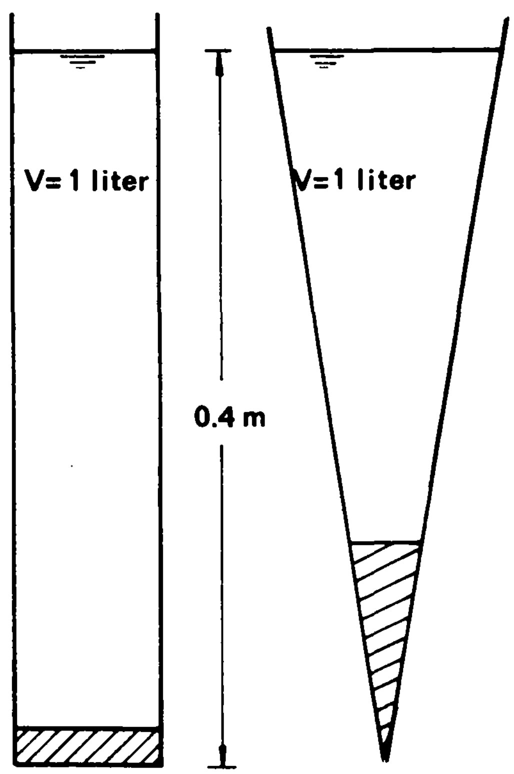


Fig. 2-17 Settling tests.

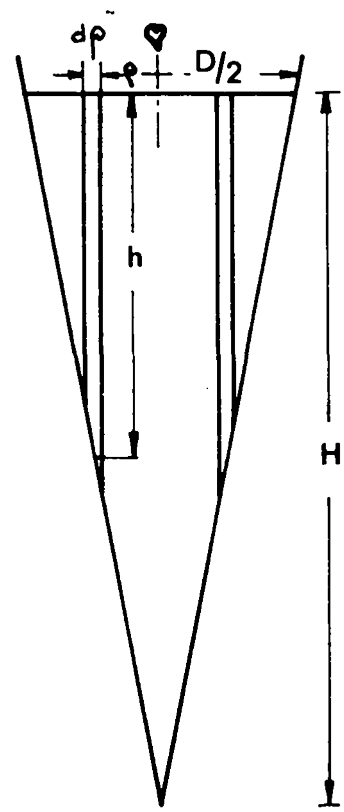


Fig. 2-18 Settling in cone-shaped vessel.

$$dS = 2\pi\rho/d\rho/hr \qquad \text{or with}$$

$$\rho = \frac{D}{2}\left(1 - \frac{h}{H}\right) \text{ , } d\rho = -\frac{D}{2H}\,dh$$

$$dS = -\frac{\pi}{2}\frac{D^2}{H^2}(H - h)hrdh$$

The relation between r and $s_o = h/T_o$ can always be represented by

$$r = 1 - \alpha_1 \frac{h}{T_o} - \alpha_2\left(\frac{h}{T_o}\right)^2 - \ldots - \alpha_n\left(\frac{h}{T_o}\right)^n$$

With $s_o < (0.5)10^{-3}$ m/sec, a good approximation for the relation shown in fig. 2.13 can be obtained by

$$r = 1 - 10^6\left(\frac{h}{T_o}\right)^2 \qquad \text{Substituted}$$

$$dS = -\frac{\pi}{2}\frac{D^2}{H^2}(H - h)h\left(1 - 10^6\frac{h^2}{T_o^2}\right)dh$$

and integrated between the limits $h = 0$ and $h = H$

$$S = \frac{\pi D^2}{4}\frac{H}{3}\left(1 - \frac{3}{10}10^6\frac{H^2}{T_o^2}\right)$$

The volume of the vessel equals

$$V = \frac{\pi D^2}{4}\frac{H}{3} \quad \text{, substituted}$$

$$\frac{S}{V} = 1 - \frac{3}{10}10^6\frac{H^2}{T_o^2}$$

while a cylindrical vessel would have given

$$\frac{S}{V} = r = 1 - 10^6\frac{H^2}{T_o^2}$$

With $H = 0.4$ m and $T_o = 15$ minutes $= 900$ seconds, this gives

	cylindrical	cone shaped
$\frac{S}{V}$ =	0.80	0.94

or with a volumetric concentration of 3000 ppm and V equal to one liter

S =	2.40	2.82 cm^3

a difference of nearly 20%.

3. Discrete settling in continuous horizontal flow basins

3.1. Introduction

As mentioned before, quiescent settling in fill-and-draw tanks is no longer applied in the field of public water and waste water engineering and here continuous-flow basins are used almost exclusively. In such basins, the suspended particles settle within the flowing liquid, which might have any direction of movement. The most important sub-division, however, is between horizontal and vertical flow tanks. As will be shown in the subsequent sections, the flow of water in horizontal basins has only a limited influence on the settling process, giving a clarification efficiency approaching that of quiescent settling. With vertical flow basins on the other hand, the movement of water is of paramount importance. In the vertical flow tank of fig. 3.1 all particles with settling velocity less than

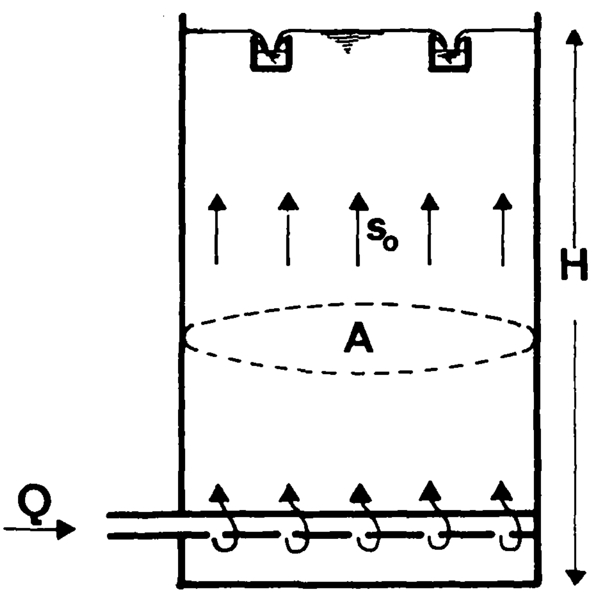


Fig. 3-1 Vertical flow settling tank.

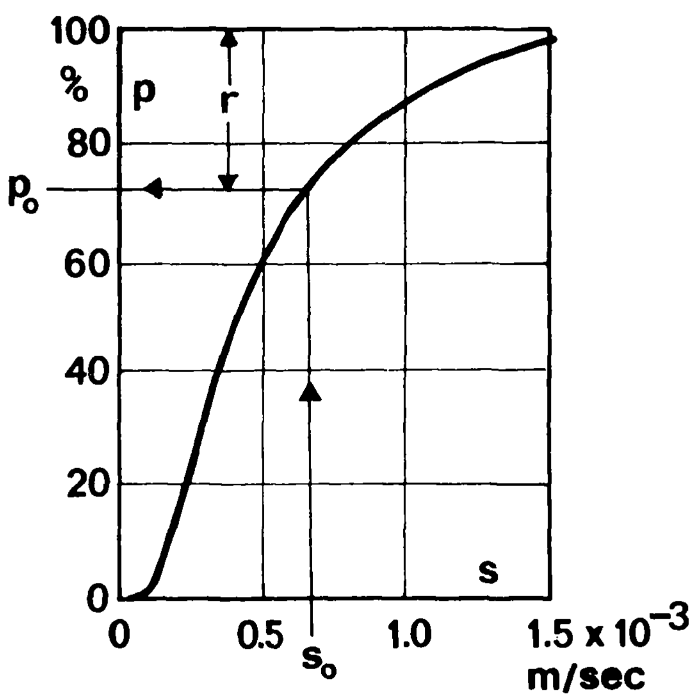


Fig. 3-2 Settling efficiency of vertical flow tanks.

$$s_o = \frac{Q}{A} = \frac{H}{T_o}$$

are entrained by the flowing water and cannot be kept back, limiting the removal ratio r to 1 - p_o in fig. 3.2. With horizontal flow tanks of the same dimensions, particles with a settling velocity $s < s_o$ are still partly retained, increasing the removal ratio to the value of r in fig. 2.12 right. Horizontal flow tanks moreover are easier and cheaper to construct and for the plain sedimentation of discrete particles, such tanks are therefore used without exception.

To develop a clarification theory for the removal of discrete particles in continuous horizontal flow basins, it is first assumed that settling in such tanks takes places under ideal circumstances (section 3.2). Conditions in real tanks differ more or less from these assumed for the ideal basin. The resulting reduction in basin efficiency will be studied separately in section 3.3 to 3.5 inclusively.

3.2. Settling in an ideal horizontal flow basin

In an ideal horizontal flow basin, settling is supposed to take place in exactly the same way as in a quiescent settling tank, without any influence of the horizontal water movement. To approach this ideal as nearly as possible, a rectangular horizontal flow tank must show the following characteristics

a. the direction of flow is horizontal and the velocity of flow is the same in all parts of the basin. This velocity moreover is so small that the horizontal water movement occurs under streamline flow conditions. As a result, the retention time for each particle of water is the same, equal to the volume of the tank divided by the rate of discharge;
b. at the basin inlet the concentration of suspended particles of each size is the same in all points of the vertical plane perpendicular to the direction of flow;
c. a particle is removed and stays removed when it strikes the bottom of the tank.

These requirements mean that in a continuous horizontal flow basin four zones must be present (fig. 3.3)

a. an inlet zone to disperse influent flow and suspended matter uniformly over the full cross-sectional area of the basin;

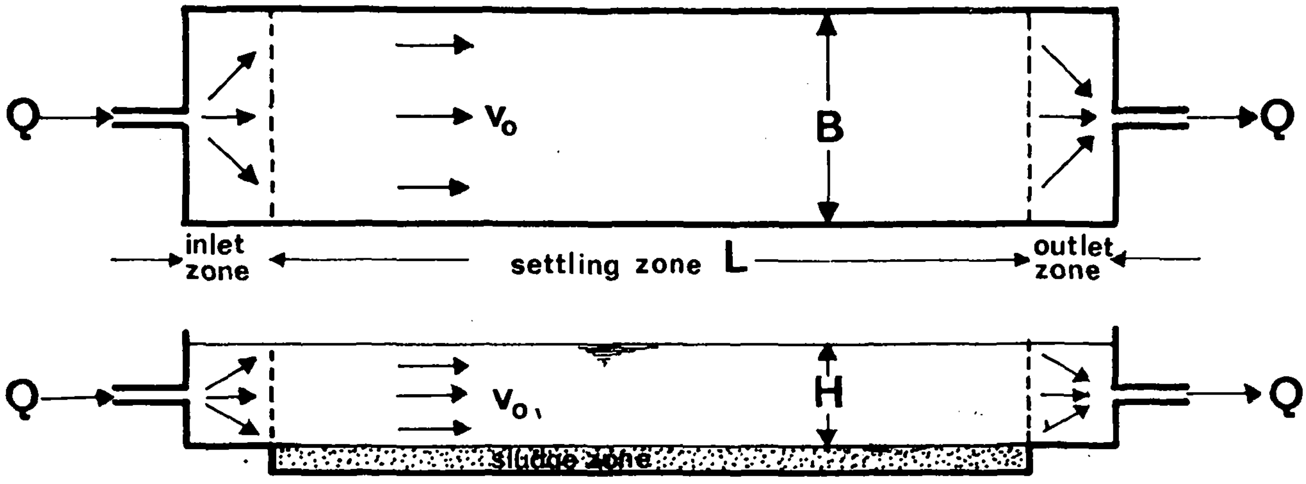


Fig. 3-3 Rectangular horizontal flow settling tank.

b. a settling zone in which the suspended particles settle within the flowing water;

c. an outlet zone to collect the clarified liquid evenly over the cross-sectional area of the basin;

d. a sludge zone at the bottom to store the removed solids without the danger of re-suspension.

The actual work of a sedimentation basin, however, is restricted to the settling zone where the discrete particles trace straight paths, following the vector sum of the settling velocity s of the particle and the displacement velocity v_o of the liquid. The horizontal velocity v_o has been assumed constant over the whole settling zone, but the settling rate s depends on particle size, shape and mass density and will vary between wide limits. According to fig. 3.4 all particles with a settling velocity equal to or larger than the critical value s_o are completely removed, while for particles with a smaller settling velocity s the removal ratio amounts to

$$\frac{h}{H} = \frac{s}{s_o}$$

The total clarification effect equals (fig. 2.12)

$$r = (1 - p_o) + \int_0^{p_o} \frac{s}{s_o} \, dp$$

and depends on two factors, the frequency distribution for the settling velocities of the suspended particles and the value of s_o. Only the last factor can be influenced by the design of the tank. With the notations of fig. 3.4

$$\frac{s_o}{v_o} = \frac{H}{L} \quad \text{in which} \quad v_o = \frac{Q}{BH}$$

$$s_o = \frac{Q}{BL} = \frac{Q}{A}$$

with A as surface area of the tank. Derived by Hazen as far back as 1904, this formula states that for a specified suspension of discrete particles and unhindered settling, the efficiency of a continuous horizontal-flow sedimentation basin solely depends on the surface area and the rate of flow, which together constitute the surface loading or overflow rate s_o. The efficiency is independent of the

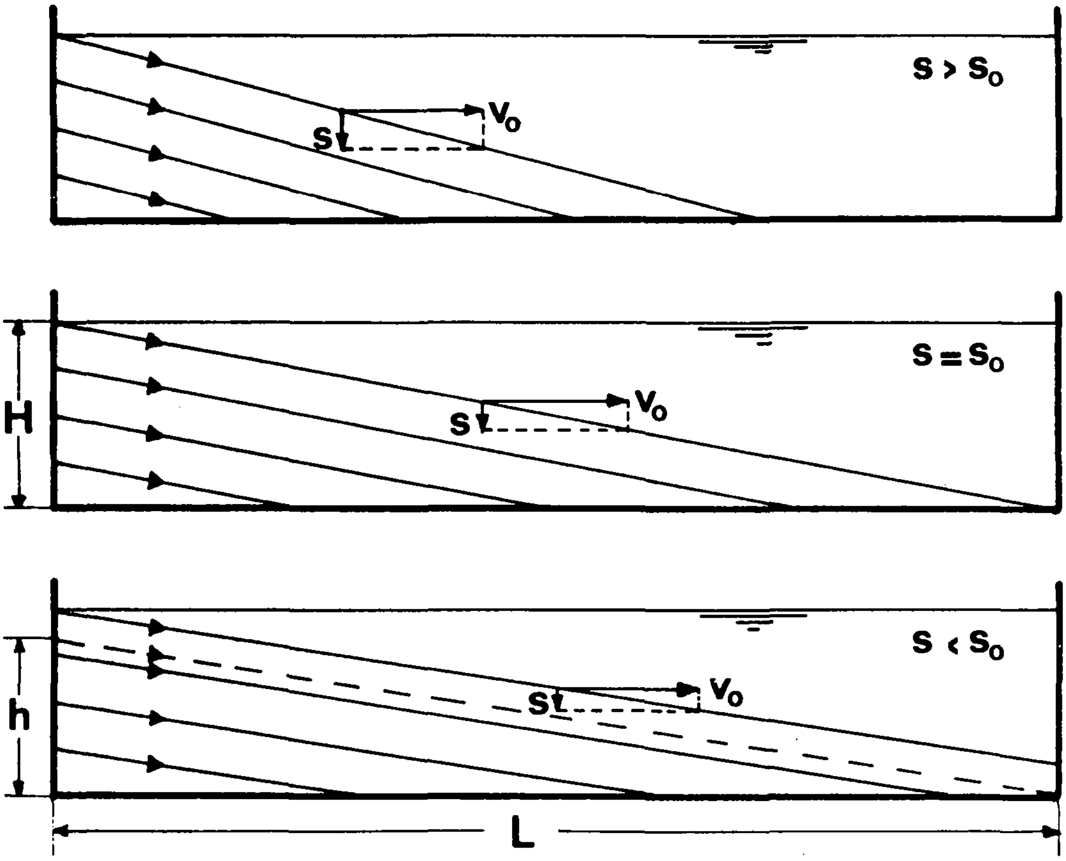


Fig. 3-4 Paths traced by discrete particles in a rectangular horizontal flow settling tank.

depth H of the basin and of the detention time T_o, results identical to those obtained in the preceding section with quiescent settling

Strictly speaking, the conclusions obtained above have been derived for horizontal flow settling tanks of rectangular plan. They hold true, however, for any shape of the tank provided that the depth H is constant. In the detention time T_o all particles reach the bottom of the tank and are removed from the flowing liquid when their settling velocity is equal to or larger than

$$s = \frac{H}{T_o}$$

With $T_o = \frac{A.H}{Q}$

$$s = \frac{Q}{A} = s_o$$

Next to rectangular tanks, circular tanks with centre feed or peripheral feed are used extensively (fig. 3.5), offering some advantages in construction. Under ideal conditions, their efficiency is the same as that

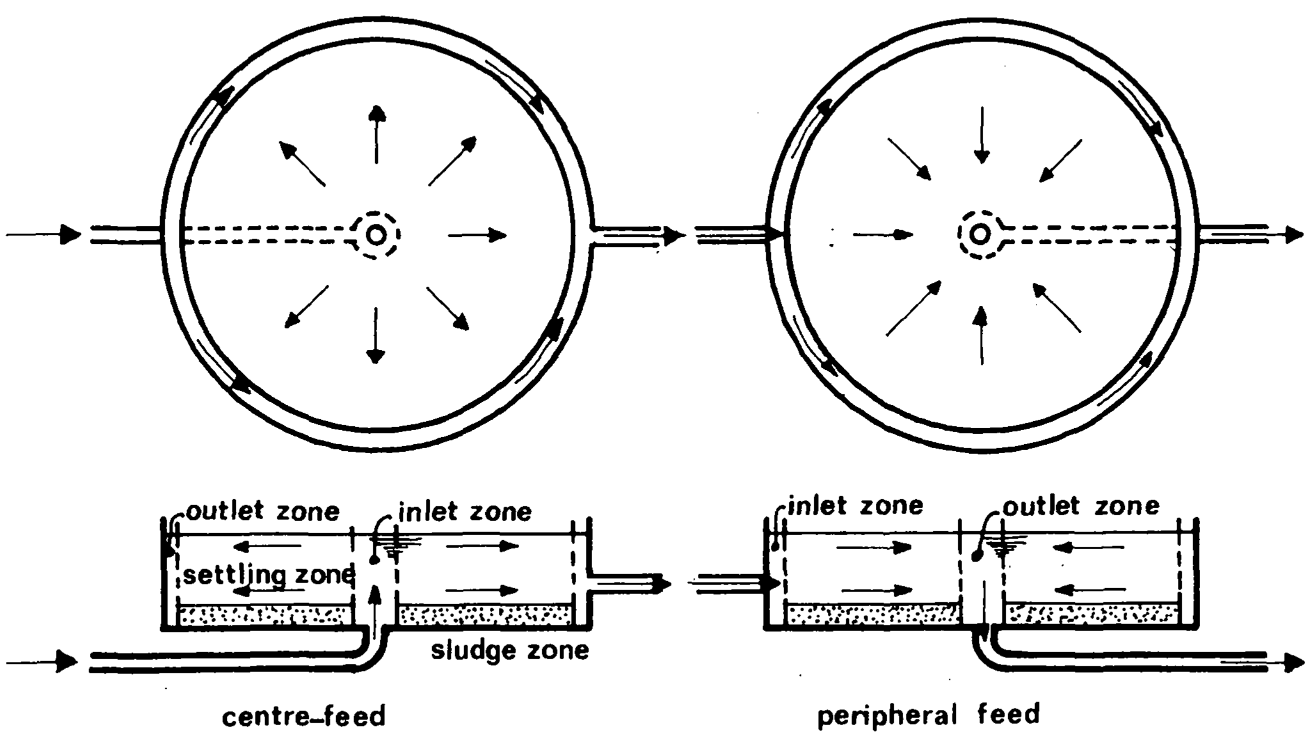


Fig. 3-5 Circular horizontal flow settling tank.

of rectangular tanks, being a function of the overflow rate s_o only. In such tanks, the paths traced by discrete particle are not straight lines, but curved ones as shown in fig. 3.6. This certainly hampers mathematical calculations, but has otherwise no influence.

With regard to the design and construction of sludge removal facilities, described in detail in section 3.9, the way in which suspended particles accumulate at the bottom of the tank is of great importance. For horizontal flow tanks of rectangular plan, the accumulation in a point at a distance l from the inlet can be calculated by first considering that particles with a settling velocity equal to or larger than

$$s_l = \frac{L}{l} s_o$$

cannot reach this point (fig. 3.7, left). Particles with a lower settling velocity s_x are divided equally over a length x (fig. 3.7, right)

$$x = \frac{s_o}{s_x} L = \frac{s_l}{s_x} l$$

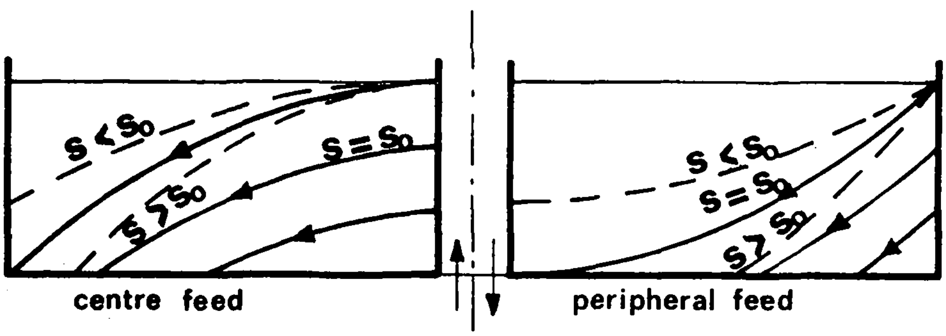


Fig. 3-6 Paths traced by discrete particles in a circular horizontal flow settling tank.

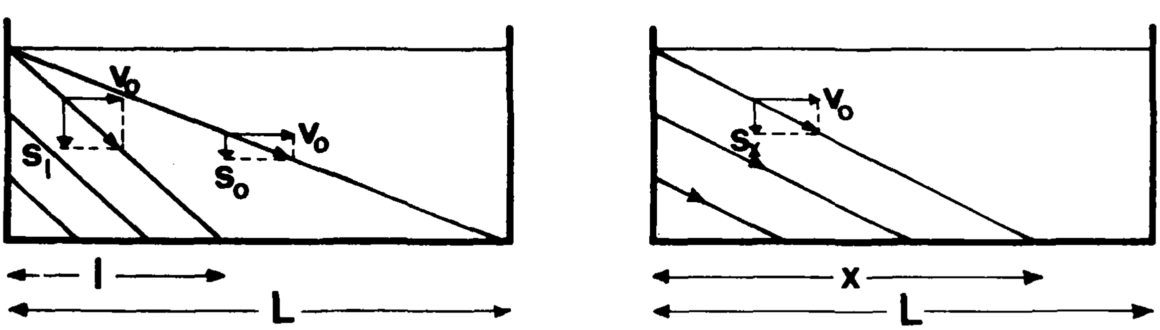


Fig. 3-7 Paths traced by discrete particles with different settling velocities.

When these particles are present in an amount dp, the accumulation per unit area at a distance l from the inlet equals

$$da = \frac{dp}{Bx} \quad \text{with B as tank width,}$$

giving as total accumulation by all particles with a settling velocity less than s_l

$$a = \int_0^{p_e} \frac{dp}{Bx} = \frac{1}{Bl}\frac{1}{s_l} \int_0^{p_e} s_x dp = \frac{r_l}{Bl}$$

in which the integral equals the shaded area in fig. 3.8 left, while the value of the dimensionless deposition factor r_l is indicated in fig. 3.8 right. For the settling velocity distribution of fig.

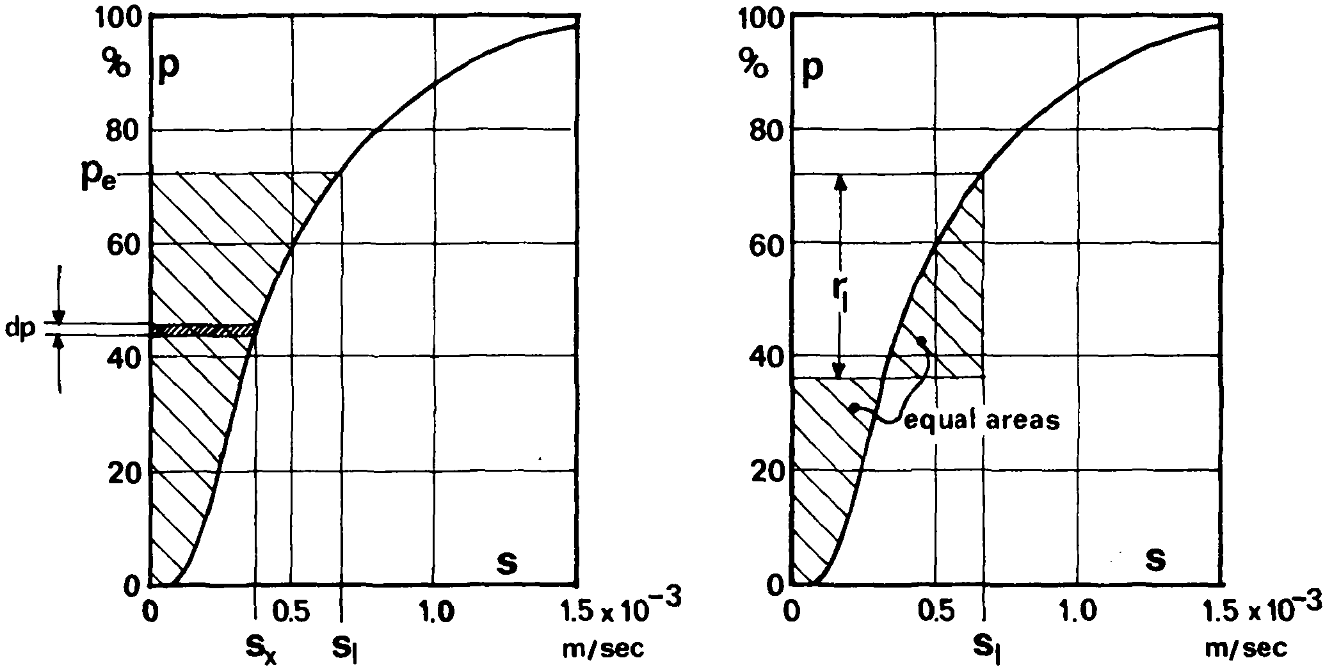


Fig. 3-8 Calculation of sludge deposits.

2.11, the value of r_l as function of

$$s_l = \frac{Q}{Bl}$$

is shown in fig. 3.9. To obtain absolute values for the sludge deposition, expressed in grams/m^2/sec, the factor a must still be multiplied with the capacity Q(m^3/sec) and with the suspended solids content

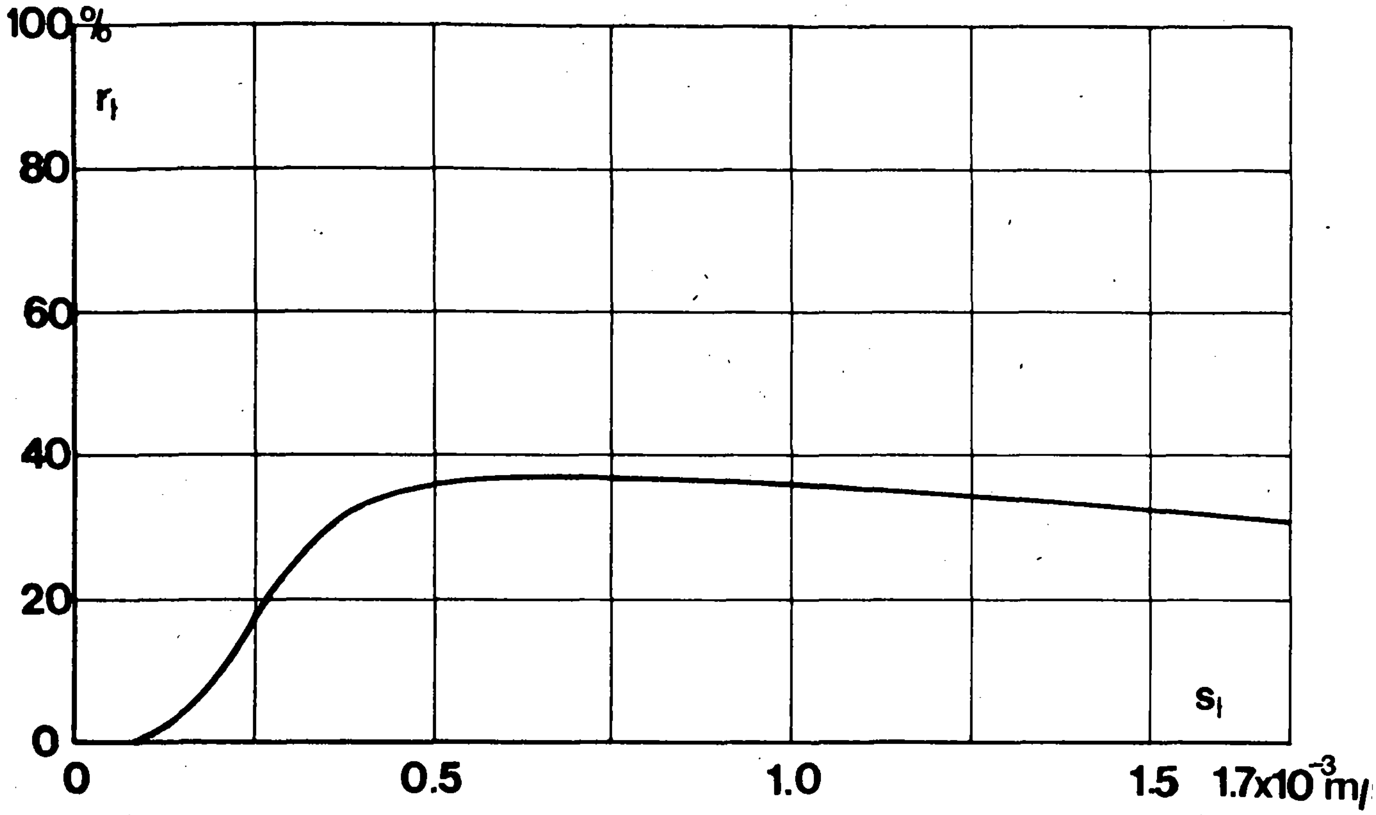


Fig. 3-9 Deposition factor r_1 for the suspension shown in fig. 2-11.

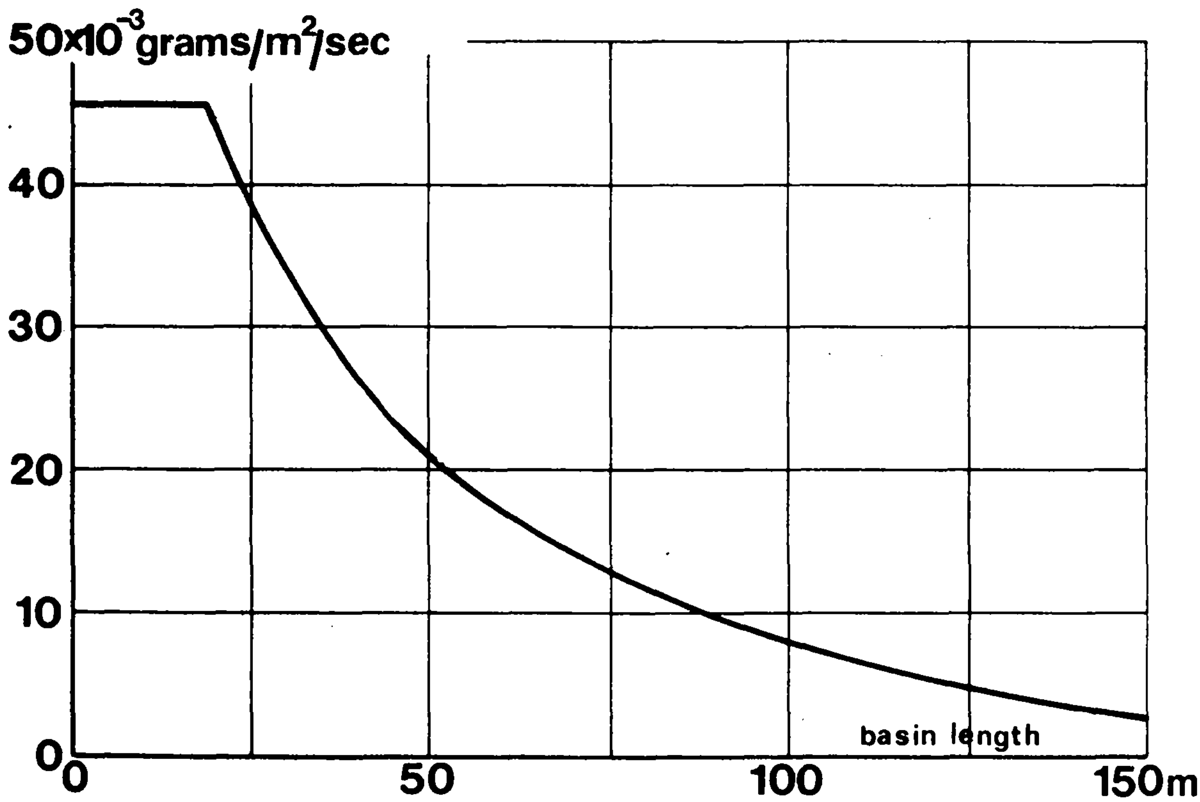


Fig. 3-10 Sludge accumulation in a rectangular basin (Q=0,5 m^3/sec, C_s=86 grams/m^3, B=15m and r_1 from fig. 3-9).

c_s (grams/m^3) of the inflowing water. An example of such a sludge curve is given in fig. 3.10, assuming a capacity Q of 0.5 m^3/sec, a rectangular basin 15 m wide, a suspended solids content in the raw water c_s of 86 grams/m^3 (table 2.1) and the values of r_1 as shown in fig. 3.9.

With circular basins, the results obtained above may again be used, remembering that the basin area traversed by the incoming water is the deciding factor. For a diameter of 50 m and the other data as mentioned above, the results are shown in fig. 3.11. In all cases, the larger part of the sludge accumulates in the first parts of the basin.

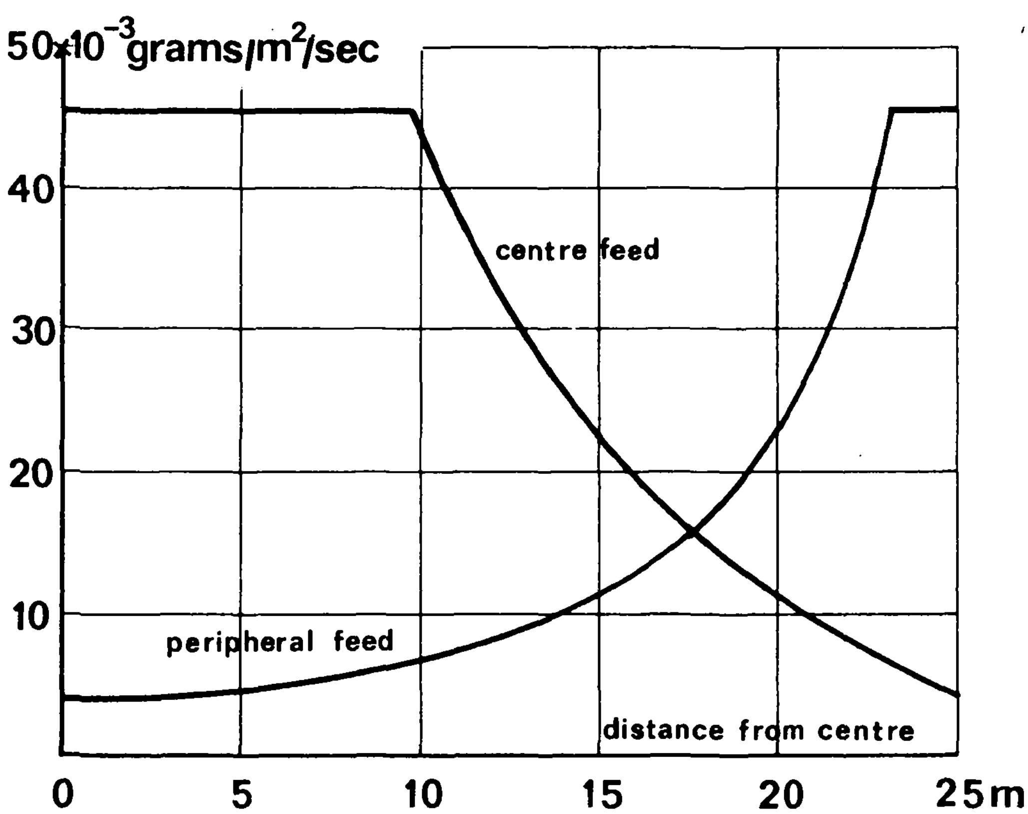


Fig. 3-11 Sludge accumulation in a circular basin (Q=0,5 m^3/sec, C_s=86 grams/m^3, D=50m and r_1 from fig. 3-9).

3.3. Reduction in basin efficiency by turbulence

In the preceding section, sedimentation efficiencies have been calculated for ideal conditions, among other things without any influence of the horizontal water movement. This may be true with streamline flow, but with turbulent flow transverse velocity components will be present, scattering the pathways of discrete particles and reducing basin efficiency in the way as shown in fig. 3.12. The horizontal flow will occur under laminar conditions when its Reynolds number

$$Re = \frac{v_o R}{\nu}$$

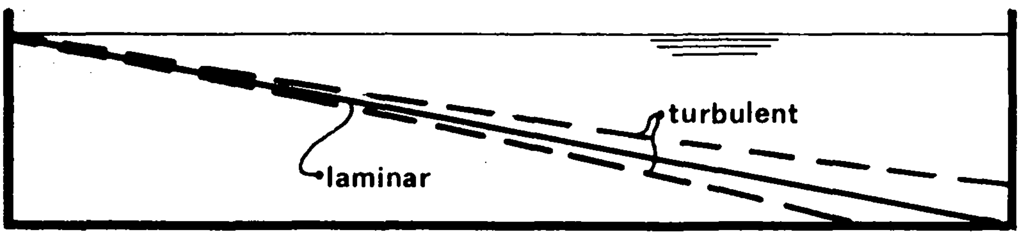


Fig. 3-12 Paths traced by discrete particles with laminar and turbulent flow.

is smaller than 580 to 2000, depending on the construction of the inletzone and on the presence of columns, cross-beams, rough walls, sludge removal equipment or other obstacles which might disturb the flow. With rectangular basins, the displacement velocity equals

$$v_o = \frac{Q}{BH}$$

and the hydraulic radius

$$R = \frac{BH}{B + 2H}$$

giving as Reynolds number

$$Re = \frac{Q}{\nu} \frac{1}{B + 2H}$$

with ν as kinematic viscosity. At 10 ^{0}C, $\nu = (1.31)10^{-6} m^2/sec$, requiring for laminar flow at $Re < 2000$

$$Q < (2.62)10^{-3}(B + 2H)$$

which is only possible when small amounts of water are treated in wide and deep basins. Unless the overflow rate

$$s_o = \frac{Q}{BL}$$

is low, the length of the basin moreover will tend to be short. For instance

$$Q = 0.03\ m^3/sec, \quad B = 7.5\ m, \quad H = 2\ m$$

$$s_o = (0.1)10^{-3}\ m/sec,\ A = 300\ m^2, \quad L = 40\ m.$$

With any larger capacity or overflow rate, however, impossible dimensions are obtained

$$Q = 0.05\ m^3/sec, \quad B = 15\ m, \quad H = 2\ m$$

$$s_o = (0.2)10^{-3} m/sec, \quad A = 250\ m^2, \quad L = 17\ m$$

or a nearly square basin, in which the flow will be unstable, giving rise to another reduction in basin efficiency as will be shown in section 3.5. With standard designs and a length several times the width of the basin, for instance in this case

$B = 6$ m, $L = 42$ m and $H =$ m as before

$$v_o = \frac{0.05}{12} = (4.2)10^{-3}\ m/sec, \qquad R = \frac{12}{10} = 1.2\ m$$

$$Re = \frac{(4.2)10^{-3}(1.2)}{(1.31)10^{-6}} = 3800$$

and turbulent conditions for the horizontal water movement through the basin.

In water and waste water engineering, rectangular basins nearly always operate under conditions of turbulent flow, with an efficiency less than calculated in the preceding section for an ideal horizontal flow settling tank. This reduction in basin efficiency can be calculated on the basis of the dispersion caused by turbulence. The classical results obtained by Camp (Sedimentation and the design of settling tanks, Thomas R. Camp, Transactions of the American Society of Civil Engineers, 1946, p. 895 - 958) are shown in fig. 3.13, giving the removal ratio r as function of the factors s/v_o and s/s_o with s as settling velocity of the particle, v_o as displacement velocity in the basin and s_o as overflow rate. For a suspension containing one type of particles

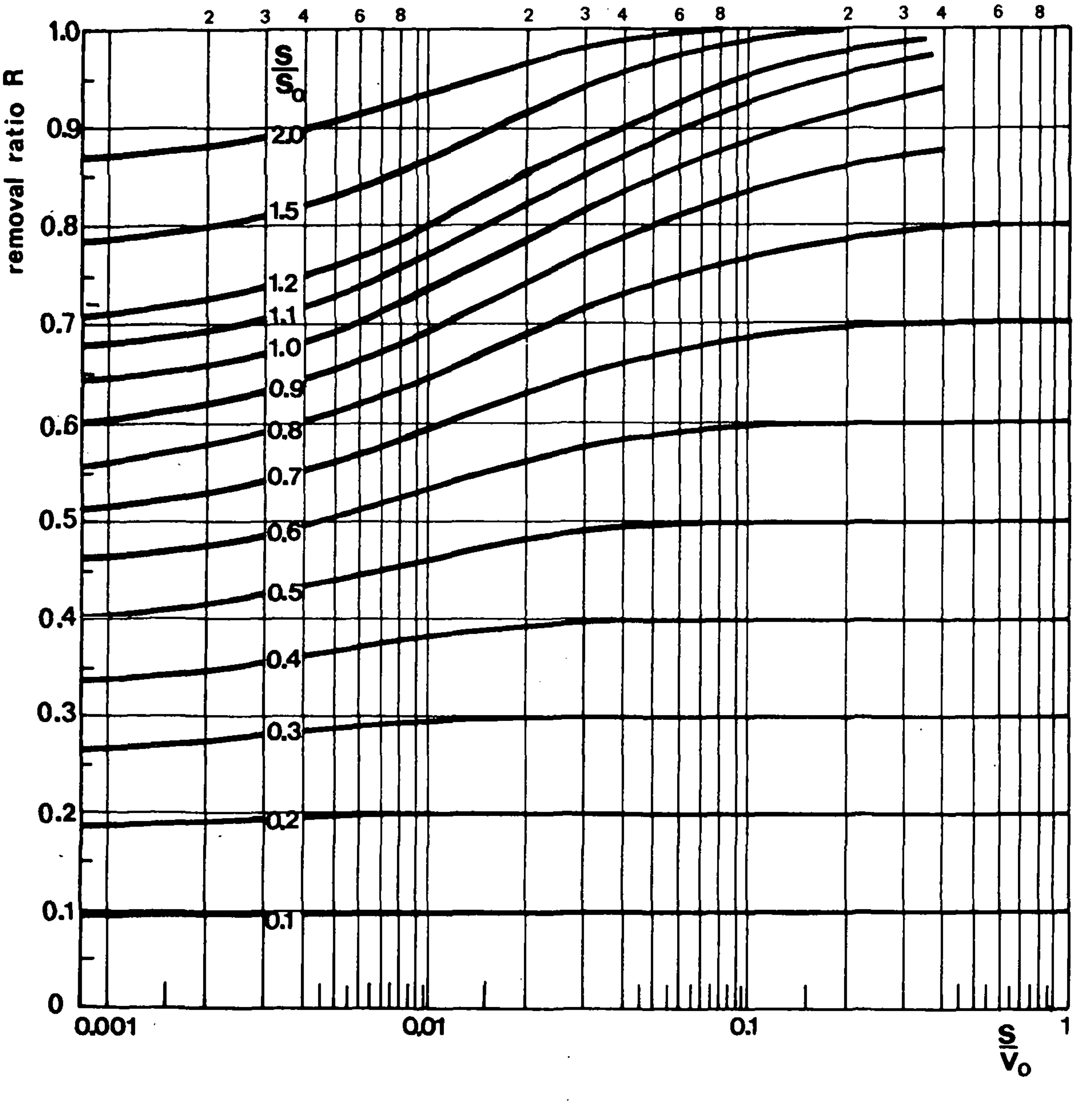


Fig. 3-13 Removal ratio with turbulent flow.

and one settling velocity s only, the calculations are quite simple. Suppose that by sedimentation 80% of these particles must be removed. With ideal settling, the overflow rate is now given by the relation

$$\frac{s}{s_o} = r = 0.8$$

According to fig. 3.13, however, an efficiency r = 0.8 with the factor $s/s_o = 0.8$ is only obtained in case

$$\frac{s}{v_o} > 0.5$$

requiring a ratio between length and depth smaller than

$$\frac{L}{H} = \frac{v_o}{s_o} = \frac{s/s_o}{s/v_o} = \frac{0.8}{0.5} = 1.6$$

that is to say a short, wide and deep basin with most probably unstable flow. With a larger ratio between length and depth, for instance

$$\frac{L}{H} = 20$$

and the same overflow rate, the deciding factors are

$$\frac{s}{s_o} = 0.8, \quad \frac{s}{v_o} = \frac{0.8}{20} = 0.04$$

giving with fig. 3.13 as removal ratio

$$r = 0.73$$

The original removal ratio of 80% can now only be obtained by decreasing the overflow rate, for instance

$$\frac{s}{s'_o} = 0.9, \quad r = 0.8, \quad \frac{s}{v_o} = 0.05 \quad \text{and } \frac{L}{H} = \frac{0.9}{0.05} = 18$$

The ratio between length and depth is quite acceptable, but the decrease in overflow rate means an increase in basin area by a factor 0.9/0.8 = 1.125 or with 12.5%, augmenting the cost of construction with 5 - 10%, depending on local circumstances.

In case the suspended particles vary strongly in settling velocities, the calculation described above must be made for a number of fractions, each with a constant settling rate. For the suspension shown in fig. 2.11, the calculations are carried out in table 3.1, going out from a capacity of 0.5 m^3/sec and a constant basin depth of 2 m. In the original design A, a width of 12 m and a length of 90 m is assumed, giving as overflow rate

$$s_o = \frac{0.5}{(12)(90)} = (0.463)10^{-3} \text{ m/sec}$$

and with fig. 2.13 an efficiency of 78% for ideal conditions. In reality, however, the efficiency amounts to 71% only. Some improvement, an efficiency of 74%, may be obtained by reducing the ratio between length and width from 90/12 = 7.5 to 45/24 = 1.9 (design B), but the original efficiency can only be regained by decreasing the overflow rate. In design C the surface area is augmented by 25%, from (12)(90) = 1080 m^2 to (15)(90) = 1350 m^2, raising the efficiency to even 79%, while maintaining an adequate ratio between length and width equal to a factor 6.

In circular settling tanks, the hydraulic radius equals the basin depth H, while the horizontal velocity of flow decreases with the distance to the centre. Using the notations of fig. 3.14

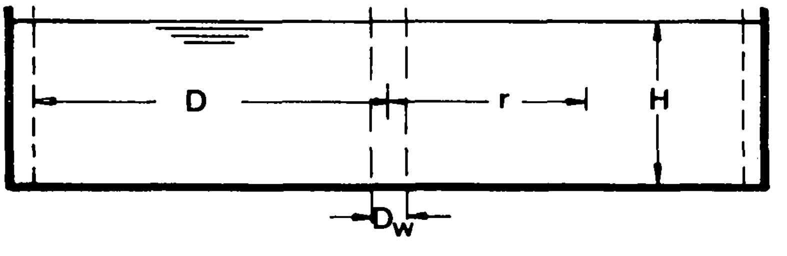


Fig. 3-14 Circular horizontal flow tank.

Table 3.1. Calculation of basin efficiency under turbulent flow conditions, $Q = 0.5$ m^3/sec, $H = 2$ m

design A,	BL = (12)(90) = 1080 m^2,	$s_o = (0.463)10^{-3}$ m/sec	
	BH = (12)(2) = 24 m^2,	$v_o = (20.8)10^{-3}$ m/sec	
	Re = 24000		
design B,	BL = (24)(45) = 1080 m^2,	$s_o = (0.463)10^{-3}$ m/sec	
	BH = (24)(2) = 48 m^2,	$v_o = (10.4)10^{-3}$ m/sec	
	Re = 14000		
design C,	BL = (15)(90) = 1350 m^2,	$s_o = (0.370)10^{-3}$ m/sec	
	BH = (15)(2) = 30 m^2,	$v_o = (16.7)10^{-3}$ m/sec	
	Re = 20000		

Suspension fig 2.11		Design A			Design B			Design C		
fraction	S in mm/sec	S/V_o	S/S_o	r	S/V_o	S/S_o	r	S/V_o	S/S_o	r
0 - 10 %	0.16	0.008	0.345	0.33	0.015	0.345	0.34	0.010	0.43	0.40
- 20 %	0.22	0.011	0.475	0.44	0.021	0.475	0.46	0.013	0.595	0.54
- 30 %	0.26	0.013	0.56	0.51	0.025	0.56	0.53	0.016	0.705	0.62
- 40 %	0.31	0.015	0.67	0.595	0.030	0.67	0.615	0.019	0.84	0.705
- 50 %	0.37	0.018	0.80	0.685	0.036	0.80	0,725	0.022	1.00	0.79
- 60 %	0.45	0.022	0.97	0.775	0.043	0.97	0.82	0.027	1.215	0.875
- 70 %	0.55	0.026	1.19	0.86	0.053	1.19	0.91	0.033	1.485	0.94
- 80 %	0.70	0.034	1.51	0.945	0.067	1.51	0.975	0.042	1.89	0.98
- 90 %	0.95	0.046	2.50	1.00	0.091	2.50	1.00	0.057	2.57	1.00
-100 %	1.35	0.065	2.91	1.00	0.130	2.91	1.00	0.081	3.65	1.00
Combined efficiency				0.71			0.74			0.79

$$v_o = \frac{Q}{2\pi rH} \quad ,R = H$$

$$Re = \frac{v_o R}{\nu} = \frac{1}{\nu}\frac{Q}{2\pi rH} H = \frac{1}{\nu}\frac{Q}{2\pi r}$$

with r varying from D/2 at the outer circumference of the tank to $D_w/2$ near the (inlet or outlet) well in the centre. With

$$Q = \frac{\pi}{4} D^2 s_o \quad \text{and } t = 10\,^0C, \ \nu = (1.31)10^{-6} \ m^2/sec$$

this gives as Reynolds numbers

$$Re_{min} = (0.19)10^6 D\,s_o, \qquad Re_{max} = (0.19)10^6 \frac{D^2}{D_w} s_o$$

In the field of water and waste water engineering, s_o is commonly in the neighbourhood of $(0.3)10^{-3}$ m/sec, while D_w is about 5 to 10% of D. Substitution of these values gives

$$Re_{min} \approx 60 \ D \ , \qquad Re_{max} \approx 1000 \ D$$

meaning that at the outer circumference of the tank the flow is mostly laminar ($Re \leq 2000$), while near the centre the flow is always turbulent.

The reduction in basin efficiency by turbulence may again be determined with the help of Camp's diagram shown in fig. 3.13. The calculation, however, is rather complicated, requiring a subdivision of the suspension into fractions with different settling velocities s as well as a subdivision of the basin into annular zones with different displacement velocities v_o. With circular tanks and peripheral feed (fig. 3.5 right), the majority of the suspended load is removed near the outer circumference of the basin (fig. 3.11), where laminar flow conditions prevail With circular tanks and centre feed (fig. 3.5 left) on the other hand, these factors are completely reversed and here the reduction in basin efficiency by turbulence will be much larger. In both cases, however, an increase in basin area of 5 - 20% is necessary to maintain the efficiency of quiescent settling shown in fig. 2.13. As mentioned before the increase in cost of construction will be appreciably less and is seldom a major factor in the design of the tank.

From the considerations in this section and in section 2.1, it will be clear that in continuous horizontal flow basins the settling process is influenced by two Reynolds numbers, one related to the upward flow of water along the downward moving particle and the other to the displacement of the body of water in horizontal direction. The value of the first or particle Reynolds number determines whether the settling is viscous or turbulent, the second or tank Reynolds number whether the horizontal flow occurs under laminar or turbulent conditions. In the field of water and waste water engineering, the settling process is mostly laminar and the horizontal water movement mostly turbulent.

3.4. Bottom scour

For the removal of discrete particles, the efficiency of a continuous horizontal flow basin was found to be a function of the overflow rate

$$s_o = \frac{Q}{BL}$$

and independent of the depth H. As this depth decreases, however, the displacement velocity

$$v_o = \frac{Q}{BH}$$

goes up and may become so high as to be able to pick up and carry away settled-out material from the sludge zone. This scouring starts at a velocity v_s, when the hydraulic shear between the flowing water and the sludge deposits equals the mechanical friction between these deposits and the bottom of the tank. Per unit area, the hydraulic shear amounts to

$$\tau = \rho_w gRJ$$

with J as slope of the water surface. According the formula of Darcy-Weisbach

$$J = \lambda \frac{1}{4R} \frac{v_s^2}{2g} .$$ Substituted

$$\tau = \frac{\lambda}{8} \rho_w v_s^2$$

in which λ is the hydraulic friction factor with an average value of 0.03. The mechanical friction per unit area is proportional to the submerged weight of the sludge layer. With the notations of fig. 3.15

$$f = \alpha_f N = \alpha_f \alpha_p (\rho_s - \rho_w) g d = \beta(\rho_s - \rho_w) g d$$

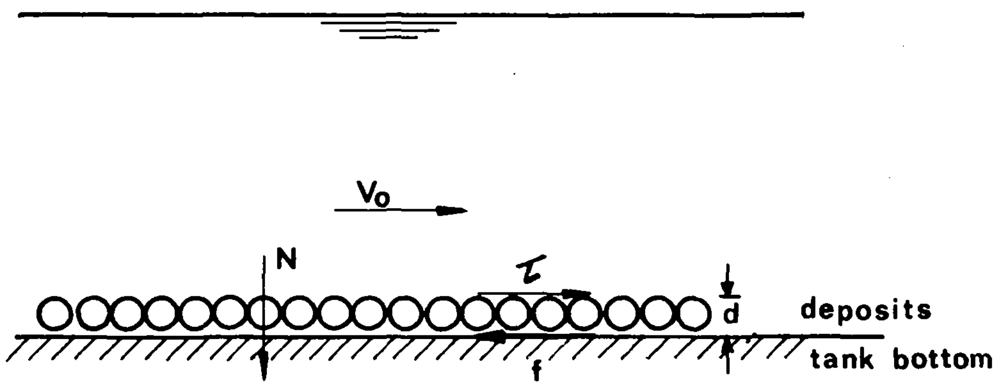


Fig. 3-15 Scour of deposited particles.

with the friction factor α_f between 0.1 and 0.2, the porosity factor α_p between 0.2 and 0.6 and the factor β varying from about 0.04 for uni-granular sand to about 0.06 for non-uniform, interlocking sticky material. Equality of the unit forces τ and f gives

$$\frac{\lambda}{8} \rho_w v_s^2 = \beta(\rho_s - \rho_w) g d$$

from which the scour velocity follows at

$$v_s = \sqrt{\frac{8\beta}{\lambda} \frac{\rho_s - \rho_w}{\rho_w} g d}$$

or with the average values of $\lambda = 0.03$, $\beta = 0.05$

$$v_s = \sqrt{\frac{40}{3} \frac{\rho_s - \rho_w}{\rho_w} g d} \qquad \text{(fig. 3.16)}$$

In section 2.1, the settling velocities of a single spherical particle have been found at

laminar settling $\quad s = \frac{1}{18} \frac{g}{\nu} \frac{\rho_s - \rho_w}{\rho_w} d^2$

turbulent settling $\quad s = \sqrt{\frac{10}{3} \frac{\rho_s - \rho_w}{\rho_w} g d}$

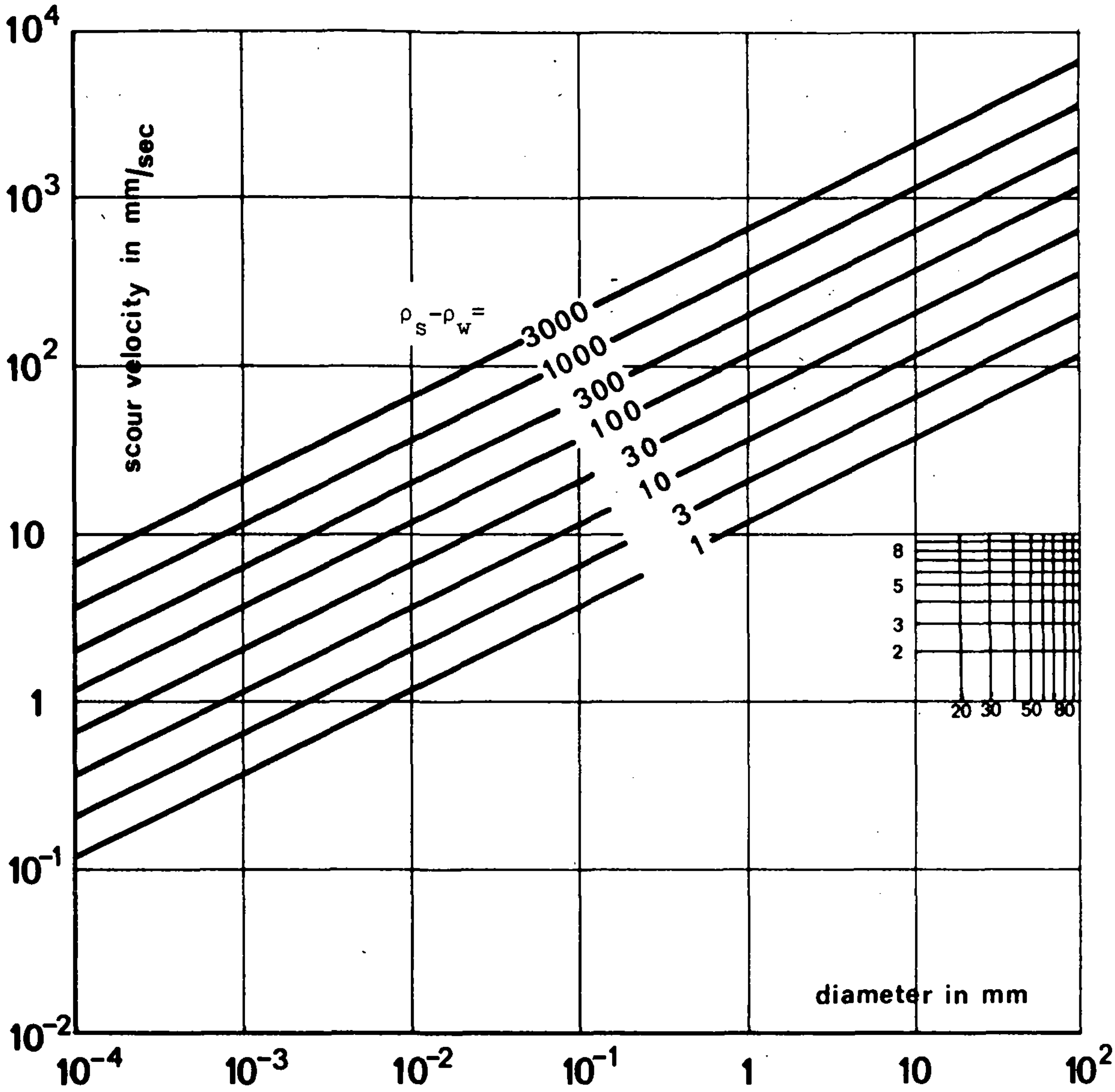


Fig. 3-16 Scour velocity of spherical particles.

This gives as ratio between the scour and the settling velocity

laminar settling $\quad \frac{v_s}{s} = 36 \sqrt{\frac{10}{3} \frac{\nu^2}{g} \frac{\rho_w}{\rho_s - \rho_w} \frac{1}{d^3}}$

and with $t = 10\ {}^0C$, $\nu = (1.31)10^{-6}\ m^2/sec$

$$\frac{v_s}{s} = (27.5)10^{-6} \left(\frac{\rho_w}{\rho_s - \rho_w}\right)^{1/2} d^{-3/2}$$

turbulent settling $\frac{v_s}{s} = 2$

which values together with those in the transition region are shown in fig. 3.17.

Reduction in basin efficiency by scour is not to be feared as long as

$$v_o < v_s$$

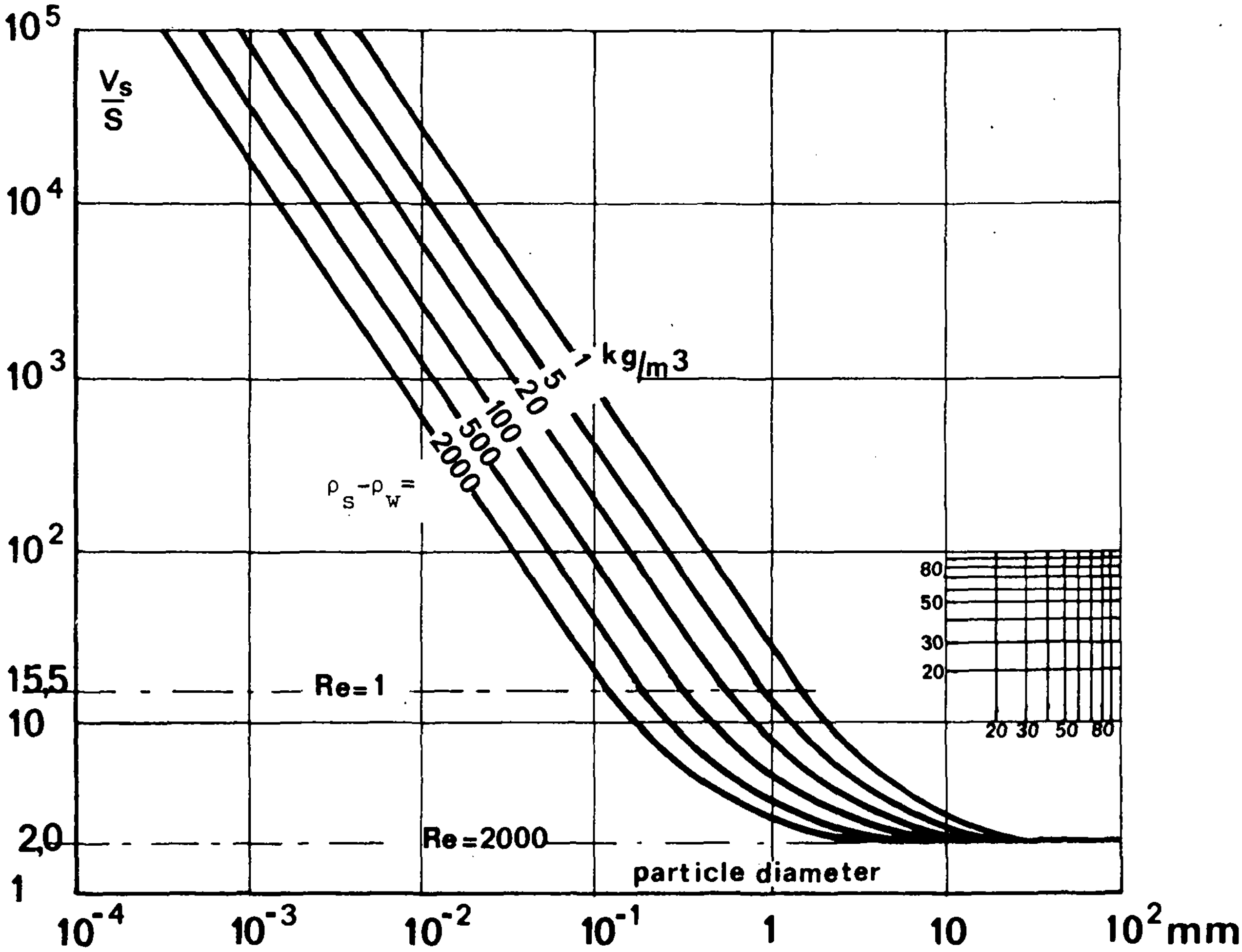


Fig. 3-17 Ratio between scour velocity V_s and settling velocity S of spherical particles in water of $10^\circ C$ as function of their diameter and mass density.

With rectangular horizontal flow basins

$$\frac{v_o}{s_o} = \frac{L}{H}$$

giving as requirement

$$\frac{L}{H} < \frac{v_s}{s} \frac{s}{s_o}$$

With turbulent settling, the factor v_s/s equals 2, asking even with uniform particles and the ratio s/s_o nearing unity for deep, short and wide basins with a length never larger than twice the depth. With viscous settling on the other hand, fig. 3.17 shows such large values of v_s/s that notwithstanding non-uniform particles and a low ratio of s/s_o, the basin length may easily be 10, 20 or 40 times the basin depth, factors which in practice are seldom surpassed for small, medium and large tanks respectively.

In many cases, all suspended particles have a common source, meaning a constant mass density and a settling velocity which only varies with the particle diameter. For the danger of scour a more direct calculation is now possible, eliminating this diameter between the formulas for the scour velocity and the viscous settling velocity

$$d = \frac{3}{40} \frac{\rho_w}{\rho_s - \rho_w} \frac{v_s^{\,2}}{g} = \sqrt{\frac{18\nu}{g} \frac{\rho_w}{\rho_s - \rho_w} s}$$

from which follows with $t = 10\ ^0C$, $\nu = (1.31)10^{-6}\ m^2/sec$ and $g = 9.81\ m/sec^2$

$$v_s = (0.45)\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{1/4} s^{1/4}$$

and for instance with $\rho_w = 1000\ kg/m^3$, $\rho_s = 1020\ kg/m^3$

$$v_s = (0.45)(0.376)s^{1/4} = 0.17\ s^{1/4}$$

In fig. 2.11 the settling velocity for 5% of the suspended load is below $(0.16)10^{-3}$ m/sec, giving as corresponding scour velocity

$$v_s = (0.17)(1.6)^{1/4}10^{-1} = (19)10^{-3}\ m/sec$$

In the example of table 3.1 this velocity is surpassed in design A, but not in the designs B and C.

In circular tanks, the horizontal velocity obtains its maximum value near the (inlet or outlet) well in the centre. With the notations of fig. 3.14

$$v_o = \frac{Q}{\pi D_w H}$$

The requirement

$$v_o < v_s$$

may now be translated as

$$D_w > \frac{Q}{\pi H v_s}$$

With for instance a capacity of 0.5 m^3/sec, a depth of 3 m and a scour velocity of $(19)10^{-3}$ m/sec as calculated above

$$D_w > \frac{0.5}{\pi(3)(19)10^{-3}} = 2.8 \text{ m}$$

For the capacity assumed, this value is fairly large but certainly not excessive.

As will be explained in the next section, a large horizontal velocity is desirable to obtain steady flow conditions. With an influent at a temperature much lower or at a suspended load much higher than that present in the tank, the inflowing water tends to plunge to the bottom and to proceed here in a shallow layer with a velocity of flow much higher than the average displacement velocity v_o. Under such conditions, the actual velocity may greatly surpass the scour velocity. Resuspension of settled out material, however, can still be prevented by using the baffles shown in fig. 3.18.

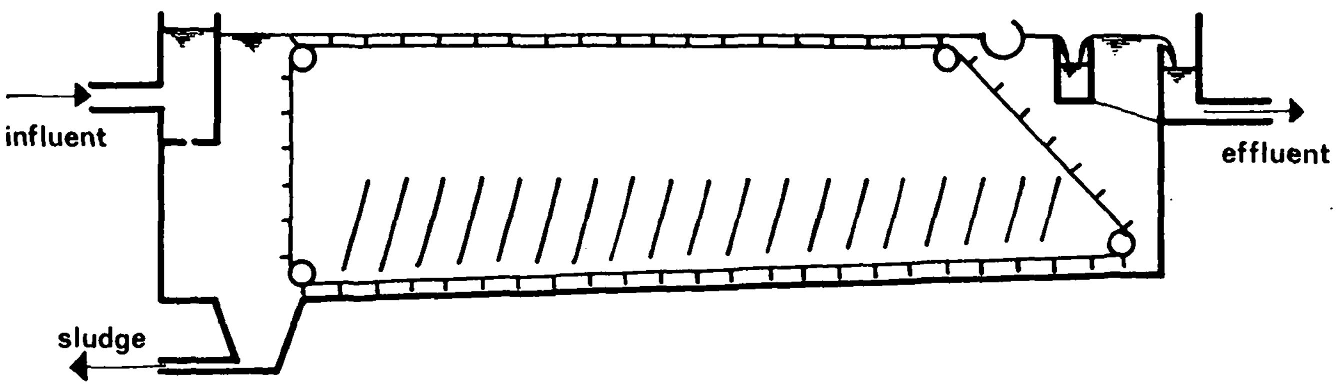


Fig. 3-18 Baffles to protect sludge deposits against scouring.

3.5. Non-uniform velocity distribution and short-circuiting

Over the full width and depth of an ideal rectangular basin, the horizontal velocity was assumed to be constant at v_o, meaning among other things that all particles of water have the same detention time T_o, equal to the volume of the basin divided by the rate of discharge

$$v_o = \frac{Q}{BH} \quad , \quad T_o = \frac{BHL}{Q}$$

This supposition, however, is unrealistic. Even when no disturbing influences are present and the flow is quite regular, the frictional drag along the walls and the floor of the basin will retard the movement of water with as consequence that near these boundaries the displacement velocity will be smaller and in the centre of the basin larger than the average value. A non-uniform velocity profile over the depth of the basin is shown schematically in fig. 3.19, lower half. In the upper picture and a constant displacement velocity, the removal ratio r for particles with a settling velocity s equals

$$r = \frac{h}{H} = \frac{s}{s_o}$$

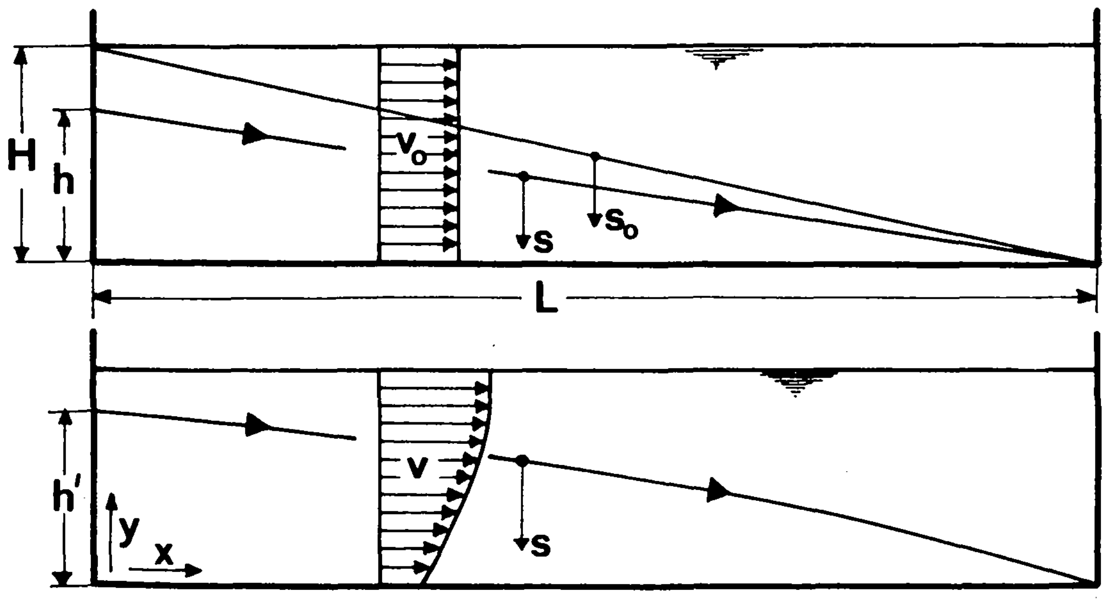


Fig. 3-19 Path traced by a discrete particles.

In the lower picture, the removal ratio equals the quotient between the amount of water entering the basin over the depth h' and the total amount of water. With B as basin width

$$r' = \frac{B}{Q} \int_{o}^{h'} v dy$$

In any time element dt, the particle moves both in horizontal as in vertical direction

$dx = vdt$

$dy = -sdt$ from which follows

$vdy = -sdx$ With the boundary conditions

$y = 0, \quad x = L \quad \text{and} \quad y = h', \quad x = 0$

the removal ratio becomes

$$r' = -\frac{B}{Q} \int_{L}^{o} s dx = \frac{B}{Q} s \int_{o}^{L} dx = \frac{BsL}{Q} = \frac{s}{s_o}$$

meaning that the variation in velocity over the depth of the basin has no influence whatsoever on clarification efficiency. A change in velocity over the width of the basin means a variation in overflow rate. Clarification efficiency may now be calculated by subdividing the basin in a number of parallel strips, each with its own surface loading and removal ratio. Schematically this procedure is shown in fig. 3.20, where for simplicity only two (equal) parts are distinguished, with overflow rates of 0.7 s_o and 1.3 s_o respectively. In case s_o is small,

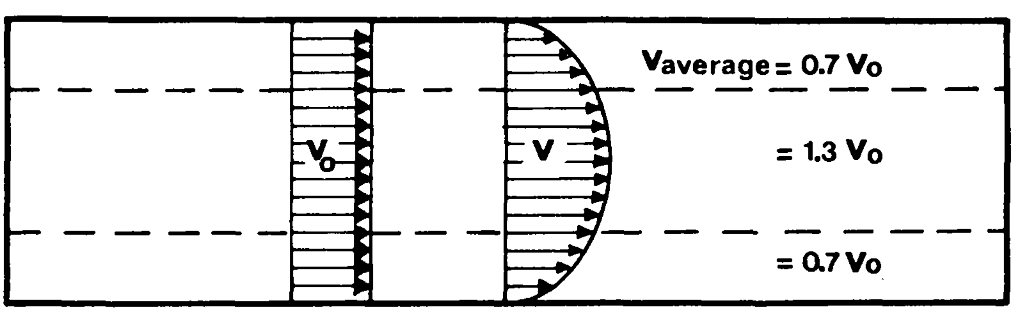


Fig. 3-20 Varation in surface loading.

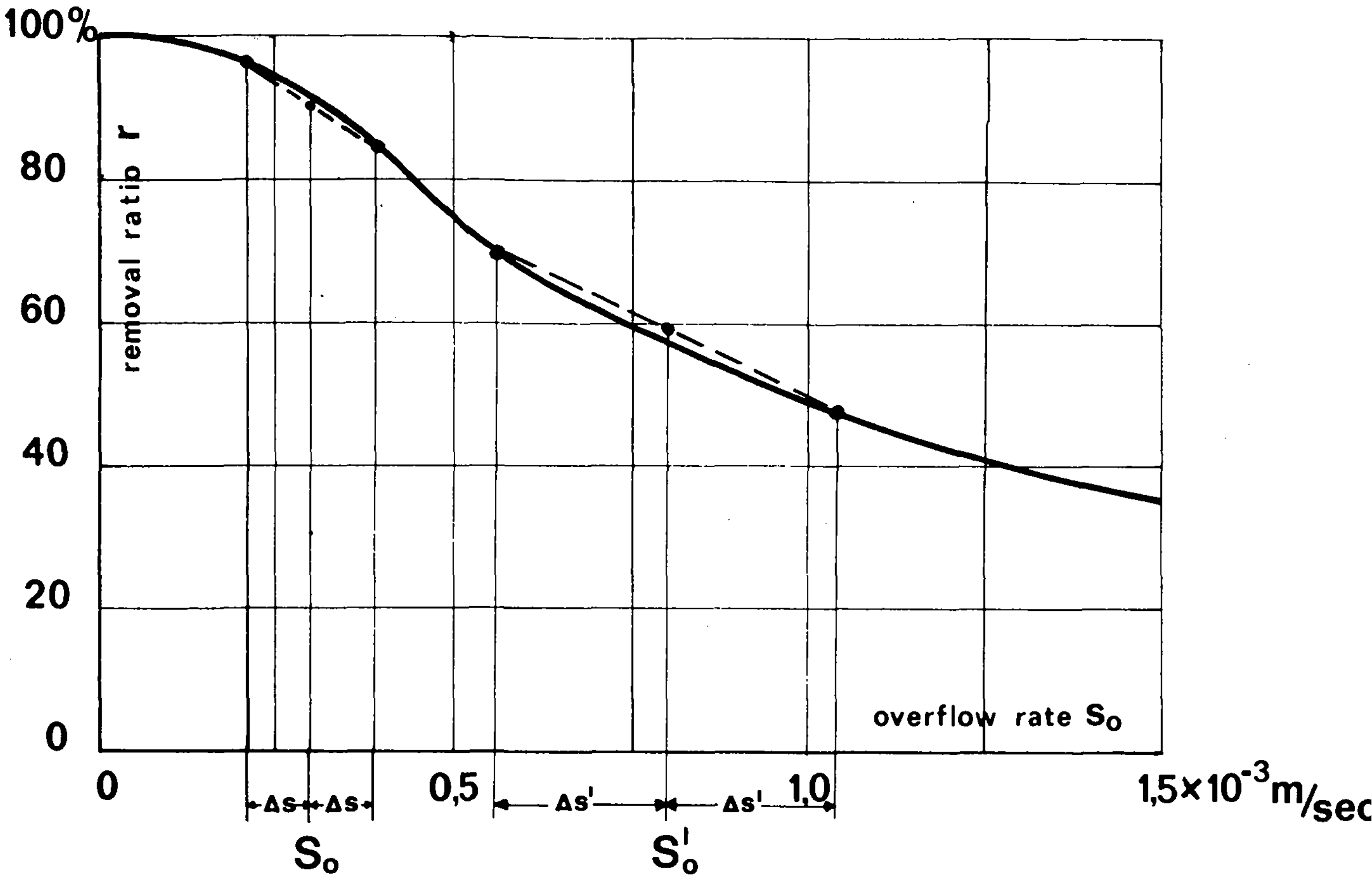


Fig. 3-21 Removal ratio as function of overflow rate (fig. 2-13.).

for instance (0.3)10^{-3} m/sec, fig. 3.21 shows a drop in basin efficiency from 92 to 91%, while with a large value of s_o equal to (0.8)10^{-3} m/sec for instance, this efficiency rises from 57 to 59%. Mostly, however, a high removal ratio is required and now a slight reduction in basin efficiency by an unequal velocity distribution over the width of the basin must be anticipated.

Due to the variation in the velocities of the horizontal water movement described above, some of the inflow will reach the basin outlet in less time than the theoretical detention period T_o and some will take much longer. This phenomenon is called short-circuiting and can best be described with the cumulative frequency distribution for the detention times of the various particles of water. Going out from the undisturbed conditions assumed above, this frequency distribution can be calculated for various types of flow (fig. 3.22). In real basins, however, some disturbance is always present, due to an unequal supply of raw water or an unequal abstraction of clarified water over the width and depth of the basin, due to straying currents caused by wind

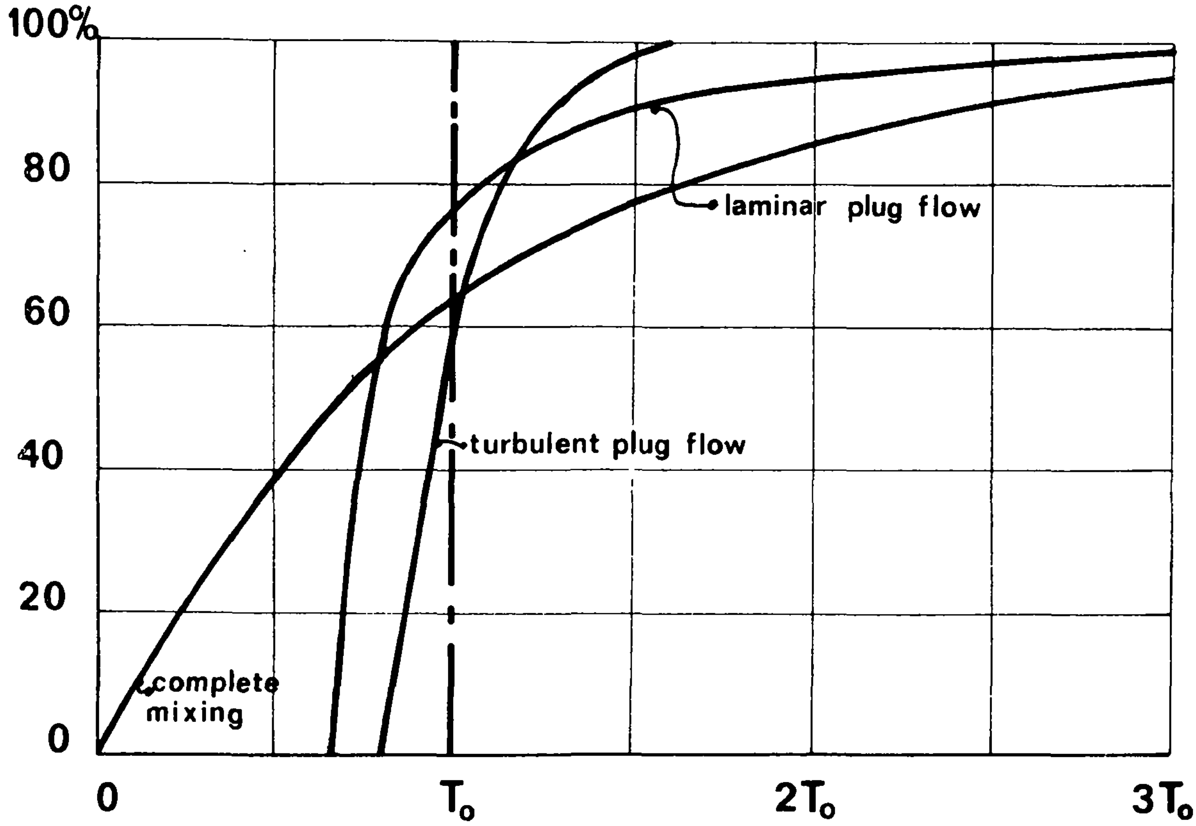


Fig. 3-22 Cumulative frequency distribution of calculated detention times.

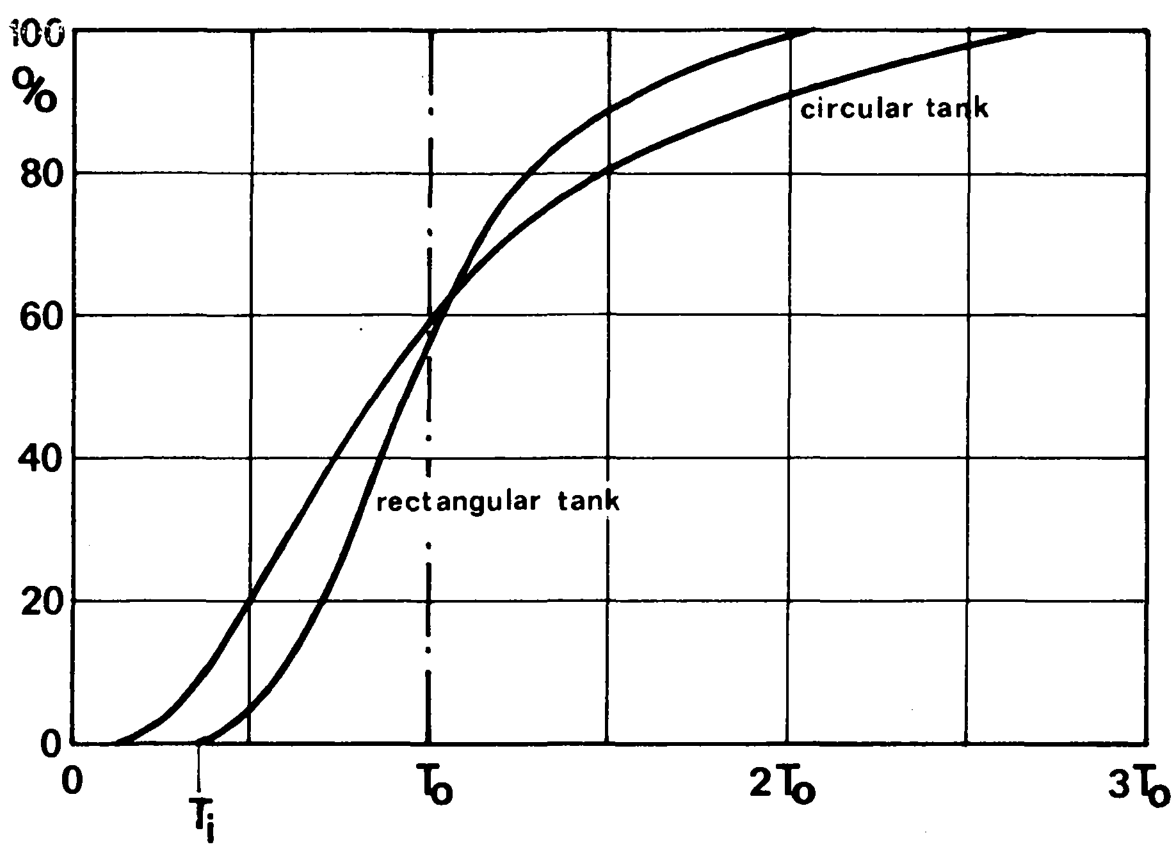


Fig. 3-23 Cumulative frequency distribution of measured detention times.

or differences in density, to eddying currents set up by the inertia of the incoming fluid, etc. A theoretical calculation is now impossible and the variation in detention times must be determined in the field. This can best be done by adding from a certain moment onward a tracer substance such as dye or kitchen salt at a constant rate to the tank influent, producing in this water a constant concentration c_o. At various time intervals thereafter, the concentration c of the tracer substance in the basin effluent is measured. The graph of c/c_o against time of observation gives directly the wanted cumulative frequency distribution of the detention times, representative examples of which are shown in fig. 3.23. Compared with the plug flows of fig. 3.22, the initial flowing through time is now much shorter, meaning that the maximum flow velocities are greatly increased. Indirectly this may cause a further reduction in basin efficiency, due to a more turbulent flow and in particular by an increased danger of bottom scour (fig. 3.24), in extreme cases asking for preventive measures as shown in fig. 3.18.

Both fig. 3.22 and 3.23 show average detention times T_a equal to the theoretical value T_o. This, however, is no longer the case when the flow rotates upon itself or is stagnant. An example of rotational flow in the vertical plane is shown in fig. 3.25, caused by a higher density of the incoming water, for instance due to a large suspended

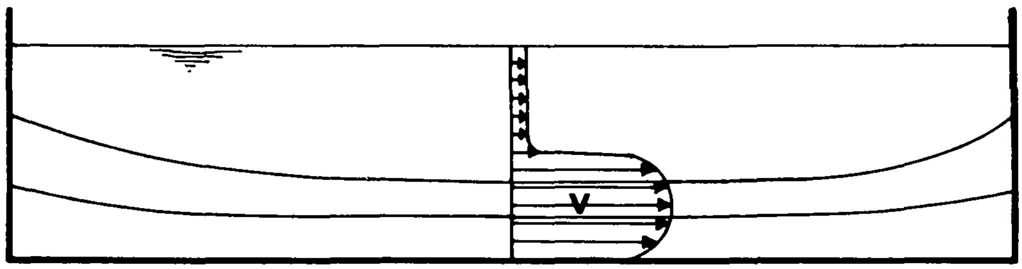


Fig. 3-24 Danger of bottom scour.

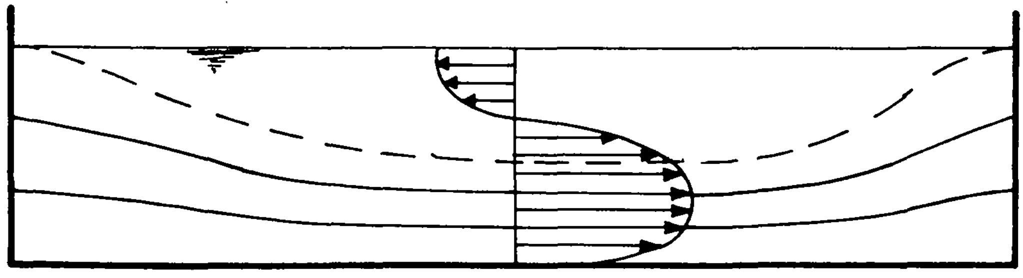


Fig. 3-25 Short-circuiting vertical plane.

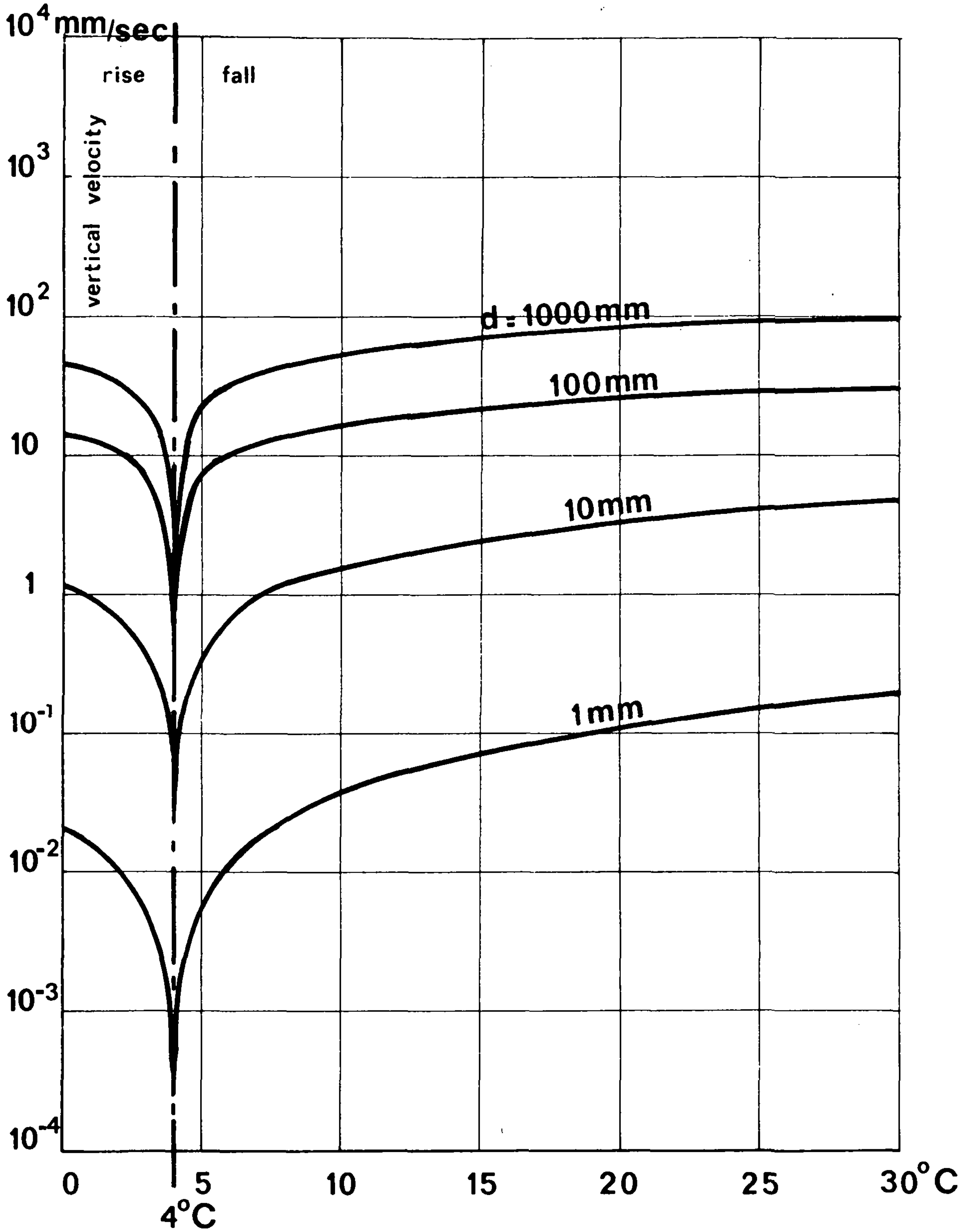


Fig. 3-26 Vertical velocity in mm/sec for a spherical mass of water of diameter d with a temperature 1°C less than that of the surrounding water.

solids content or a lower temperature (fig. 3.26). The tank area above the dotted line in fig. 3.25 does not contribute to the horizontal displacement of the inflowing water, resulting in an average velocity of flow much higher and an average detention time much smaller than corresponds with the theoretical values of v_o and T_o. Adverse effects in the meanwhile do not need to occur as according to section 3.2 the removal ratio is independent of the basin depth. A reduction in basin efficiency can only arise by the secondary factors of turbulence and bottom scour already mentioned above. A direct reduction in basin efficiency does take place when the plan of the basin shows areas with stagnant water or eddying currents, caused by an unequal distribution of the incoming water, by wind induced currents, etc. When in fig. 3.27 the effective surface area of the basin is thus lowered by 20%, the average overflow rate will show a similar increase, rising for instance from $(0.4)10^{-3}$ m/sec to $(0.5)10^{-3}$ m/sec. According to fig. 3.21, the removal ratio will now drop from 84 to 75% and even more when the unequal velocity distribution over the remaining basin width, the increased turbulence and the greater danger of bottom scour are taken into account.

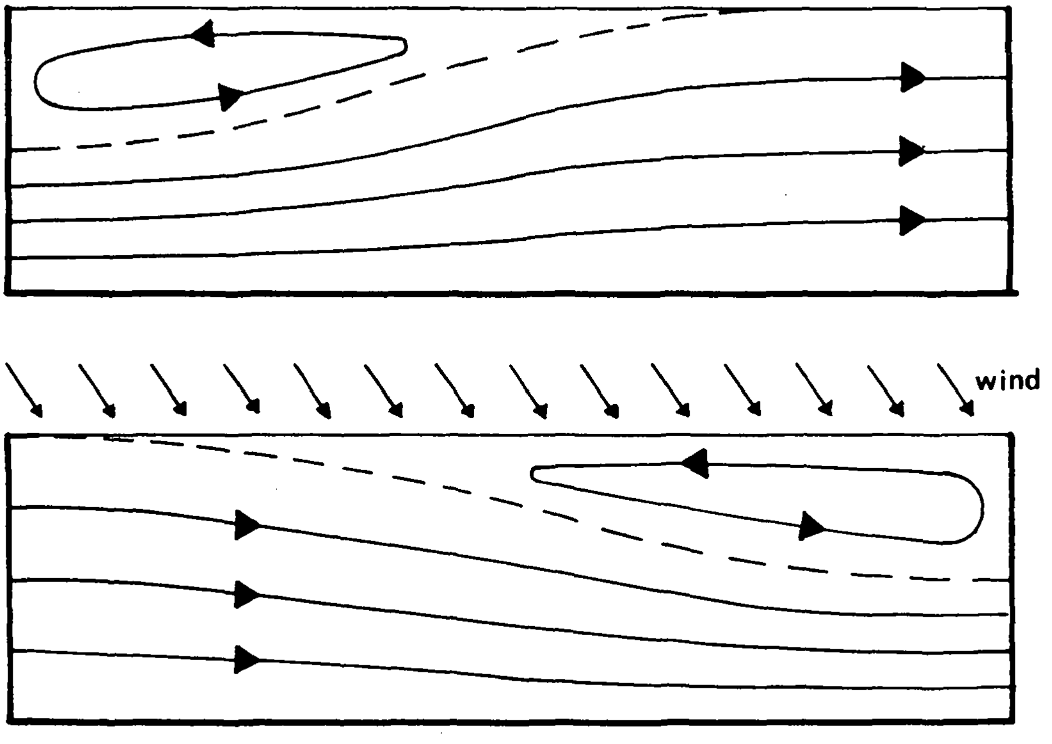


Fig. 3-27 Short-circuiting in horizontal plane.

From the description given above, it will be clear that short-circuiting cannot be avoided. As long as the average detention time equals the theoretical one (fig. 3.23), the resulting reduction in basin efficiency is only indirect and small. Direct and appreciable reductions in basin efficiency, however, must be anticipated when the average detention time T_a is smaller than the theoretical value T_o (fig. 3.28) and in particular this type of short-circuiting must therefore be reduced as much as possible. To a large degree, short-circuiting can be

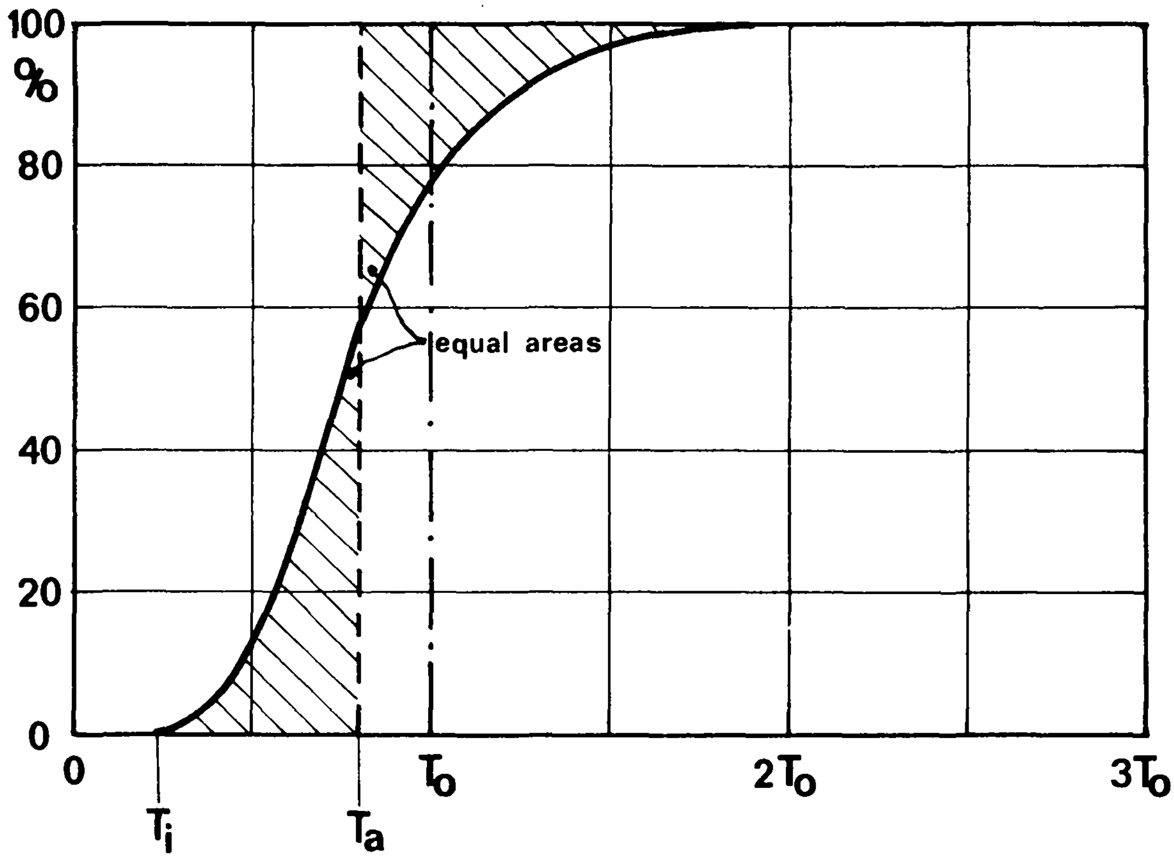


Fig. 3-28 Cumulative frequency distribution of detention times for the basin shown in fig. 3-25.

diminished by a judicious design, supplying and abstracting the water equally over the full width and depth of the basin, preventing concentrated inlets with high velocities of flow, mixing the incoming water intimately with the tank content so as to prevent density currents, etc. As regards the settling zone itself, stable flow conditions must be pursued, able to withstand as much as possible the disturbing influence of wind, differences in density, etc., which otherwise would result in random currents and a larger variation in displacement velocities. This stability can be promoted by augmenting the ratio between inertia forces and gravity, expressed by the dimensionless Froude number

$$Fr = \frac{v_o^2}{gR}$$

in which v_o is the average displacement velocity, g the gravity constant ($9.81\ m/sec^2$) and R the hydraulic radius. With rectangular horizontal flow basins

$$v_o = \frac{Q}{BH} \quad \text{and} \quad R = \frac{BH}{B + H}$$. Substituted

$$Fr = \frac{Q^2}{g} \frac{B + 2H}{B^3H^3}$$

meaning that the Froude number will be higher as the capacity is larger and the basin has a smaller width and a smaller depth. With the chosen surface loading

$$s_o = \frac{Q}{BL}$$

the Froude number becomes

$$Fr = \frac{s_o^2}{g} \frac{L^2}{H^3} (1 + \frac{2H}{B})$$

according to which stability is promoted when the basin has a high overflow rate and a great length. Some idea about the influence of the Froude number on basin stability is given by the diagram of fig. 3.29 , taken from the paper by Camp quoted in section 3.3. A minimum acceptable figure is difficult to fix, but generally a value of 10^{-5} is adopted, giving a ratio between initial and theoretical flowing-through time of about 0.4 and a ratio between average and theoretical flowing-through time of about 0.9. This reduces the adverse effects of short-circuiting to more or less acceptable values, but certainly not to zero. Whenever possible, higher Froude numbers should be applied, but not so high as to endanger basin efficiency by turbulence or bottom scour. For the example dealt with in table 3.1, the Froude number amounts to $(2.9)10^{-5}$ for design A, $(0.6)10^{-5}$ for design B and $(1.8)10^{-5}$ for the chosen design C.

With circular horizontal flow basins the Froude number is not constant, but decreases with the distance to the centre of the tank. Using the notations of fig. 3.14, this number varies from

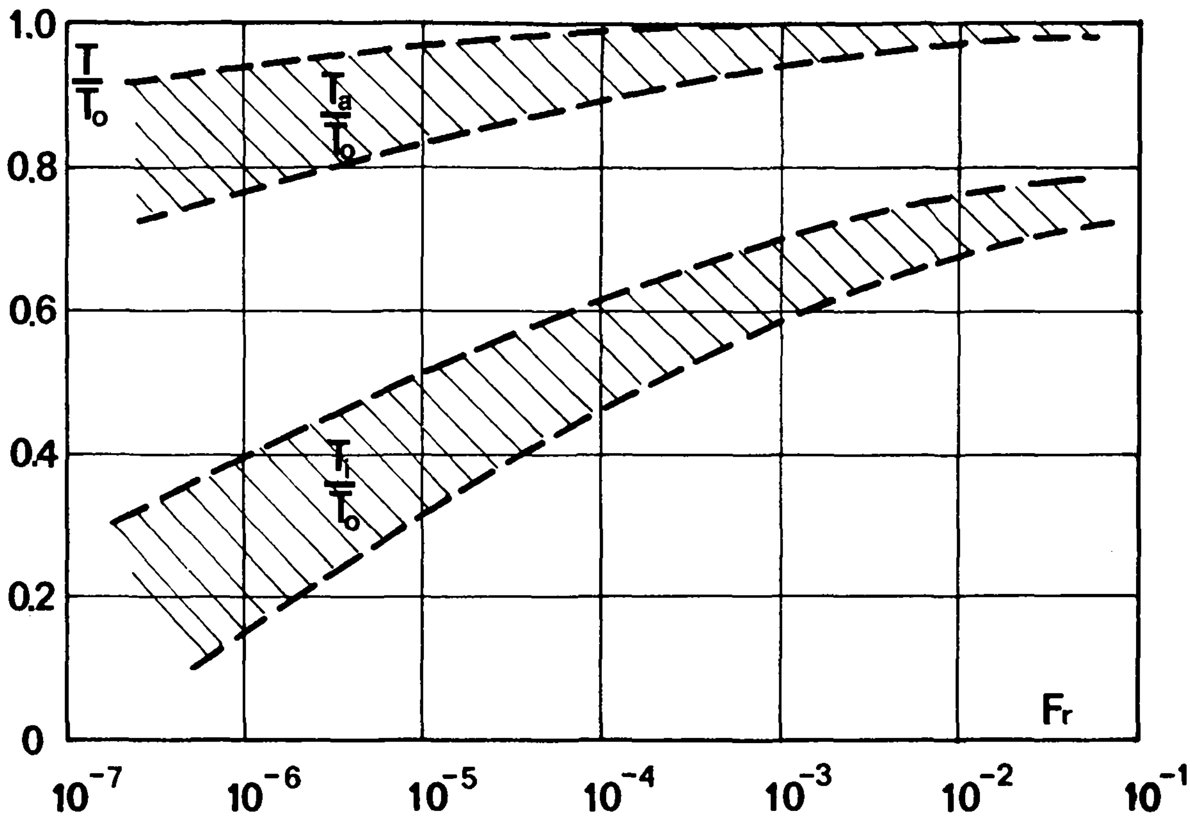


Fig. 3-29 Infuence of the Froude number on the degree of short-circuiting.

$$Fr_{min} = \frac{Q^2}{\pi^2 g} \frac{1}{D^2 H^3} = \frac{s_o^{\ 2}}{16g} \frac{D^2}{H^3}$$

near the outer circumference to

$$Fr_{max} = \frac{Q^2}{\pi^2 g} \frac{1}{D_w^{\ 2} H^3} = \frac{s_o^{\ 2}}{16g} \frac{D^4}{D_w^{\ 2} H^3}$$

near the inlet or outlet well in the centre. In the field of water and waste water engineering, s_o is commonly in the neighbourhood of $(0.3)10^{-3}$ m/sec, while D_w is about 5 to 10% of D. With a normal tank depth of 3 m the Froude numbers thus become

$$Fr_{min} \simeq (0.02)10^{-9}\ D^2$$

$$Fr_{max} \simeq (4)10^{-9}\ D^2$$

or with a large diameter of 50 m

$$Fr_{min} \simeq (0.005)10^{-5}, \quad Fr_{max} \simeq 10^{-5}$$

Already at a short distance from the centre, the flow in circular tanks is unstable. Even a slight disturbance by wind or difference in density (due to insolation for instance) will change the radial flow pattern, resulting in short-circuiting and an quite erratic tank behaviour (fig. 3.30).

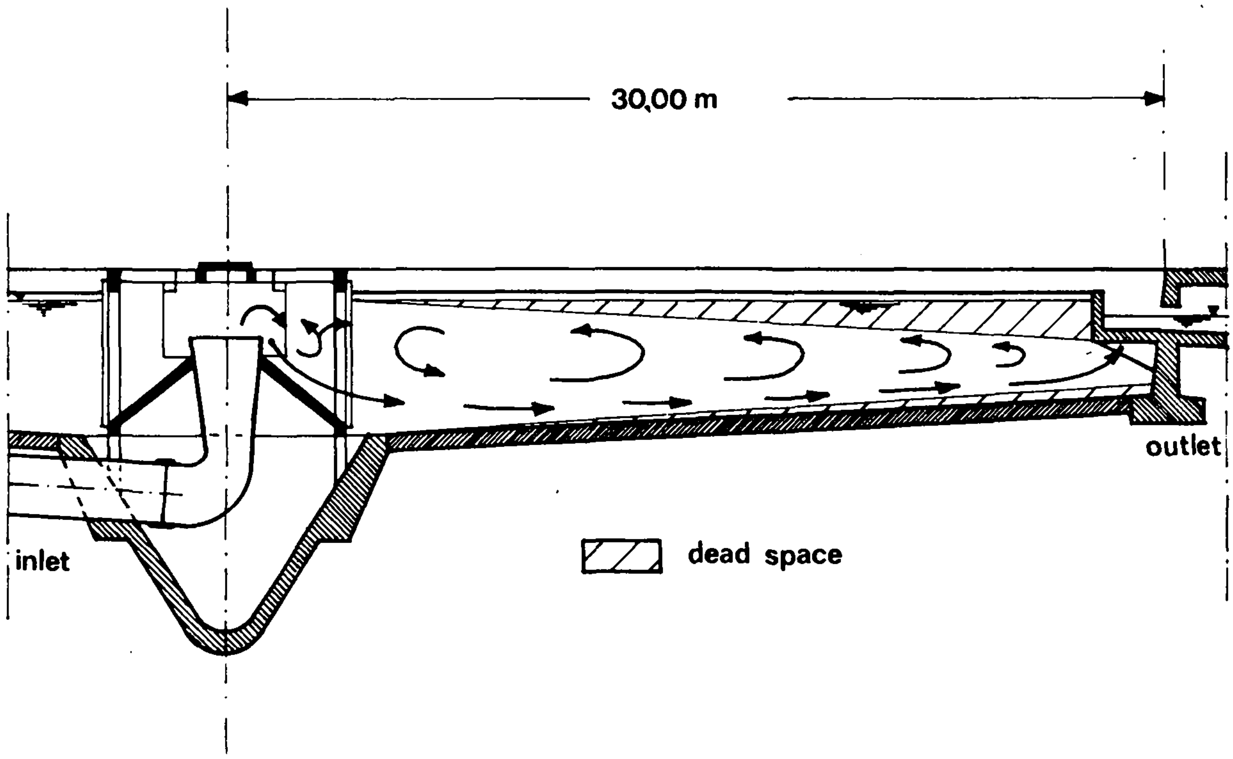


Fig. 3-30 Erratic behaviour of a circular horizontal flow tank.

3.6. Design of the settling zone

When designing a horizontal flow settling tank for the removal of discrete particles, the principal factors are on one hand the amount Q of water to be treated and on the other hand the settling characteristics of the suspension and the desired removal ratio which together determine the surface loading s_o to be applied. Once these factors are known, the required surface area $A = Q/s_o$ is fixed, leaving for circular tanks only the depth to be chosen. With mechanical sludge removal, already a small depth will satisfy all requirements. As rule of the thumb, applicable for preliminary designs

$$H = 0.17\ A^{1/3}$$

for manual cleaning to be increased by about 1 m for sludge storage, depending on local circumstances.

With rectangular tanks there is more freedom as not only the depth, but also the ratio between length and width need still to be fixed. Needless to say that these factors ought to be chosen such as to simulate quiescent settling conditions as much as possible. According to the preceding sections, however, this asks for short, wide and deep basins to prevent reduction in basin efficiency by turbulence and bottom scour and for long, narrow and shallow basins to prevent such a reduction by basin instability and short-circuiting. As mentioned before, scour is commonly not a problem leaving as requirements

$$Re = \frac{v_o R}{\nu} < 2000 \quad \text{and} \quad Fr = \frac{v_o^2}{gR} > 10^{-5}$$

Strange as it may seem, both these conditions can be satisfied, at a temperature of 10 ^{0}C, $\nu = (1.31)10^{-6}$ m^2/sec by

$$v_o = (6.4)10^{-3} \text{ m/sec}, \quad R < 0.41 \text{ m}$$

Such basins are shown in fig. 3.31 and are either short, wide and shallow or long, narrow and deep, while the horizontal velocity of flow is so small that bottom scour is indeed not to be feared. With regard to its enormous depth, the cost of constructing the basin shown in fig. 3.31 at the right will be prohibitive, while the

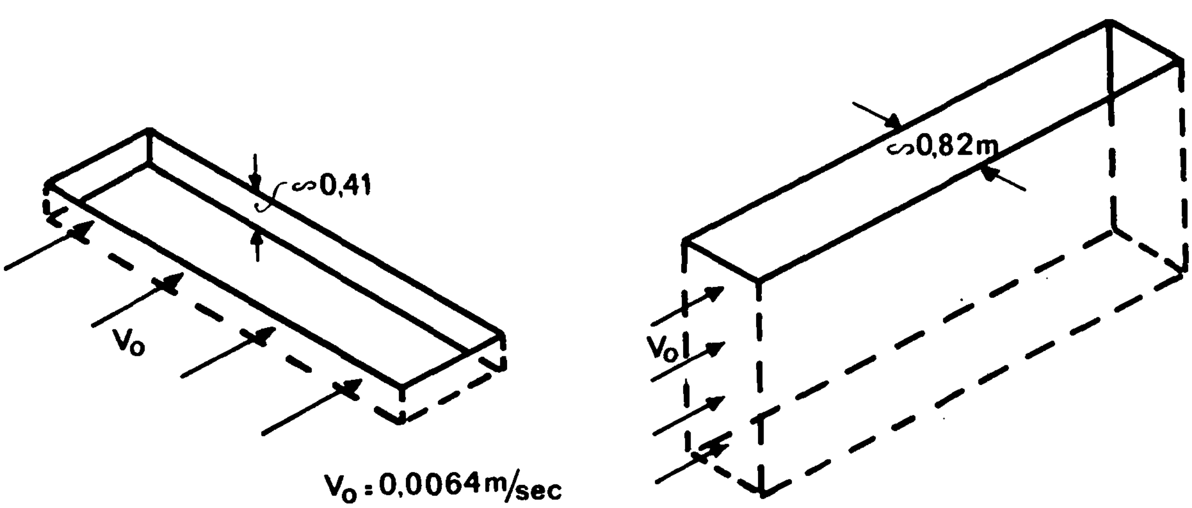


Fig. 3-31 Rectangular settling tanks with laminar and stable flow.

dimensions of the left hand basin will be rather odd. With a capacity of for instance 0.5 m^3/sec and an overflow rate of $(0.37)10^{-3}$ m/sec (design C in table 3.1)

$$H = 0.41 \text{ m}, \qquad B = \frac{0.5}{(0.41)(0.0064)} = 190 \text{ m}$$

$$L = \frac{0.5}{(190)(0.37)10^{-3}} = 7.1 \text{ m}$$

making sludge removal extremely expensive.

In practice, the shape of the settling zone is a compromise between on one hand the hydrodynamic requirements of a high Froude number and a low Reynolds number and on the other hand economic considerations asking for a limited ratio between length and width and for a shallow depth . This depth does not need to be larger than is necessary for installation and proper functioning of sludge collecting mechanisms, giving a value of about

$$H = \frac{1}{12} L^{0.8}$$

while the ratio between length and width commonly varies from 6 to 10. With a capacity of 0.5 m^3/sec and an overflow rate of $(0.37)10^{-3}$ m/sec as mentioned above, the required surface area equals

$$A = \frac{0.5}{(0.37)10^{-3}} = 1350 \text{ m}^2$$

giving with for instance L = 6B

$$B^2 = \frac{1350}{6} = 225, \qquad B = 15 \text{ m} \qquad \text{and}$$

$$L = \frac{1350}{15} = 90 \text{ m} \qquad H = \frac{1}{12}(90)^{0.8} = 3 \text{ m}$$

$$R = \frac{(3)(15)}{21} = 2.14 \text{ m}, \qquad v_o = \frac{0.5}{(3)(15)} = 0.0111 \text{ m/sec}$$

The hydrodynamic parameters thus become $(t = 10\ {}^0C, \nu = (1.31)10^{-6}\ m^2/sec)$

$$Re = \frac{(0.0111)(2.14)}{(1.31)10^{-6}} = 18000$$

$$Fr = \frac{(0.0111)^2}{(9.81)(2.14)} = (0.6)10^{-5}$$

The Froude number is only a little too low, but the Reynolds number is much too high, reducing basin efficiency by turbulence as described in section 3.3. When this reduction is not acceptable, a larger width could be applied. With B = 25 m in stead of 15 m

$$L = \frac{1350}{25} = 54 \text{ m}, \qquad H = \frac{1}{12}(54)^{0.8} = 2 \text{ m}$$

$$R = \frac{(2)(25)}{29} = 1.72 \text{ m}, \quad v_o = \frac{0.5}{(2)(25)} = 0.010 \text{ m/sec}$$

$$Re = \frac{(0.010(1.72)}{(1.31)10^{-6}} = 13000$$

$$Fr = \frac{(0.010)^2}{(9.81)(1.72)} = (0.6)10^{-5}$$

giving the same Froude number and only a moderate reduction in Reynolds number. Much better results could be obtained by using the longitudinal baffles of fig. 3.32. With 3 baffles, the basin width of 15 m is divided in 4 strips, each 3.75 m wide, reducing the hydraulic radius to

$$R = \frac{(3)(3.75)}{9.75} = 1.15 \text{ m}$$

With the horizontal velocity unchanged

$$Re = 9700 \qquad Fr = (1.1)10^{-5}$$

A further improvement may be obtained by subdividing the width of 25 m in 7 strips each 3.57 m wide

$$R = \frac{(2)(3.57)}{7.57} = 0.94 \text{ m}$$

$$Re = 7200 \qquad Fr = (1.1)10^{-5}$$

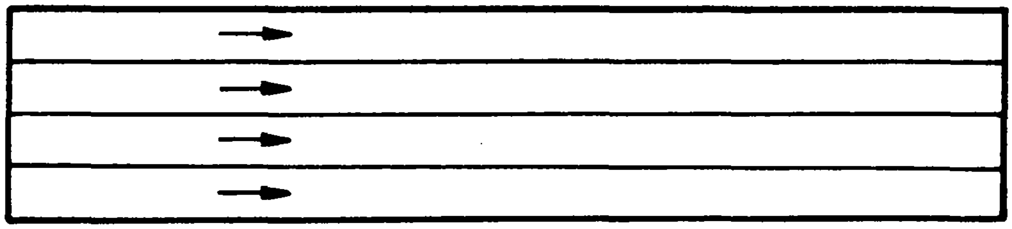


Fig. 3-32 Continuous longitudinal baffles.

The total length of basin walls is now reduced from 2(15 + 90) = 210 m to 2(25 + 54) = 158 m, while the total length of baffles has grown from (3)(90) = 270 m to (6)(54) = 324 m. These baffles, however, only guide the water, ironing out irregularities in flow, but the water pressure at both sides is the same. A small structural strength is consequently sufficient, allowing the use of cheap materials such as wood in a light-weight construction. As regards the cost of the basin alone, the cheapest construction will therefore be had with a nearly square plan, for instance

$$B = 36 \text{ m}, \quad L = 37.5 \text{ m}, \quad H = 1.5 \text{ m}$$

subdivided by 8 baffles in 9 strips, each 4 m wide

$$R = \frac{(4)(1.5)}{7} = 0.86 \text{ m}, \quad v_o = \frac{0.5}{(36)(1.5)} = 0.0093 \text{ m/sec}$$

$$Re = \frac{(0.0093)(0.86)}{(1.31)10^{-6}} = 6100$$

$$Fr = \frac{(0.0093)^2}{(9.81)(0.86)} = 10^{-5}$$

The total wall length now equals 2(36 + 37.5) = 147 m and the total length of baffles (9)(37.5) = 337 m. Compared with a basin width of 25 m, however, the gain is only slight, while a heavy increase in the cost of mechanical sludge removal equipment must be anticipated.

When in exceptional cases the Froude number is much too low and an increase in Reynolds number is still acceptable, the round-the-end baffles of fig. 3.33 may be applied. As serious dis-

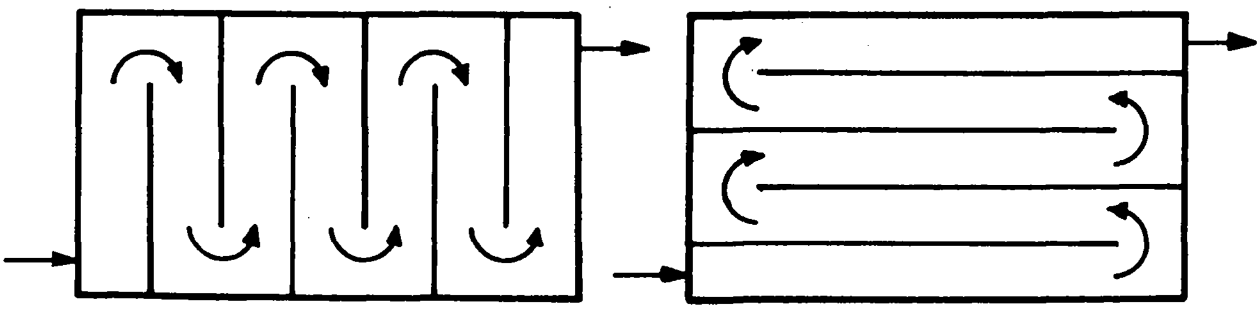


Fig. 3-33 Round-the-end baffles.

advantage must be mentioned, however, that around their ends eddying currents will arise, creating dead spaces and disturbing the sludge deposits at the bottom of the tank. Their application should therefore be avoided as much as possible, using channel-type settling tanks with a large ratio between length and width.

Instability of the basin and an erratic behaviour are in particular due to straying currents caused by the wind (water velocity 1 - 4% of the wind velocity) and to density currents caused by the heat of the sun. These effects can partly be prevented by a judicious selection of basin site and orientation, by the application of wind screens in the form of hedges, etc. Complete inactivation of these factors can be obtained by covering the basin, but in practice this is only done when strictly necessary for other reasons, for instance to prevent freezing in winter time. For the purification of drinking water, covering has the advantage that contamination by wind blown material is no longer to be feared.

3.7. Tray settling tanks, tilted plate separators and tube settlers

Not only with the vertical baffles of the preceding section, but also with horizontal baffles a favorable change in the Reynolds and Froude numbers may be obtained. With continuous horizontal baffles only the hydraulic radius R is lowered, while next to this horizontal baffles of the round-the-end type also increase the horizontal velocity of flow v_o. The most important effect of these horizontal baffles or trays, however, is an enlargement of the area on which sludge may accumulate (fig. 3.34), in this way reducing the overflow rate and greatly promoting clarification efficiency.

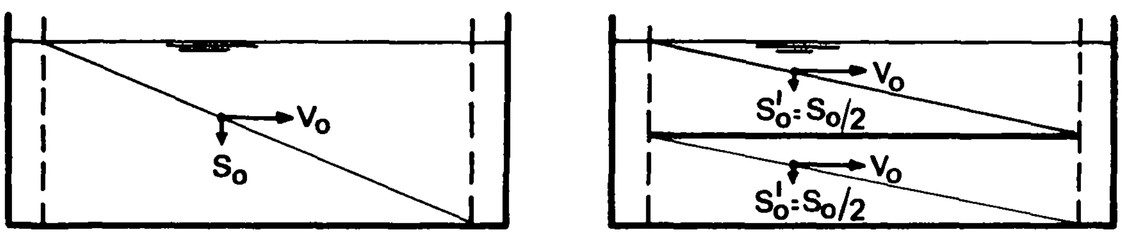


Fig. 3-34 Reduction in overflow rate by continuous horizontal baffles.

Tray settling tanks (fig. 3.35) consequently pack an enormous capacity in a small volume, while the additional cost of installing the trays is small, their structural strength only being necessary to carry their own weight next to the submerged weight of the sludge deposits. In the field of public and industrial water supplies tray settling tanks are used extensively. For the treatment of sewage, however, they are seldom applied as the larger volume and the putrescibility of the suspended matter asks for a continuous sludge removal, for which the mechanical installations are difficult to accomodate and to maintain in tray settling tanks. This difficulty in the meanwhile may be overcome by setting the trays at a steep

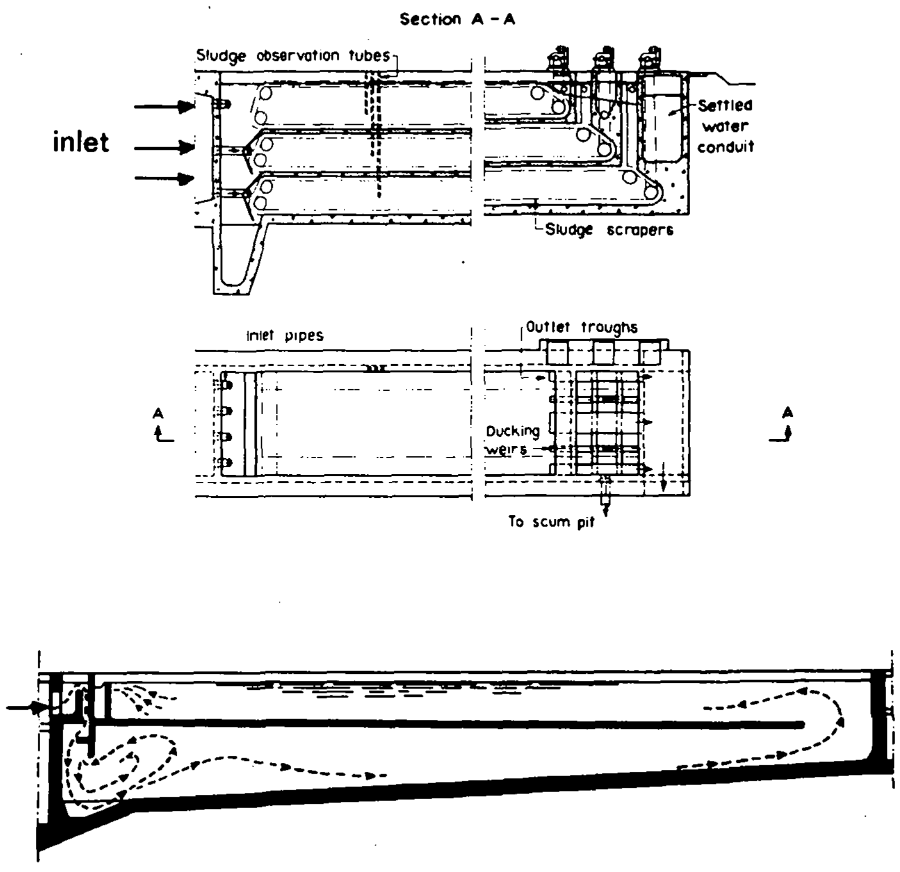


Fig. 3-35 Tray settling tanks.

angle, 60^0 for instance with the horizontal. In these so-called tilted plate separators (fig. 3.36), the sludge does not adhere to the plates, but slides down to the space below, for subsequent removal by normal means (compare section 3.9). The hydrodynamics of a tilted plate separator are shown in fig. 3.37, in which the value of s_o again determines the removal ratio. In the detention time T, the particle travels from A to B, which movement may be subdivided in a travel from A to C at a velocity v_o and a travel from C to D at a velocity s_o. With the notations of fig. 3.37

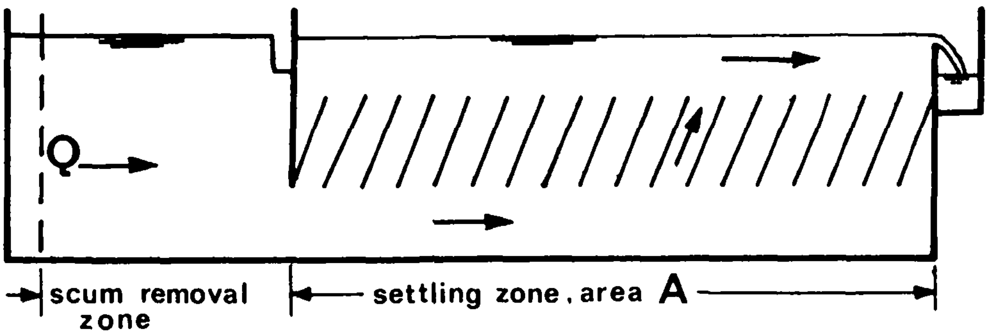


Fig. 3-36 Tilted plate separator.

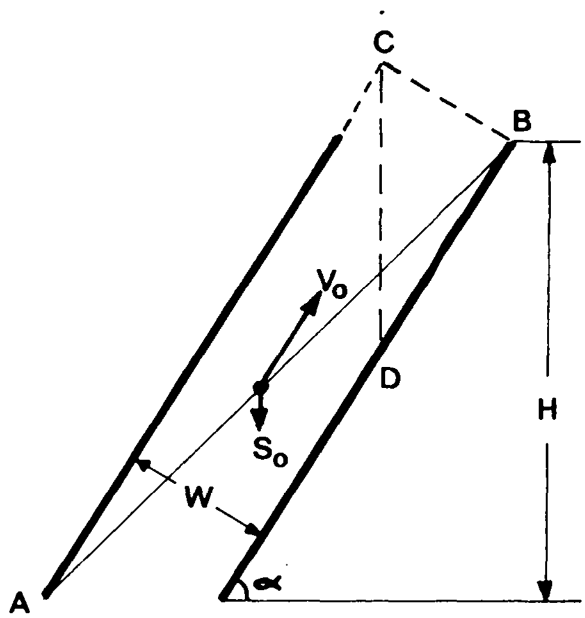


Fig. 3-37 Principle of tilted plate separation.

$$AC = \frac{h}{\sin\alpha} + \frac{w}{tg\,\alpha} = v_o T$$

$$CD = \frac{w}{\cos\alpha} = s_o T$$

from which follows

$$\frac{s_o}{v_o} = \frac{w \sin\alpha}{h\cos\alpha + w\cos^2\alpha}$$

When A is again the surface area of the settling zone and Q the capacity

than $v_o = \frac{Q}{A\sin\alpha}$ Substituted

$$s_o = \frac{Q}{A}\,\frac{w}{h\cos\alpha + w\cos^2\alpha}$$

Assuming for instance $w = 0.1$ m, $h = 1$ m and $\alpha = 60^0$ gives

$$s_o = \frac{Q}{A}\,\frac{0.1}{(1)(0.5)+(0.1)(0.5)^2} = 0.19\,\frac{Q}{A}$$

or an overflow rate which is 5.25 times as small as in a conventional tank of the same area. Disregarding a reduction in basin efficiency by turbulence or instability, fig. 2.13 requires for 95% removal an overflow rate less than $(0.25)10^{-3}$ m/sec. With a capacity of 0.5 m^3/sec, the required area thus becomes

$$A = 0.19\,\frac{Q}{s_o} = 0.19\,\frac{0.5}{(0.25)10^{-3}} = 380\ m^2$$

which now may be accomodated in a square tank, further reducing the cost of construction. The flow between the tilted plates is governed by

$$v_o = \frac{Q}{A\sin\alpha} = \frac{0.5}{(380)(0.866)} = (1.52)10^{-3}\ m/sec$$

$$R = \frac{w}{2} = 0.05\ m$$

$$t = 10\ ^0C,\quad \nu = (1.31)10^{-6}\ m^2/sec$$

$$Re = \frac{(1.52)10^{-3}(0.05)}{(1.31)10^{-6}} = 58$$

$$Fr = \frac{(1.52)^2 10^{-6}}{(9.81)(0.05)} = (0.5)10^{-5}$$

that is to say a flow which is fairly stable and completely laminar, assuring satisfactory operation.

A technical difficulty with tilted plate separators concerns the sagging of the plates by their own weight, asking for an additional stiffening to prevent difficulties during operation. In fig. 3.38 the desired stiffness is obtained by the use of corrugated sheets, while

Fig. 3-38 Corrugated plates.

in fig. 3.39 modules composed from square tubes are shown. The original set-up of these so-called tube settlers used a nearly horizontal flow (fig. 3.40 top) with a tube inclination of only 5^0 with the horizontal. The efficiency was extremely good, but for sludge removal the tank had to be drained and flushed out with water. With the steeply inclined tubes of fig. 3.40 bottom this is not required, allowing a continuous operation. The diameter of the square tubes is commonly 0.05 m, with a length of in average 0.9 m. The effective overflow rate thus becomes

$$s_o = \frac{Q}{A} \frac{0.05}{(0.9)(0.5)+(0.05)(0.5)^2} = 0.108 \frac{Q}{A}$$

giving for the example quoted above

$$A = (0.108)\frac{0.5}{(0.25)10^{-3}} = 216 \text{ m}^2$$

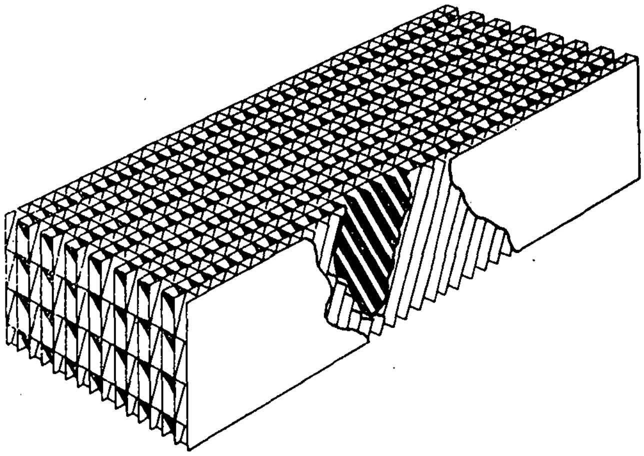


Fig. 3-39 Modules of tube settlers.

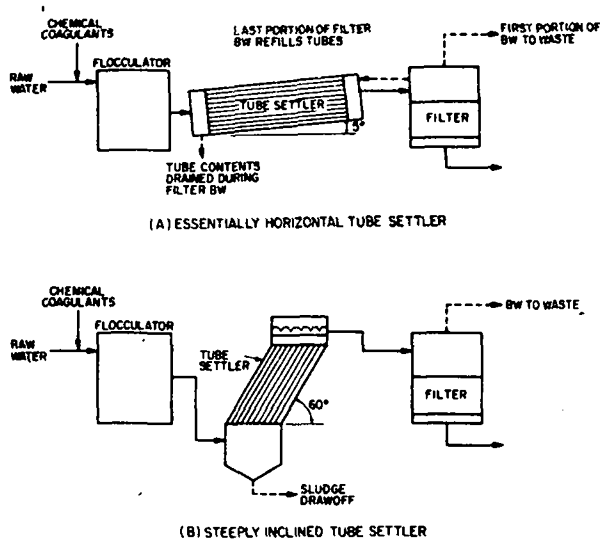


Fig. 3-40 Basic tube settler configuration.

$$v_o = \frac{0.5}{(216)(0.866)} = (2.67)10^{-3}\text{m/sec}$$

$$R = \frac{(0.05)^2}{(4)(0.05)} = 0.0125 \text{ m}$$

$$Re = \frac{(2.67)10^{-3}(0.0125)}{(1.31)10^{-6}} = 26$$

$$Fr = \frac{(2.67)^2 10^{-6}}{(9.81)(0.0125)} = (5.8)10^{-5}$$

assuring not only laminar but also a completely stable flow.

The use of trays, tilted plates or inclined tubes is not restricted to rectangular tanks, also in circular tanks they may be used to advantage. Fig. 3.41 shows such a tank into which afterwards tube modules have been installed to increase the capacity and to raise clarification efficiency.

Experience with tilted plate separators and tube settlers is still rather limited and mostly concerns experimental plants, not subject to the whims of strongly varying raw water quality and desired capacity. In economic respect they are very attractive indeed, but to quarantee a sound engineering solution, more research and above all more practical experience are needed.

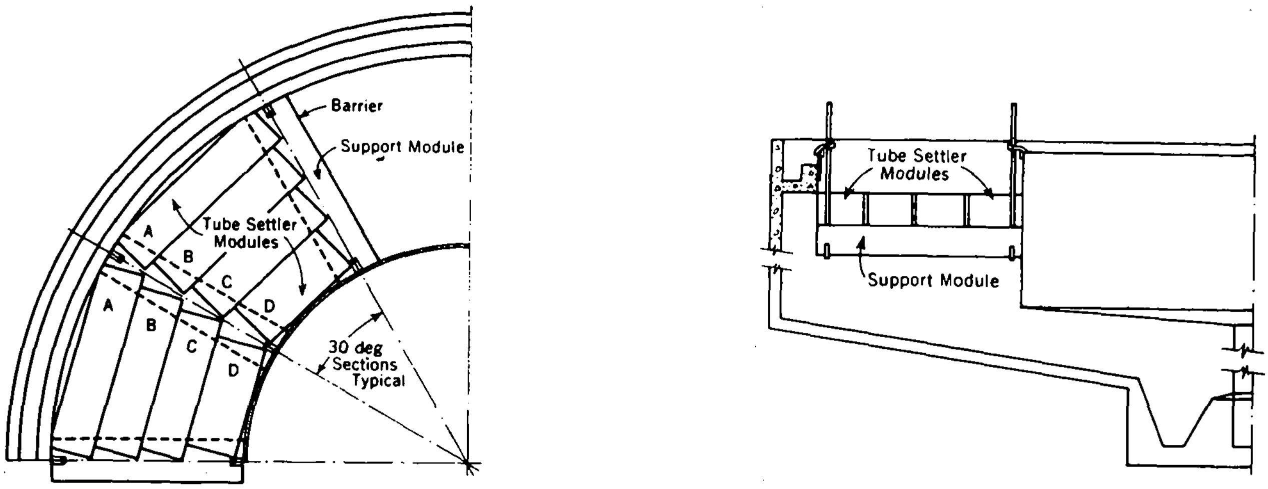


Fig. 3-41 Circular tanks with tube settling.

3.8. Inlet and outlet constructions

For ideal conditions (section 3.2.), the settling zone proper ought to be preceded by an inletzone, to divide incoming water and suspended particles equally over the full cross-sectional area of the tank and to mix incoming water with part of the tank content so as to prevent as much as possible the occurrence of density currents due to differences in temperature and suspended matter content. It goes without saying that these purposes should be achieved without creating an undue amount of turbulence and in particular without the introduction of concentrated jets of water which after shooting through would disturb and hinder the settling process.

An equal division of the water is especially important with regard to the width of the tank (fig. 3.42). Theoretically at least, the depth of the tank has no influence on clarification efficiency, neither an unequal division of the inflow over this depth (fig. 3.43). An equal distribution of the incoming water over the width of the tank can be obtained by subjecting this water to equal losses of head and can be approximated by subjecting the water to head losses which are large compared with the variation in piezometric level over the length of the inlet channel. The principle of equal head losses is shown in fig. 3.44, where by symmetry the water at each bifurcation is split in two equal parts. With regard to the cost of the pipes or channels involved, the number of inlets tends to be small, requiring a diffusor wall (fig. 3.45) for a finer sub-division over the full width and depth of the settling zone.

The use of controlling head losses large in comparison to the

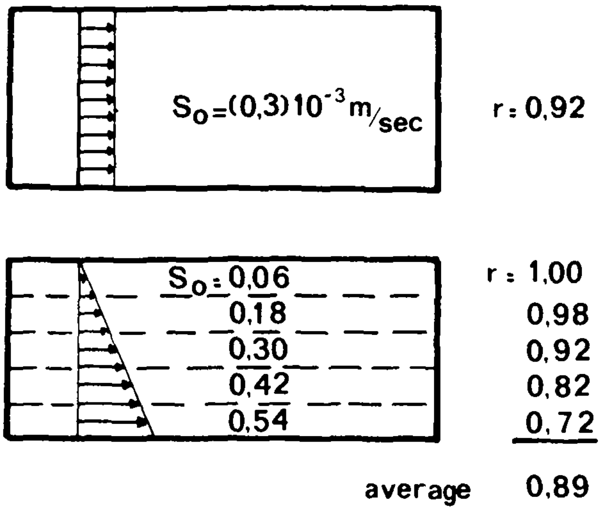


Fig. 3-42 Plan of tanks with different efficiencies. (according to fig. 3-21)

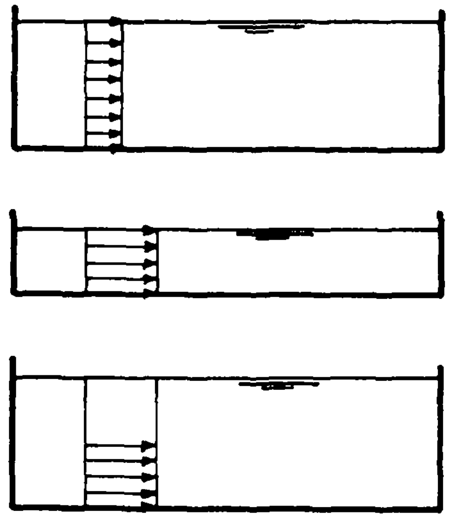
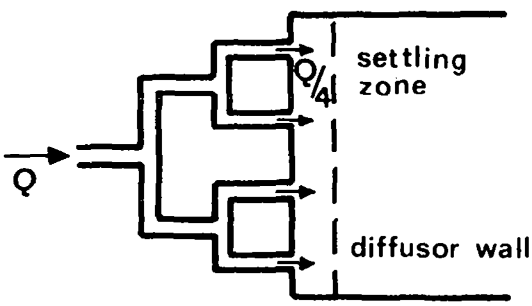


Fig. 3-43 Longitudinal section over tanks with equal efficiency.



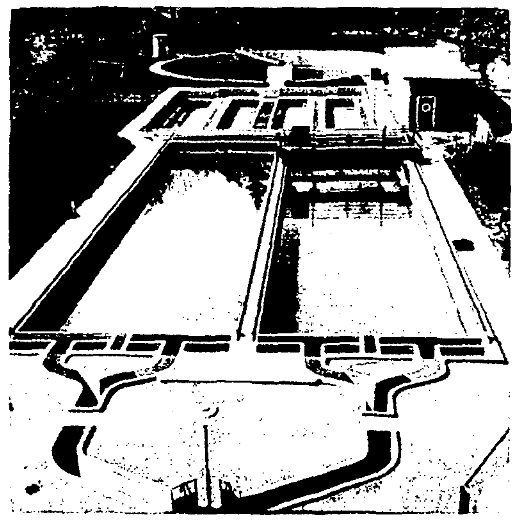


Fig. 3-44 Inletconstruction with equal losses of head.

Fig. 3-45 Diffusor wall.

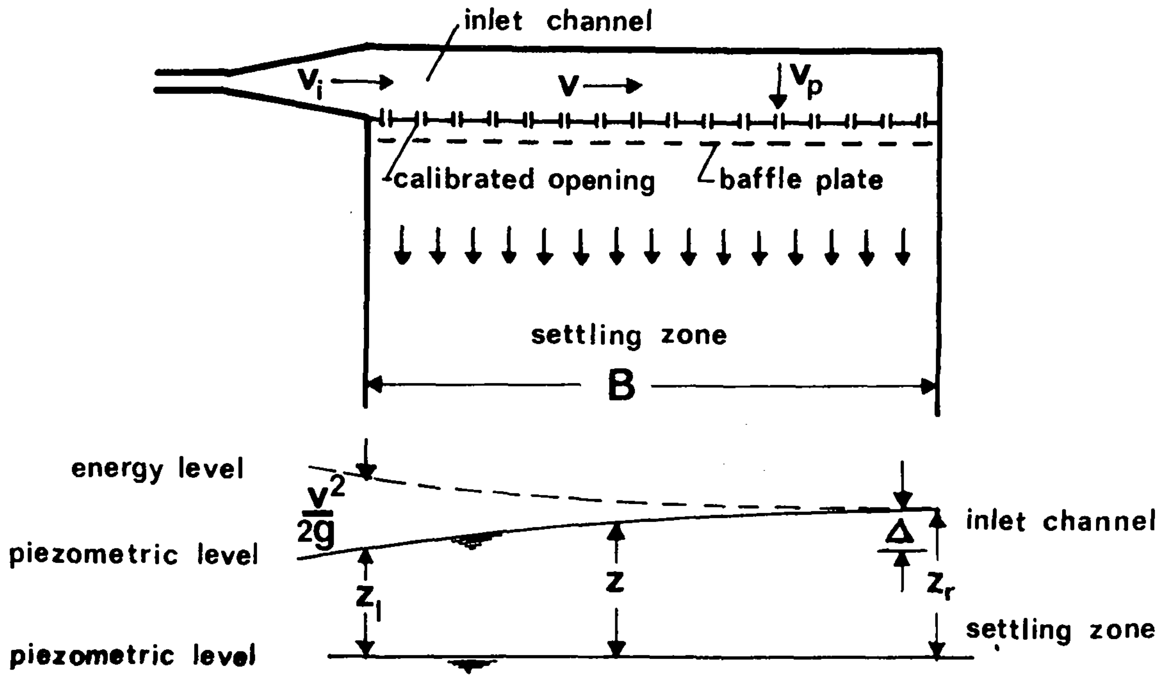


Fig. 3-46 Inletconstruction with calibrated openings.

variation in piezometric level ahead of the various inlet openings, is demonstrated in fig. 3.46. Over the length B of the inlet channel, the energy level drops by losses of friction and turbulence, while next to this the piezometric level (water level) rises due to recovery of velocity head. With the notations of fig. 3.46 the net rise of piezometric level amounts to

$$\Delta = \frac{v_i^2}{2g} - \frac{\lambda}{3}\frac{B}{D_h}\frac{v_i^2}{2g} - n\,\frac{(v_i/n)^2}{2g}$$

with λ as friction coefficient, D_h as the hydraulic diameter of the inlet channel and n as the number of calibrated openings. From the first to the last opening, the discharge will now vary from

$$Q_p = \mu F\sqrt{2gz_l} \qquad \text{to}$$

$$Q_p = \mu F\sqrt{2gz_r} = \mu F\sqrt{2g(z_l + \Delta)}$$

with μ as discharge coefficient and F as area of the opening. In case this variation in discharge must be limited to 5%, the allowable variation in controlling head loss equals 10%. In formulae

$$z_1 + \Delta < 1.10\, z_1 \qquad \text{or} \qquad z_1 > 10\,\Delta$$

For design C in table 3.1, the capacity equalled 0.5 m^3/sec and the tank width 15 m. To reduce sludge accumulation in the inlet channel as much as possible, the velocity v_i may not be to small. With $v_i \approx$ 0.6 m/sec, the required cross-sectional area amounts to 0.5/0.6 = 0.83 m^2, for instance a depth of 1.2 m and a width of 0.7 m. The hydraulic diameter of the inlet channel thus becomes

$$D_m = 4\,\frac{(0.7)(1.2)}{0.7 + 2(1.2)} = 1.17 \text{ m}$$

With λ = 0.04 and n = 30 openings

$$\Delta = \frac{v_i^2}{2g}\left(1 - \frac{0.04}{3}\,\frac{15}{1.17} - \frac{1}{30}\right) = 0.79\,\frac{v_i^2}{2g}$$

and with

$$v_i = \frac{0.5}{(0.7)(1.2)} = 0.595 \text{ m/sec}, \qquad \frac{v_i^2}{2g} = 0.018 \text{ m}$$

$$\Delta = (0.79)(0.018) = 0.0143 \text{ m} \qquad \text{and}$$

$$z > (10)(0.0143) \quad \text{or} \quad z > 0.143 \text{ m}$$

With 30 openings

$$Q_p = \frac{0.5}{30} = 0.0167 \text{ m}^3\text{/sec}$$

$$\mu F = \frac{Q_p}{\sqrt{2gz}} = \frac{0.0167}{\sqrt{(2)(9.81)(0.143)}} = 0.010 \text{ m}^2$$

or with μ = 0.3 to 0.7 circular openings with diameters of 0.135 to 0.205 m. The velocity in these openings is quite high

$$v_p = \sqrt{(2)(9.81)(0.143)} = 1.67 \text{ m/sec}$$

and baffle plates are consequently necessary to prevent a shooting-through of this jet of water. Various constructions can be applied, but under all circumstances the passage ways should not to be too small and should allow an easy cleaning (fig. 3.47 left). With more elaborate design as shown in fig. 3.47 on the right, such a fine distribution

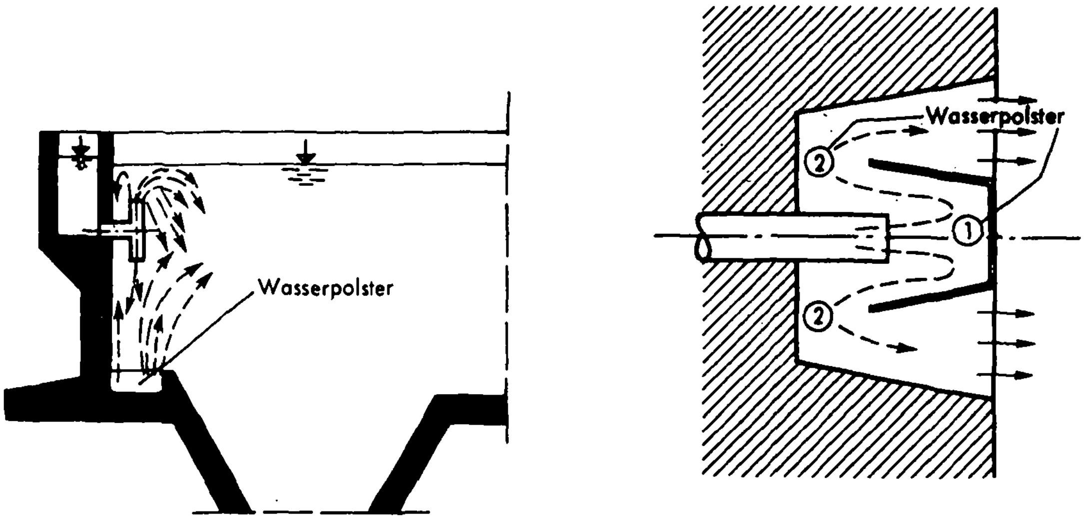


Fig. 3-47 Clifford inlet and Stuttgarter inlet.

of the water can be obtained, that a diffusor wall is no longer required for this purpose. Notwithstanding a higher value of v_i, the velocity v near the end of the inlet channel will always be low. Sludge accumulation here can still be prevented by blowing in air (fig. 3.48) which for sewage treatment has the added advantage of freshening the raw liquor, preventing putrescense and bad odors during the subsequent settling process.

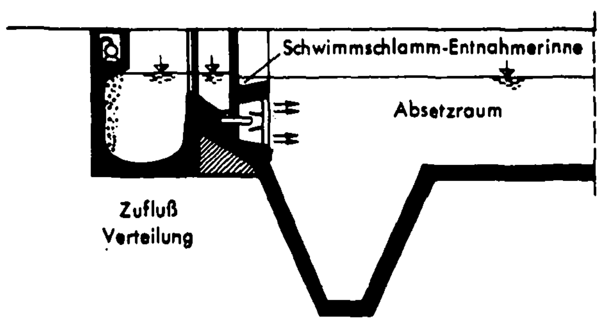


Fig. 3-48 Aerated inlet channel.

With circular settling tanks and centre feed, all inlets are at the same distance from the point of supply. Relatively large openings and small outflow velocities are thus allowable and simple baffle constructions will suffice (fig. 3.49). In case the centre well has a larger diameter, to avoid turbulent flow in the settling zone (section 3.3), more certainty of equal distribution can be obtained by a tangential inlet, increasing the Froude number and stabilising the flow as well as by smaller openings with a larger controlling loss of head, which require however, a more complicated baffle construction (fig. 3.50). With peri-

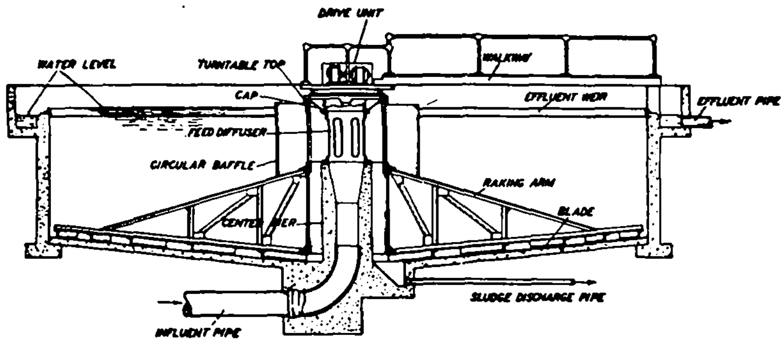


Fig. 3-49 Feed well with simple inlet construction.

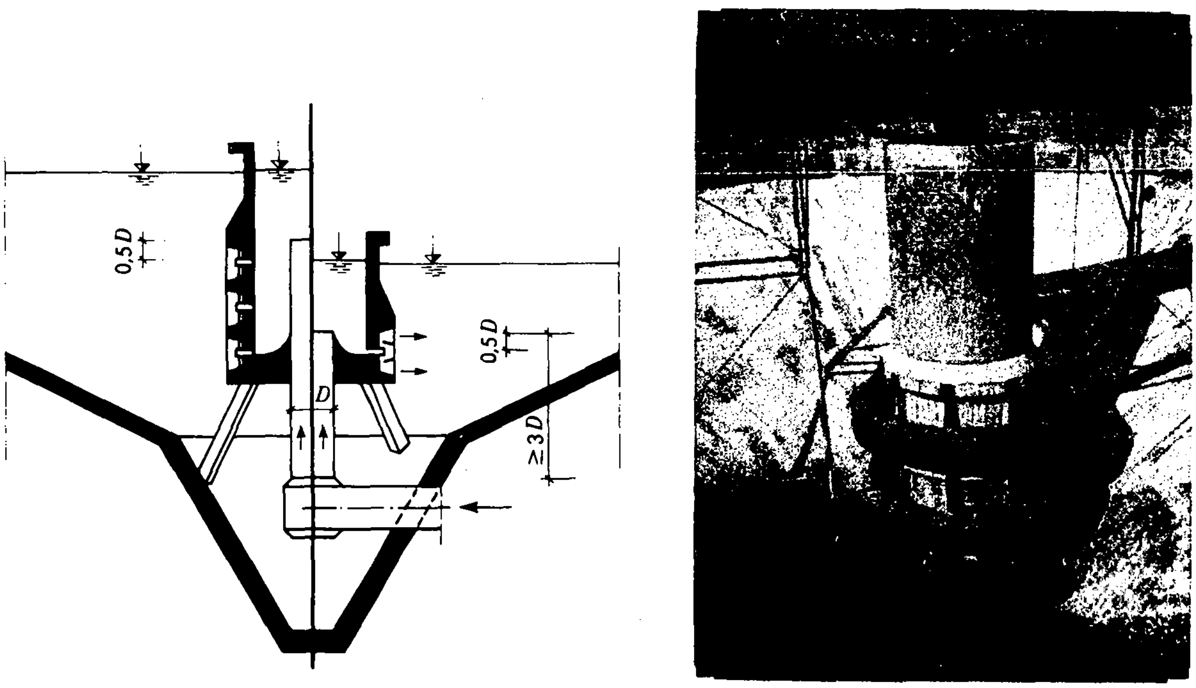


Fig. 3-50 Feed well with Stuttgarter inlet.

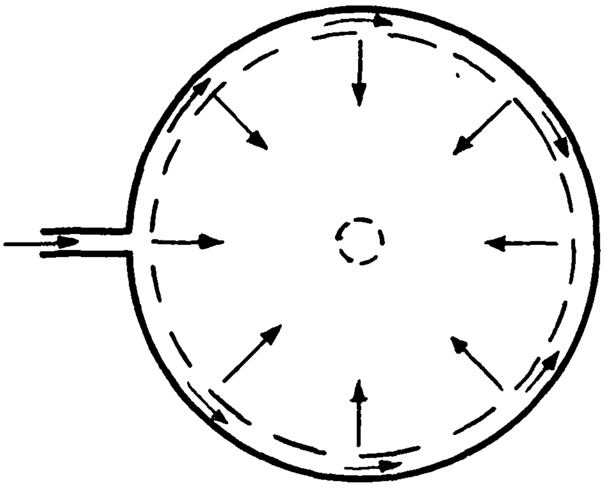


Fig. 3-51 Peripheral feed.

pheral feed, the distance travelled by the raw water from the point of supply to the various inlet openings will vary strongly (fig. 3.15). An equal distribution of the inflow over the full circumferential length of the tank can now only be obtained by the use of small inlet openings and a large controlling loss of head. This also means concentrated jets of water, the energy of which must be dissipated by baffles to prevent a disturbance of the settling process.

The outlet zone of an ideal settling tank serves to collect the clarified liquid evenly over the full cross-sectional area of the basin. As in this zone the water is accelerated, the flow will be quite stable and even with concentrated outlets good results can be obtained. For rectangular tanks weirs over the full width of the tank are commonly applied. Hydrodynamically the single weir of fig. 3.52 satisfies all requirements, but the weir loading (discharge per unit length) may not be too high as otherwise the settling of suspended matter near the end

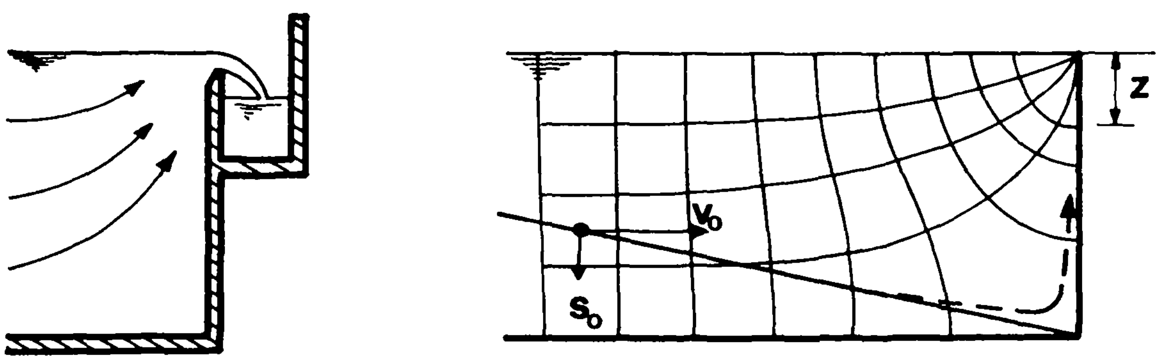


Fig. 3-52 Outlet weir over full width of tank.

of the tank will be disturbed. This will occur when the downward movement of the particle due to its settling velocity is undone by the updraft velocity created by the weir discharge. At a small distance z from the weir crest (fig. 3.52 right), this updraft velocity equals

$$v_z = \frac{Q}{B \frac{\pi}{2} z}$$

near the bottom of the tank, the updraft velocity will be smaller than follows from this formula. With an estimated reduction factor 3

$$v_H = \frac{1}{3} \frac{Q}{B \frac{\pi}{2} H} = \frac{1}{5} \frac{Q}{BH}$$

When this velocity must be smaller than the overflow rate s_o, the allowable weir loading becomes

$$\frac{Q}{B} < 5Hs_o \qquad \text{With}$$

$$s_o = \frac{Q}{BL} \qquad \text{this requirement is}$$

automatically fullfilled in case

$$\frac{L}{H} < 5$$

With the exception of turbulent settling in grit chambers for instance, the ratio between length and depth of the settling basin is mostly larger than 5. A smaller weir loading is now required, which can easily be obtained by installing additional weirs (fig. 3.53) with a total length nB

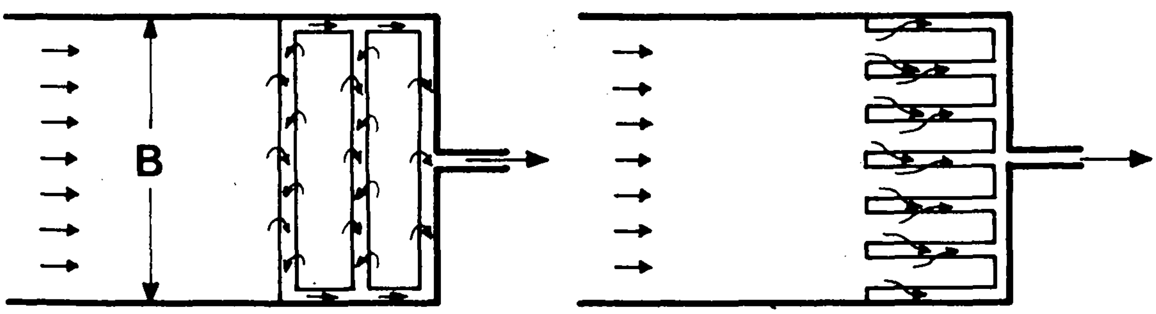


Fig. 3-53 Multiple outlet weirs, combined length nB.

$$\frac{Q}{nB} < 5Hs_o$$

In design C of table 3.1, $Q = 0.5$ m^3/sec, $B = 15$ m, $H = 2$ m and $s_o = (0.37)10^{-3}$ m/sec. Substituted

$$\frac{0.5}{n(15)} < (5)(2)(0.37)10^{-3} \quad \text{or} \quad n > 9$$

that is to say a total weir length of 135 m and a weir loading of $(3.7)10^{-3}$ m/sec.

With circular tanks and centre feed, a single weir around the outer circumference will be adequate when

$$\frac{Q}{\pi D} \leq 5Hs_o \qquad \text{or} \qquad \frac{D}{H} < 20$$

which mostly will be the case. If not, multiple weirs are necessary, for instance as shown in fig. 3.54. With peripheral feed, the updraft velocities near the points of discharge in the centre of the tank will

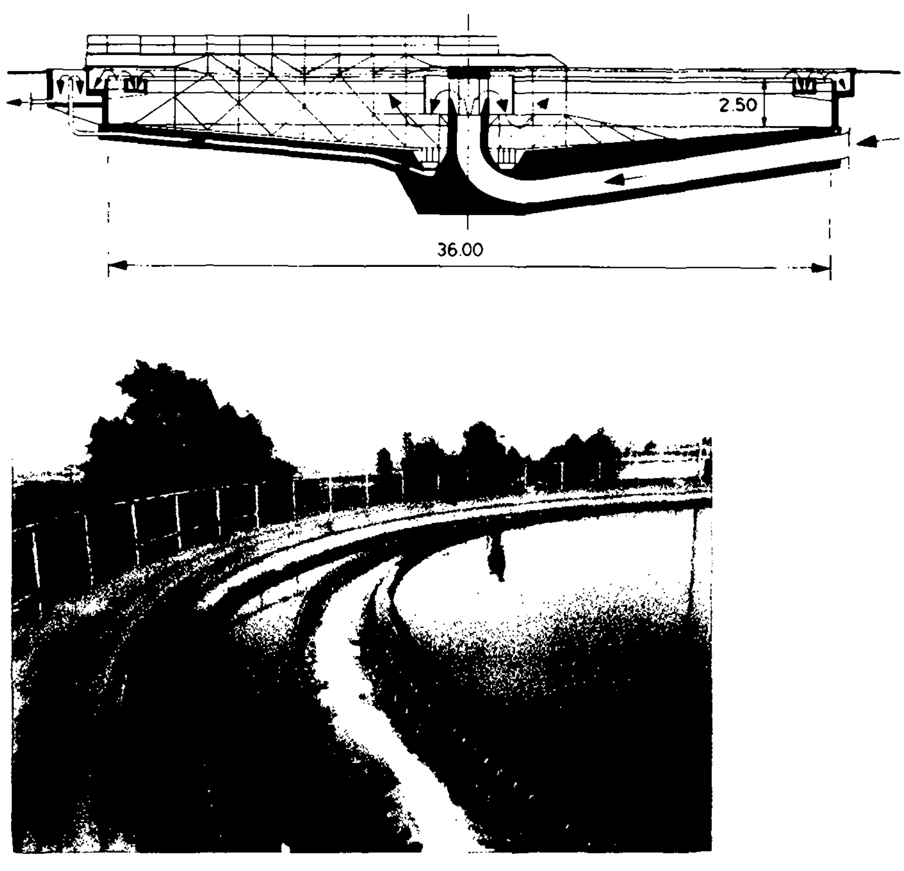


Fig. 3-54 Multiple outlet weirs in circular tanks with centre feed.

be so high that a disturbance of the settling process cannot be prevented. This disturbance can be inactivated, however, by choosing a low surface loading so that near the centre of the tank the settling process has been completed and no further deposition occurs.

In the field of water and waste water engineering, common weir loadings q are $(2 - 3)10^{-3}m^3/m'/sec$. With straight weirs this means very small overflow heights h (fig. 3.55) of 10 - 14 mm only, where a deviation by 1 mm from the horizontal already means an unequality in discharge of 10 - 15%. This is the reason that commonly notched weirs are applied, made of steel and fastened to the concrete tank structure in such a way that an easy and accurate adjustment of the horizontal position is possible. V-notches are most popular as they tend to be self-cleaning, but with rectangular openings a deviation in the vertical position has less influence on the discharge

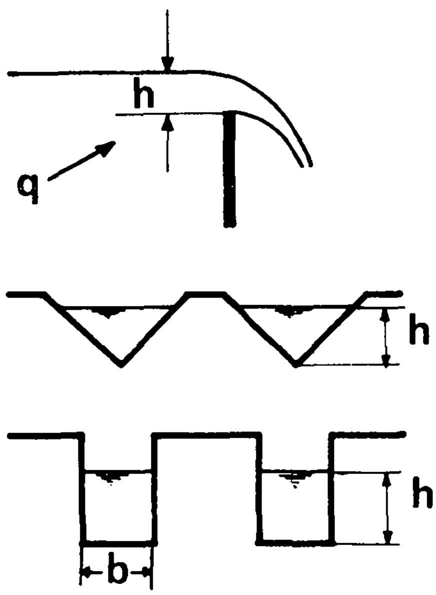


Fig. 3-55 Notched weirs.

V - notch, 90^0, $q_o = 1.4\ h^{5/2}$ $\qquad \frac{dq_o}{q_o} = 2.5\ \frac{dh}{h}$

U - notch, $q_o = 1.8\ bh^{3/2}$ $\qquad \frac{dq_o}{q_o} = 1.5\ \frac{dh}{h}$

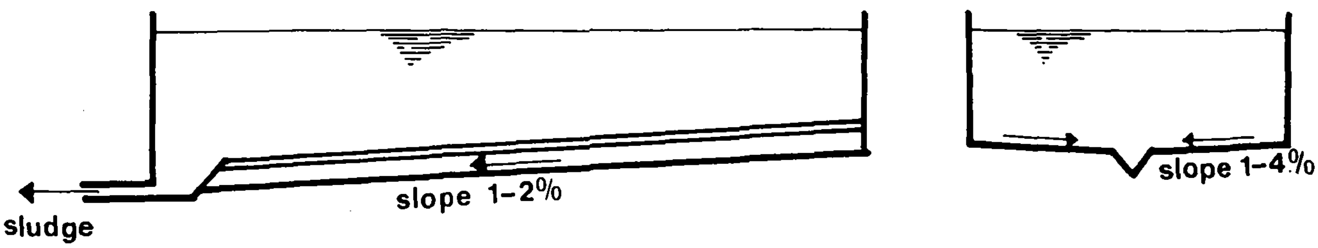
with q_o as discharge per opening

It should be emphasized once again, that the area of the settling zone equals the full area of the tank minus the areas occupied by the inlet and the outlet zone. In rectangular tanks the inlet zone extends to 0.5 - 1 times the tank depth H beyond the supply openings, while the outlet zone begins at a distance of about the tank depth ahead of the first discharge weir.

3.9. Sludge removal and skimming devices

During the settling process, the suspended particles heavier than water accumulate in the sludge zone at the bottom of the tank, from which they are removed periodically or more or less continuously, depending on local circumstances. Removal at intervals is only possible when the volume of the suspended matter retained is not too large and when this material is stable, not subject to putrefaction. These conditions are often fullfilled in the field of public and industrial water supply engineering, where the turbidity of the river water is rather low and entirely due to mineral soil particles. In the example dealt with in fig. 3.10, the maximum sludge accumulation amounts to $(45.5)10^{-3}$ grams/m^2/sec and with an assumed water content of 95% to about $(900)10^{-9}$ m^3/m^2/sec or per month (2.6 million seconds) a thickness of 2.3 m. When an adequate tank depth is provided, for instance in this case 3.5 m over the first 20 m length of the tank, cleaning may be done at intervals of about 4 weeks. Most effectively this cleaning can be achieved by cutting the tank out of service and dewatering, after which the sludge is flushed with pressure water taken from hydrants to a hopper at the inlet end of the tank, from which it is subsequently removed by gravity or by pumping. To facilitate this hydraulic cleaning, the tank bottom should slope in two directions and be provided with a longitudinal channel as indicated in fig. 3.56. With the exception of the hydrants around the rim of the basin, this manual cleaning does not ask for additional equipment. As such it is cheap in cost of construction, but expensive in terms of labor. This is the reason that it is not longer applied in western-type countries. When here periodic cleaning is to be used, it is mostly done with some kind of mechanical equipment such as the suction dredger of figs. 3.57 and 3.58

Continuous sludge removal is a necessity when sludge volumes are large or the sludge is unstable, resulting in anaerobic decomposition during storage in the sludge zone. By this process taste



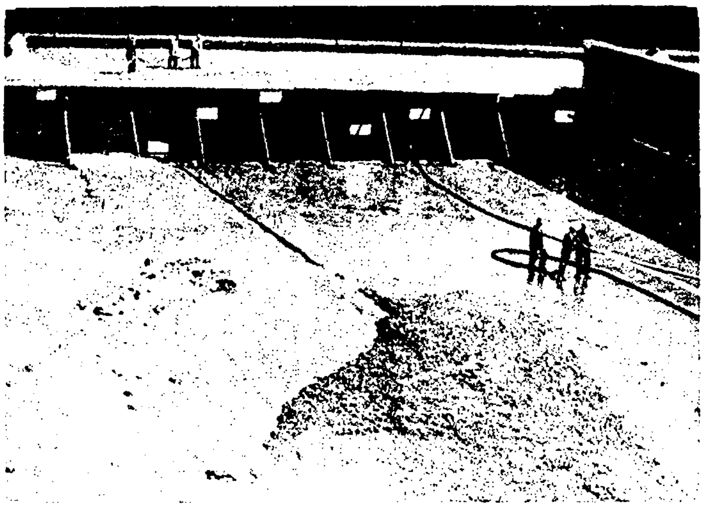
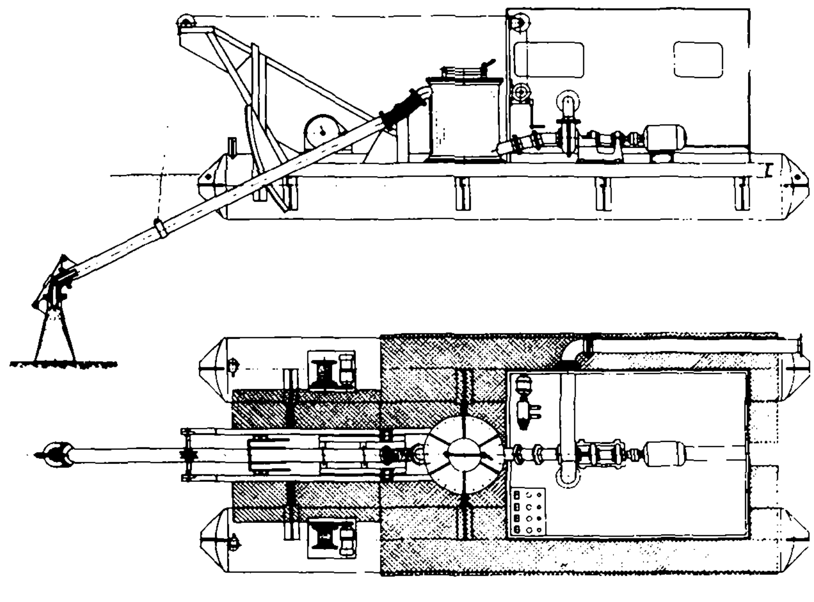


Fig. 3-56 Manual sludge removal.

and odor producing substances are formed and gases are generated which might lift the sludge and when rising through the supernatant water will disturb the settling process. Continuous sludge removal has the added advantages of a saving in the cost of labor, of a shallower sludge zone reducing the depth of the basin and its cost of construction, while additional basins to replace these taken out of operation for periodic cleaning are no longer required. On the other hand it must be realized, that mechanical sludge removal is expensive, increasing the building cost of a settling tank by about 10 % . Generally speaking it is economically attractive when the suspended load is larger than 1 kg/m^3. In the field of sewage treatment, however, continuous sludge removal is used without exception, moreover because here manual cleaning would be an extremely unpleasant task.

Almost without exception, continuous sludge removal in rectangular tanks is effected by mechanical means, using scrapers carried by pairs of endless chains or supported from a travelling bridge which push the sludge to a hopper near the inlet end of



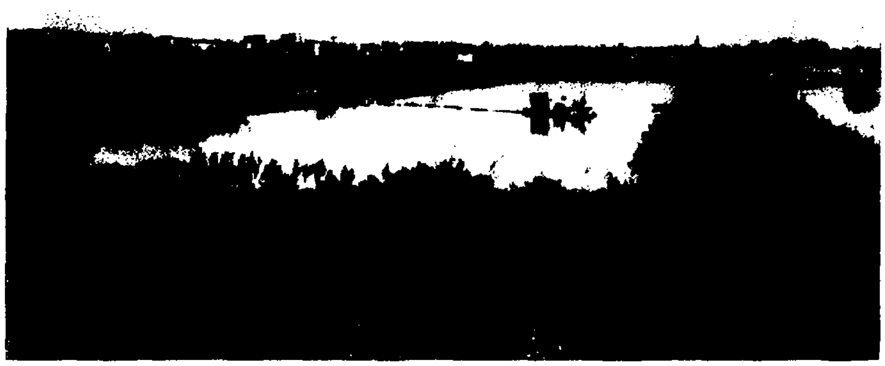
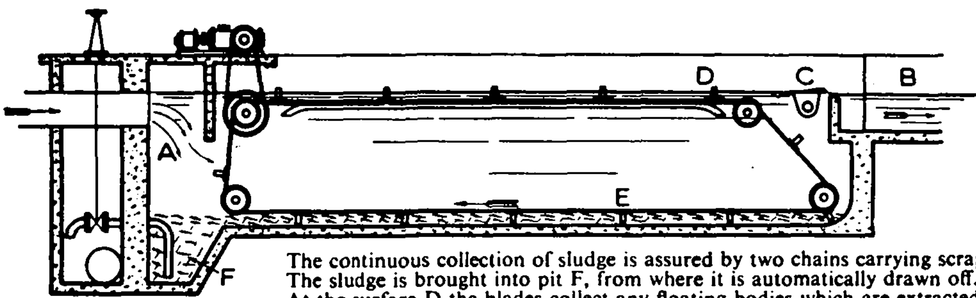


Fig. 3-57 Floating suction dredger.

Fig. 3-58 Suction dredger suspended from a travelling crane.

the tank, where the sludge accumulation is highest. Chain carried scraper blades are shown in fig. 3.59 and when necessary may also be installed entirely below water, in a small depth just above the tank bottom (fig. 3.60). The intervals between the scraper flights is commonly about 3 m and the speed of movement (5 - 15)10^{-3} m/sec, depending on local circumstances. With regard to the structural strength, the span of the flights is limited to 6 m, tanks of greater width to be subdivided in a number of raking compartments. Chain carried scrapers offer the cheapest solution for mechanical sludge removal, but the presence of moving parts below water makes them more vulnerable, while for repairs the tank must first be drained. A better solution in technical respect can be obtained by supporting the rakes from a travelling bridge, running on rails set on the



The continuous collection of sludge is assured by two chains carrying scraper blades E.
The sludge is brought into pit F, from where it is automatically drawn off.
At the surface D the blades collect any floating bodies which are extracted via the trough C.
A. Water inlet. B. Clarified water outlet.

Fig. 3-59 Chain carried scrapers in a plain settling tank.

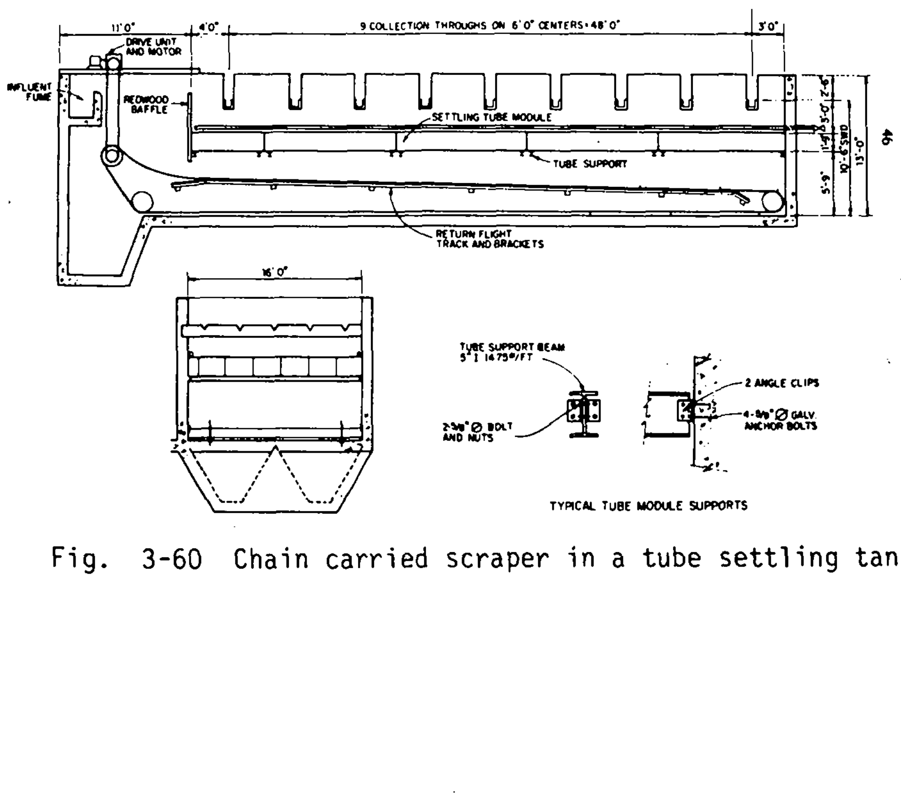


Fig. 3-60 Chain carried scraper in a tube settling tank.

longitudinal walls of the tank (fig. 3.61). Cleaning now only occurs when the carriage moves back to the inlet, asking for somewhat higher speeds of $(10 - 50)10^{-3}$ m/sec, but the equipment is always accessible for maintenance and repairs, the scraper blades after they have been lifted out of the water. To facilitate the movement of sludge, the bottom of the tank may be constructed with a longitudinal slope of 1% or more when possible.

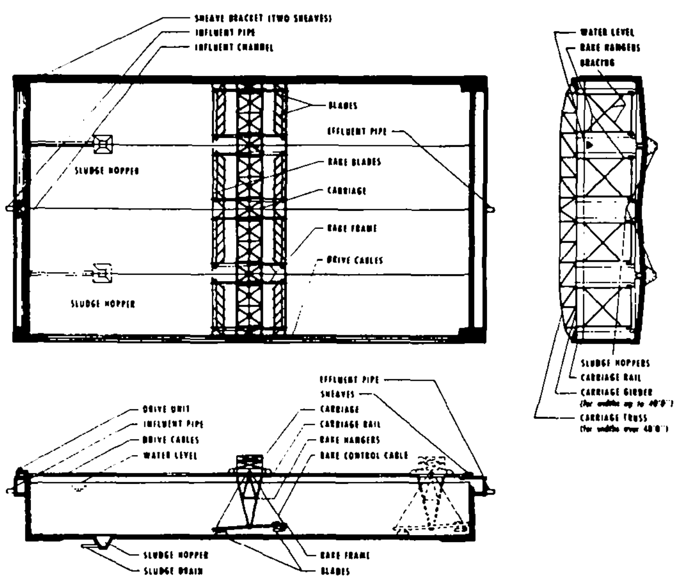


Fig. 3-61 Rakes suspended from a travelling bridge.

Mechanical sludge removal in circular tanks is of the rotating type, having radial arms to which the scraper blades are fastened at an angle so as to move the sludge to a hopper in the centre. Formerly the whole construction was below water (fig. 3.62), subject to additional wear and bear. Nowadays the rakes are suspended from a revolving

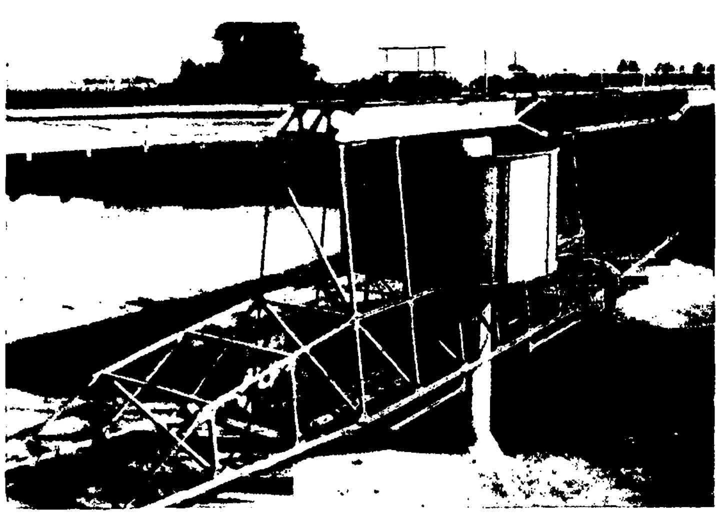


Fig. 3-62 Rotating sludge scrapers below water.

bridge above water (fig. 3.63), in such a way that they can be easily lifted for maintenance and repairs. The rotational speed is rather low, about 1 revolution per hour. The detention time of the sludge in the tank, however, is still small as by a slope of the tank bottom at

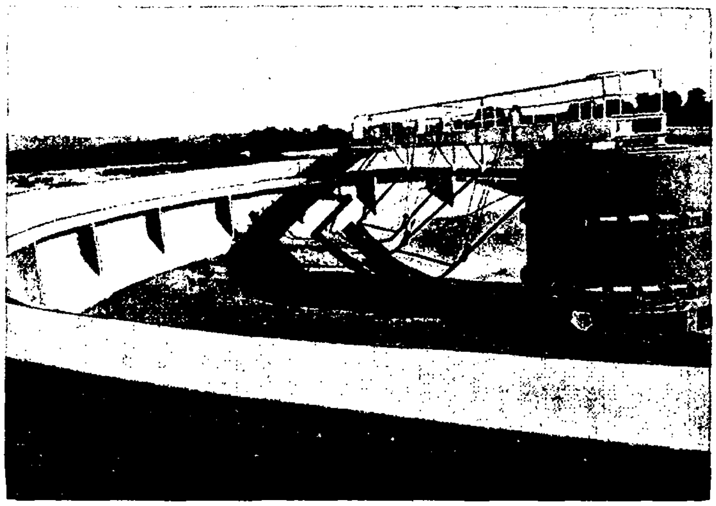


Fig. 3-63 Rotating sludge scraper suspended from revolving bridge.

8% the greater part of the sludge flows by gravity to the centre hopper, the scrapers only serving to overcome inertia and to avoid sludge adherence to the tank bottom. When the mass density of the sludge is small, as sometimes occurs in activated sludge secondary tanks, the rotating scrapers only whirl up the settled out material, without moving it to the sludge hopper in the centre. Better results can now be obtained with the suction sludge collector of fig. 3.64, in principle consisting of

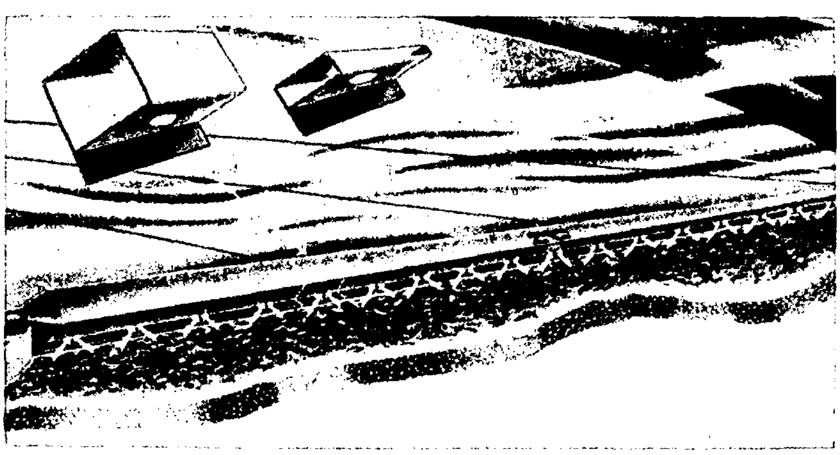


Fig. 3-64 Suction sludge collector in circular tanks.

rotating tubes provided with a serie of orifices through which the sludge is withdrawn by the difference in level between the water surface in the tank and the sludge outlet. Without any doubt, revolving scrapers in circular tanks do a better job than travelling scrapers in rectangular tanks. When to avoid putrescence a rapid sludge removal is desired, circular tanks are at a premium. This is the reason that notwithstanding their deficiencies in other respects, they are often applied for the treatment of sewage and industrial wastes. Nowadays, however, rotating scrapers can also be accomodated in rectangular tanks, for instance as demonstrated in fig. 3.65.

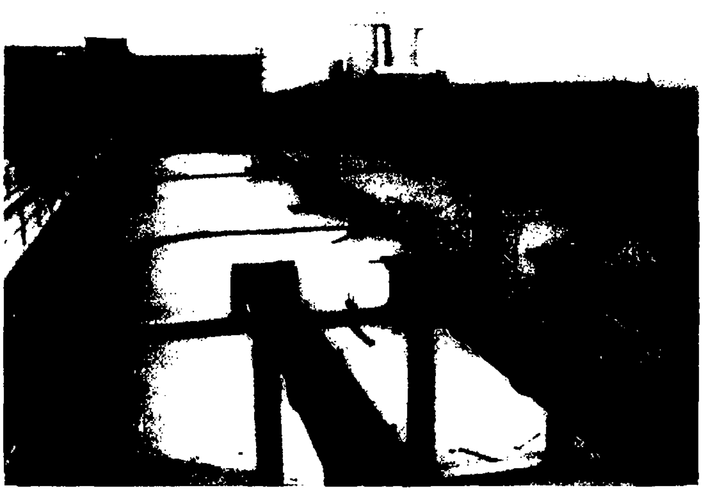
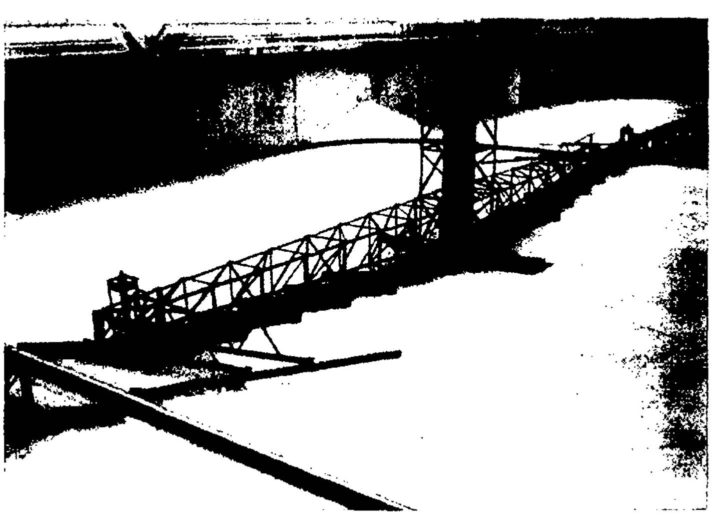


Fig. 3-65 Rotating scrapers in rectangular tanks.

During the settling process, suspended matter with a mass density less than that of the surrounding fluid will rise upward and accumulate at the water surface. A scum baffle, directly ahead of the outlet weirs (fig. 3.66), prevents this floating matter to escape with the



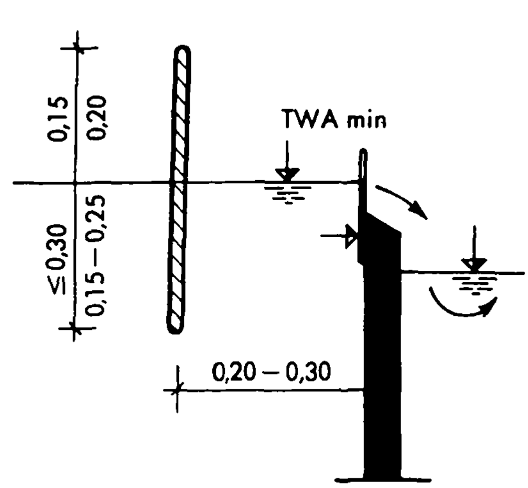


Fig. 3-66 Scum board.

effluent. In rectangular tanks, the scum board is often combined with a scum trough, into which the scum is conveyed by the sludge collecting mechanism on their return travel (fig. 3.67 and 3.68). In circular tanks,

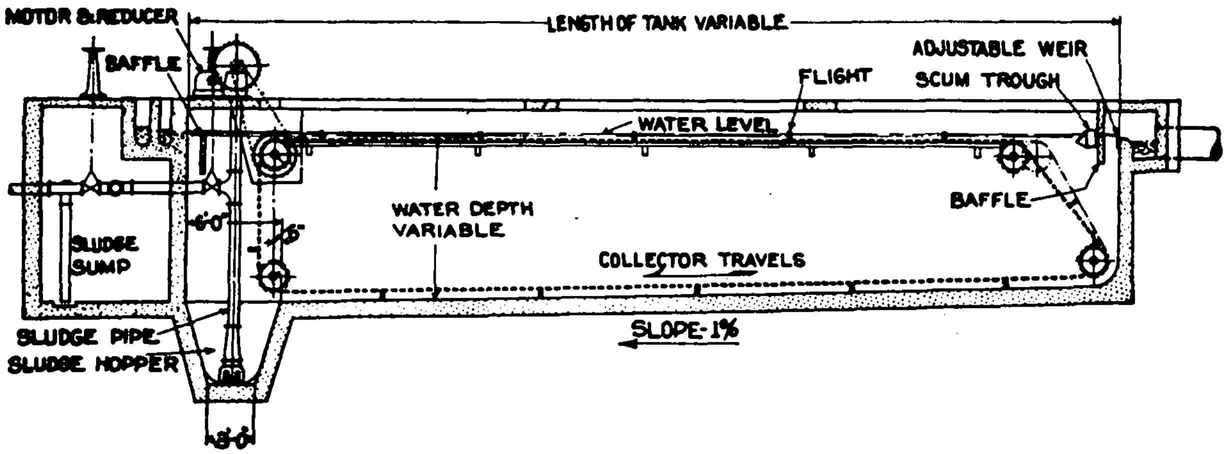


Fig. 3-67 Chain carried scrapers for sludge removal and skimming.

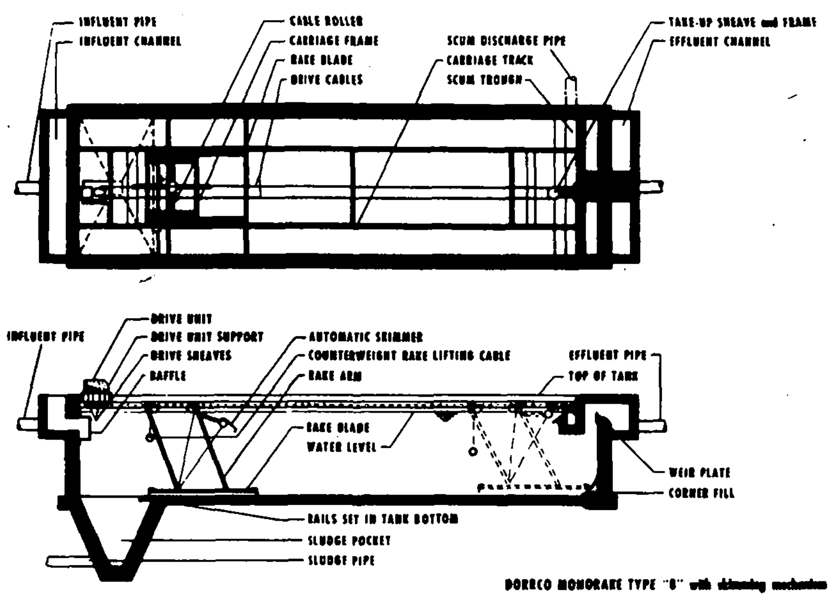


Fig. 3-68 Travelling bridge for sludge removal and skimming.

the skimming blade is commonly suspended from the rotating bridge and the scum deposited in a special sump, as shown in fig. 3.69. Disposal of the scum is often a difficult problem, that can best be solved by pumping it together with the sludge to the digester tanks.

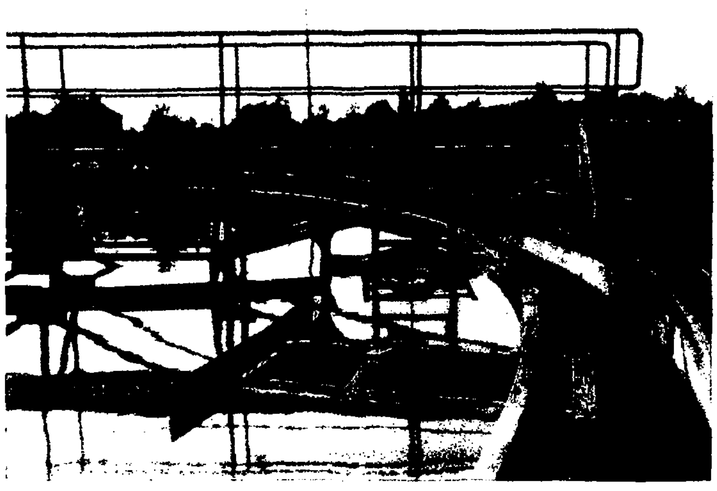


Fig. 3-69 Skimming in circular horizontal flow settling tanks.

4. Flocculent settling

4.1. Principles of flocculent settling

When a suspension containing particles with different velocities of subsidence, is subjected to some settling process, the smaller and lighter particles will be overtaken by those of larger size or higher mass density, resulting in a number of collisions. With flocculent settling, part of the colliding particles will coalesce to form aggregates which settle at rates higher than did the parent particles (fig. 1.3). This primary aggregation process continues as long as there are particles in suspension, the average settling velocities becoming higher and higher (fig. 4.1), while next to this the combinations formed in first instance will unite to larger and larger flocs, augmenting also the maximum settling velocities (fig. 4.2.). With depth below the water surface the settling process becomes faster and faster, greatly increasing settling efficiency. As a consequence, the removal ratio with flocculent settling is not only larger as the overflow rate s_o is smaller, but will also increase as the basin depth H is greater, which factors together constitute the detention time $T_o = H/s_o$. With floccu-

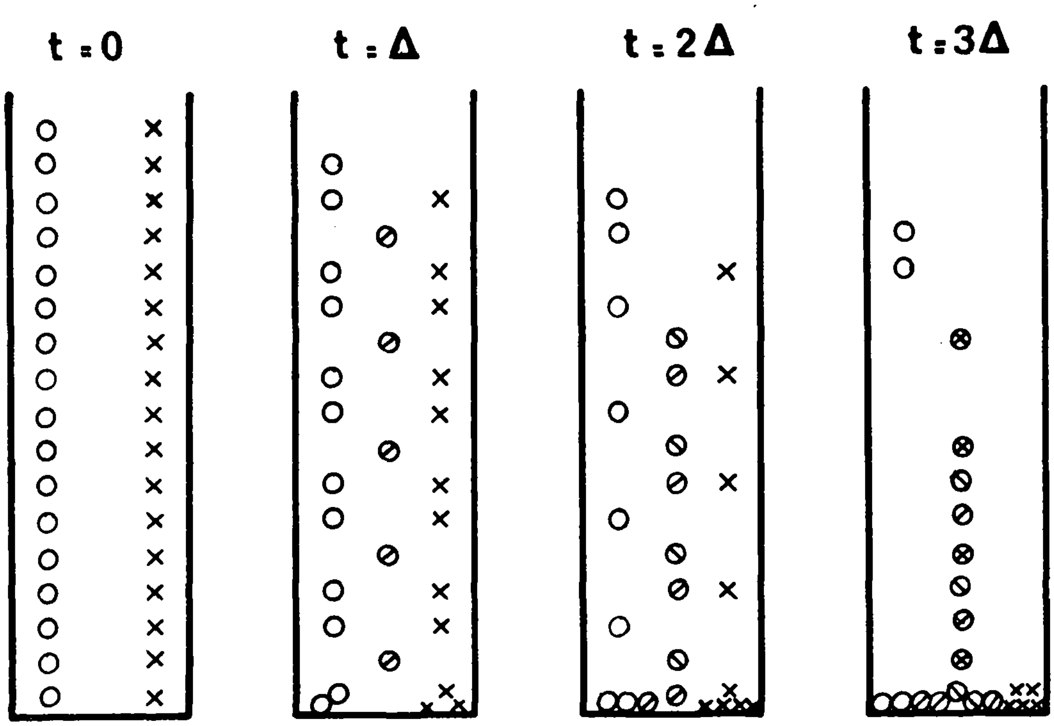


Fig. 4-1 Primary flocculation.

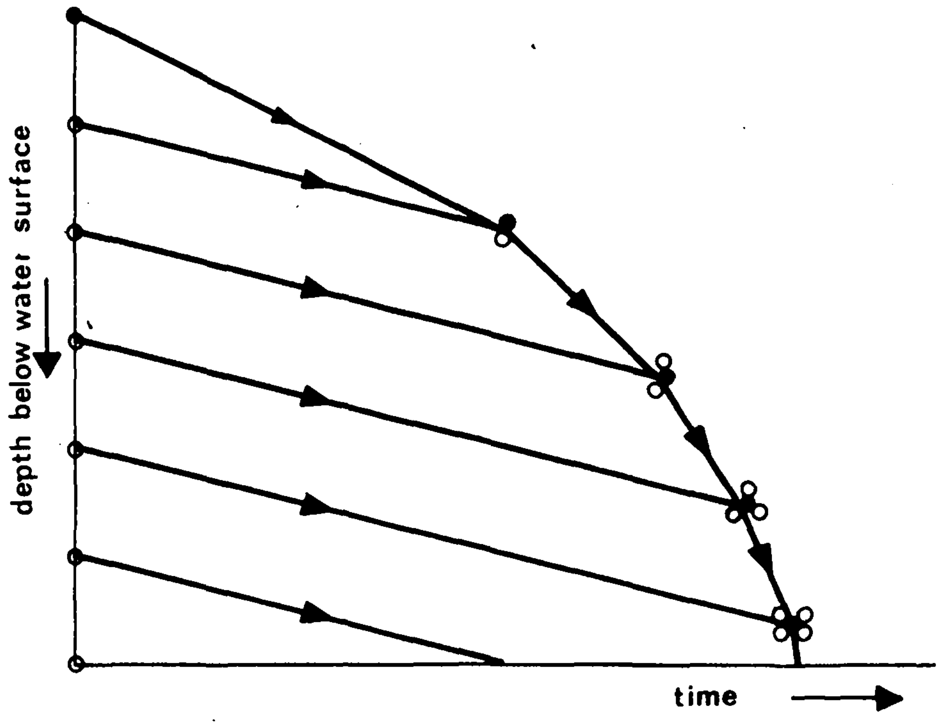


Fig. 4-2 Continued flocculation.

lent settling this detention time is of paramount importance.

With the mathematical theory of flocculation by differential settling, the number of contacts per unit volume and per unit time is found to be proportional to the number of particles and to the difference between their settling velocities. Otherwise, however, this theory is of little help. It fails to predict which part of the contacts will result in unison and is neither able to ascertain the ultimate floc size as resultant of aggregation on one hand and break-down on the other hand. The first part of this statement goes without saying, while as regard the second one it should be remembered that with increasing floc size also the settling velocity goes up, subjecting the floc to larger forces exerted by the upward flowing water on the downward moving particle. These forces are equal to the submerged weight of the floc, that is to say are proportional to the third power of the floc diameter. The shearing force per unit surface area of the floc is consequently linear proportional to this diameter. At a certain floc size the shearing force becomes so high that it is able to tear the floc apart, preventing a further growth to take place.

Lacking the help of theoretical considerations, the data necessary for the design of settling tanks meant for the clarification of flocculent suspensions, can only be found by laboratory tests. Again the apparatus of fig. 2.9 is used, but now of a much greater height, exceeding the maximum depth envisaged for the settling tank to be built. At a number of distances h below the water surface the suspended solids content, or any other property that is reduced by settling, is measured from samples taken at various time intervals t after the start of the test. As an example, table 4.1 shows the results of such an experiment. As with discrete settling, the data may be analyzed by plotting the suspended solids content of the samples, expressed as percentage p of the initial value, against the quotient s = h/t (fig. 4.3). With flocculent settling, however, a separate curve is obtained for each sampling depth. The upper one, incidentally, coincides with the curve shown in fig. 2.11 for discrete settling at depths of both 0.5 and 1.25 m.

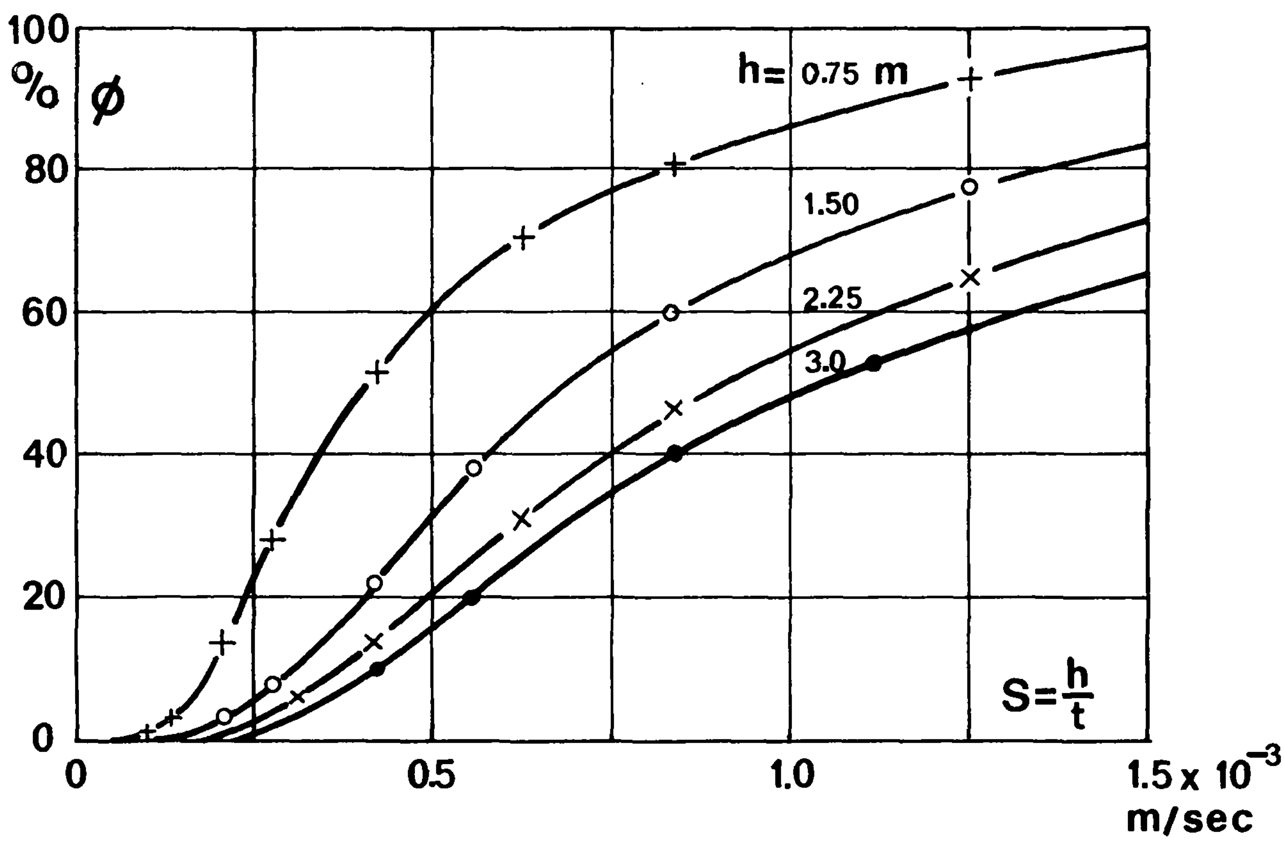


Fig. 4-3 Cumulative frequency distribution of apparent settling velocities calculated from the data of table 4.1.

Table 4.1 - Suspended solids content of samples taken at various times and depths from the quiescent settling tank of fig. 2.9, expressed in % of the starting value at 86 grams/m^3

time t elapsed since start of test (sec)	depth h below water level			
	0.75 m	1.5 m	2.25 m	3.0 m
0	100	100	100	100
600	93	96	98	99
900	81	86	88.5	89.5
1200	70.5	77.5	81	83
1800	51.5	60	65	69
2700	28	38	46.5	53
3600	13.5	22	31	40
5400	3	8	13.5	20
7200	1.5	3	6	9.5

4.2. Flocculent settling in quiescent basins

With discrete settling and unchanging velocities of subsidence, the design of a quiescent sedimentation basin is simple once the frequency distribution for the settling velocities of the suspended particles is known. As explained in section 2.4, the deciding factor is the overflow rate s_o as ratio between the tank depth H and the detention time T_o. This finding, however, is based on the circumstance that discrete particles settle at a constant rate, while the settling velocity of a flocculent particle changes continuously, increasing in some way or another with time and depth. This already becomes clear from fig. 4.4 where the settling velocities s = h/t from fig. 4.3. are plotted against the depth h below the water surface. Moreover it should be realised, that with flocculent settling the quotient h/t does not represent the real settling velocity s_r, but only the apparent one s_a. A particle starting at the top of the tank with a real settling velocity

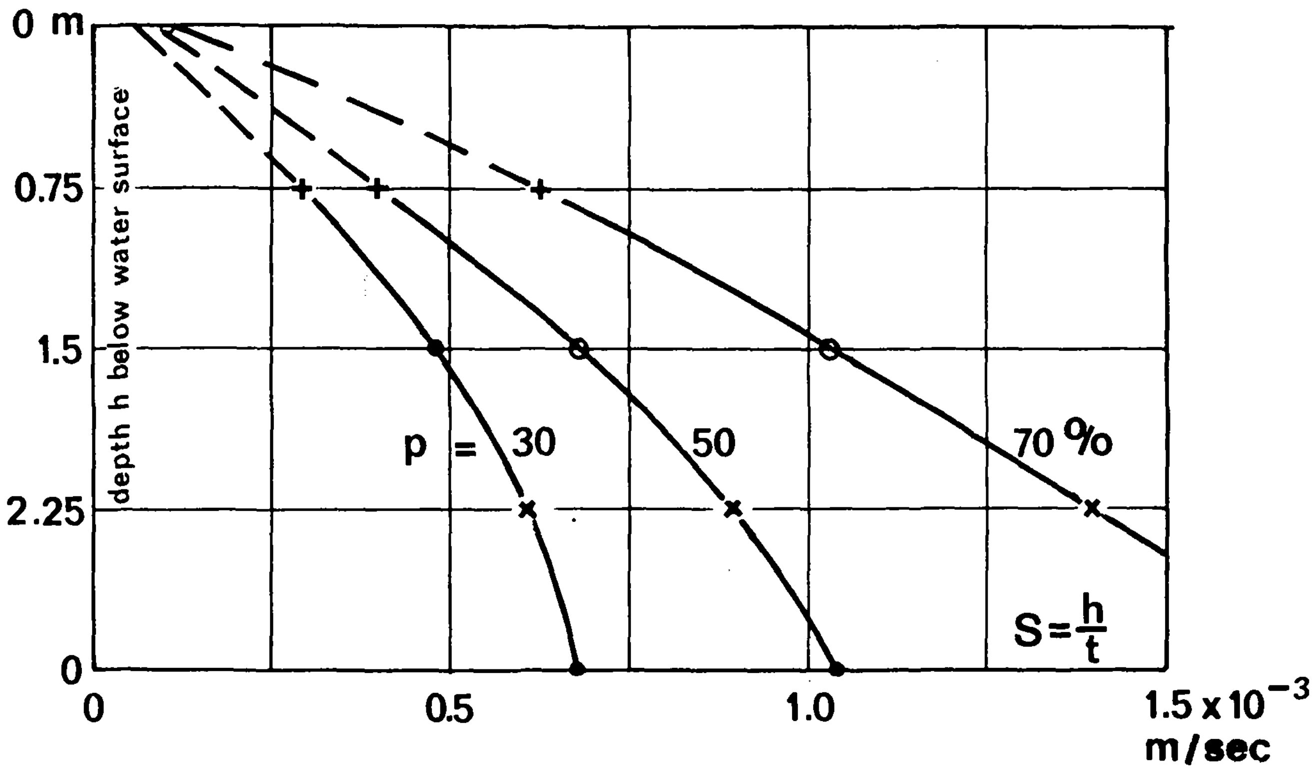


Fig. 4-4 Increase of apparent settling velocity with depth according to the data of fig. 4-3.

$$s_r = f(h)$$

traverses a depth h in a time

$$t = \int_0^h \frac{dh}{s_r}$$

giving as apparent settling velocity

$$s_a = \frac{h}{t} = \frac{h}{\int_0^h \frac{dh}{s_r}}$$

from which the unknown real settling velocity could be calculated at

$$s_r = \frac{s_a}{1 - \frac{h}{s_a}\frac{ds_a}{dh}}$$

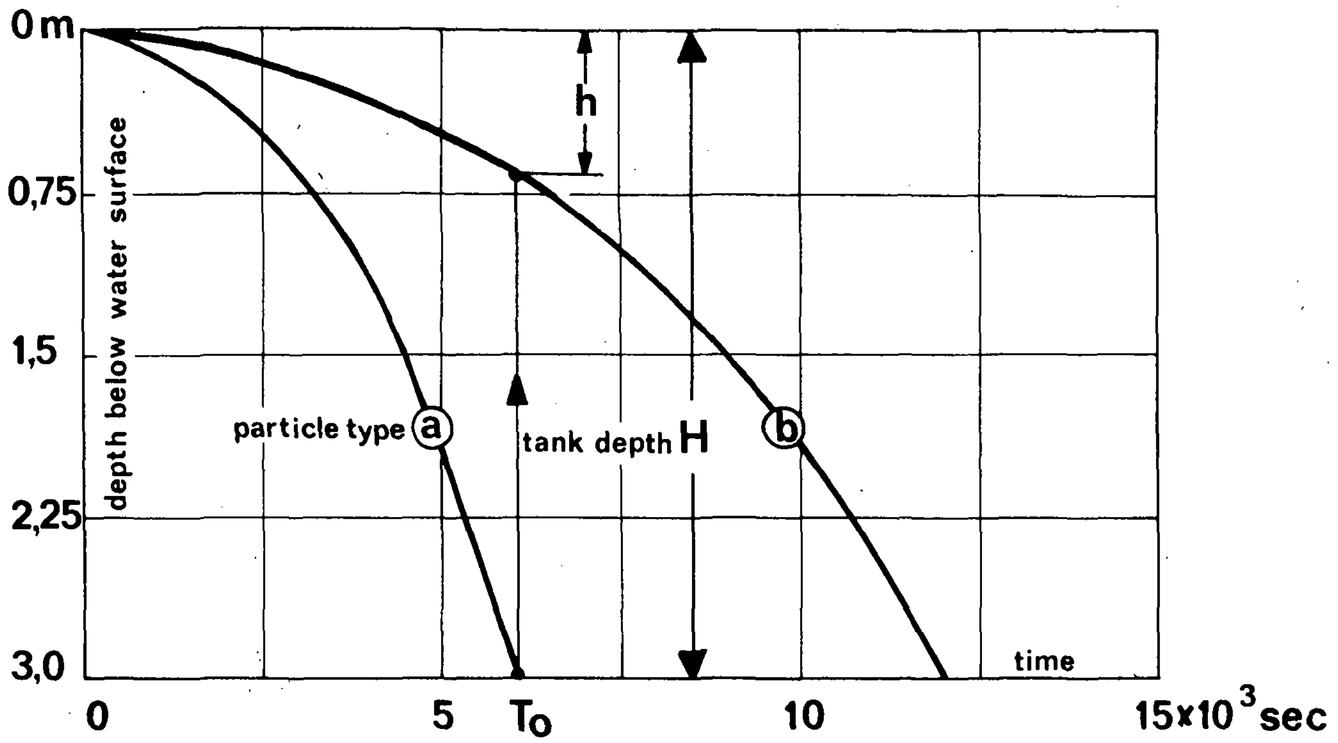


Fig. 4-5 Removal of flocculent particles.

Once these real settling velocities are known, the time-depth diagram of fig. 4.5 may be constructed for the various types of particles present in the suspension. The settling efficiency may next be calculated by considering that in the detention time T_o all particles settling faster than particles type (a) are completely removed, while for particles type (b) with a smaller settling velocity the removal ratio equals h/H (fig. 4.5).

The procedure indicated above for the calculation of the removal ratio for flocculent suspensions, is scientifically correct, but too complicated to be used in actual practice. There a more simple method is commonly applied, plotting the data of table 4.1 directly against the depth of the sampling point (fig. 4.6), thus showing the remaining suspended solids content over the full depth of the container at various times after the test started. Although the dotted lines between the water surface and the first sampling point are rather speculative, it is still possible to determine the suspended solids content as average over a random depth H with fair accuracy.
The removal as difference between the original and remaining suspended solids content is plotted in fig. 4.7 for various tank depths H, as function on the detention time T_o. Going out from

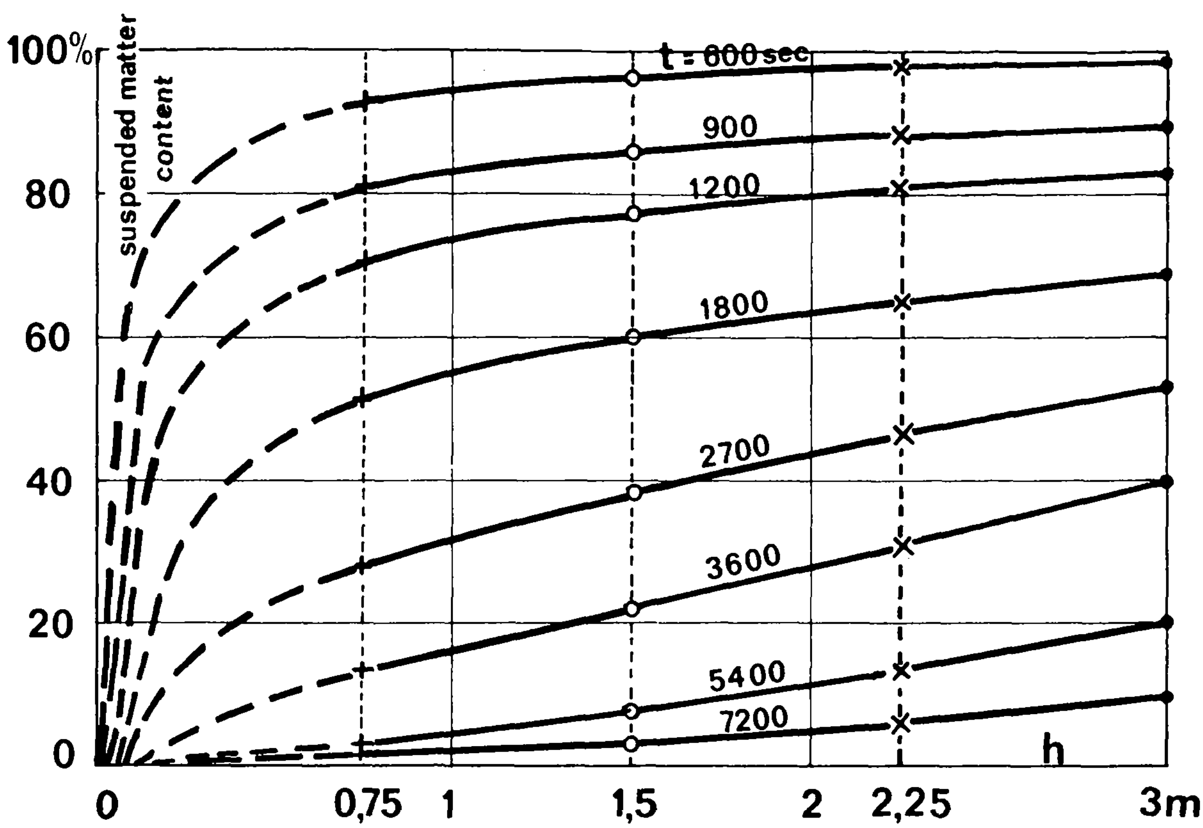


Fig. 4-6 Concentration remaining in suspension at various times and depths.

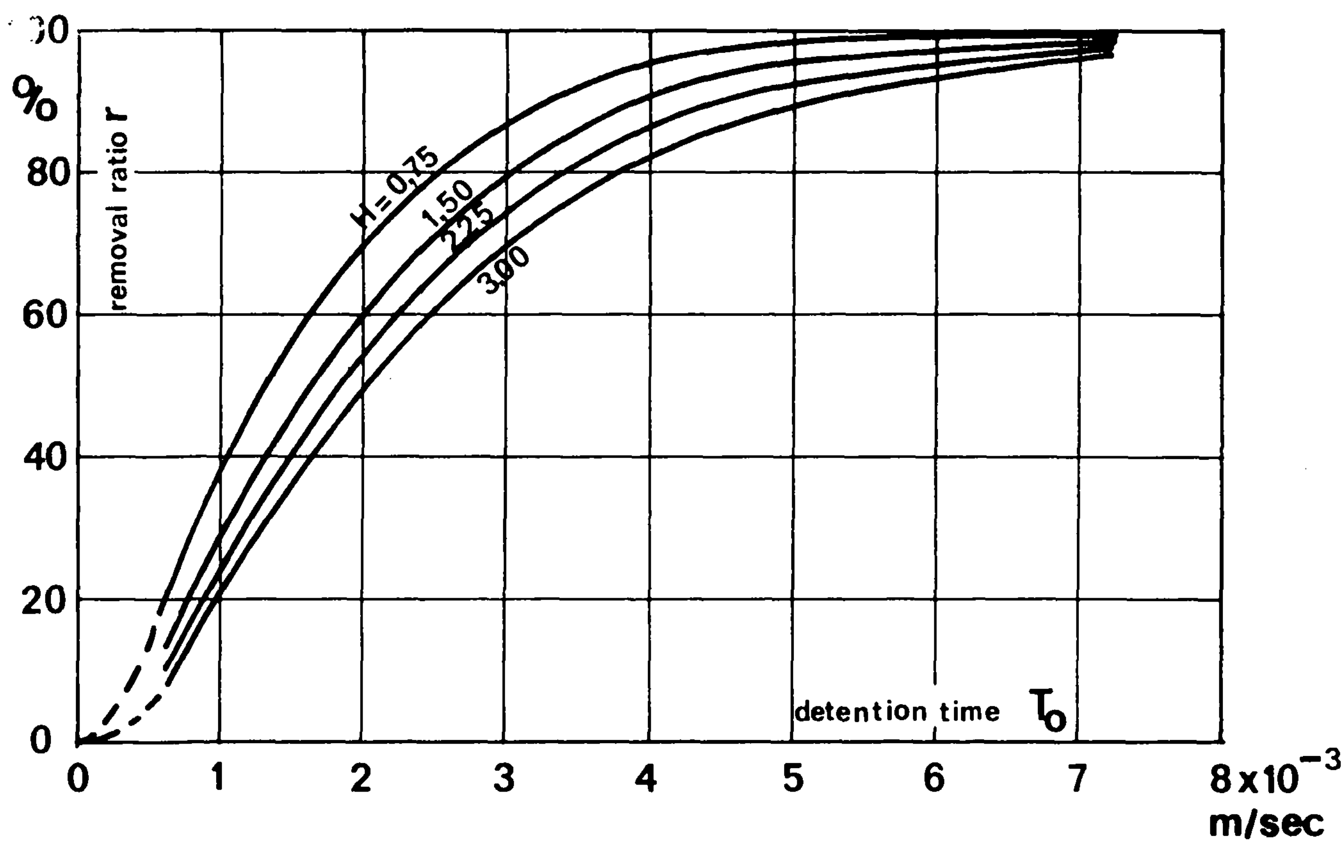


Fig. 4-7 Removal ratio as function of detention time for various tank depths.

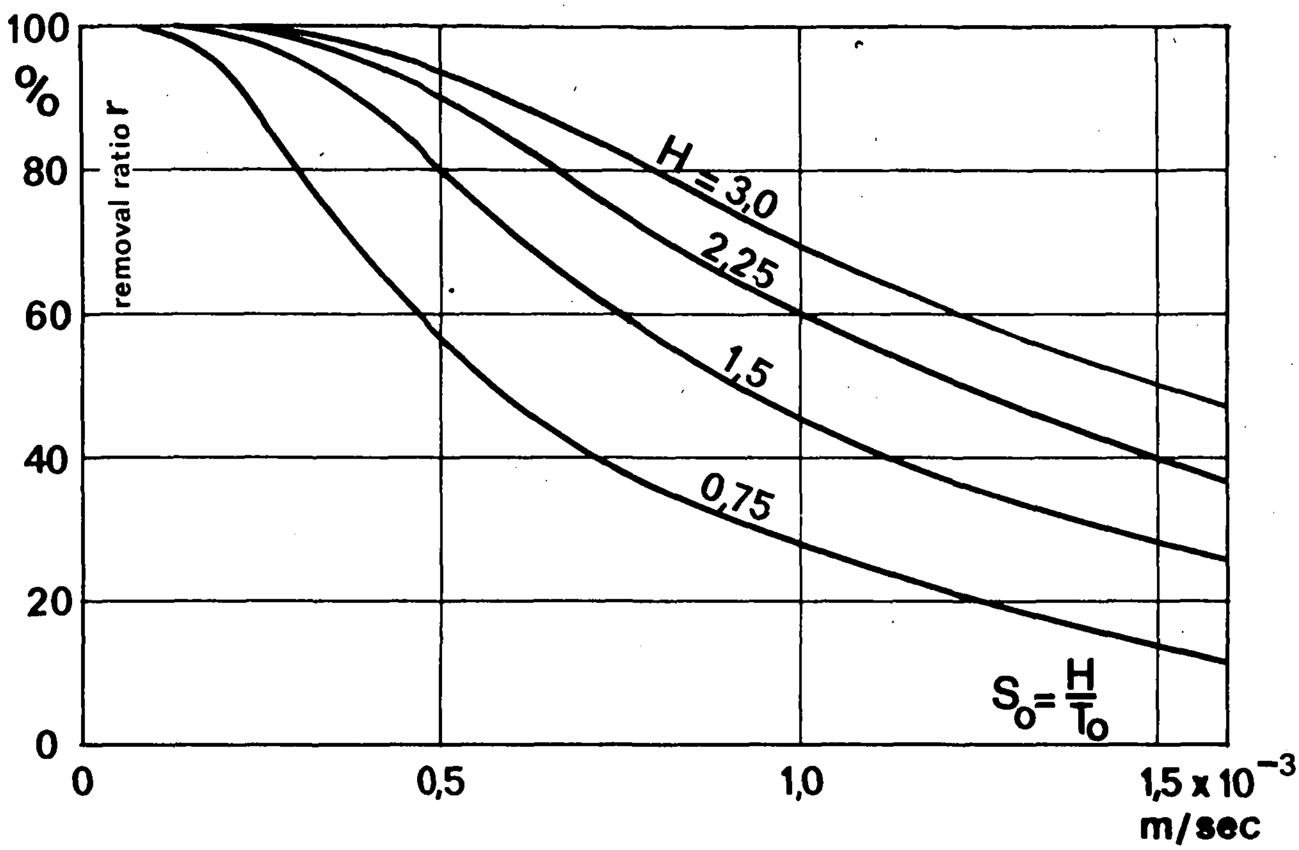


Fig. 4-8 Removal ratio as function of overflow rate for various tank depths.

this diagram, the removal ratio as function of the overflow rate $s_o = H/T_o$ may next be calculated (fig. 4.8), again giving for each depth a different result.

From figs. 4.7 and 4.8 it will be clear that with flocculent particles the settling efficiency depends on two factors, on the overflow rate s_o and on the detention time T_o or the tank depth $H = s_oT_o$. The influence of both factors together is shown in fig. 4.9 from which follows that for deep tanks with long detention times and more or less completed flocculation the overflow rate is the most important factor, while for shallow tanks the detention time is of paramount importance.

When fill-and-draw tanks are used for the clarification of Q m^3/sec during a detention time T_o at an overflow rate s_o, the required tank dimensions are

volume $V = QT_o$

area $A = \frac{Q}{s_o}$

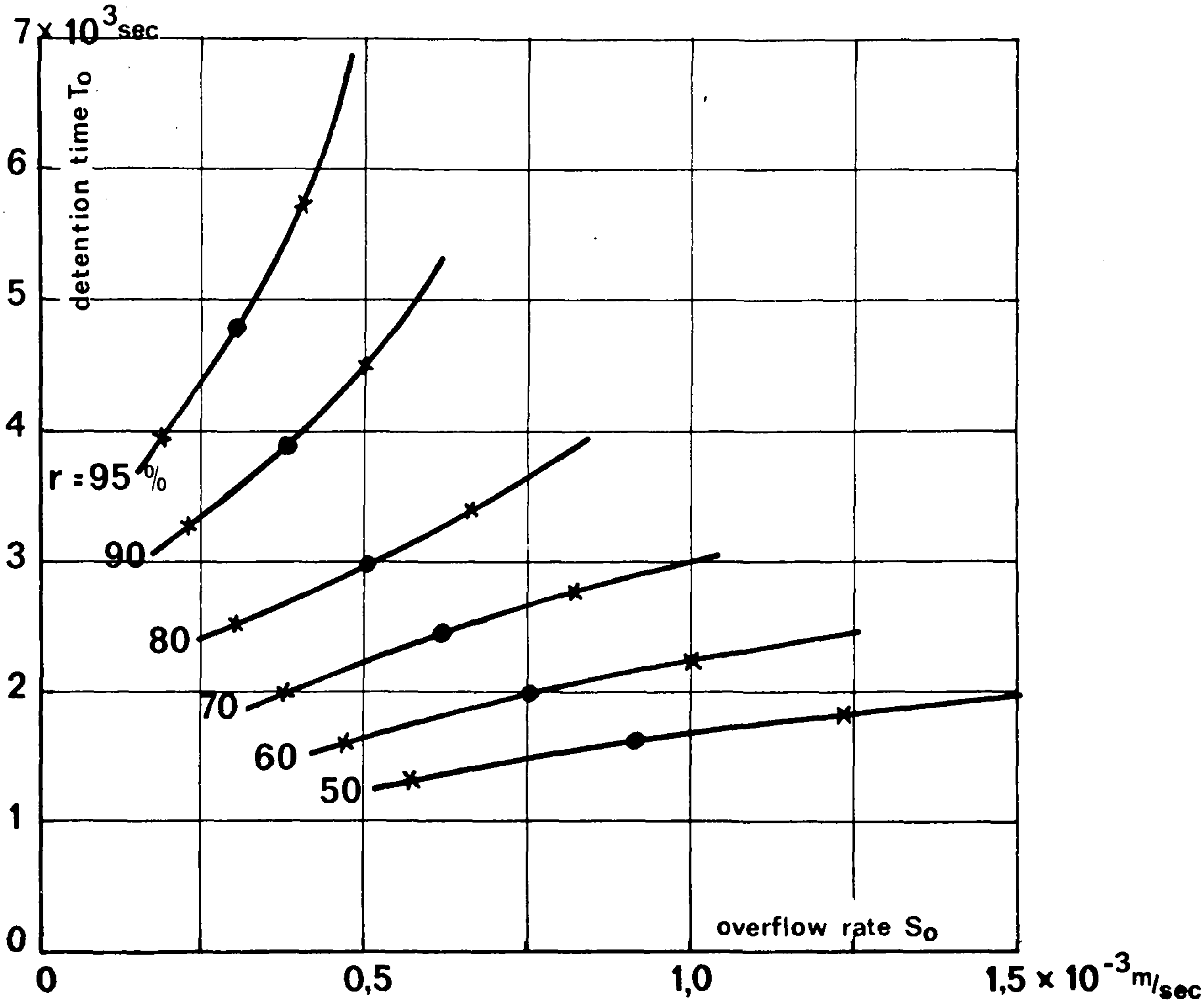


Fig. 4-9 Removal ratio as function of overflow rate and detention time.

depth $H = s_o T_o$

As an example, table 4.2 shows the possibilities for the treatment of 0.5 m^3/sec, removing 90% of the particles meant in fig. 4.9.

Table 4.2 - Tank dimensions for the treatment of 0.5 m^3/sec, removing 90% of the particles shown in fig. 4.9

s_o m/sec	T_o sec	A m^2	V m^3	H m	$A^{1.5}H$ m^4
$(0.2)10^{-3}$	3150	2500	1580	0.63	$(79.0)10^3$
$(0.3)10^{-3}$	3550	1670	1780	1.07	$(72.6)10^3$
$(0.4)10^{-3}$	4000	1250	2000	1.60	$(70.8)10^3$
$(0.5)10^{-3}$	4500	1000	2250	2.25	$(71.1)10^3$
$(0.6)10^{-3}$	5200	830	2600	3.13	$(74.9)10^3$

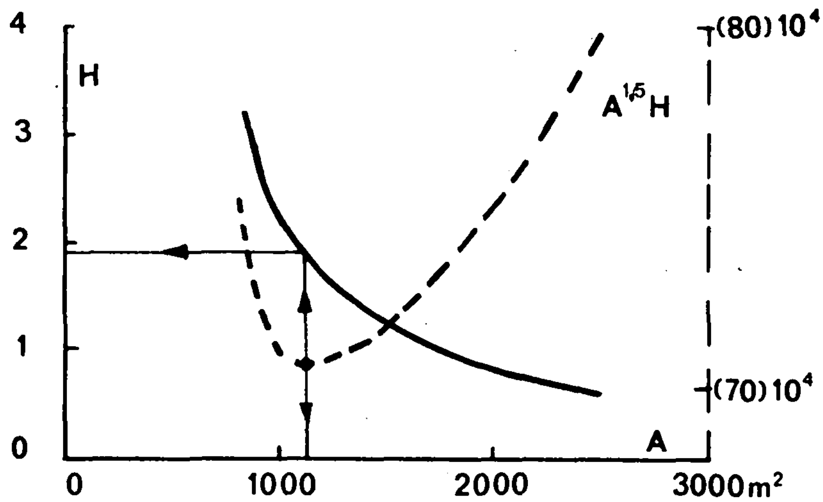


Fig. 4-10 Relation between tank area and tank depth for the treatment of 0,5 m^3/sec, removimg 90% of the particles meant in fig. 4-9.

Which solution should be chosen, depends on the cost of tank construction. This cost increases more sharply with area than with depth. In case the most economical solution is obtained when the optimization factor

$$A^{1.5}H$$

reaches its minimum value, a tank with a depth of 1.9 m and a surface area of 1100 m^2 should be chosen (fig. 4.10). Also with regard to the problems of building the tank, this is quite an acceptable proposition.

Indeed, the removal ratio as function of the detention time (fig. 4.7) varies less with tank depth than is the case with the removal ratio as function of the overflow rate (fig. 4.8). The dominant position of detention time as prime factor in the design of settling tanks for sewage treatment (fig. 4.11), however, is due to

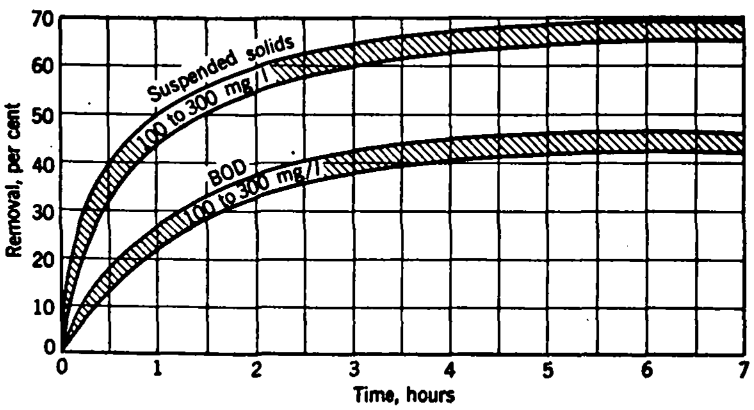


Fig. 4-11 Removal ratio for the plain sedimentation of sewage in primary tanks.

another factor, the danger of putrefaction. When here detention times are larger than 2 to 3 hours, the liquid may become septic during treatment, contaminating the plant and its surrounding with very unpleasant odors.

4.3. Zone settling in quiescent basins

When a uniform suspension contains particles with one and the same settling velocity, the suspended matter will move down as a whole (fig. 4.12), at a constant rate with discrete particles and at an increasing

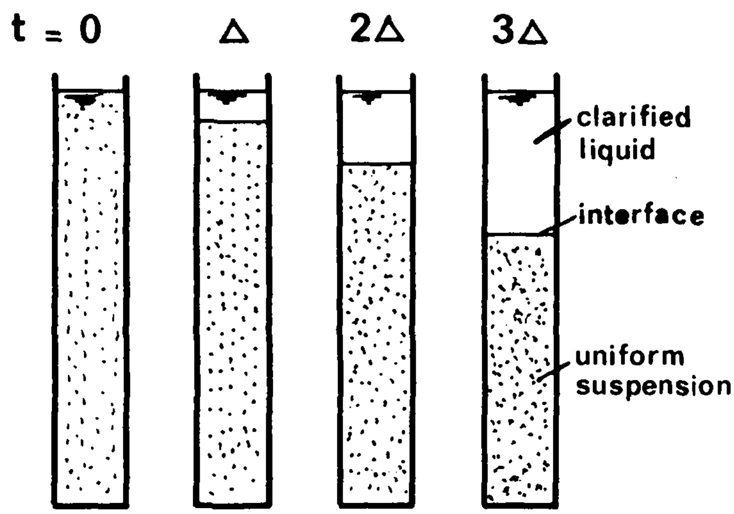


Fig. 4-12 Clarification of a uniform suspension.

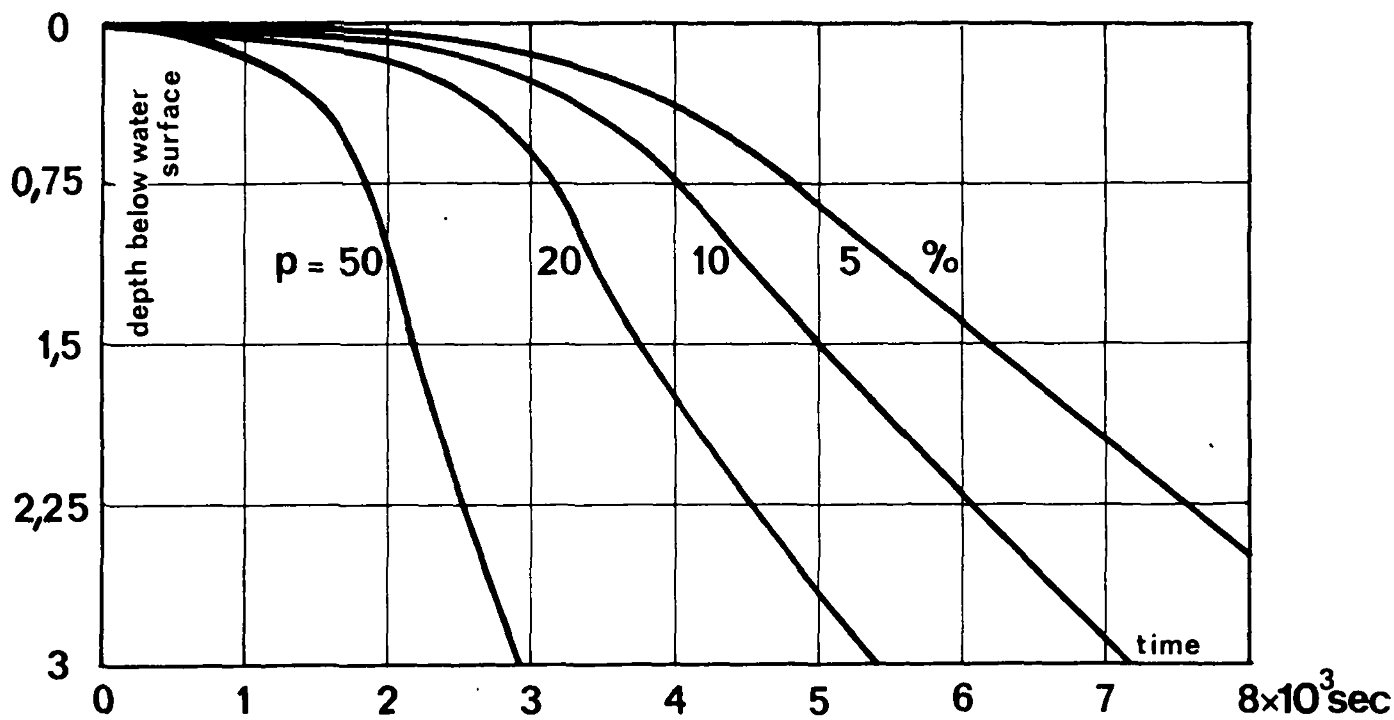


Fig. 4-13 Clarification of the suspension from fig. 4-6 as function of time and depth.

rate with flocculent ones. In both cases, however, there is a distinct interface between the suspension and the clear supernatant water. Such a sharp interface cannot be expected for the suspensions dealt with in the preceeding section, neither with the discrete particles of table 2.1 (fig. 2.15), nor with the flocculent particles of table 4.1 (fig. 4.13). Clarification here is more gradual, the supernatant water brightening as time goes on.

When the water to be treated has a high suspended matter content, say larger than 500 gram/m^3, and in particular when the solids are of the flocculent type, the interparticulate forces tend to hold the particles fixed relative to one another and again the suspension settles as one unit (fig. 4.12). With a large suspended matter content, a rapid accumulation of sludge at the bottom of the tank will furthermore occur and already after a short time the rising interface between sludge and suspension will meet the falling interface between suspension and clarified liquid (fig. 4.14). After this moment, subsidence proceeds by compaction

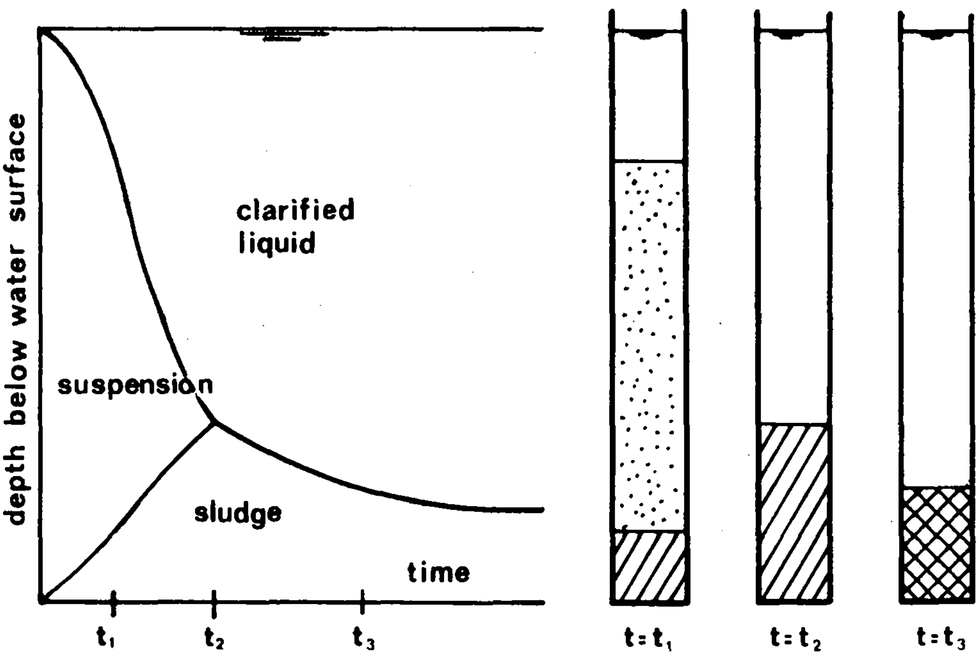


Fig. 4-14 Zone settling.

of the sludge, squeezing out the interstitial water (fig. 4.15), a rather slow process.

As an example of zone settling may be mentioned the quiescent clarification of activated sludge. With an initial suspended matter content of 2000 - 7000 grams/m^3 and deposition on the bottom with 98 to 99% intersti-

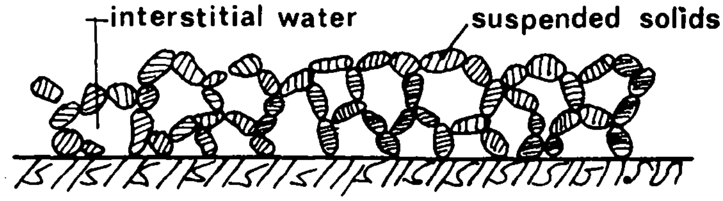


Fig. 4-15 Built-up of sludge deposits.

tial water, the maximum sludge volume at $t = t_2$ in fig. 4.14 amounts to about 300000 ppm or 30%. For $t < t_2$ the top of the suspension will fall at a rate of about $(10)10^{-3}$ m/sec and the top of the sludge layer will rise at a rate of roughly $(4)10^{-3}$m/sec. Compaction for $t > t_2$ proceeds much slower, with rates of $(2 - 0.02)10^{-3}$ m/sec, decreasing as time goes on.

4.4. Flocculent settling in continuous horizontal-flow tanks

In an ideal sedimentation tank, settling is assumed to take place without any influence of the horizontal water movement. This process parenthetically may be visualized by assuming the water to be confined in a vertical container, which moves through the tank at a speed equal to the average displacement velocity v_o (fig. 4.16). The effect

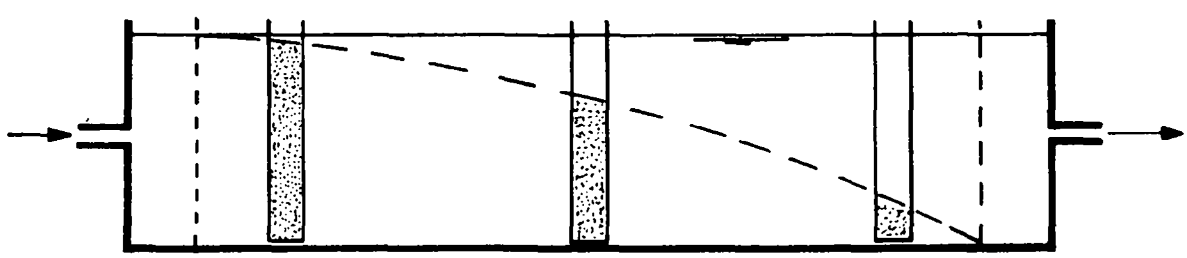


Fig. 4-16 Flocculent settling in a ideal tank.

of such an ideal settling process may be calculated in the same way as described in section 4.2 for quiescent settling, the efficiency being a function of both the overflow rate s_o and the detention time T_o, which factors together constitute the tank depth $H = s_oT_o$. For any desired value of the removal ratio r, various combinations of s_o and T_o are available, each one leading to a different tank depth H. In first instance this depth should be chosen on the basis of technical and economical considerations, so as to provide the lowest cost of construction.

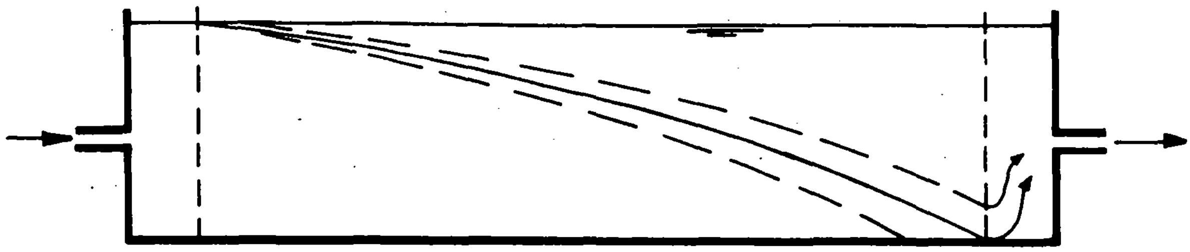


Fig. 4-17 Adverse effect of turbulence.

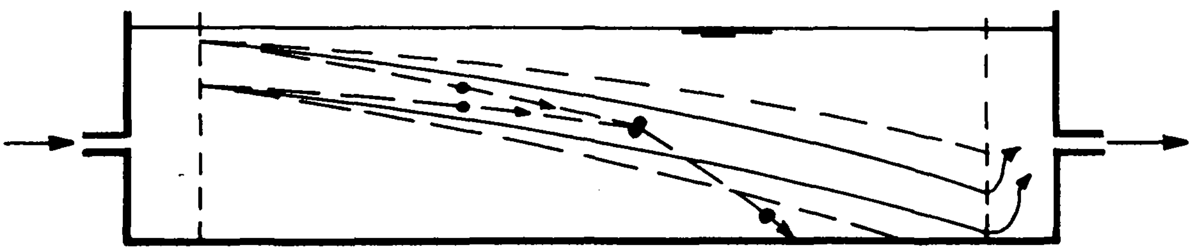


Fig. 4-18 Benificial effect of turbulence.

In practice, ideal settling tanks do not exist and there the disturbing influence of turbulence, bottom scour and short-circuiting should be taken into account. With flocculent particles, the effects of bottom scour and short-circuiting are exactly the same as with dis-discrete particles and the considerations of section 3.4 and 3.5 may therefore be applied without change. The dispersion caused by turbulence, however, has now two aspects. On one hand this dispersion will refrain part of the suspended matter to reach the bottom of the tank, reducing settling efficiency as shown in fig. 4.17, but the same dispersion will also promote the aggregation of finely divided suspended matter into larger flocs with higher settling rates, increasing settling efficiency as shown in fig. 4.18. The net effect of turbulence on a flocculent settling process is difficult to predict, but in general adverse results are absent or so small as to be negligeable.

When indeed the influence of turbulence may be neglected, the design of a settling tank is quite simple. For a high Froude number and stable flow conditions, this tank should be long, narrow and shallow, with an as high displacement velocity v_o as is consistent with the requirement of no bottom scour. As derived in section 3.4, the scour velocity

$$v_s = \sqrt{\frac{8\beta}{\lambda} \frac{\rho_s - \rho_w}{\rho_w} gd}$$

is proportional to the root of the particle diameter, meaning that when the displacement velocity v_o goes up, first the finest particles are resuspended. These settle well within the laminar region, at a rate (section 2.1)

$$s = \frac{1}{18}\,\frac{g}{\nu}\,\frac{\rho_s - \rho_w}{\rho_w}\,d^2$$

With $\lambda = 0.03$, $\beta = 0.06$ and $t = 10\ ^0C$, $\nu = (1.31)10^{-6}\ m^2/sec$, elimination of the diameter d gives

$$v_s = (0.49)\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{1/4} s^{1/4}$$

and for flocculated mud particles with a mass density of 1030 kg/m^3 for instance

$$v_s = (0.215)s^{1/4}$$

with s as settling velocity of the finest particle to be retained. For the suspension of fig. 4.3 and a tank depth of about 1.5 m, the 5% settling velocity may be estimated at $(0.25)10^{-3}$ m/sec, giving finally as requirement

$$v_o < (27)10^{-3}\ m/sec$$

The design procedure based on this limitation, can best be explained with an example, assuming that 0.5 m^3/sec of the suspension mentioned above should be treated for 95% clarification. The combinations of overflow rate s_o and detention time T_o satisfying these requirements can be taken from fig. 4.9, curve upper left. These combinations are shown in table 4.3, where next the tank depth $H = s_o T_o$ and the surface area $A = Q/s_o$ are calculated. The width of the tank finally should be chosen so large that the displacement velocity v_o equals the critical value of $(27)10^{-3}$ m/sec, mentioned above.

TABLE 4.3

s_o 10^{-3} m/sec	T_o sec	$H=s_oT_o$ m	$A=0.5/s_o$ m^2	$B=0.5/0.027H$ m	$L=A/B$ m	Fr	Re
0.15	3700	0.56	3330	33.1	101	$(14)10^{-5}$	11000
0.25	4400	1.10	2000	16.9	118	$(7.8)10^{-5}$	20000
0.35	5200	1.82	1430	10.2	140	$(5.6)10^{-5}$	28000
0.45	6400	2.88	1110	6.4	173	$(5.0)10^{-5}$	31000

Dimensions of a rectangular horizontal-flow tank for the treatment of 0.5 m^3/sec, removing 95% of the particles shown in fig. 4-9.

As regards hydrodynamic considerations, there is little to choose between the various combinations of table 4.3. All have stable flow conditions ($Fr > 10^{-5}$), while the Reynolds numbers of 11000-31000 are not excessive. The final choice must therefore be made on economical considerations, in such a way that the building costs are at a minimum. Off-hand a depth of 1.5 m (increased to 2 m to accomodate sludge raking equipment) looks attractive, requiring (fig. 4.19) a width of 12.5 m (subdivided in 2 raking compartments) and a length of 130 m (increased to 135 m to accomodate inlet and outletzones).

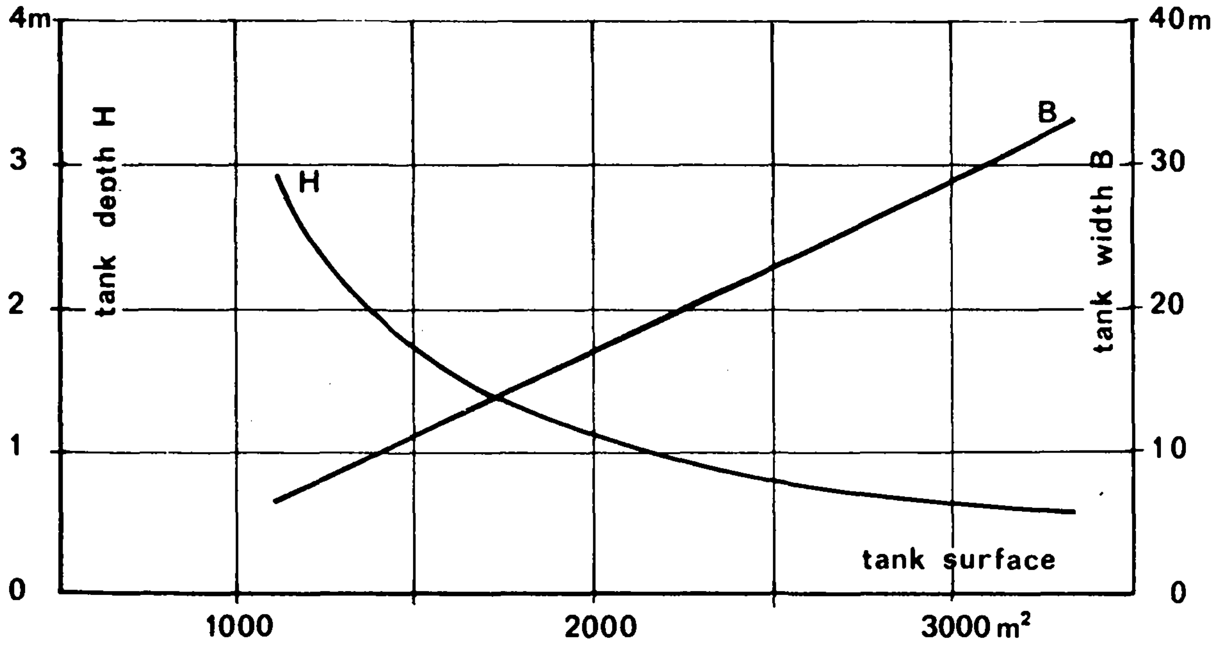


Fig. 4-19 Tank dimensions for the treatment of 0,5 m^3/sec, removing 95% of the particles shown in fig. 4-9.

With circular horizontal flow tanks, the displacement velocity reaches its maximum value near the inlet or outlet well in the centre. With the notations of fig. 3.14 this velocity equals

$$v_o = \frac{Q}{\pi D_w H}$$

To keep this velocity below the scour velocity, the depth of the tank and/or the diameter of the centre well should be rather large. With the same example as discussed above (table 4.3), an average depth of 2.5 m and an outer diameter of 40 m looks attractive. With a bottom slope of 8%, the depth in the centre will be 3.5 m, from which follows

$$v_o = \frac{0.5}{\pi D_w (3.5)} < (27)10^{-3} \quad \text{or } D_w > 1.7 \text{ m},$$

quite an acceptable value

4.5. Flocculent settling in continuous vertical flow tanks

With vertical flow tanks, the displacement velocity v_o and the overflow rate s_o are identical, both being equal to Q/A (fig. 4.20 left). The consequence of this is that discrete

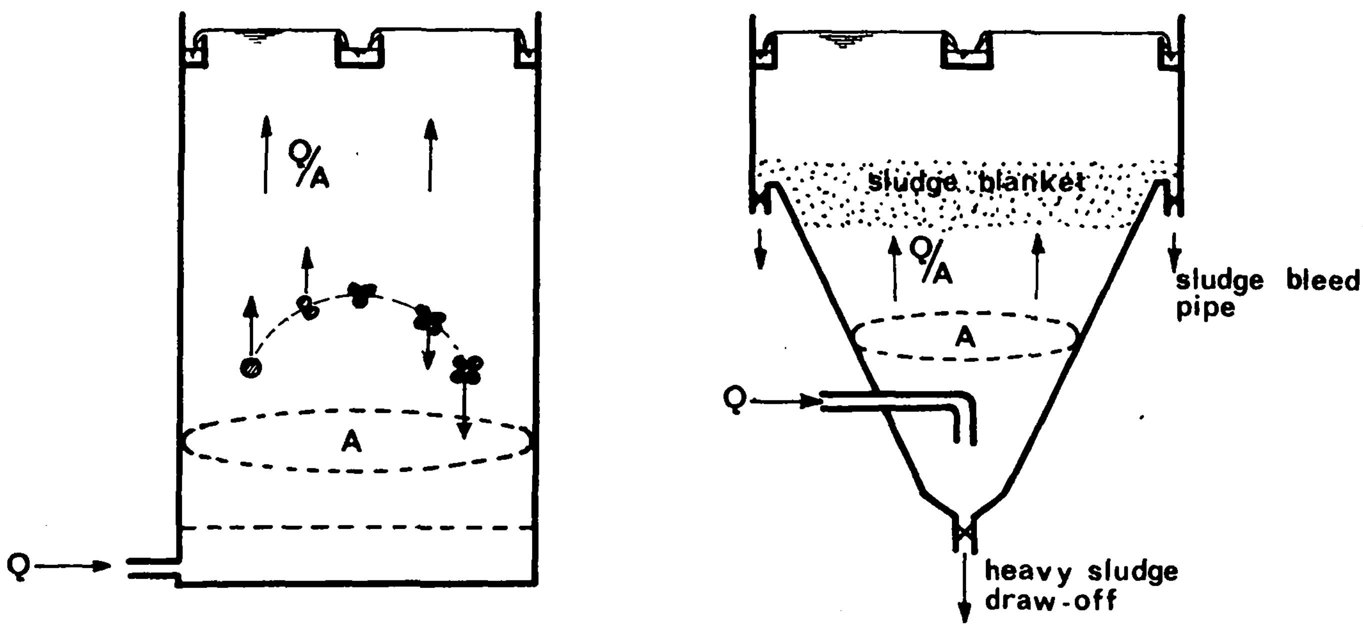


Fig. 4-20 Vertical flow tanks, cylindrical and flaring, for flocculent settling.

particles can only be retained when their settling velocity s is larger than the overflow rate s_o. As demonstrated in chapter 3, continuous horizontal flow tanks of the same dimension also remove discrete particles with a settling velocity $s < s_o$ (in a ratio s/s_o), appreciably increasing settling efficiency. This is the reason that for discrete particles vertical flow tanks should never be applied.

The behavior of flocculent particles in a continuous vertical flow tank is quite different. After entering the tank they will again move upward if their initial settling velocity s is smaller than the overflow rate s_o. During this upward travel, however, they will contact and coalesce with other particles, increasing their size and settling velocity. The effective rate $s_o - s$ of upward movement thus decreases, while further aggregation will even make this rate negative, after which the particle sinks to the bottom of the tank (fig. 4.20 left). In horizontal flow tanks, the particle cannot remain in the tank for a period longer than the detention time $T_o = AH/Q = H/s_o$, but with vertical flow tanks of the same dimensions and capacity, the detention time of the flocculent particle $T_f = H/(s - s_o)$ is much greater. The number of contacts with other particles will be correspondingly larger, promoting floc formation and increasing settling velocities and sedimentation efficiency. In many cases vertical flow tanks can thus operate at a higher to much higher surface loading than horizontal flow tanks with the same removal ratio.

The flocculation process described above may be greatly enhanced by the application of flaring vertical flow tanks, as shown in fig. 4.20 on the right. When moving upward in such a tank, the cross-sectional area becomes larger and larger, continuously decreasing the displacement velocity of the water. Even when the flocculation process has been completed and the settling velocity of the floc has reached its ultimate value s_f, the net rate $s_o - s_f$ at which this floc travels upward becomes smaller and smaller. At a certain level in the tank, this rate is so small that the flocs are virtually at rest, forming a stationary sludge blanket. In this blanket the suspended matter concentration is very high. On one hand this enables the interparticulate forces to keep the flocs together (section 4.4), making the blanket to behave as one unit, while on the other hand fine particles entrained by the upward flowing water cannot fail to come into contact and to coalesce with particles already present, removing these fine particles in the same way as in the bed of a rapid or slow filter (blanket filtration).

To understand the hydrodynamics of a sludge blanket, it should be remembered that flocculation is able to built up the floc to a certain size only (section 4.1), limiting the settling velocity to a specified value s_f, depending on floc properties. Due to the interparticulate forces mentioned above, all flocs in the blanket will have about the same settling velocity s_f, meaning that the blanket wants to move down at this rate. To keep such a blanket stationary requires an upward flow of water, at a rate s_d which will be larger as the volumetric concentration c_v of flocs in the blanket is smaller. According to the theory of hindered settling (section 2.2), the 3 factors mentioned above are interrelated by the formula

$$\frac{s_f}{s_d} = \frac{1}{1 - 2c_v^{2/3}} \quad \text{(fig. 4.21, curve A)}$$

This formula, however, is not meant for larger concentrations and there a better relation can be taken from the theory of filterbed expansion when backwashing a rapid filter

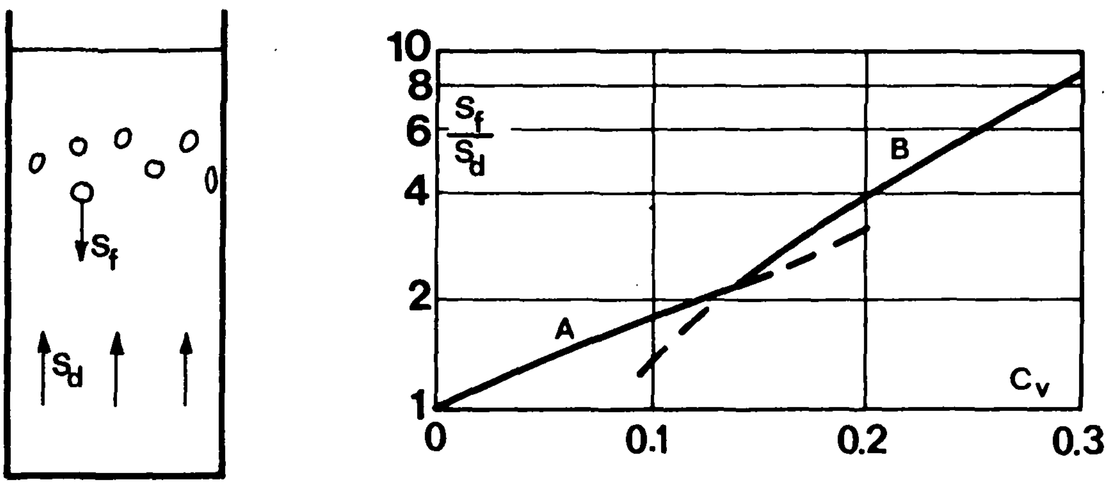


Fig. 4-21 Relation between the settling velocity S_f of the sludge blanket and the displacement velocity S_d of the water, as function of the volumetric concentration C_v of suspended matter in the blanket.

$$\frac{s_f}{s_d} = \frac{10\ c_v}{(1 - c_v)^3} \qquad \text{(fig. 4.21, curve B)}$$

With these formulae or the graphical representation in fig. 4.21, it is easy to determine the volumetric concentration c_v of flocs in the blanket, which is able to keep this blanket with a specified settling rate s_f stationary at the desired level in a vertical flow tank with a chosen surface loading s_o. This concentration can subsequently be obtained by adjusting the sludge bleed shown in fig. 4.20 on the right. Small mistakes in adjustment have little effect, the blanket moving upward or downward till the displacement velocity of the water has become so low or so high that a new equilibrium is achieved. This also means that adjustments can be made at intervals and that expert supervision is not constantly required, very important in developing countries. In developed countries and a high cost of labor, flaring vertical flow tanks have the disadvantage that they are very expensive to build. According to the formulae given above, however, sludge blanket filtration is also possible with cylindrical tanks and a constant velocity of upward water movement, equal to the overflow rate s_o, if only the volumetric concentration c_v of the flocs is closely adapted to the settling velocity s_f of the blanket. This settling velocity in the meanwhile is not constant, but will show appreciable variations during the day, due to changes in the amount and character of the suspended matter in the raw water, diurnal variations in temperature, etc, requiring careful adjustments at short intervals, either by experts which are constantly available or by complicated measuring and regulating devices. In developed countries this is not a great drawback, but it nearly prevents the use of cylindrical sludge blanket tanks in developing countries.

For the design of continuous vertical flow tanks, with or without a sludge blanket, the major factor is again the overflow rate s_o. Once this is known, the surface area may be calculated from the desired capacity as $A = Q/s_o$. With the same flocculent suspension this overflow rate may now be 2 or 3 times as high as with continuous horizontal flow tanks, with a corresponding reduction in surface area and cost of construction. In the field of water and waste water engineering, this overflow rate is commonly in the neighbourhood of

$(1)10^{-3}$ m/sec for conditions of average flow and during periods of peak flow 1.5 times as large. From one raw water to another, however, the maximum allowable overflow rate may show large variations and the value to be applied should therefore be determined each time anew in the laboratory. Quiescent settling tests are now inadequate and the experiments should be carried out with a continuous flow of water, through a cylindrical or conical vessel with a depth equal to that of the tank to be built. Such tests are rather expensive and economically only warranted with larger installations where the saving obtained by application of vertical flow tanks in stead of horizontal flow tanks may be expected to be appreciable. The required surface area finally may be accomodated in circular or square tanks, depending on local circumstances and the preference of the designer, while with a number of tanks built next to each other a rectangular plan may be more attractive (fig. 4.22).

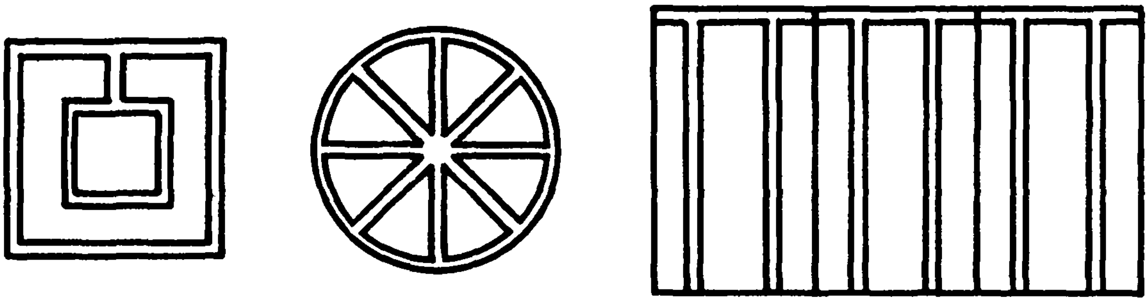


Fig. 4-22 Plans for vertical flow tanks.

The depth of the tank should be adequate to allow the inlet zone, the settling zone and the outlet zone to be built vertically above one another, while next to this it should be large enough to permit installation and proper functioning of sludge collecting mechanisms. The inlet zone is meant to divide the incoming water equally over the full cross-sectional area of the settling zone. With flaring square or circular tanks, simple constructions already suffice this purpose (fig. 4.23), but when the tank area is constant over the full depth, more elaborate provisions are necessary (fig. 4.24 and 4.25). A saving in the required depth of the inlet zone can be obtained by a detailed sub-division of the influent, using a grid of perforated pipes or channels (fig. 4.26). The additional cost of such a grid is an obvious draw-back, while the danger of clogging and subsequent putrescence forbids its use with sewage and many industrial waste waters.

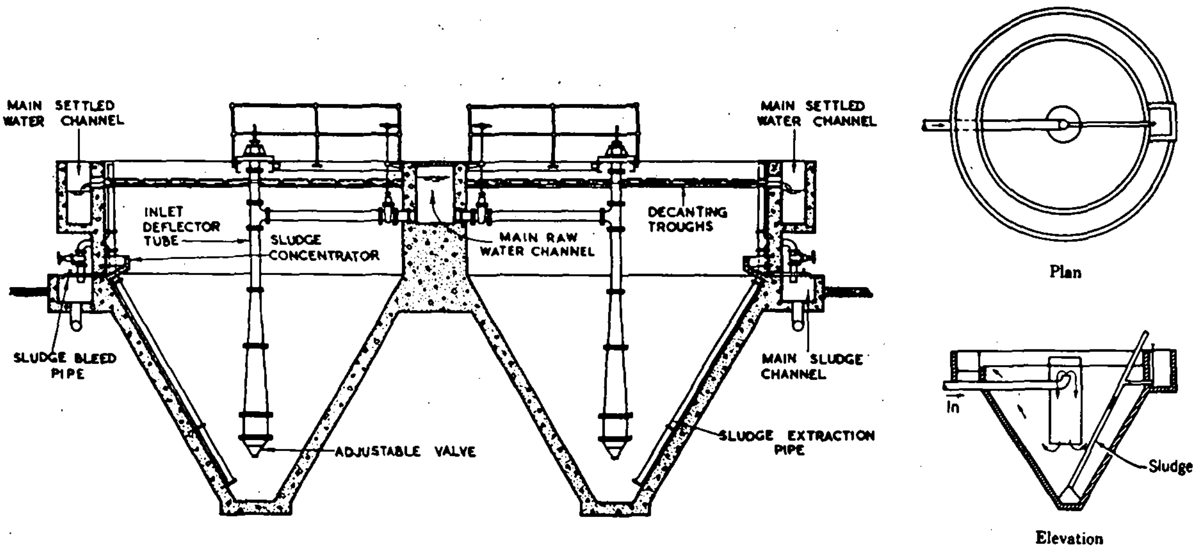


Fig. 4-23 Flaring vertical flow tanks.

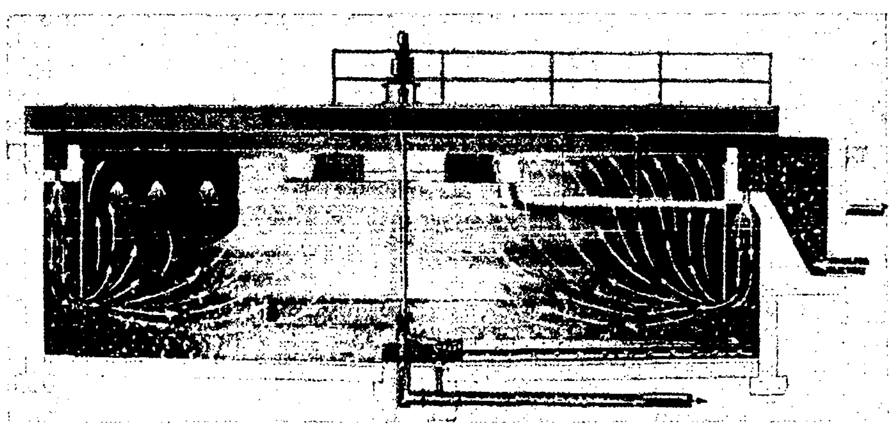


Fig. 4-24 Cylindrical vertical flow tank.

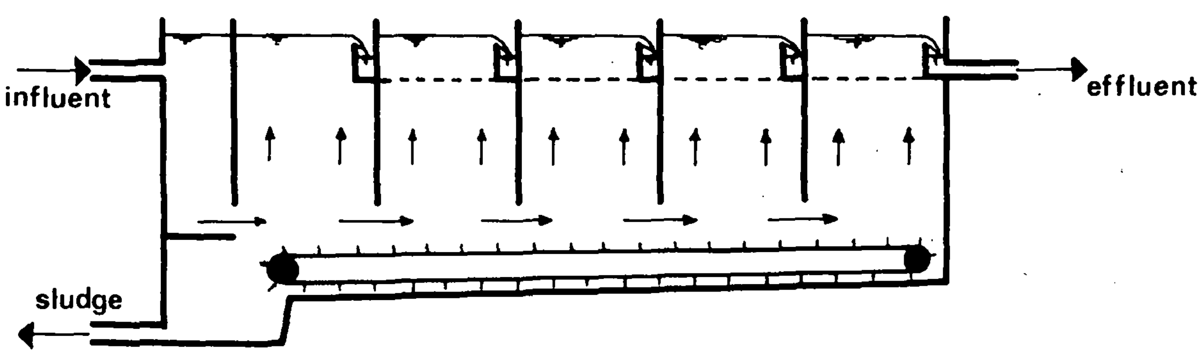


Fig. 4-25 Rectangular vertical flow tank.

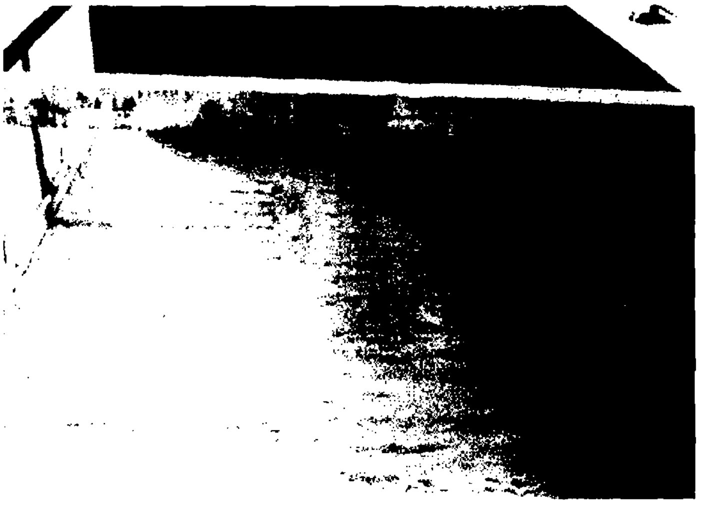


Fig. 4-26 Supply grid in vertical flow tanks.

The depth of the settling zone should be large enough to allow completion of the flocculation process. Generally speaking this means a detention time of 1500-3000 seconds and a depth of roughly 2 m. This, however, only represents the order of magnitude and the true value to be applied in a particular case, should be taken from the laboratory tests mentioned above. With flaring tanks and the walls at 60^0 with the horizontal, the transition from inlet zone to settling zone takes quite a height, in particular with tanks of circular or square cross-section (fig. 4.27), making them very expensive to build.

The outlet zone finally should be of such a depth that an even abstraction of clarified water from the settling zone is assured. As

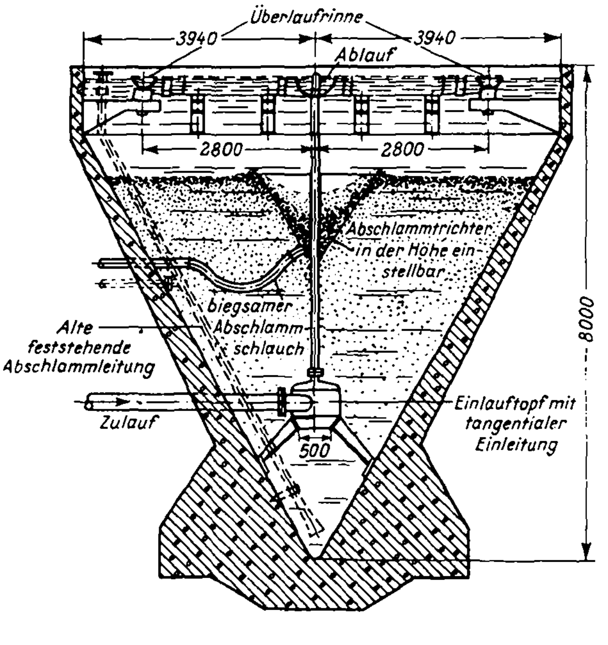


Fig. 4-27 Flaring square settling tank.

demonstrated in fig. 4.28, this will be the case when the depth h is not too small compared to the width (a) covered by a discharge trough. In practice values of h equal to (0.5-1) (a) are applied, asking either for an appreciable depth of the outlet zone or for the discharge

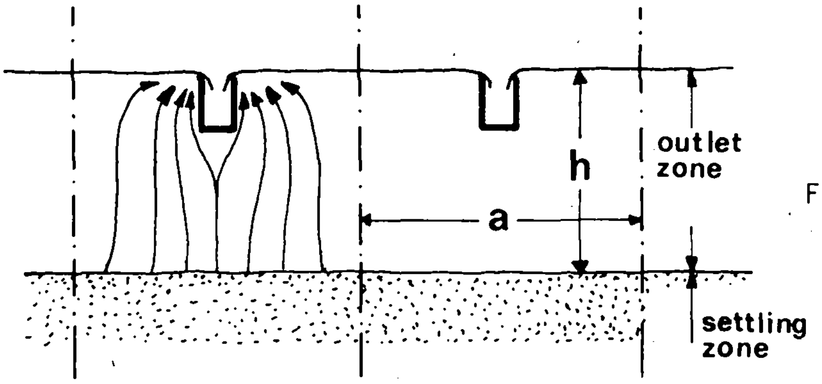
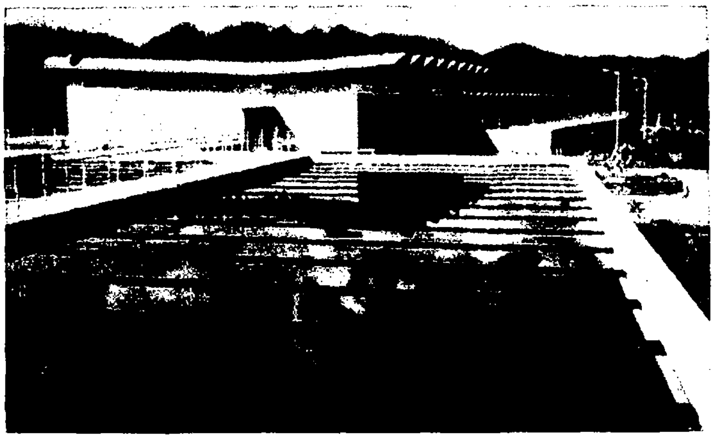


Fig. 4-28 Outlet zone.

troughs to be set close together. The latter solution is commonly cheaper (fig. 4.29 and fig. 4.30) and has the added advantage



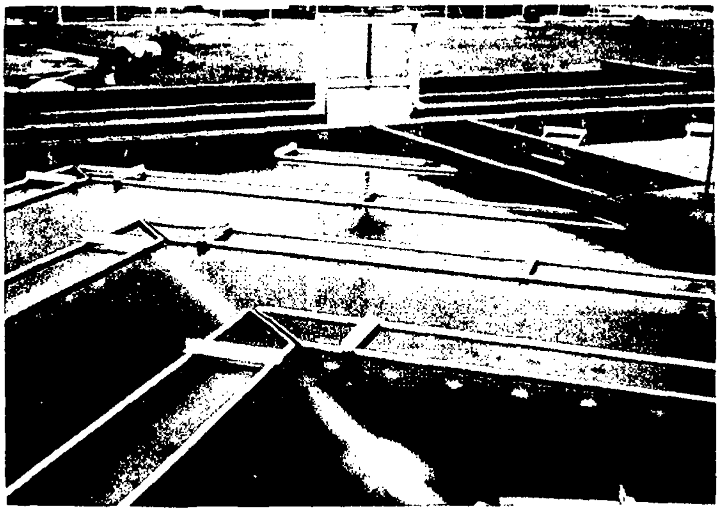


Fig. 4-29 Effluent discharge in rectangular vertical flow tanks.

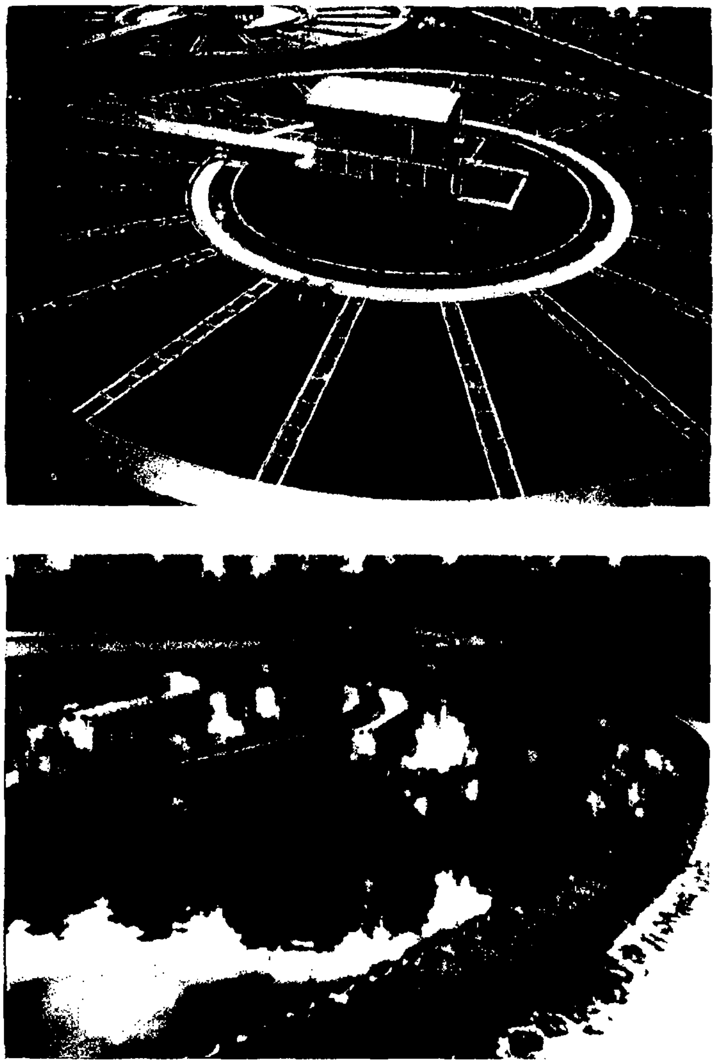


Fig. 4-30 Effluent discharge in circular vertical flow tank.

of a small weir loading and less danger of flow disturbances. With straight weirs the overflow height is now extremely small and a uniform discharge difficult to obtain. Notched weirs should therefore be used, while in the field of drinking water treatment also small diameter openings are applied, set below water level in the vertical walls of the troughs or even in the top of submerged pipes (fig. 4.31).

With flaring tanks sludge removal is simple. The suspended matter from the blanket is removed with sludge bleed pipes (fig. 4.20 right and fig. 4.23 left) and the heavy sludge at the bottom with draw-off or extraction pipes, without the need for any collecting mechanism. With tanks of a constant cross-sectional area, mechanical sludge removal equipment is again required, in rectangular tanks as chain carried scrapers (fig. 4.25) and in circular

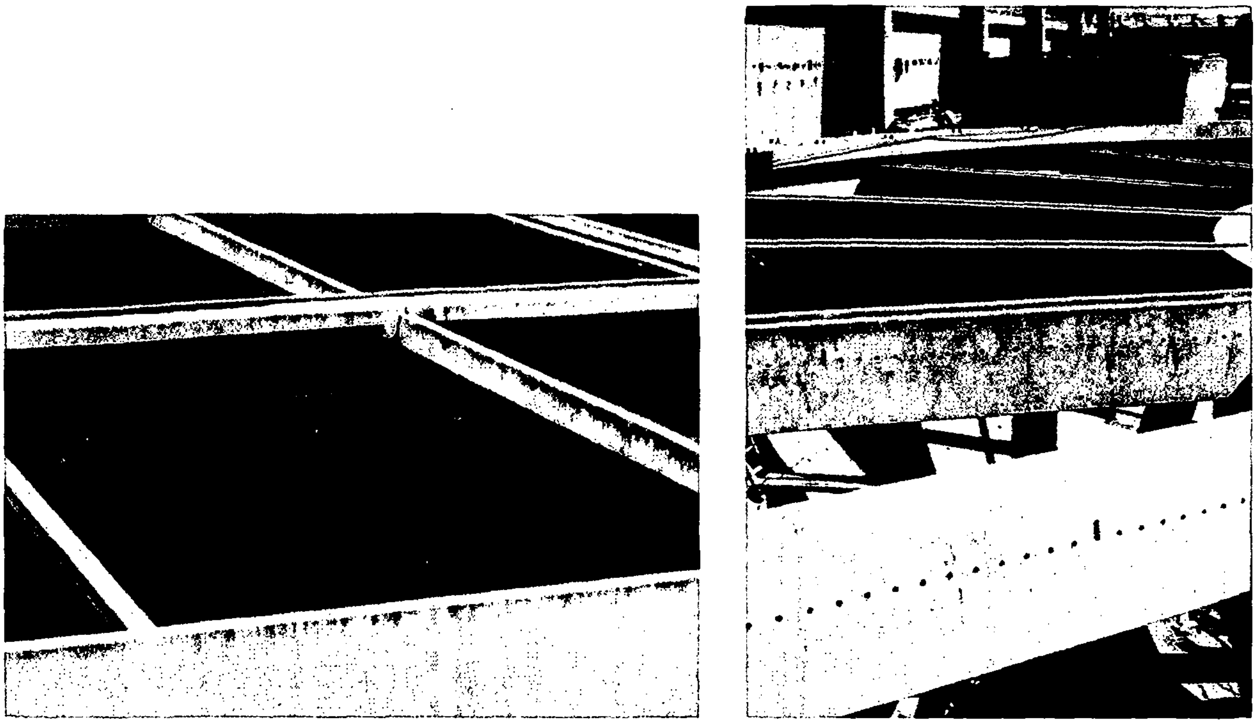


Fig. 4-31 Effluent discharge through submerged openings.

tanks as revolving rakes (fig. 4.32) or as revolving suction sludge collectors (fig. 4.24). Skimming devices are difficult to accomodate in vertical flow tanks. Small amounts are removed by adherence to the suspended flocs, but when large amounts are present horizontal flow tanks should be used or the vertical flow tank preceded by a flotation tank (chapter 6).

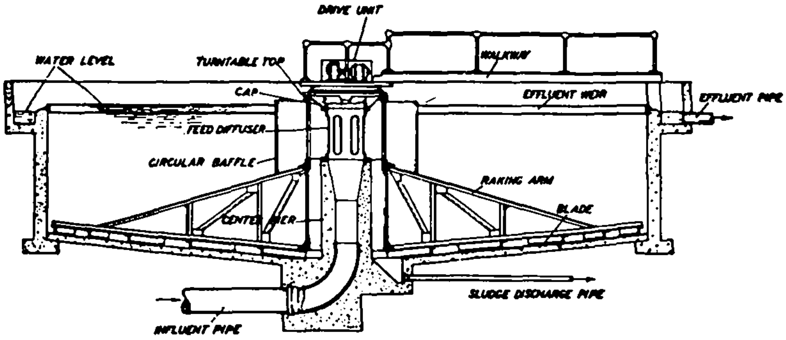


Fig. 4-32 Sludge collection by revolving rakes in circular vertical flow tank.

5. Special service equipment

5.1. Flow splitters

With regard to maintenance and repairs, a single settling tank is never sufficient and at least two tanks must be provided, while in larger plants a greater number may be necessary or economically attractive. This brings with it, however, the problem of equally dividing the total capacity over the various units. If this is not achieved, some tanks will operate at a higher overflow rate and some at a lower one, resulting in a (slight) reduction in settling efficiency as already demonstrated in section 3.5. In rare cases this equal division can be obtained by a symmetrical lay-out of the raw water supply pipes, comparable to an equal division over the width of the basin as shown in fig. 3.44.

In water supply engineering the equal division is commonly obtained without special equipment, by a judicious consideration of the hydraulics involved. As an example fig. 5.1 shows 2 tanks,

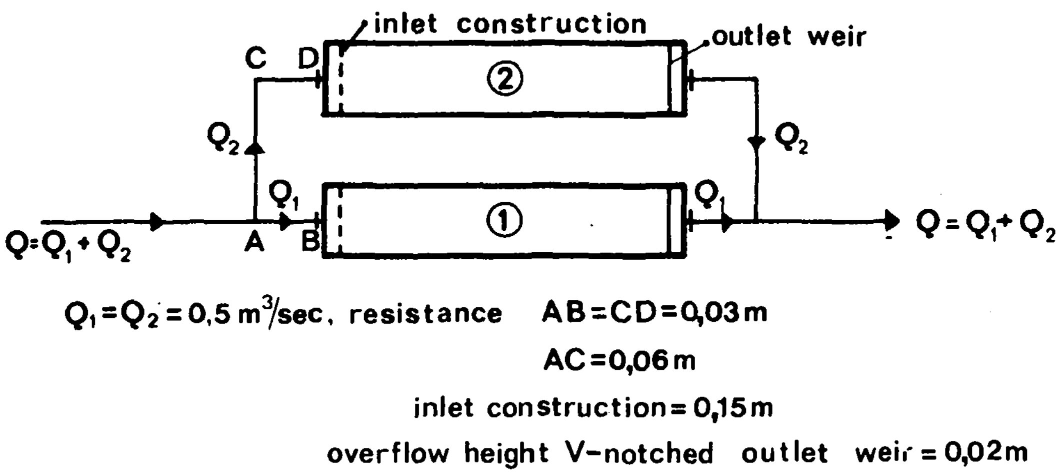


Fig. 5-1 Lay-out of settling tanks and raw water supply lines.

where the resistance of 0.06 m over the pipe section AC will result in tank (2) having a lower rate than tank (1). This can be prevented in two ways

a. by providing pipesection AB with an additional resistance of 0.06 m for the design flow of 0.5 m^3/sec, most simply by partly closing the inlet valve at B;

b. by placing the outlet weir in tank (2) 0.06 m lower than in tank (1).

The last solution is fool-proof, but with other capacities than the design flow a slight deviation in the equal division must be expected. This division can be calculated by considering that the drop in piezometric level from the bifurcation A over C and D to the outlet weir in tank (2) must equal the drop in piezometric level from A over B to the outlet weir in tank (1), augmented with the difference in weir level of 0.06 m. In formula

$$\left(\frac{Q_2}{0.5}\right)^2 (0.06 + 0.03 + 0.15) + \left(\frac{Q_2}{0.5}\right)^{2.5} (0.02) =$$

$$\left(\frac{Q_1}{0.5}\right)^2 (0.03 + 0.15) + \left(\frac{Q_1}{0.5}\right)^{2.5} (0.02) + 0.06$$

from which follows at 150% of the design capacity

$$Q_1 = 0.725\ m^3/sec = 0.483(Q_1 + Q_2)$$

$$Q_2 = 0.775\ m^3/sec = 0.517(Q_1 + Q_2)$$

indeed a negligeable deviation.

The method described above uses the (quadratic) flow resistance of the supply lines to obtain an equal distribution. This method will therefore fail when these resistances undergo uncontrolable changes, for instance by deposition of settled out material from the raw water. Such a clogging of the pipelines may easily occur in sewage and waste water treatment plants and this is the reason that there the desired equal division is commonly obtained by flow splitters (fig. 5.2). These devices consist of a circular or otherwise symmetrical weir, divided in a number of equal parts, each with its own discharge pipe. With the lay-out of fig. 5.2, the additional cost is only that of the flow splitter itself, but with the lay-out of fig. 5.3 additional piping is required, making this solution quite expensive. It should therefore only be applied when strictly necessary.

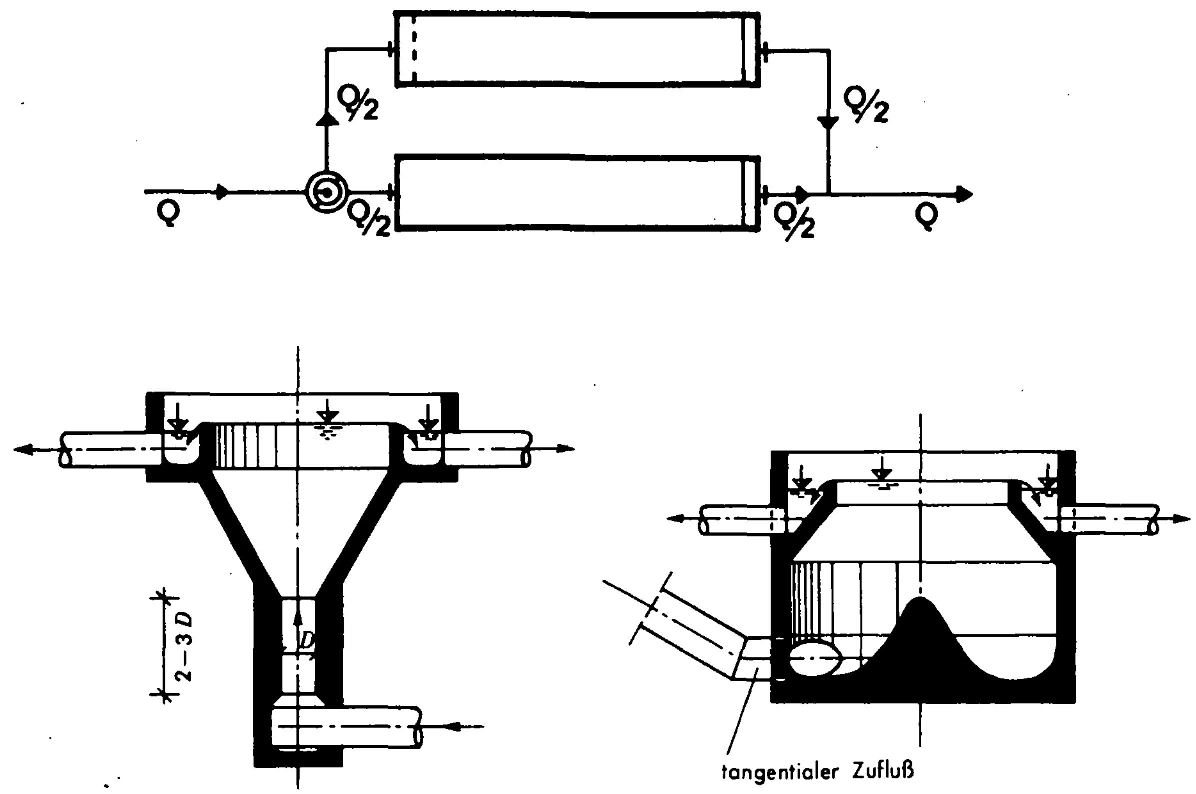


Fig. 5-2 Flow splitters and their application.

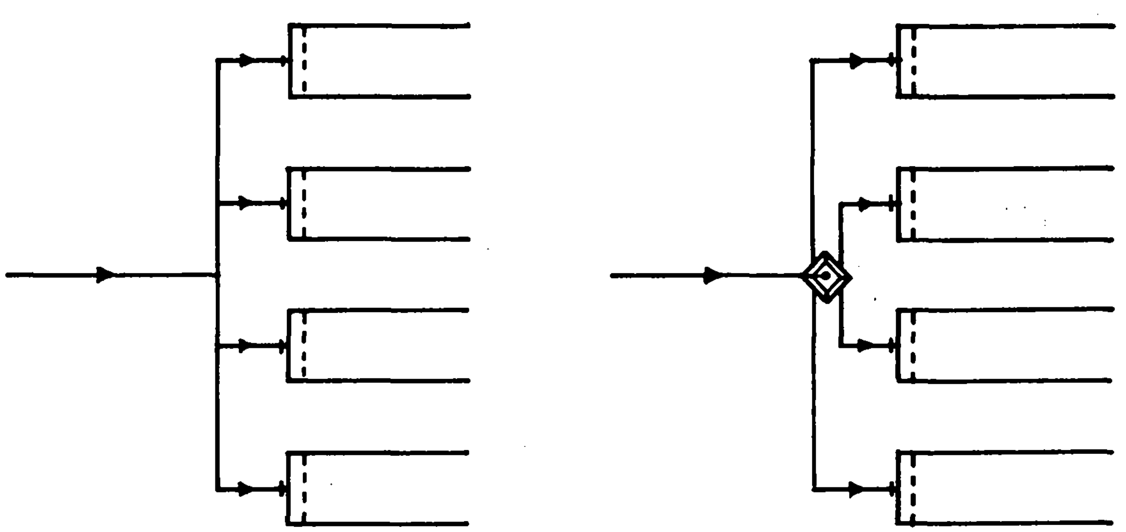


Fig. 5-3 Equal division without or with a flow splitter.

5.2. Imhoff tanks

Sewage purification in the past was often limited to mechanical treatment, using settling tanks to remove the major part of the suspended load. Next to this, however, digesters were required to take care of the sludge retained by the settling process (fig. 5.4). With

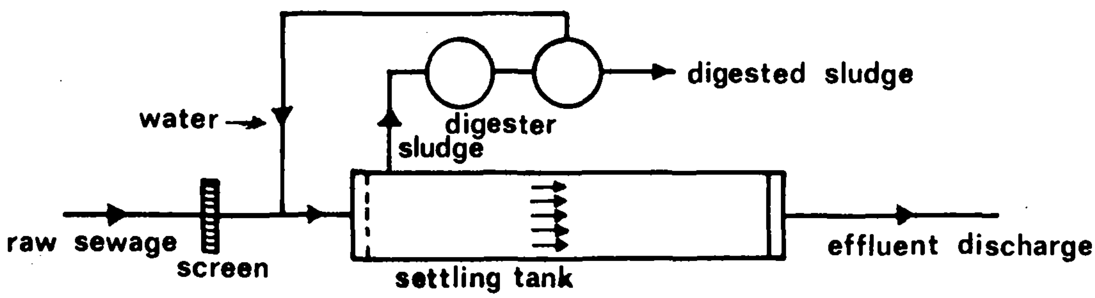


Fig. 5-4 Mechanical sewage treatment.

mechanical treatment only, these digesters constitute a major part of the plant. Some saving in the cost of construction may now be obtained by combining both processes in a two-storied tank, the upper level of which is used for clarifying the water and the lower level for digesting the sludge thus produced (fig. 5.5). These tanks were first applied by Imhoff for the Emscher district in Germany, by which names they are also known. Following his example, they have been built all over the world (fig. 5.6) also in the form of circular tanks (fig. 5.7).

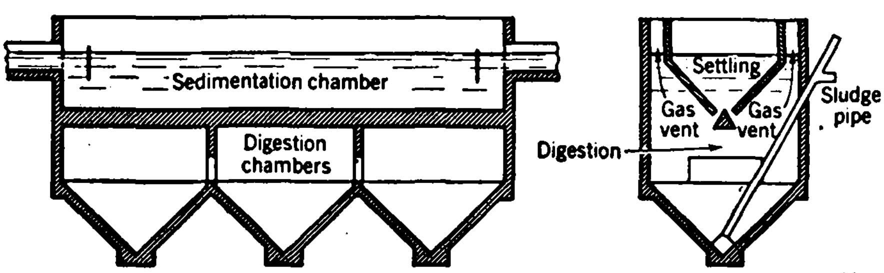


Fig. 5-5 Two-storied Imhoff or Emscher tank.

The purification provided by mechanical treatment in the meanwhile is rather limited, removing only 60% of the total suspended matter content and not more than 35% of the biochemical oxygen demand. Today, the

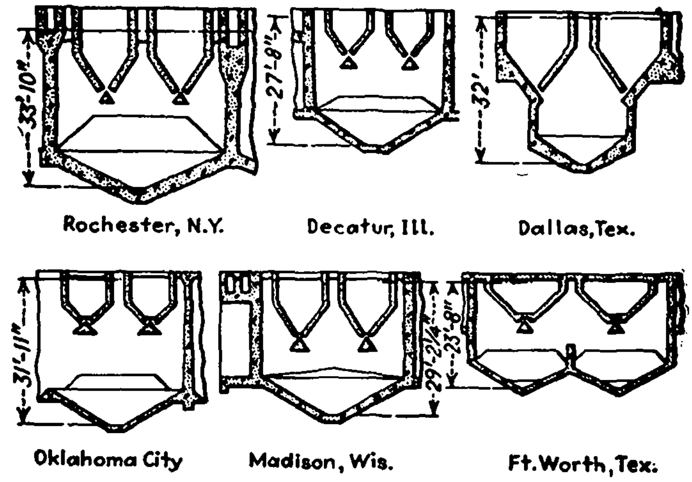


Fig. 5-6 Large Imhoff tanks.

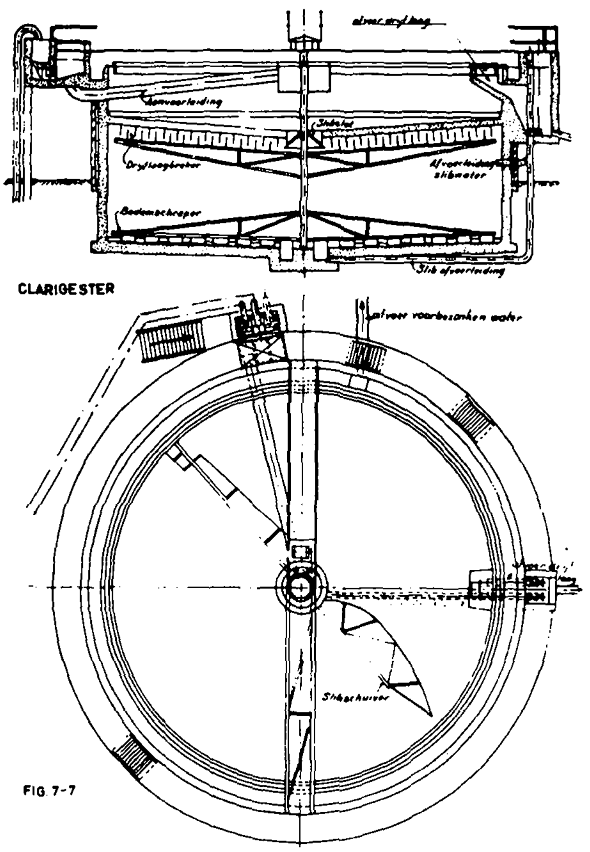


Fig. 5-7 Circular Imhoff tank or clarigester.

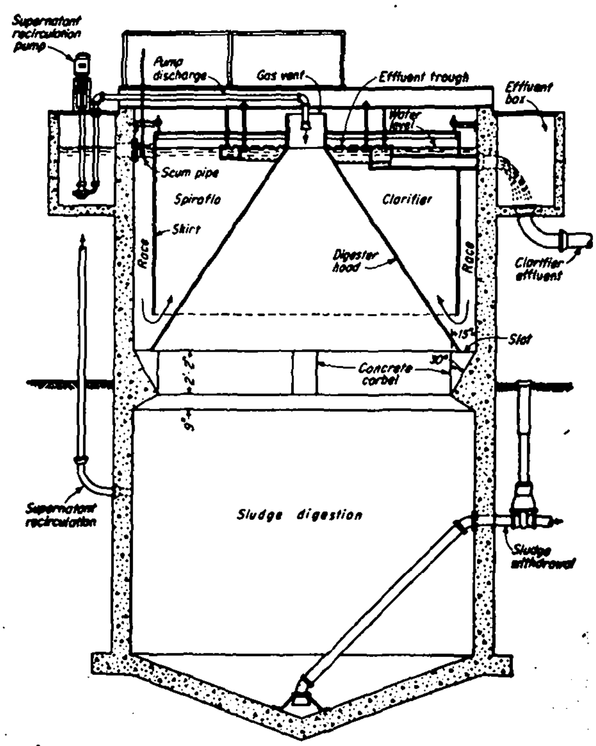


Fig. 5-8 Small Imhoff tank or spiragester.

self-purification capacity of many receiving waters is inadequate to cope with the remaining pollutional load and there the primary settling process has to be followed by some sort of aerobic biological treatment. Much more sludge is now produced, requiring extended sludge digestion facilities. Speeding up this process by heating now becomes very attractive. With Imhoff tanks, however, and the digestion chambers at a great distance below ground surface, completely surrounded by soil and water, the heat losses would become excessive and this is the reason that for larger installations such tanks are no longer applied. They still offer an excellent solution when for small quantities of sewage a partial treatment will suffice , for instance with isolated hotels, camping places, etc, replacing septic tanks formerly used for this purpose (fig. 5.8).

5.3. Channel-type grit chambers

Next to putrescible organic solids, domestic sewage contains heavy and inert materials such as ashes and clinker, egg shells, bone chips etc, while in a combined sewerage system also significant amounts of sand may be present. Removal of this so-called grit ahead of the primary settling tanks offers several advantages , reducing settling tank and digester tanks volumes, preventing clogging of pipelines and damage to moving mechanical equipment such as pumps, scrapers etc.

Grit as defined above settles discretely and sedimentation efficiency consequently only depends on the overflow rate. Grit chambers are generally designed to capture sand particles with a mass density of 2650 kg/m^3 and a diameter of 0.2 mm or more. With settling in the first part of the transition region (particle Reynolds number between 1 and 50), the overflow rate becomes (section 2.1)

$$s_o = s = \frac{1}{10} \frac{g^{0.8}}{\nu^{0.6}} \left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{0.8} d^{1.4}$$

or in the case under consideration and a temperature of 10 ^{0}C, $\nu = (1.31)10^{-6}$ m^2/sec

$$s_o = \frac{1}{10} \frac{(9.81)^{0.8}}{(1.31)^{0.6}(10)^{-3.6}} (1.65)^{0.8}(2)^{1.4}(10)^{-5.6}$$

$$s_o = \frac{1}{10} \frac{6.21}{1.176} (1.493)(2.64)(10)^{-2} = (21)10^{-3} \text{ m/sec}$$

To prevent re-suspension of settled out material, the displacement velocity v_o must remain below the scour velocity (section 3.4)

$$v_s = \sqrt{\frac{40}{3} \frac{\rho_s - \rho_w}{\rho_w} gd}$$

or in this case

$$v_s = \sqrt{\frac{40}{3} (1.65)(9.81)(2)(10^{-4})} = (210)10^{-3} \text{ m/sec}$$

This means a maximum ratio between length and depth of

$$\frac{L}{H} = \frac{v_o}{s_o} = \frac{(210)10^{-3}}{(21)10^{-3}} = 10$$

For a capacity of 0.5 m^3/sec, the required surface area equals

$$BL = \frac{Q}{s_o} = \frac{0.5}{0.021} = 24 \text{ m}^2 \quad , \text{ for instance}$$

a width of 2 m, a length of 12 m and a depth larger than 1.2 m. These are quite acceptable dimensions, but with larger capacities (combined sewerage systems!) and a still limited depth the plan tends to become square. With Q = 3 m^3/sec and H = 1.2 m as before, the maximum length equals L = 12 m, giving as minimum width

$$B = \frac{Q}{s_o L} = \frac{3}{(0.021)(12)} = 12 \text{ m}$$

Such tanks are called detritus tanks and have as disadvantage that next to the heavy and inert grit, also part of the decomposable organic load is retained. When such material is disposed of by dumping, putrescence will set in, spreading unpleasant odors, attracting flies and making the place rather unsightly. To facilitate disposal of the grit in land-fills, sludge drying beds, etc, clean grit is necessary, with a washable organic matter content less than 3%, preferably below 1%. From detritus tanks clean grit can only be obtained by washing the retained material after removal from the tank, for instance as shown in fig. 5.9. A more straight-forward

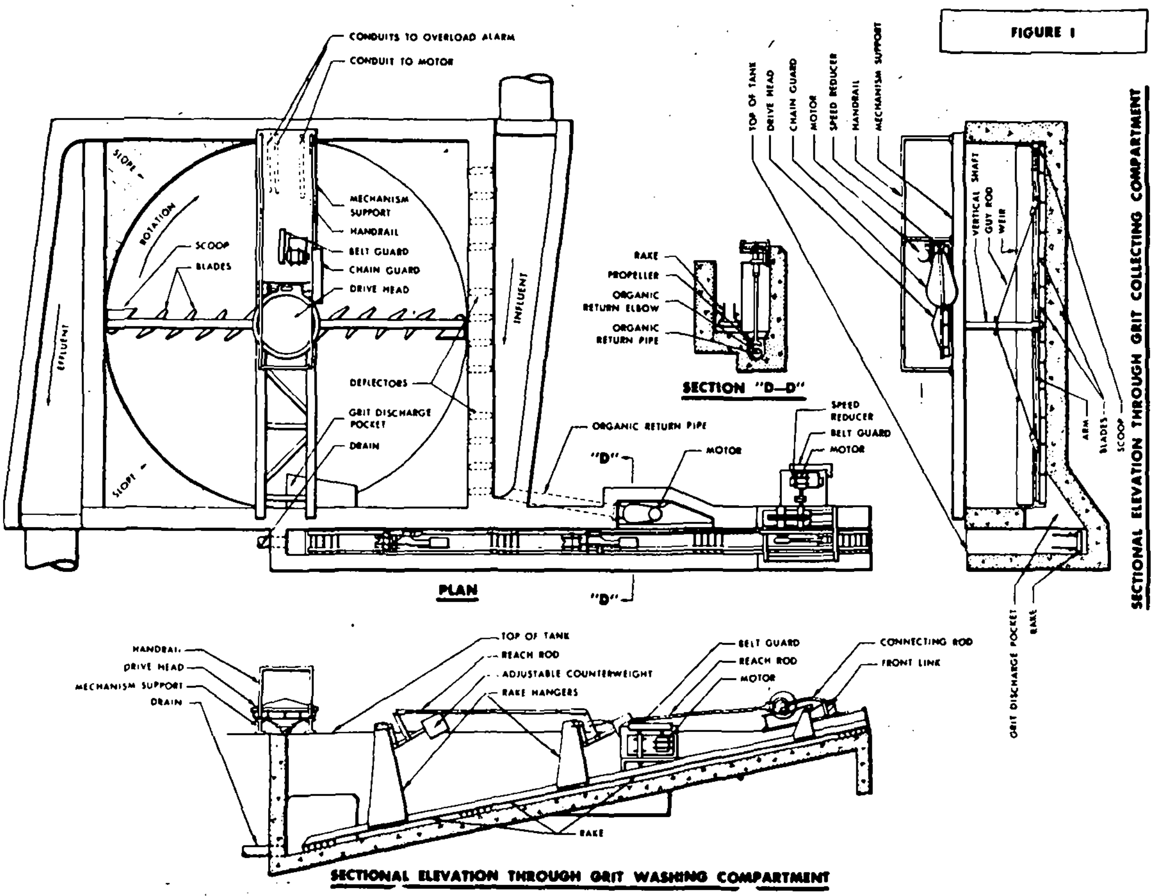


Fig. 5-9 Detritus tank with grit washer.

solution, however, would be to prevent a falling out of the lighter organic particles by maintaining such a high velocity of horizontal water movement that these particles are continuously removed by scour. Commonly a displacement velocity of 0.3 m/sec is chosen for this purpose. Indeed, such a velocity already initiates the scour of sand grains with a diameter

$$d = \left(\frac{0.3}{0.21}\right)^2 0.2 = 0.4 \text{ mm}$$

but in practice the loss of material between 0.2 and 0.4 has been found to be small. With the design factors

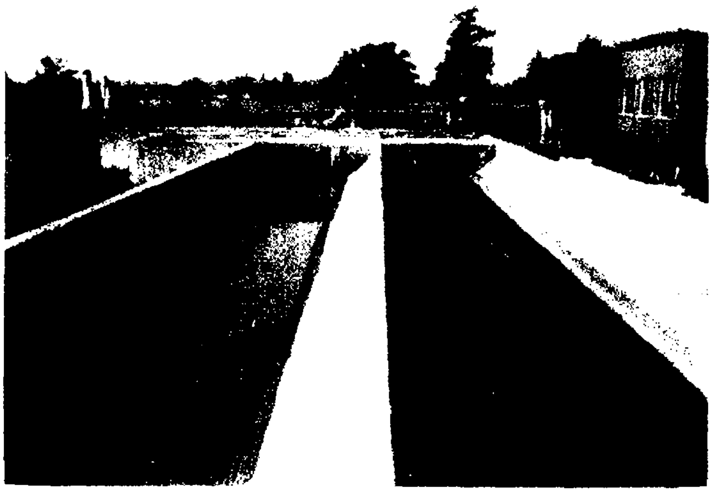


Fig. 5-10 Channel-type grit chamber.

$$s_o = \frac{Q}{BL} = (21)10^{-3}$$

$$v_o = \frac{Q}{BH} = 0.3$$

the ratio between length and depth equals

$$\frac{L}{H} = \frac{v_o}{s_o} = \frac{0.3}{0.021} = 14$$

giving for instance

$Q = 0.5\ m^3/sec$, $H = 1.2$ m, $L = 17$ m, $B = 1.4$ m

$Q = 3\ m^3/sec$, $H = 2$ m, $L = 28$ m, $B = 5.1$ m.

These channel type grit chambers only operate to satisfaction when the horizontal velocity is indeed constant, independent of variations in capacity. With a rectangular cross-section (fig. 5.11), this can

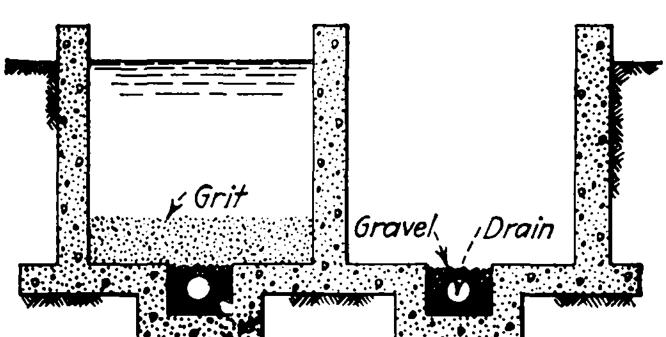


Fig. 5-11 Channel-type grit chamber with rectangular cross-section.

be obtained with a proportional flow weir (fig. 5.12), while with an horizontal weir a parabolic cross-section is required. With the notations of fig. 5.13 the amount of water flowing over such a weir equals

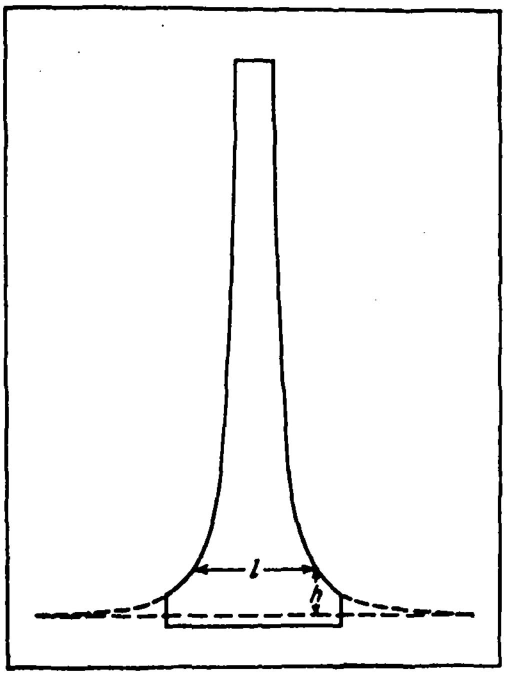


Fig. 5-12 Proportional flow weir. weir.

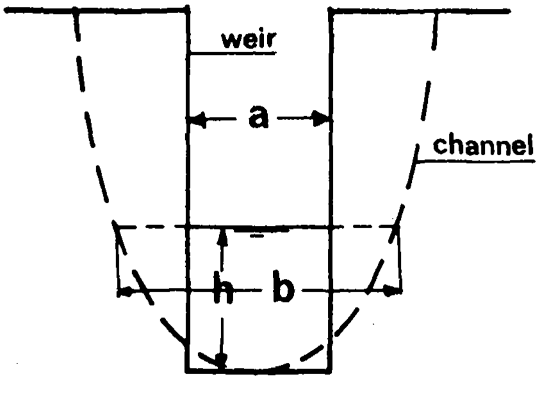


Fig. 5-13 Channel-type grit chamber with parabolic cross-section.

$$Q = 1.8\ ah^{3/2}$$

with h as overflow height. In case h is also the depth of the grit chamber

$$Q = v_o \int_o^h b dh$$

This gives

$$1.8\, a h^{\frac{3}{2}} = v_o \int_o^h b\, dh \quad \text{After differentiation}$$

$$1.8\, a\, \frac{3}{2}\, h^{\frac{1}{2}} = v_o b \qquad \text{or} \quad b = \frac{2.7\, a}{v_o} \sqrt{h}$$

The horizontal velocity v_o in the meanwhile is only constant when grit is removed continuously, for instance as shown in fig. 5.14.

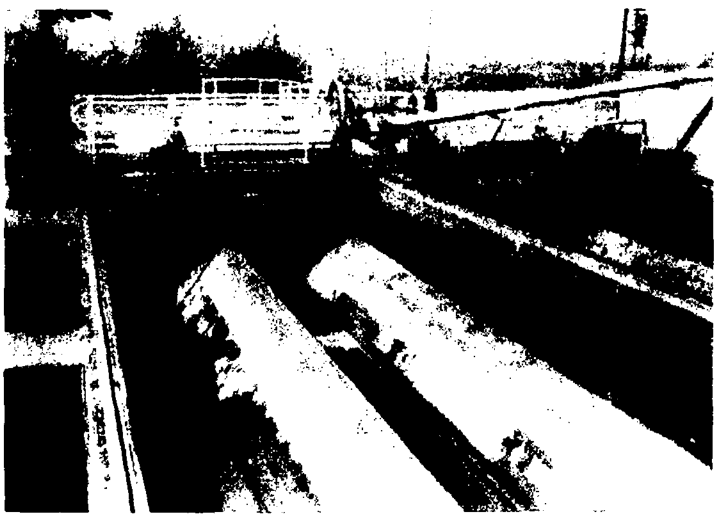


Fig. 5-14 Grit chamber with continuous cleaning by suction dredging.

With grit removal at intervals, the additional depth of the sludge zone (fig. 5.11) will decrease the horizontal velocity in the beginning of the run. Deposition of organic matter may still be prevented by blowing in air (fig. 5.15), also having the advantage of freshening the raw liquor, preventing putrescence during subsequent

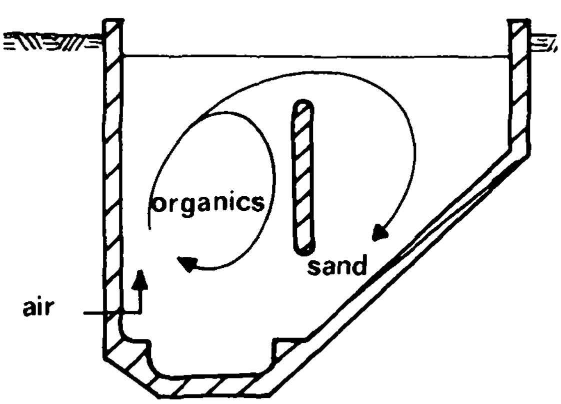


Fig. 5-15 Aerated grit chamber.

treatment. When properly designed, this aeration is so effective that the horizontal velocity of flow may be reduced to 0.2 or 0.25 m/sec, eliminating completely the loss of sand grains larger than 0.2 mm by scour.

With a grit chamber of rectangular cross-section and constant width, the overflow rate s_o is directly proportional to the amount Q of water to be treated, while with a parabolic cross-section the overflow rate is proportional to the capacity to a power $^2/_3$. Generally speaking, however, this variation has little influence, the major factor in the clarification process being the scour produced by the horizontal velocity v_o.

5.4. Thickeners

The water content of the sludge as deposited on the bottom of the settling tank (fig. 4.15) will be high, comprising up to 99% of the total sludge amount. This not only means an enormous volume, but also a mass density not more than slightly higher compared to water. Alumn floc $Al_2O_3.20\ H_2O$ for instance, has a mass density of its own equal to 1180 kg/m^3, meaning a volume of 0.85 l per kg mass. With 99% water, however, the volume of the same kg mass increases to 0.85 + 99 = 99.9 l, while the over-all density drops to

$$\rho = (0.01)(1180) + (0.99)(1000) = 1001.8 \text{ kg/m}^3.$$

Reduction of the water content from 99 to 98% already gives an enormous decrease in volume, from 99.9 to 0.85 + 49 = 49.9 l, and a slight increase in mass density, from 1001.8 to 1003.6 kg/m^3. According to fig. 5.16 a further lowering of the water content has similar effects, although below 85% water and a volume of 6.5 l the results are not impressive. A reduction in volume from 99.9 l to 6.5 l, however, is of great economic importance when for disposal transport of the sludge over great distances is required.

As regards the water present in freshly deposited sludge, a subdivision can be made in 3 types

a. the freely moving interstitial water that can be removed without the intervention of external forces, reducing the water content to about 85%;

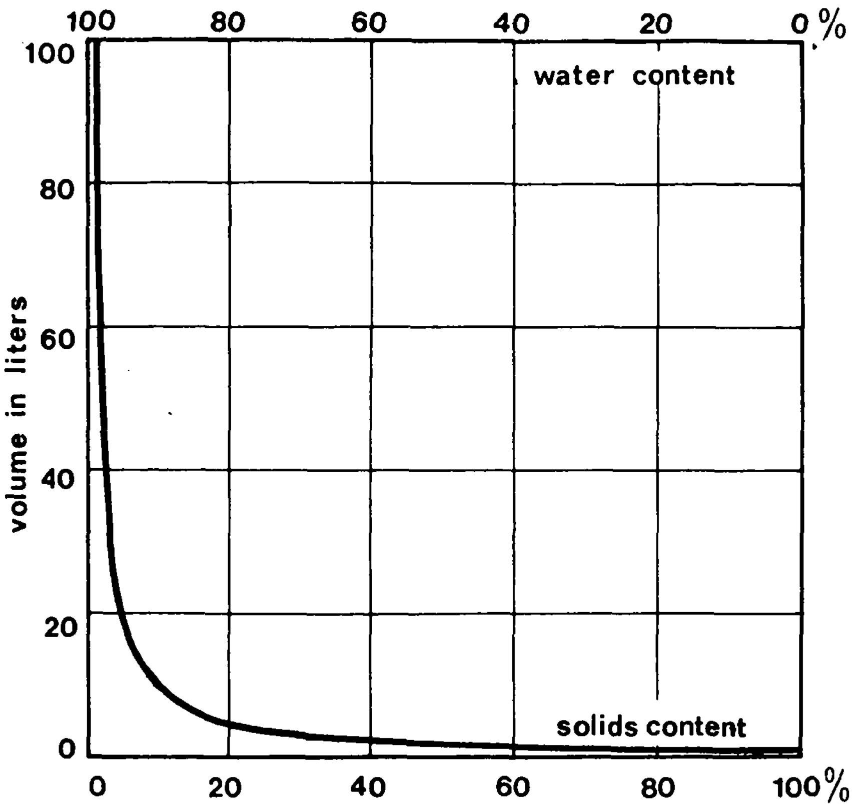


Fig. 5-16 Sludge volume containing 1 kg $Al_2O_3.20\ H_2O$.

b. the interstitial water fixed to the sludge particles by capillary forces. Removal is still possible by natural forces such as evaporation, lowering the water content to about 60%;

c. the water bound to the sludge particles by adsorbtion, chemical bonds etc. Removal down to zero percent can be obtained, but requires enormous amounts of (thermal) energy.

With the batch settling process of fig. 4.14, the amount of freely moving interstitial water is decreased with time by gravity compaction. Taking into account the low mass density of the sludge, only slightly higher than that of the surrounding liquid and in particular the resistance the sludge offers to the upward flowing pore water, it will be clear that this process is extremely slow and even more so as compaction progresses and the passage ways for the water to be removed become smaller and smaller. The resistance against upward water movement, however, may be decreased by stirring, creating temporary channels through which the pore water can escape more freely. This already occurs in every continuous flow settling tank by the motion of the mechanical sludge removal equipment, while it may further be promoted by additional agitation as shown in fig. 5.17.

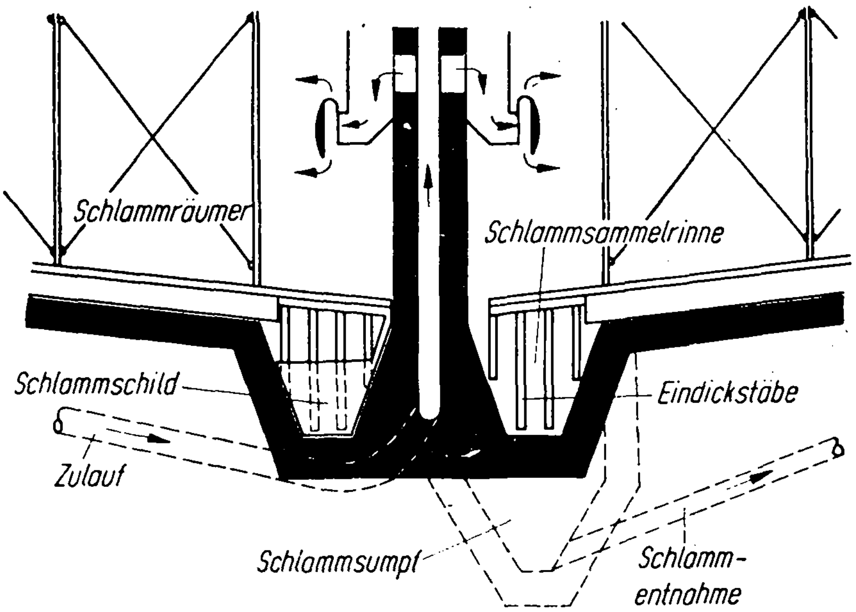


Fig. 5-17 Thickening in the sludge sump of a circular tank.

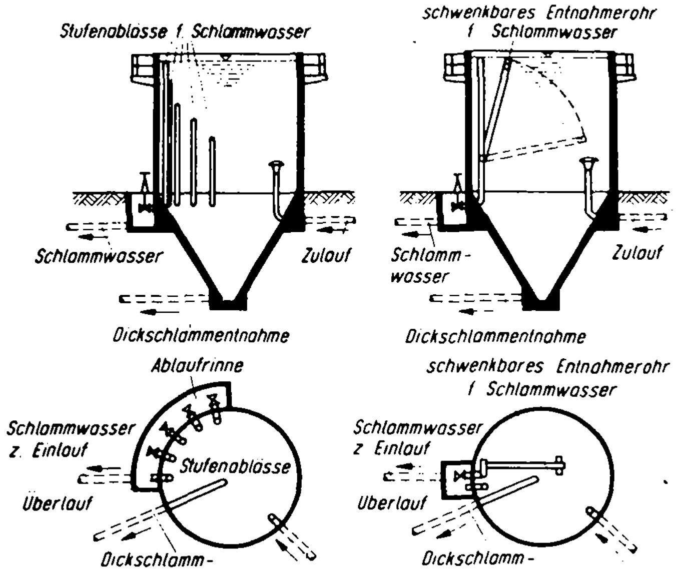
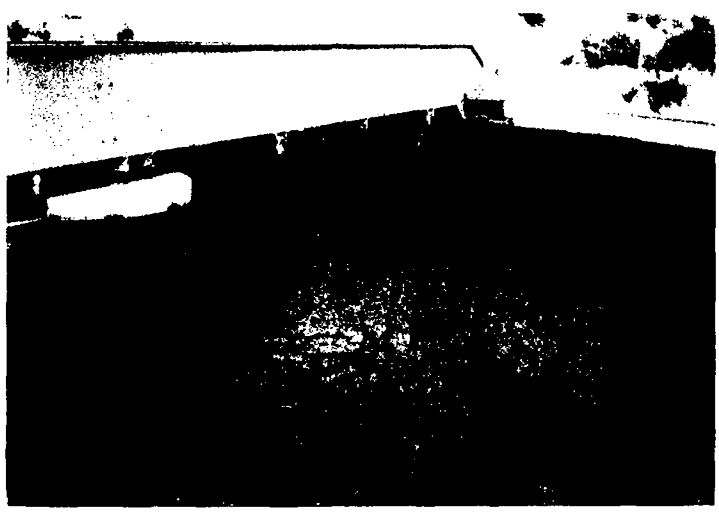


Fig. 5-18 Fill-and-draw thickeners.

When settling tanks are equipped with mechanical sludge removal devices, the detention time of the sludge in the tank will be small and the reduction in the water content of the sludge by gravity compaction and stirring of little importance. Much better results can be obtained by resettling the sludge in separate tanks, equipped with specially designed stirring mechanisms and allowing the addition of chemical conditioners to speed up the process. Fill-and-draw type thickeners are shown in fig. 5.18 and abstract the liberated pore water near the surface, either with multiple outlets at various heights or with a moving outlet supported by a float. The detention time is now between $\frac{1}{2}$ and 2 days and excellent results can be obtained, reducing the water content to below 90%. Continuous flow type thickeners (fig. 5.19) are better suited when an uninterrupted supply of sludge must be handled, but clarification effect is decidedly smaller.



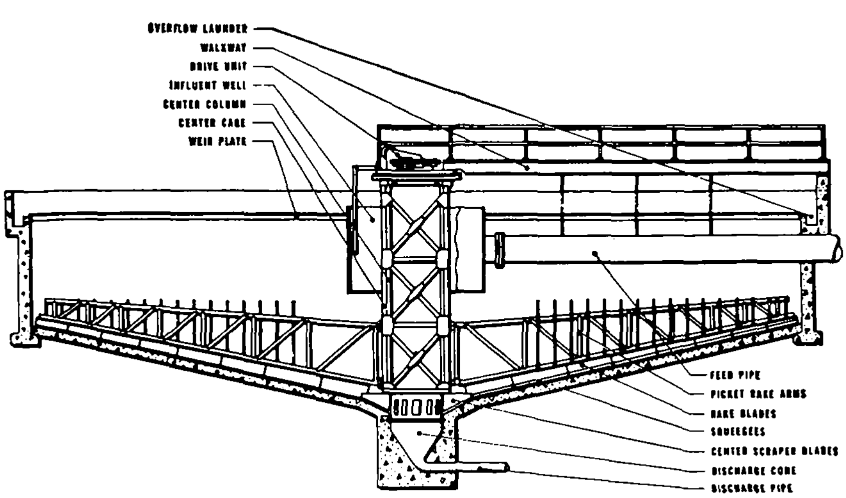


Fig. 5-19 Continuous flow thickeners.

6. Flotation

Flotation is the reverse process of sedimentation, the removal by gravitational rising of suspended particles that are lighter than water. Flotation occurs in every sedimentation tank where the horizontal velocity is decreased and oil, grease and other matters ascend naturally to the surface during substantial quiescence. Such basins must therefore be equipped with skimming devices to remove the accumulated scum, as explained in section 3.9. In the field of water and waste water engineering, flotation as a separate process is applied in three different ways

a. with grease traps (fig. 6.1) to keep back inflammable matter such as oil or clogging matter such as fat before discharge into the sewerage system;

b. as sole treatment of oil contaminated bilge water on board of ships (fig. 6.2) or oil containing drainage water of refineries (fig. 6.3).

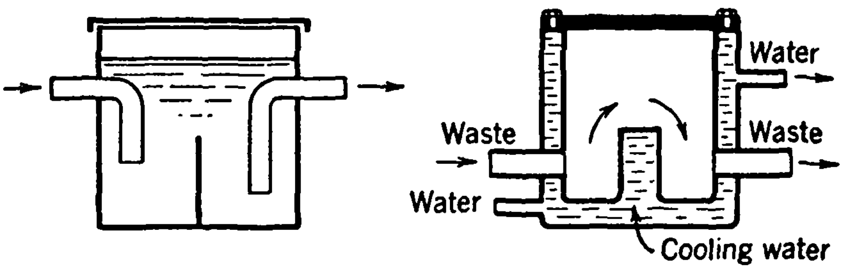


Fig. 6-1 Grease traps.

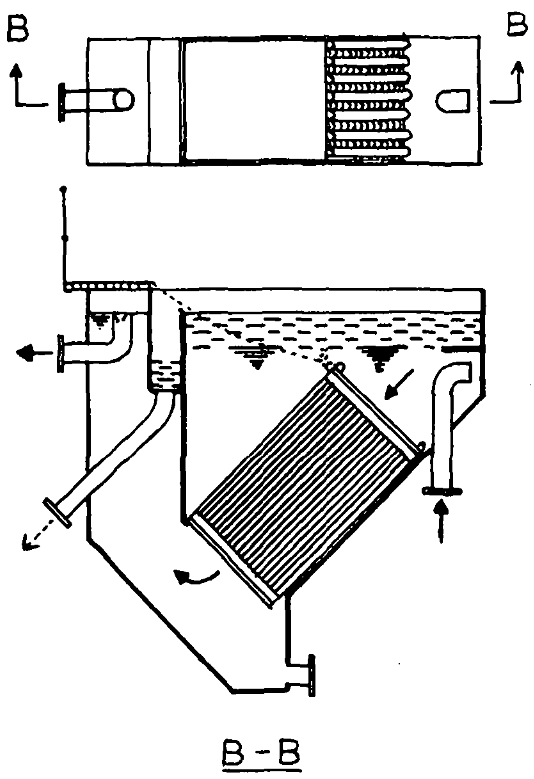


Fig. 6-2 Tilted plate separator for oil removal.

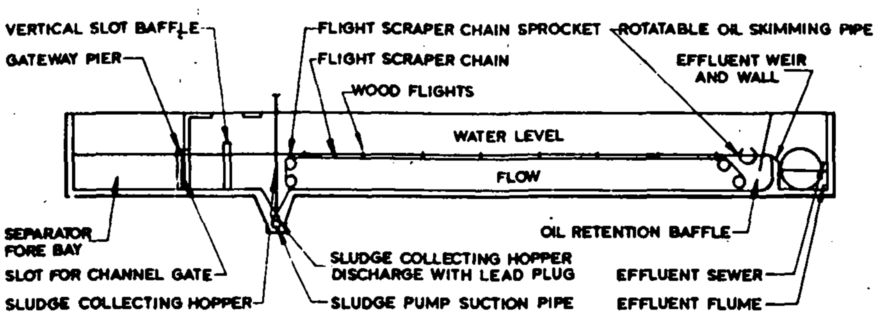


Fig. 6-3 Rectangular horizontal flow tank for oil removal. (American Petroleum Institute)

c. as preliminary treatment to lighten the load on subsequent settling tanks when large amounts of scum are expected, as may be the case with many industrial effluents.

Skimming tanks are commonly designed on the basis of overflow rate with the surface area of the tank as deciding factor. Surface loadings vary from $(1)10^{-3}$ m/sec to $(3)10^{-3}$ m/sec for domestic sewage with much higher values for special applications. As regards the shape of the tank, turbulence and bottom scour do not need to be feared and these tanks are therefore designed as long, narrow, trough-shaped structures with a high velocity of horizontal flow. The depth is commonly between 1 and 2 m, giving detention period varying from 300 to about 1500 sec. With the greater depth and larger detention periods, flotation efficiency is improved by flocculation, but there is also more danger that settled out material accumulates on the bottom of the tank. Short detention periods, 300-500 seconds are therefore commonly preferred and flocculation when necessary is promoted by the addition of various chemicals. In case settling cannot be prevented, removal of detritus must be incorporated in the flotation process by equipping the unit with sludge removal facilities. When desired, greater widths and shorter lengths may now also be applied.

Flotation can be promoted and extended to particles that are slightly heavier than water, by introducing finely divided air or gas bubbles at the bottom of the tank. These bubbles attach themselves to the suspended particles, increasing their buoyancy and entangling them in the surface foam that the bubbles create. An example of such an aerated skimming tank is shown in fig. 6.4, where vertical baffles separate the tank into a central aerated channel and two lateral stilling chambers in which oil and grease gather at the surface. The

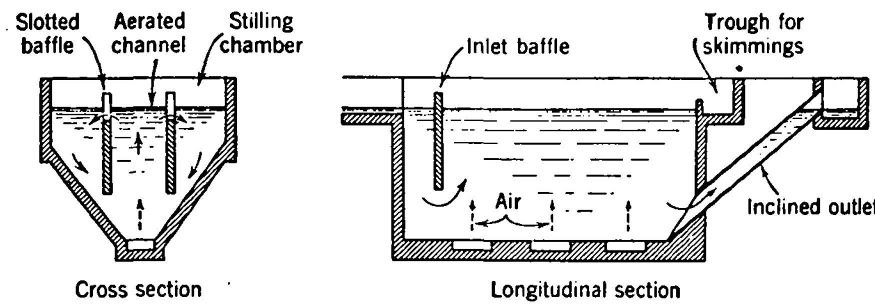


Fig. 6-4 Aerated skimming tank using dispersed air.

baffles are slotted near the water level to provide entrance into the stilling compartments. Settleable matter slides back into the centre channel where it is resuspended and moved forward till it ultimately reaches the inclined outlet from the tank. Air requirements are commonly small, about 0.2 m^3 of air per m^3 of water, but when this aeration is also meant to freshen the raw liquor, larger amounts of air and larger detention times, 2000 sec for instance, are required. Air as a flotation agent in the meanwhile is more effective as air bubbles are smaller, increasing their number and the chance of contact with suspended particles. Extremely fine air bubbles can be obtained when part of the clarified effluent is pressurized to 2 or 4 atmosphere in contact with sufficient air to approach saturation (fig. 6.5). When this air-saturated liquid is brought into or mixed

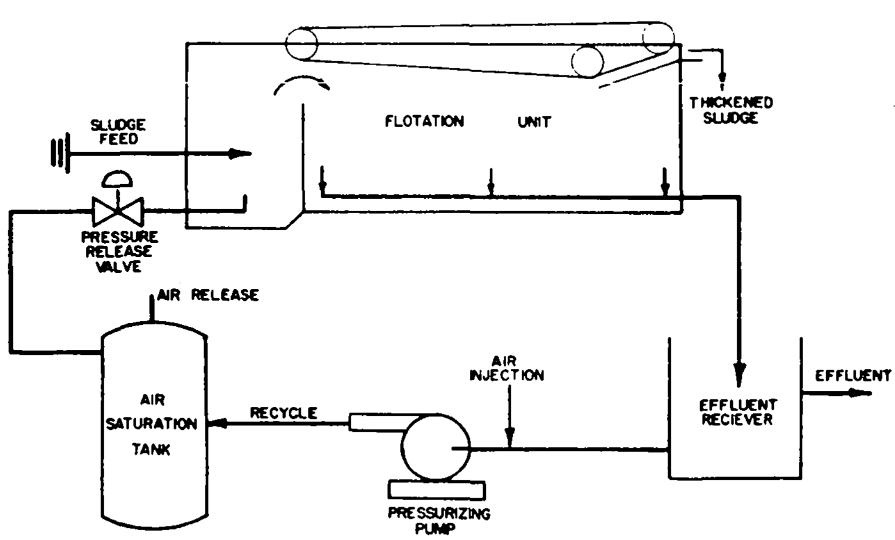


Fig. 6-5 Aerated skimming tank using dissolved air.

with the water to be treated in the flotation tank under atmospheric pressure extremely fine air bubbles are released from solution with the suspended particles as nuclei, increasing their buoyandy and thus promoting flotation efficiency.

Disposal of skimmings is again a difficult problem. When larger amounts of mineral oil are present, it can best be burned or buried together with rakings and screenings. When grease predominates, the skimmings can be added to the sludge in digester tanks, increasing gas production.

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Mechanical filtration

Contents

1. Definitions and terms

Mechanical filtration is the process whereby the water to be treated is passed through a porous membrane, consisting of closely spaced bars or gratings, perforated plates, woven wirecloth or fabric, etc, retaining the floating and suspended particles that are larger in size than the openings in this screening device. With time an accumulation of strained-out material occurs, by which the effective openings in the membrane are reduced and also smaller particles can be retained. This process may be enhanced by adding to the raw water inert fibrous or powdered material, so-called filter aid, capable of forming a mat of intertwined threads or a granular layer which now does the actual work, the porous membrane itself only serving to support this layer of filtering material. Under all circumstances, however, the thickness of this filtering layer is small and its action entirely mechanical.

Mechanical filtration may serve different purposes. In the field of water and waste water engineering the most important ones are

a. protection of the treatment plant by removal of the grosser floating and suspended impurities which otherwise might clog pipelines and channels, damage pumps and other mechanical equipment or interfere with the satisfactory operation of the various unit operations;
b. clarification of the passing liquid by removal of more finely divided particulate matter. Mechanical filtration alone is seldom able to give the desired amount of purification, but it may be of invaluable help to lighten the load on subsequent treatment processes, to reduce the volume of the sludge zone in settling tanks, to prevent a rapid clogging of slow sand filtres, etc.

Next to this, mechanical filtration is used extensively for the dewatering of sludge in water and waste water treatment plants.

When during operation retained material accumulates on the porous membrane, the openings decrease in size and combined area, increasing the resistance against the passage of water. After some time this resistance becomes so high that cleaning of the filter is necessary. With regard to the way this cleaning is effected, three types of mechanical filters may be distinguished, screens, strainers and pre-coat filters. Screens may further be subdivided in fixed screens, cleaned *in situ* and movable screens which are stationary during operation but are lifted from the water for the purpose of cleaning. Strainers are in continuous motion, but only part of the fabric is submerged in the water, available for operation, while the other part is above the surface for cleaning. Pre-coat filters finally may be stationary or moving with cleaning at intervals or continuously by washing away the coating of filter aid and retained impurities.

2. Screens

Fixed screens are commonly constructed from parallel bars (fig.1) rectangular in cross-section, about 10 mm thick and 50 mm wide. They are not meant to purify the water, only to prevent chokage and damage of the subsequent installation. As a consequence, the size of the openings between the bars is not smaller than strictly necessary with regard to their function of equipment protection. According to the size of these openings, bar screens may further be subdivided in

a. coarse screens or racks, clear openings 50 - 150 mm, to prevent logs and timber, dead dogs and cats, etc, from entering the plant;
b. medium-size screens, openings 20 - 50 mm, to keep back those grosser impurities which even in larger plants might clog pipelines, injure pumps and other mechanical equipment;
c. fine screens, openings 5 - 20 mm, for the protection of plants with limited capacity, where small diameter pipelines and small sized pumpbowls must be used.

The approach velocity of the water in the channel upstreams of the screen may not be smaller than 0.3 - 0.5 m/sec to prevent settling out of suspended matter, while the passing velocity of the water in the openings between the bars may not be larger than 0.7 - 1 m/sec to prevent that soft, deformable matter is forced through the screen openings. The ratio between these two velocities depends primarily on the size of the openings compared to the width of the bars, but may further be decreased by setting the bars at a smaller angle (α in fig 4) with the horizontal. When clean, the resistance of a

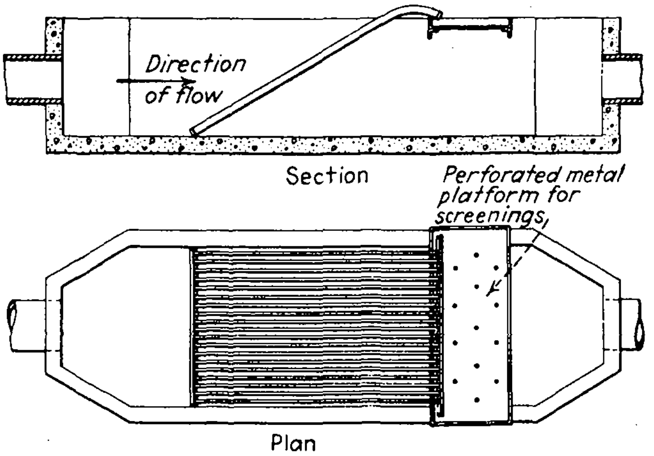


Fig. 1 Fixed bar screen

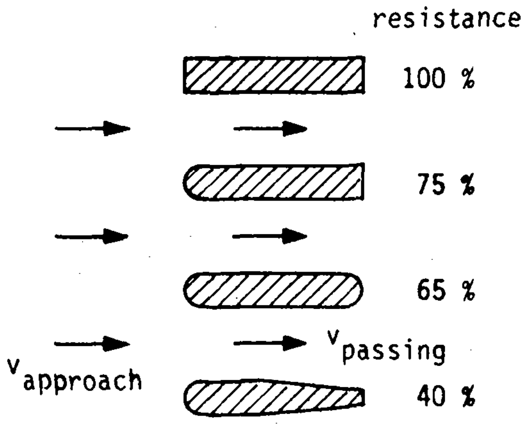


Fig. 2 Streamlining of bars

bar screen is small, a few centimetres only. This resistance may be reduced by streamlining the cross-section of the bars (fig. 2), but generally this is not worth the cost involved. The shape of the bars in fig. 3 has the additional advantage that the openings are self-cleaning and clogging is less. Due to this clogging, the resistance will increase sharply, from H_o for a clean screen with a percentage open area p_o to

$$H = \left(\frac{p_o}{p}\right)^2 H_o$$

when the percentage open area is reduced to p. This head loss will produce a large load on the bars (fig. 4), asking for sufficient

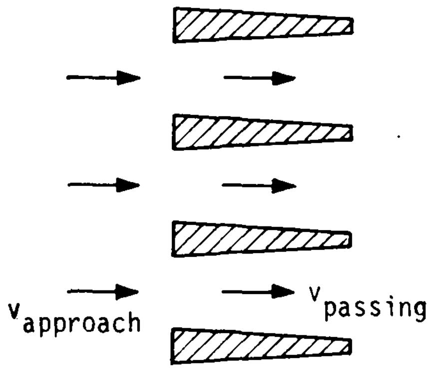


Fig. 3 Bar screen with self-cleaning openings.

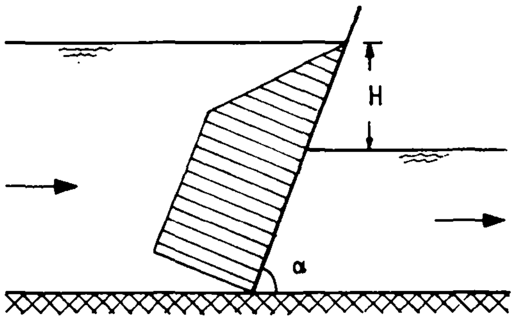


Fig. 4 Hydraulic loading of a bar screen.

structural strength, which in the course of time may even decrease by corrosion. By regular cleaning this hydraulic load must be limited, keeping the maximum value of H below 0.5 m for instance. With regard to unforeseen circumstances, however, it is good practice to base the structural design of the bars on a resistance of 1 to 2 m, depending on local circumstances. Sometimes the maximum resistance is limited by providing the screen with a bye-pass, receiving the water from a side weir with rack (fig. 5).

In water supply practice, bar screens are set at the inlet of surface water from rivers or lakes. Due to their velocity of flow, rivers may carry a high suspended load. The intake works, however, are constructed in such a way that only a minimum amount

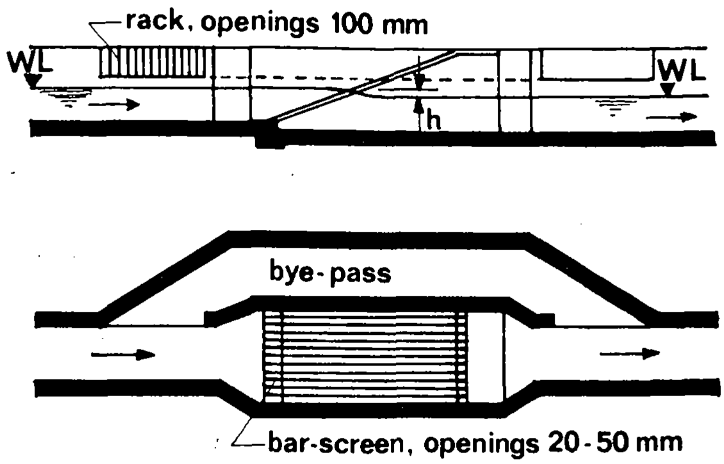


Fig. 5 Bar screen with bye-pass.

of impurities is entrained by the water abstracted. In fig. 6 the entrance velocity is less than 0.1 - 0.2 m/sec, the bottom preferably more than 0.5 m above the river bed, while floating matter is kept back with a dip or floating boom. As a consequence only little material is retained by the screen, allowing this screen to be set at a steep angle, 60° - 75° with the horizontal and making hand cleaning with rakes (fig. 7) a perfectly sound solution. The entry of fishes can be prevented with electrical shock devices (fig. 8), while in cold climates heating arrangements, steam jets or electric wiring, are necessary to prevent a blocking of the

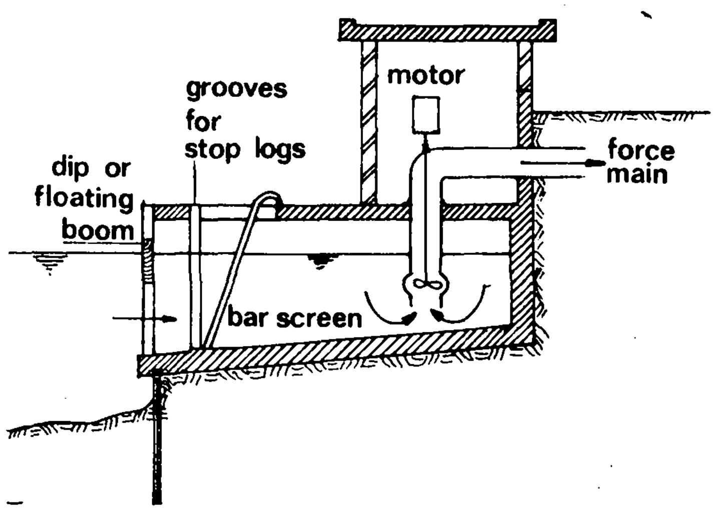


Fig. 6 River water intake with pumping station

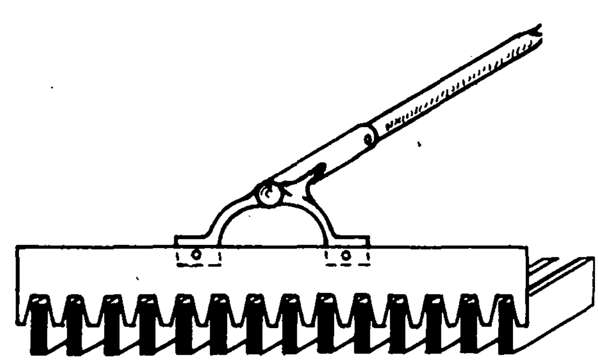


Fig. 7 Rake for manual cleaning of bar screens

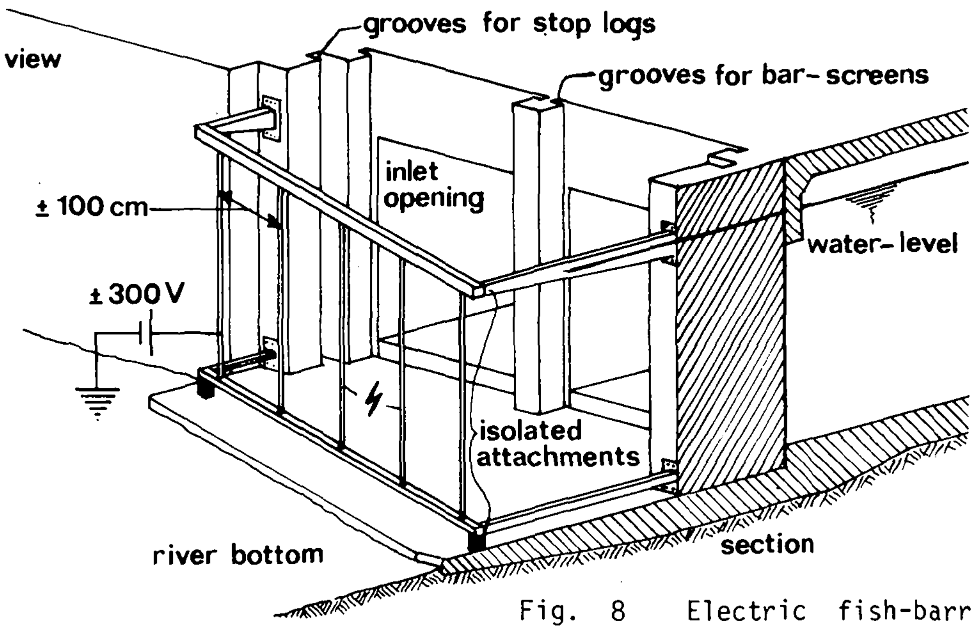


Fig. 8 Electric fish-barrier.

screen by anchor or frazil ice. Disposal of the rakings is commonly not a problem. The amount is small, their nature not offensive, often allowing a downstream return to the river. In smaller plants, intake works as described above are too elaborate and only the inlet to the pump needs a strainer (fig. 9). When this strainer is not accessible for cleaning and blocking cannot be excluded completely, a hydraulic backwash of the strainer may give an attractive solution (fig 10). Similar constructions may be used for abstracting water from lakes where by the absence of flow grosser suspended impurities will not be present. Even when screening is completely superfluous, a rack is required for protection of the operating personnel. Commonly such racks are made of circular bars, for instance with a diameter of 20 mm and openings of 100 - 150 mm. Wire-mesh screens are cheaper and less deformable, but they tend to retain more material as for instance leaves in autumn. Lifting them above water, however, is

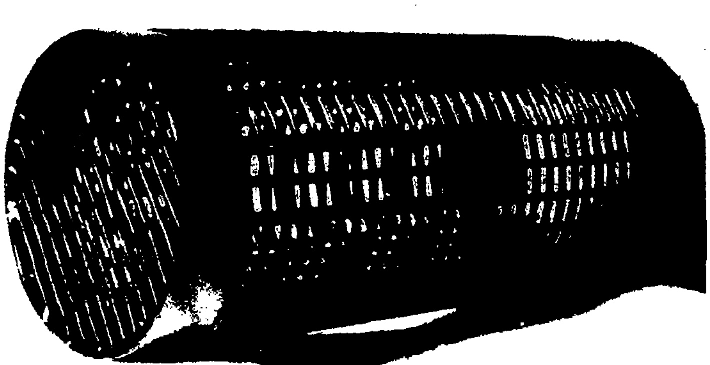


Fig. 9 Pump strainer

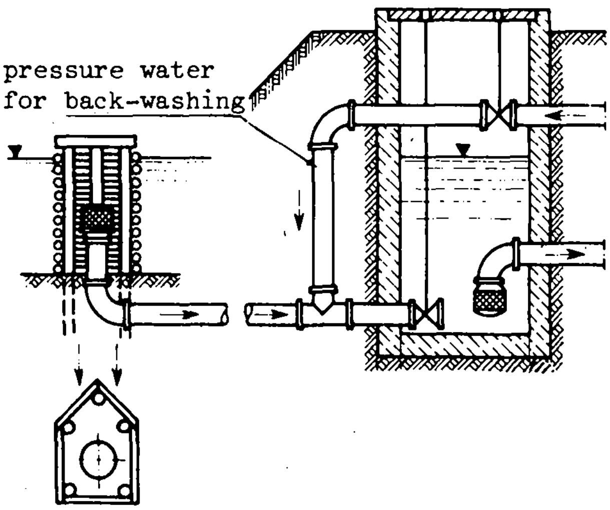


Fig. 10 Riverwater intake with a hydraulically back-washed screen.

easy after which they can be cleaned with a jet of pressure water. Very conscientious engineers construct such movable screens in duplicate, so that one is always in the water for operation while the other is above water for cleaning.

In waste water treatment plants, bar screens for equipment protection are set before the pumps and when no pumping is required before the primary settling tank or grit chamber when present. Compared with river water, the amount of floating and suspended matter is much larger, while furthermore no selection can be made and all this material must be processed. As a consequence, large amounts of screenings must be expected, for municipal sewage roughly

size openings	5	10	20	50	mm
screenings	15	10	5	2 × 10^{-3}	m^3/capita /year.

A plant handling the domestic sewage from 100.000 people will have a maximum hydraulic capacity of about 0.6 m^3/sec, asking for a screen area of 1 - 1.5 m^2 on to which an amount of 1 - 3 m^3 of suspended matter will be retained each day. This amount is so large, that hand cleaning with long-tined rakes is only possible when the screens are set with a small angle, down to 20^0 or 25^0, with the horizontal. Such a flat position has the added advantage that the flowing water pushes the retained material to the top of the screen, facilitating cleaning, while the damming effect of a partially blocked screen raises the upstream water level, by which the wetted screen area will increase appreciably. With regard to the origin and putrescibility of the screenings, manual cleaning is an unpleasant task to say the least. In sewage treatment plants bar screens are therefore mostly cleaned mechanically. In developed countries with a high cost of labor, mechanical cleaning will also be more economical, moreover because the screens can be set at a steeper angle, 60^{o} to 75^{o} with the horizontal, reducing space requirements.

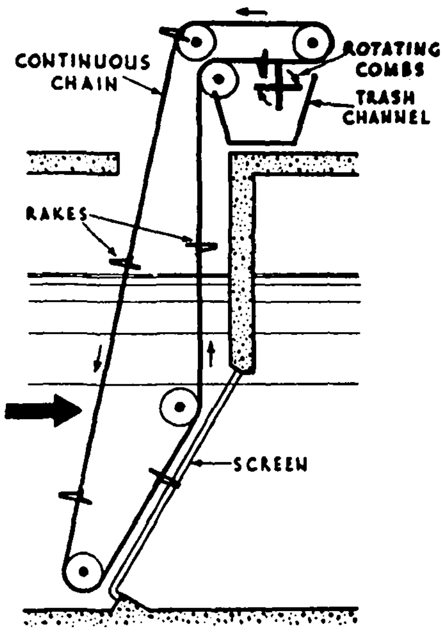


Fig. 11 Diagram of a mechanically raking screen.

The principle of cleaning bar screens mechanically is shown in fig. 11, but as regards the actual construction innumerous proprietary designs are available of which fig. 12 and 13 only show a few examples.

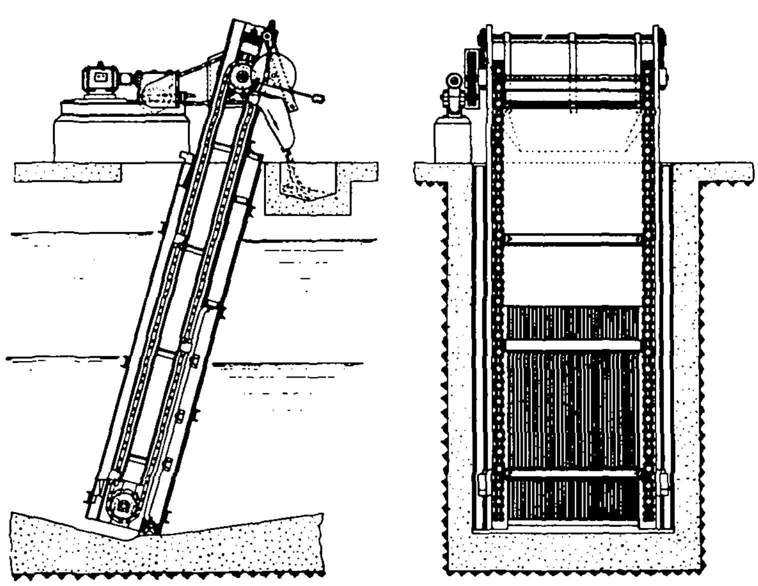


Fig. 12 Mechanical rakes mounted on an end-less chain (Passavant).

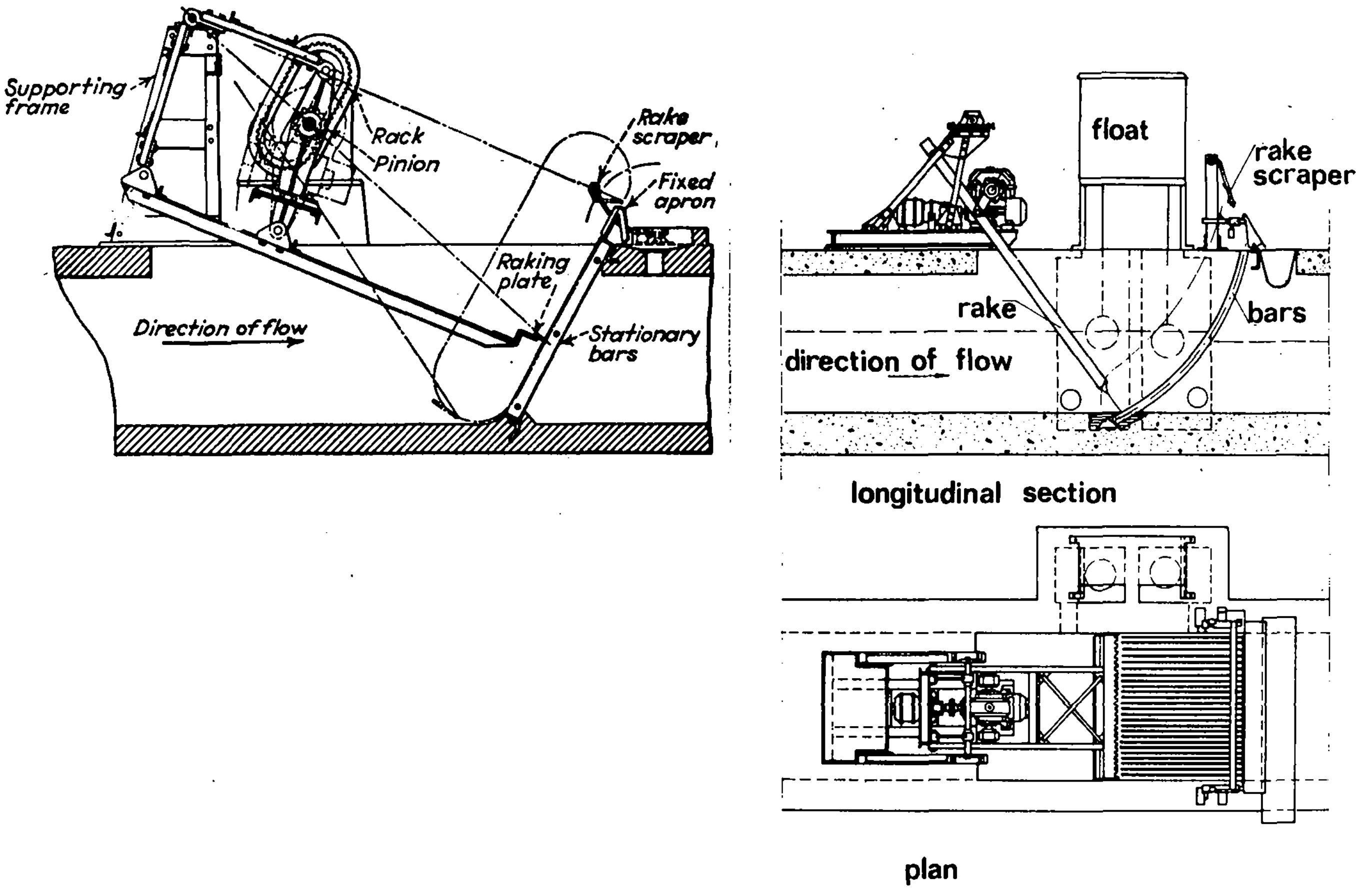


Fig. 13 Dorrco bar screens.

The common feature is the presence of rakes which periodically sweep the entire screen area, removing the retained material, in this way keeping the hydraulic head loss down to values of about 0.1 m and preventing surges of high flow after a strongly clogged screen is cleaned and restored to its original capacity. The rakes can be operated by a timing device or by a float on the upstream water level measuring the amount of ponding. Preferably both controls are installed, working independent of each other, to prevent damage when one of them fails. A remaining difficulty concerns the amount and nature of the retained materials. Disposal may be effected by burying, burning, addition to digester tanks or municipal refuse, but again this work is unpleasant and labor intensive. Keeping in mind the purpose of equipment protection, a solution may also be obtained by grinding the screenings and returning the pulverized material to the sewage for subsequent treatment in settling tanks. The shredder

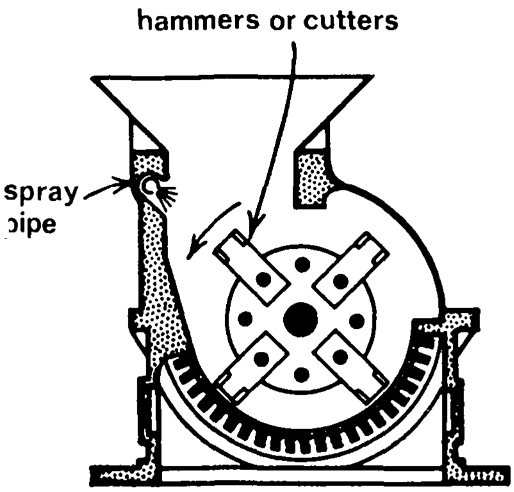


Fig. 14 Shredder.

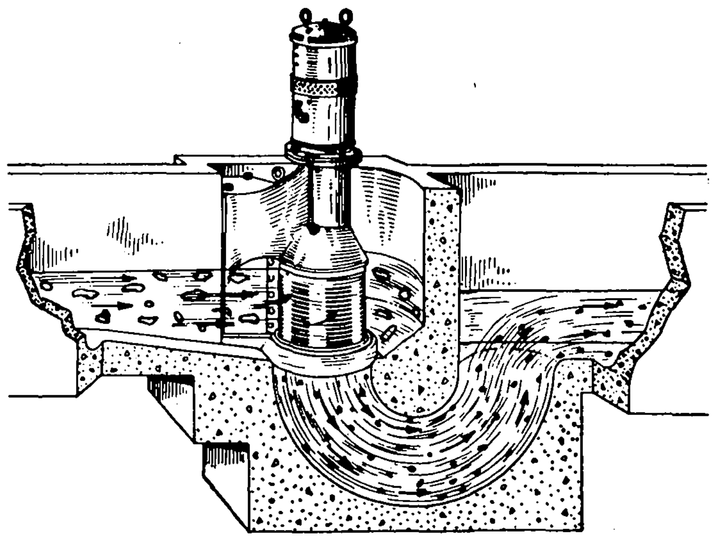


Fig. 15 Comminutor.

of fig. 14 is set above the water level, making it difficult to prevent nuisance from odors or moisture. A more elegant and straightforward solution is to pass the whole flow of raw sewage through comminuting devices (fig. 15), which desintegrate the grosser impurities without removal from the water. The screenings remain submerged, reducing flies, odors and unsightliness. With regard to the small clearance between moving and stationary parts, comminutors are preferably set behind the grit chamber, after the hard and sharp grit particles have been removed from the liquid. In both cases, however, racks remain necessary, but they keep back only minute amount of materials, mostly of an inoffensive nature.

In the beginning of this section it was stated that bar screens are only used for equipment protection. In exceptional cases, however, mechanically raked fine screens are used for purification purposes. In drinking water practice they may be applied to remove suspended particles of the same density as water ahead of settling banks. Where sewage is discharged without treatment, fine screens may still be desired to keep back coarse floating objects, which otherwise would render receiving waters unsightly or cause objectionable shore-line conditions. In terms of surface water pollution, however, their effect is negligeable, being a few percent only. In both cases much better results can be obtained with strainers, having much smaller openings. To prevent freezing in winter time and with sewage to prevent nuisance of flies, odors, etc, fine screens must be installed in a screen house.

3. Strainers

Strainers are moving screens with part of the porous membrane below water for operation and the other part above water for cleaning. The membrane travels at an appropriate speed of for instance $(50)10^{-3}$ m/sec, so that blocking of the openings by the arrested solids is only partial and the hydraulic resistance against the passage of water fairly low. After leaving the water, the membrane is liberated from the retained impurities by cleaning arrangements, after which it re-enters the water in a clean condition. This continuous cleaning allows the use of fine openings, even with heavily contaminated water. Common constructions are

parallel bars with openings of 2-5 mm;

perforated metal plates with slots 1-2 mm wide;

fine meshed wire cloth with openings 0.2-1 mm.

The larger openings can be cleaned with rakes or brushes, but for the finer openings jets of water, steam or air must be applied. With regard to their more delicate construction, strainers are usually housed, which in cold climates is even a necessity to prevent freezing in winter time. Strainers are easily damaged by corrosion and great attention must consequently be given to the selection of suitable materials or coatings.

Strainers are commonly bought as complete units from firms specializing in this field. This has resulted in an enormous variety of proprietary constructions of which fig. 16 and 17 shows perfo-

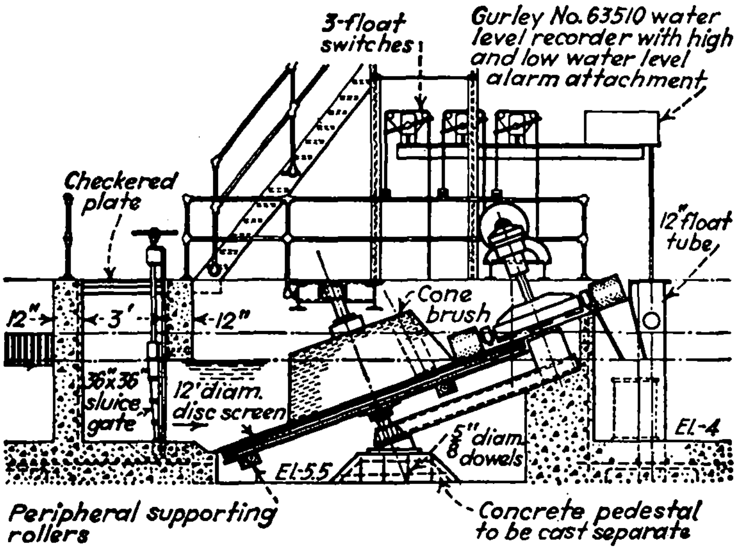


Fig. 16 Riensch-Wurl disk screen.

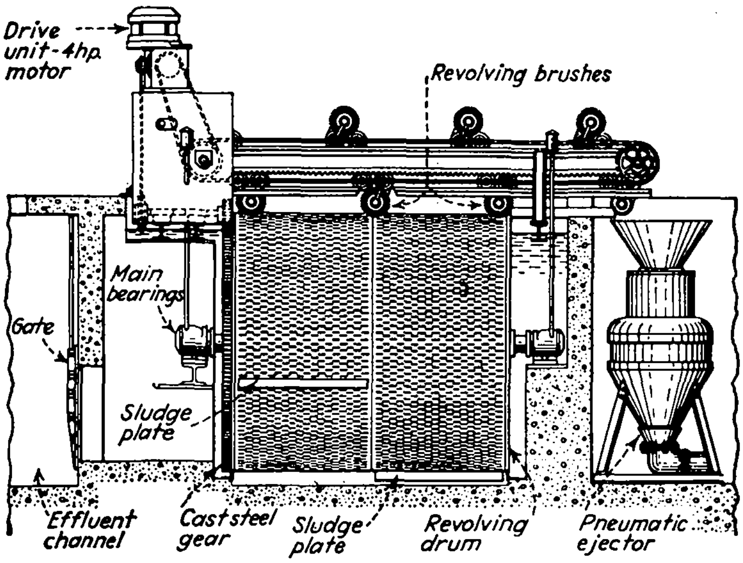


Fig. 17 Link-Belt drum screen.

rated plate strainers cleaned with brushes and fig. 18 to 22 wire mesh strainers cleaned with wash-water jets. Travelling band strainers are very popular, but the installation of fig. 19 has the disadvantage that inadequate back-washing results in retained impurities being carried into the downstream side of the screen. In this respect, the construction of fig. 23 is better suited. As an added advantage, the wire gauze is now stretched over curved frames, increasing the area of contact with the water and the capacity per unit width. Especially with fine openings, a failure of the washwater supply will result in a rapid clogging, increasing the resistance against the passage of water. To prevent damage of the fabric by this hydraulic loading, relief weirs must be present or the strainers installed in duplicate (fig. 24).

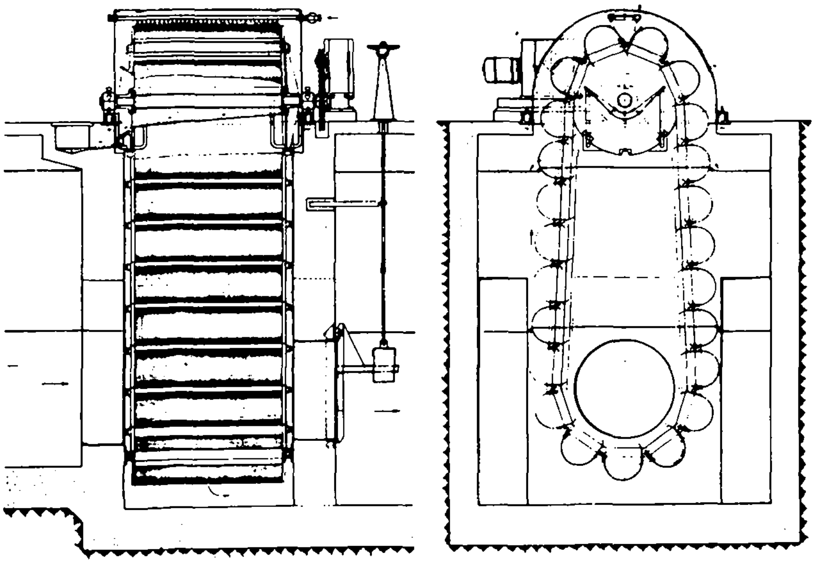


Fig. 23 Travelling band strainer.

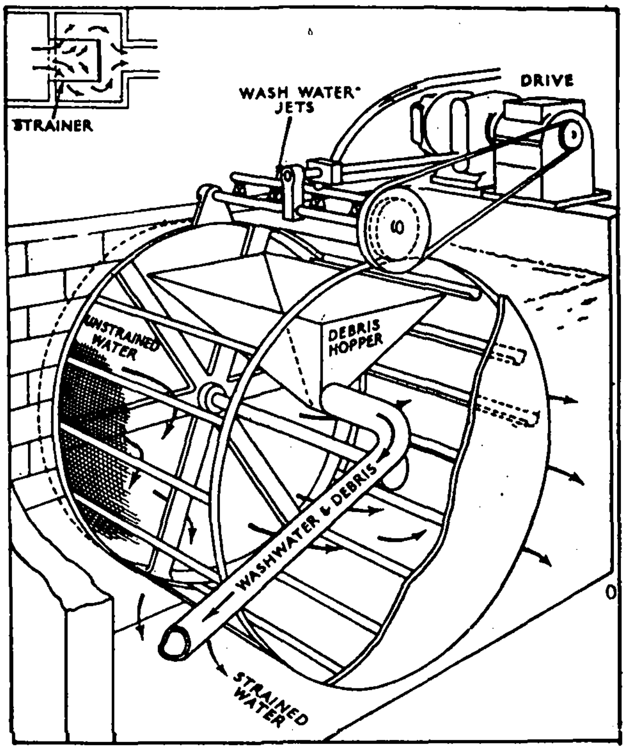


Fig. 18 Rotary drum strainer

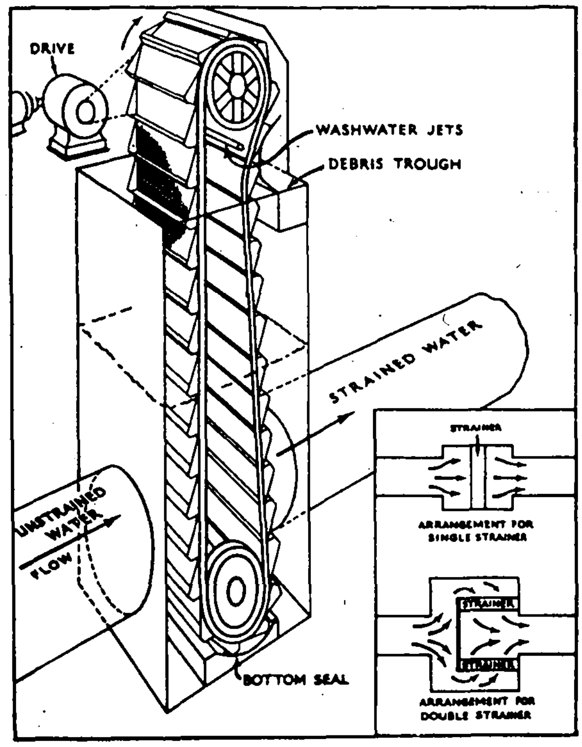


Fig. 19 Travelling band strainer

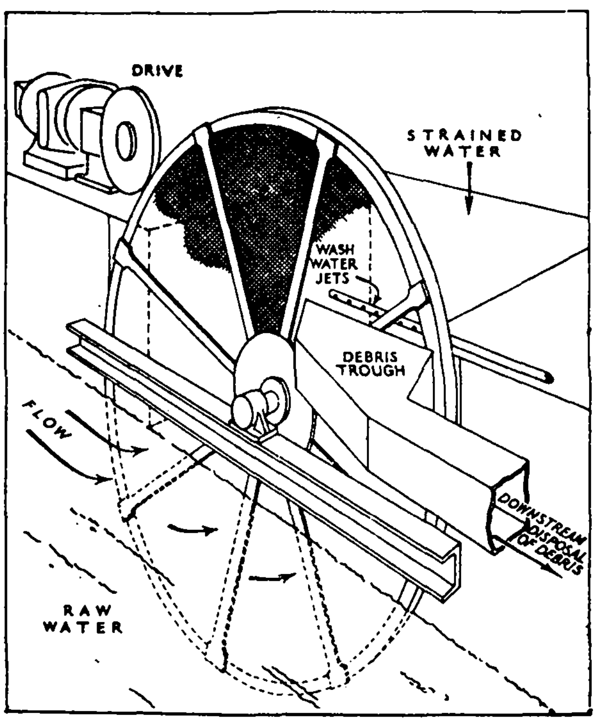


Fig. 20 Rotary disk strainer

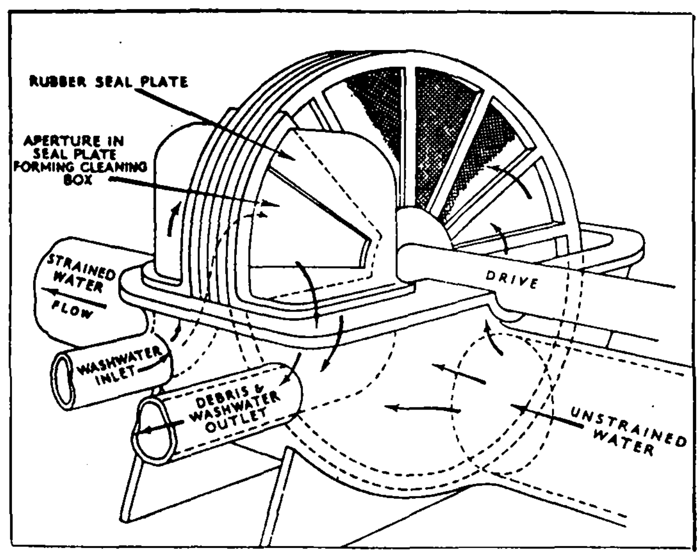


Fig. 21 Pressure strainer

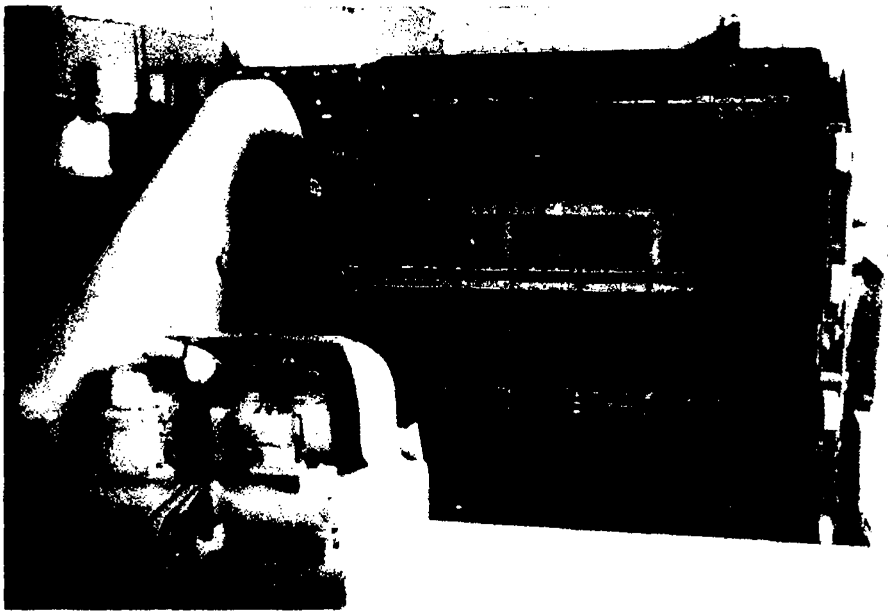


Fig. 22 Travelling-band strainer at the Skudai treatment plant for Singapore.

In water supply practice, strainers may be used directly after the intake screens, to assure that subsequent treatment processes such as sedimentation and filtration may proceed unhindered. In terms of real purification, their effect is small and hardly worth the cost involved, but they do have value in nuisance prevention in assuring a smooth operation of the plant. Disposal of the retained material is usually not a problem. The washings are collected in a launder and are led back to the river at a point well below the intake. In waste water treatment, strainers are sometimes used in stead of primary settling tanks. This gives an appreciable saving in the cost of construction, but on the other hand the efficiency in removing suspended solids is much smaller, 10 to a maximum of 20% in stead of 50% and more. The most elegant disposal of the retained material is now by pumping it to the digester tanks. In exceptional cases strainers are used as sole treatment of waste water, to prevent a visible contamination of the receiving water, the formation of sludge banks and septic conditions. With the larger amount and the putrescibility of the strainings, disposal is a difficult problem. Common solutions are burying, incineration or addition to the municipal refuse.

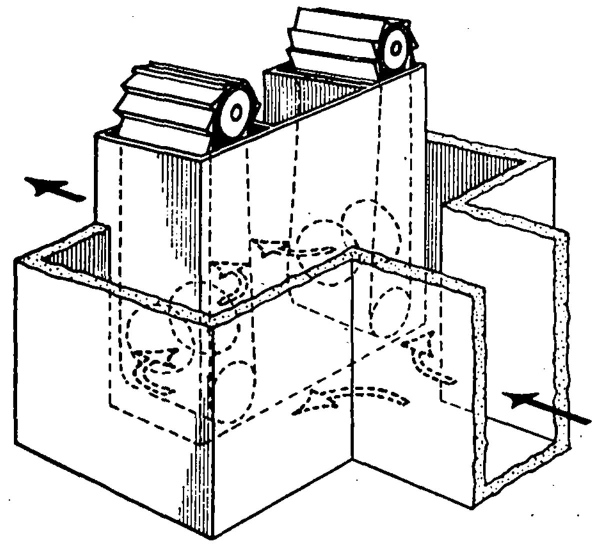


Fig. 24 Travelling-band strainers in duplicate.

4. Principles of micro-straining

Micro-strainers are hydraulically cleaned rotary drum strainers (fig. 25), covered with a finely woven metallic fabric. This fabric

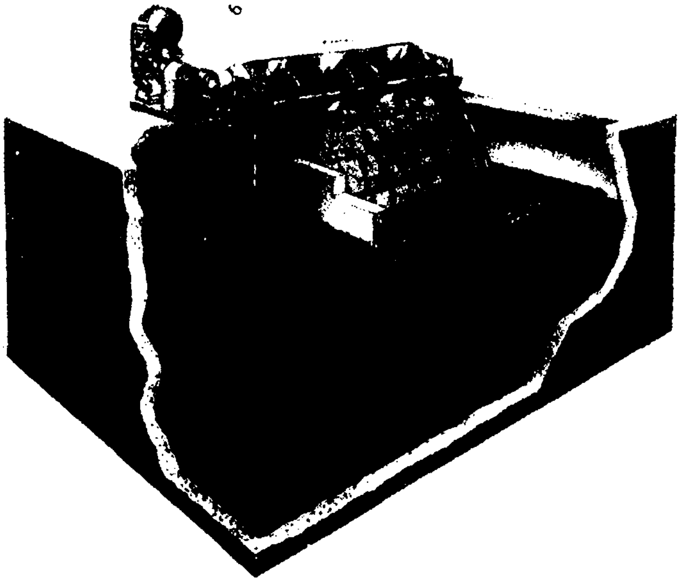
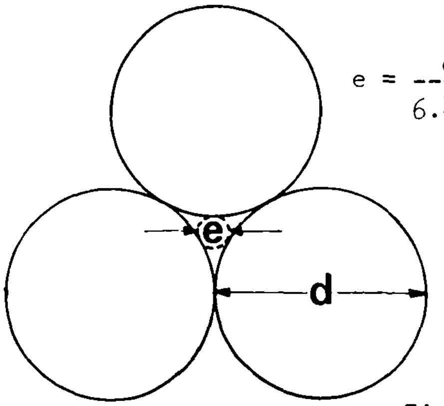


Fig. 25 Perspective drawing of a micro-strainer.

has extremely small openings, 0.02-0.06 mm, that is to say smaller as the interstices between the grains in a rapid or slow sand filterbed (fig. 26). The fabric moreover is woven in such a way that viewed normally no open area is shown (fig. 27), promoting the formation of a matt of retained material (fig. 28). In this way the openings are further reduced, enabling micro-strainers to keep back solids of sizes still smaller than the minute apertures of the fabric. The remaining openings are so small indeed, that with fixed screens and even the clearest of raw waters, they would become blocked in a matter of minutes, whilst with ordinary good quality water containing larger amounts of suspended matter, their useful life would be reckoned in seconds. Micro-strainers must therefore be



$$e = \frac{d}{6.46}$$

	d	e	
rapid filters	o.6	0.093	mm
slow filters	0.2	0.031	mm

Fig. 26 Openings between the grains of a filterbed

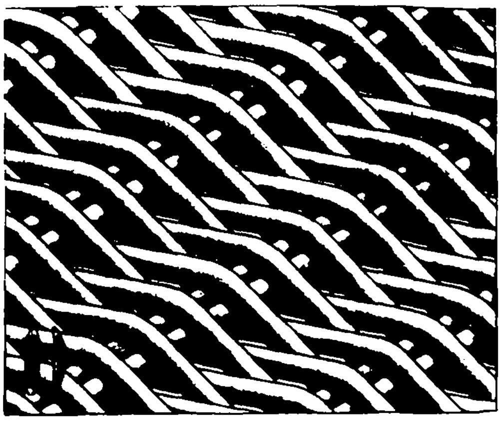


Fig. 27 Micro-straining fabric

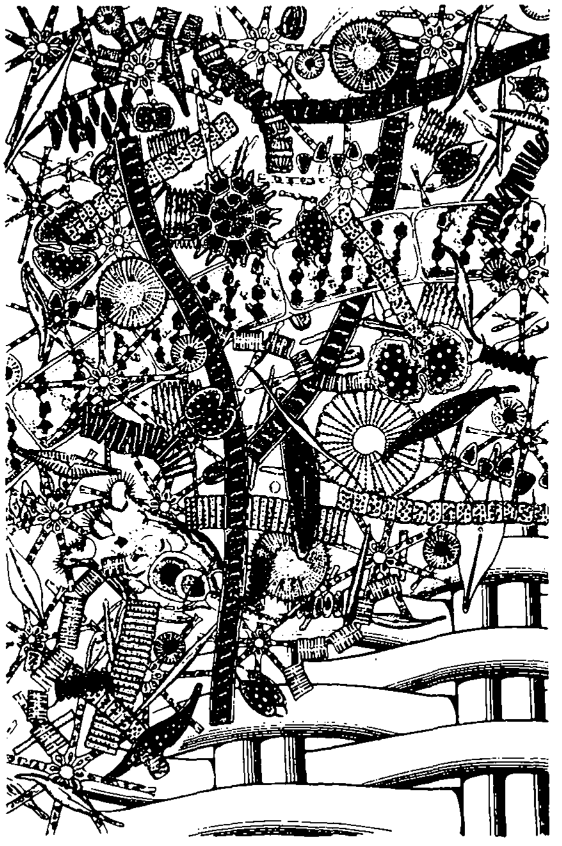


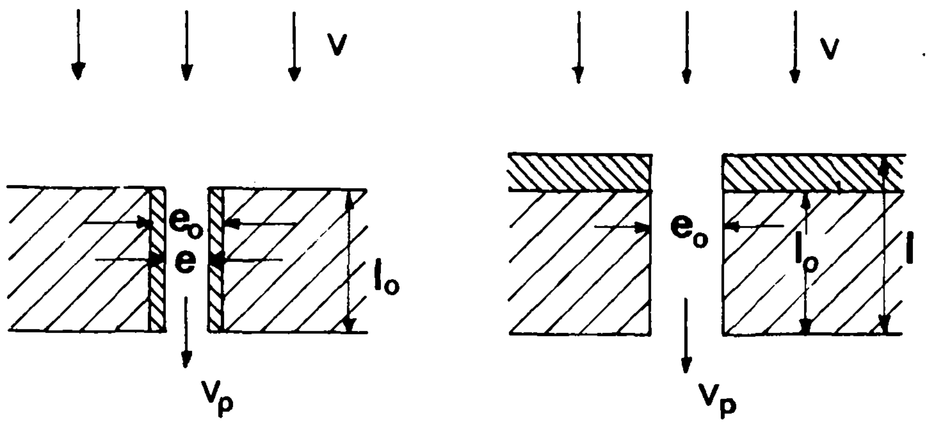
Fig. 28 Matt of microscopic plankton organisms on micro-straining fabric

build as automatic selfcleaning machines, which run continuously, eliminating solids from the water flowing through them and disposing of these solids at the same time.

With the continuous back-washing, the capacity of a micro-strainer not only depends on the area and construction of the micro-mesh fabric, the clogging properties of the raw water and the maximum allowable flow resistance, but also on the area of fabric cleaned in unit time. To draw up a relationship between the six factors involved, various models may be visualized. In fig. 29a it is assumed that the clean fabric has n pores of length l_o and diameter e_o per area A, while clogging reduces this diameter uniformly to e. According to Poiseuille's law for the laminar flow of water through capillary tubes, the flow resistance than equals

$$H = \frac{32\nu}{g} l_o \frac{v_p}{e^2}$$

with ν as kinematic viscosity of the water, g as gravity constant and v_p as actual velocity of the water inside the pores. With v_o as constant capacity per unit area



a. deposition in the pores b. deposition on the surface

Fig. 29 Models for clogging of micro-straining fabric.

$$v_p = \frac{v_o A}{n \frac{\pi}{4} e^2} \qquad \text{, substituted}$$

$$H = \frac{128\nu}{\pi g} \frac{A}{n} l_o \frac{v_o}{e^4}$$

The resistance of the clean fabric thus becomes

$$H_o = \frac{128\nu}{\pi g} \frac{A}{n} l_o \frac{v_o}{e_o^4} = \alpha v_o$$

and after clogging

$$H = H_o \left(\frac{e_o}{e}\right)^4$$

With c_a as gravimetric and $\gamma_a c_a$ as volumetric concentration of suspended matter retained by the micro-mesh fabric, the reduction in pore size from e_o to e in time t is determined by

$$v_o c_a \gamma_a tA = n\, l_o \frac{\pi}{4} (e_o^2 - e^2)$$

from which follows

$$1 - \left(\frac{e}{e_o}\right)^2 = \frac{v_o c_a \gamma_a tA}{n l_o \frac{\pi}{4} e_o^2} = \beta_a v_o t \qquad \text{or}$$

$$\left(\frac{e}{e_o}\right)^4 = (1 - \beta_a v_o t)^2 \qquad \text{and} \qquad \frac{H}{H_o} = \frac{1}{(1 - \beta_a v_o t)^2}$$

In fig. 29b another model is shown with the retained material being deposited on the membrane, increasing uniformily the length of the pores from l_o to l, but leaving their diameter unchanged at e_o. According to the formulas given above, the increase in flow resistance is now determined by

$$H = \alpha v_o \qquad\qquad H = H_o \frac{l}{l_o}$$

$$v_o c_b \gamma_b t A = (A - n \tfrac{\pi}{4} e_o^2)(l - l_o)$$

from which follows

$$\frac{l}{l_o} - 1 = \frac{v_o c_b \gamma_b t A}{l_o (A - n \frac{\pi}{4} e_o^2)} = \beta_b v_o t \qquad \text{and}$$

$$\frac{H}{H_o} = 1 + \beta_b v_o t$$

In reality, however, the two models will occur simultaneously. The increase in resistance is now due to both phenomena, in formula

$$\frac{H}{H_o} = \frac{1 + \beta_b v_o t}{(1 - \beta_a v_o t)^2}$$

Taking the logarithme gives

$$\ln \frac{H}{H_o} = \ln (1 + \beta_b v_o t) - 2 \ln (1 - \beta_a v_o t)$$

For small values of t this may be approximated by

$$\ln \frac{H}{H_o} = \beta_b v_o t + 2\beta_a v_o t = \beta v_o t \qquad \text{and}$$

$$\frac{H}{H_o} = e^{\beta v_o t}$$

that is to say an exponential increase of flow resistance with time. For larger values of t, this will not be completely true. The difference, however, will be small as according to the first model the resistance rises more rapidly and according to the second model more slowly with time than corresponds with an exponential increase.

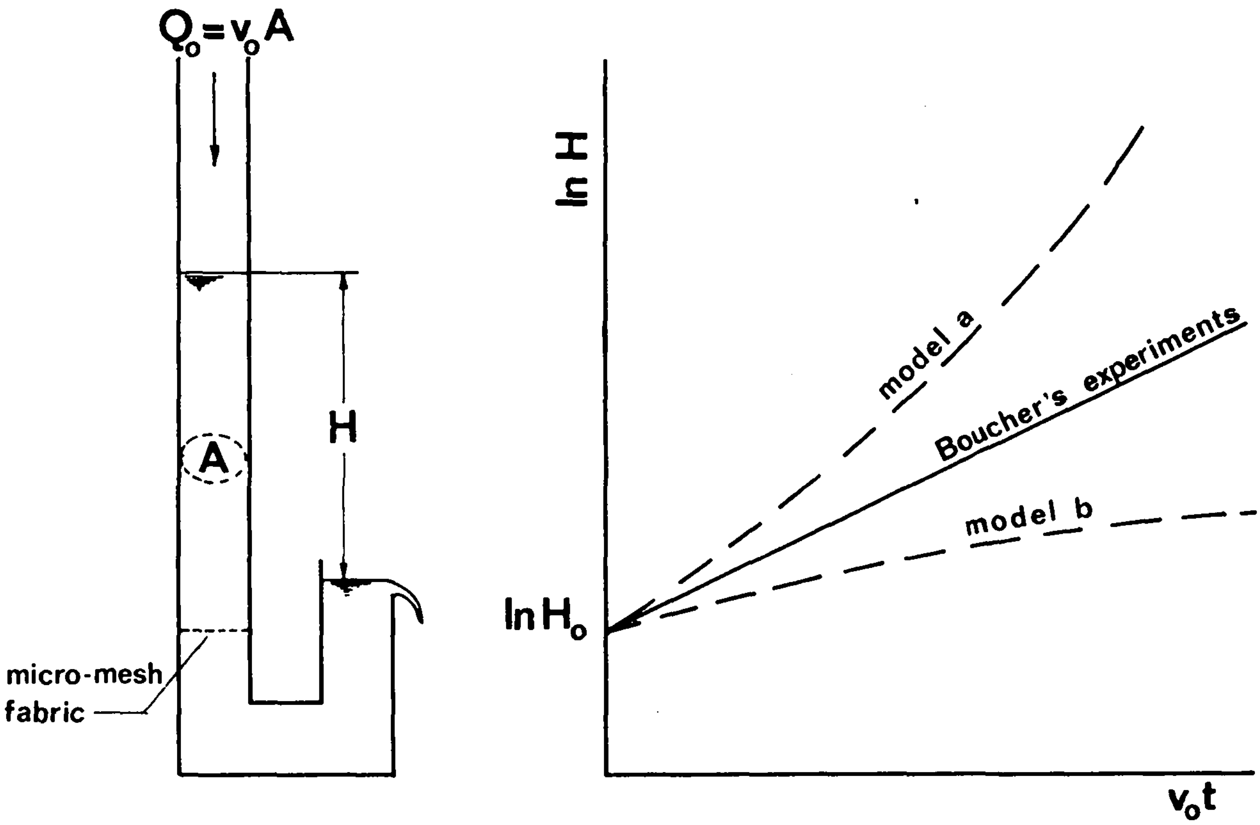


Fig. 30 Experimental determination of filtrability according to Boucher.

The straining law derived above has been checked experimentally by Boucher, measuring the resistance of the micro-mesh fabric as function of the volume of water passed. As shown schematically in fig. 30, the fabric to be applied is percolated at a constant rate v_o. The resistance H is plotted on logarithmic scale against the time t on linear scale giving for most types of water indeed a straight-line relationship with as formula

$$\ln H = \ln H_o + \beta v_o t \qquad \text{or}$$

$$\frac{H}{H_o} = e^{\beta v_o t}$$

In this formula the initial resistance H_o depends on the construction of the fabric

$$H_o = \alpha v_o$$

and on the viscosity of the water, that is on the temperature

$$\frac{\alpha_1}{\alpha_2} = \frac{\nu_1}{\nu_2} \simeq \frac{0.7 + 0.03\ \tau_1}{0.7 + 0.03\ \tau_2}$$

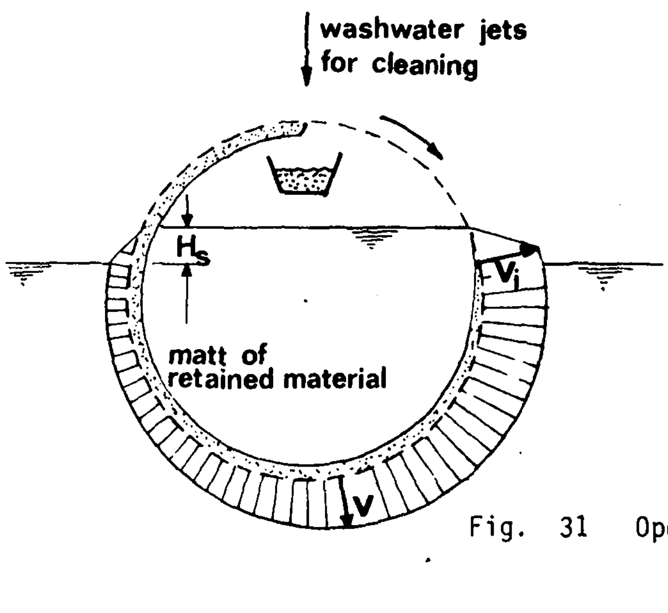
with τ as temperature in degrees centigrade. The value of β is a function of the clogging properties of the raw water in combination with the micro-mesh fabric to be used. It may show great seasonal fluctuations and for an adequate design, it is therefore essential to measure the value of β during at least a full year. Boucher's formula in the meanwhile does not correspond completely with the operating conditions of a micro-strainer. Here the straining medium passes into the water in a clean state, is submerged progressively over the whole of its working area as the drum rotates and emerges from the water in a partly blocked condition. With a constant head loss H_s, this means a decrease in straining rate with the time of submergence (fig. 31). Keeping in mind the laminar flow conditions and the proportionality between resistance and the total amount of water passed, Boucher's law may now be written as

$$\frac{v}{v_i} = e^{-\beta \int v dt} \quad \text{with } v_i = \frac{H_s}{\alpha} \text{ as initial rate of straining.}$$

Differentiating this formula to t gives

$$\frac{1}{v_i}\frac{dv}{dt} = -e^{-\beta \int v dt}\beta v = -\beta \frac{v^2}{v_i} \qquad \text{or}$$

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



fabric area A
wetted fabric area pA
time of revolution T

Fig. 31 Operating conditions of a micro-strainer

Integration between the limits $t = 0$, $v = v_i$ and $t = t$, $v = v$ yields

$$-\frac{1}{v} + \frac{1}{v_i} = -\beta t \quad \text{or } v = \frac{v_i}{1 + \beta v_i t}$$

With the notations of fig. 31, the average straining rate will equal

$$v_a = \frac{1}{pT}\int_o^{pT} v\,dt = \frac{v_i}{pT}\int_o^{pT} \frac{dt}{1 + \beta v_i t}$$

$$v_a = \frac{1}{\beta pT}\ln\,(1 + \beta v_i pT) \qquad \text{With}$$

$v_i = \frac{H_s}{\alpha}$ the capacity of the micro-strainer thus becomes

$$Q = v_a pA = \frac{A}{\beta T}\ln\,(1 + \beta\,\frac{H_s}{\alpha}\,pT)$$

in which the constants α and β have to be determined each time anew. In practice the straining rate v_a varies from values as low as $(2 - 3)10^{-3}$ m/sec with rather contaminated water such as a humus laden effluent from a sewage treatment plant, up to about $(25)10^{-3}$ m/sec with good quality water from impounding reservoirs. This straining rate can be raised in different ways

a. by selection of a coarser fabric with larger openings, decreasing the value of α. At the same time, however, the efficiency of the micro-straining process is lowered and more suspended matter will pass into the effluent;
b. by the abstraction of a less turbid water, lowering the value of β;
c. by shortening the time of revolution T, augmenting the circumferential speed from a low value of 0.05 or 0.1 m/sec to a high of 0.5 m/sec;
d. by increasing the allowable flow resistance H_s from normally applied values of 0.1 - 0.15 m to a maximum value of 0.25 m. In many cases, however, such an increase will result in a break-through of the fabric supported mat of retained material - indicated in the diagram of fig. 30 by a sudden fall-off from the straight-line relationship - again lowering effluent quality.

The straining capacity can also be increased by choosing a larger drum area. With a diameter D and width B, the gross area of the drum equals πDB and varies in practice from minimum $D = 0.75$ m, $B = 0.5$ m , $\pi DB = 1.2\ m^2$

maximum $D = 3$ m, $B = 3$ m , $\pi DB = 28\ m^2$

The net area of the fabric is about 10% smaller, while with 60 - 65% submergence the value of p is about 0.6. Best results are obtained when the size of the drum, respectively the number of drums in larger installations, is chosen such as to ensure that under normal circumstances satisfactory operation is obtained at low to average speeds and with a normal head loss. This leaves the possibility of increasing drum speeds and allowing a larger head loss for periods of deteriorating raw water quality.

5. Construction, operation and application of micro-strainers

As described in the preceding section, a micro-strainer is a revolving drum strainer - operating under open gravity conditions in a rectangular tank, usually made of reinforced concrete, occasionally of steel. Microstrainers were develloped by the firm of Glenfield and Kennedy Ltd, Hydraulic Engineers in Kilmarnock, Scotland. Their construction is shown schematically in fig. 32, where in principle two parts can be distinguished

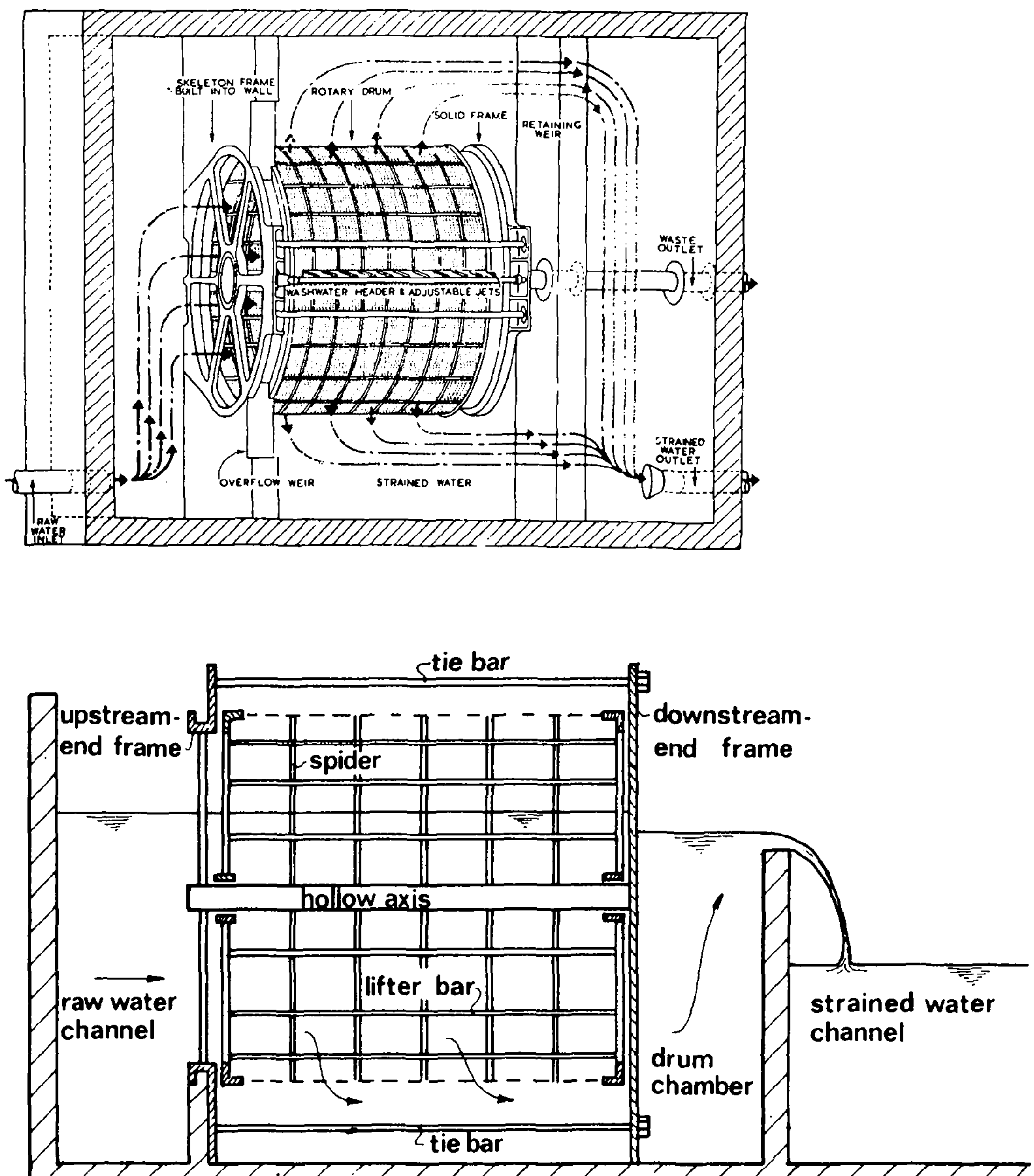


Fig. 32 Schematic construction of micro-strainers

a. a stationary part, consisting of the upstream and downstream endframes, interconnected by a hollow axle and 4 tie bars. The upstream endframe is fitted with a spoked wheel, providing large segmental openings and is built into the concrete or other division wall between the raw water channel and the drum chamber. The downstream endframe is blanked off by plating and the water pressure acting upon it is transmitted by the tie bars to the upstream endframe and the division wall;

b. a rotating part, consisting of two open endframes, supported on the hollow axle by ball and roller bearings and interconnected by lifter bars, stiffened with circumferential strips and spiders when necessary. In this way a cage is constructed (fig. 33) over which the micro-mesh fabric is stretched and fastened down by metal strips. The running clearence between the stationary and rotating parts is closed by adjustable felt-lined sealing bands, spring loaded to resist the differential head of water across the drum.

The actual work of the micro-strainer is done by the micro-mesh fabric, which must satisfy the contradictory requirements of providing on one hand fine openings, capable of matting and on the other hand an as large mechanical strength as possible. The original

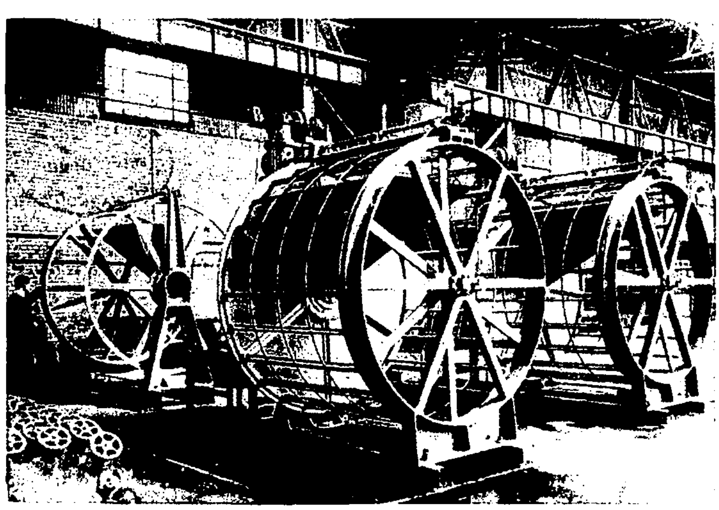


Fig. 33 Micro-strainer body.

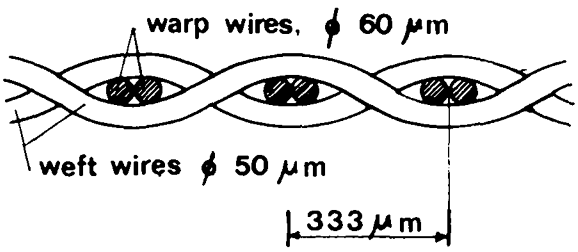
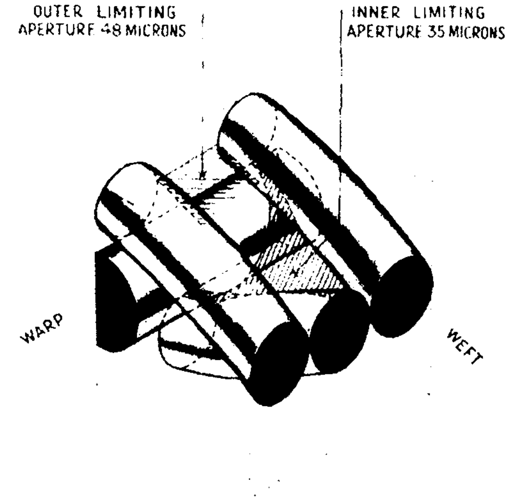


Fig. 34 Construction of Mark I micro-mesh fabric.



ISOMETRIC SKETCH OF MK: I

construction (mark I fabric) is shown in fig. 34 and consists of stainless steel wires with diameters of 50 and 60 μm, woven in special manner with 3 pairs of warp wires and 20 weft wires per mm, giving per mm^2 135 openings with an inner limiting aperture of 35 μm. Nowadays 3 types of fabric are available

mark	0	I	II
size of opening in μm	23	35	60
number of openings per mm^2	231	135	92
percentage open area	35.0	40.6	57.6

Notwithstanding the particular way of weaving, the micro-mesh fabric is still quite delicate. For additional mechanical support it is therefore sandwiched between square woven coarse mesh stainless steel wire cloth, pinned together by stainless steel screws, nuts and washers and fastened to the drum cage by longitudinal and circumferential straps (fig. 35). Stainless steel in the meanwhile has water repel-

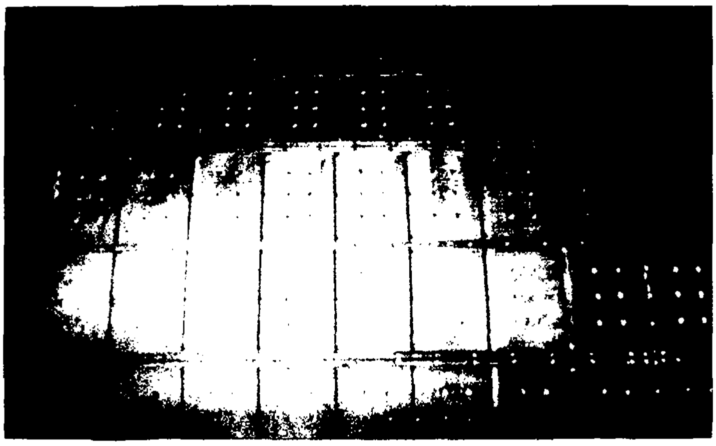


Fig. 35 View on micro-mesh fabric.

lent properties. To prevent entrainment of air by the backing fabric with the air blanking off large areas of the drum, seriously reducing the capacity, this fabric must be made from dull wires with openings of at least 2 - 2.5 mm or other materials such as monel metal must be used for its construction. In case clogging of the sandwich by filamentous algae is anticipated, the inner supporting fabric can be left out.

During operation the raw water enters the drum axially through the open frame at the upstream end, turns 90° and passes the micromesh fabric radially from inside to outside the drum. The strained water leaves the drum chamber over a fixed weir, the level of which is chosen such that the drum is submerged for about 60% of its diameter The drum is rotated by an electric motor, through worm-and-spur reduction gearing and speed-change gearbox, terminating in a pinion which meshes with a large spur-ring bolted to one side of the drum. Speeds measured on the drum periphery vary between 0.05 and 0.5 m/sec, higher as the diameter is larger. The number of revolutions is between 0.5 and 5 per minute, for which an electric input of 1 kW is always sufficient.

During operation the micro-strainer is cleaned continuously, using a row of wash-water jets over the full width of the drum (fig. 36). The wash-water nozzles are adjustable and self-cleaning and

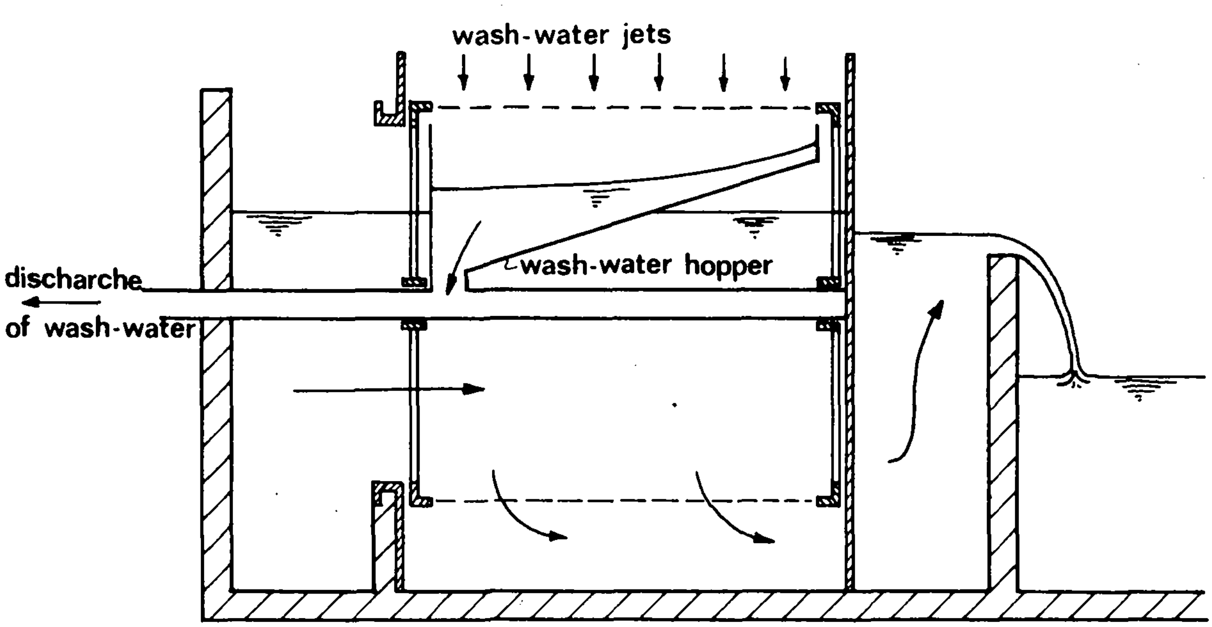


Fig. 36 Continuous hydraulic cleaning of micro-strainers.

designed for producing a thin continuous vane of water (fig. 37 left), which strikes vertically downward through the fabric, thus washing away the retained material from the inside of the drum into the waste water hopper and away through the hollow axle (fig. 36 and 37 right). Solids which do not adhere to the fabric are picked up by longitudinal bars inside the drum and are again deposited into the waste hopper. Preferably filtered and chlorinated water should be used for back-washing, with a jet pressure of 3 to 7 m watercolumn, depending on local circumstances and in an amount of 0.5 - 1.5% of the volume of water strained. This low-pressure back-washing in the meanwhile is only possible by the presence of the outer backing fabric

Fig. 37 Wash-water jets and waste-water hopper.

described above, which assures that the micro-mesh fabric below the wash-water nozzles is immersed in a nappe of retained water, in this way breaking down the surface tension which would otherwise deflect the wash-water jets sideways, preventing them from penetrating and cleaning the micro-mesh fabric. To keep the wash-water from splashing over the machine, the wash-water jets are covered by a plastic shield (fig. 38). In case the wash-water supply fails or the drum stops, a continuous clogging of the micro-mesh fabric will occur. Excess differential pressures which might damage the fabric can be avoided by providing the division wall between raw water inlet and the drum chamber with a weir (fig. 32), in this way limiting the upstream water level. Bacterial slimes are difficult to remove by back-washing alone and when they occur a blinding of the fabric and reduced flow capacity will result. Bacterial slimes can be destroyed by washing the strainer drum with a strong chlorine solution at intervals of one to a few weeks, but their occurence can also be prevented by equipping the micro-strainer with high-intensity ultra-violet lights (fig. 39).

From the description given above, it will finally be clear that corrosion is a serious danger to micro-strainers. The selection of suitable high-quality materials in conformity with the best water engineering practice is therefore of the utmost importance.

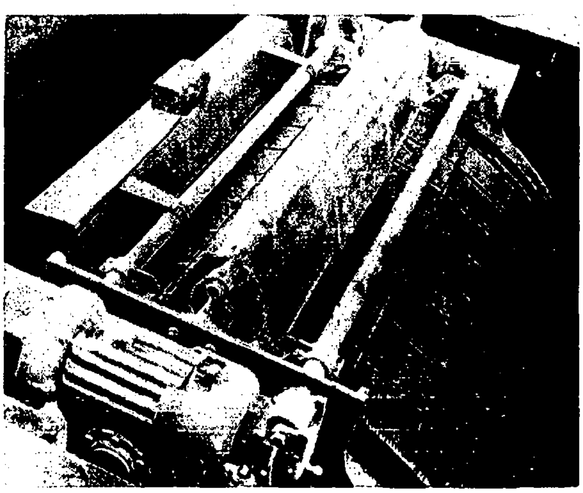


Fig. 38 Plastic shield covering wash-water jets to prevent splashing.

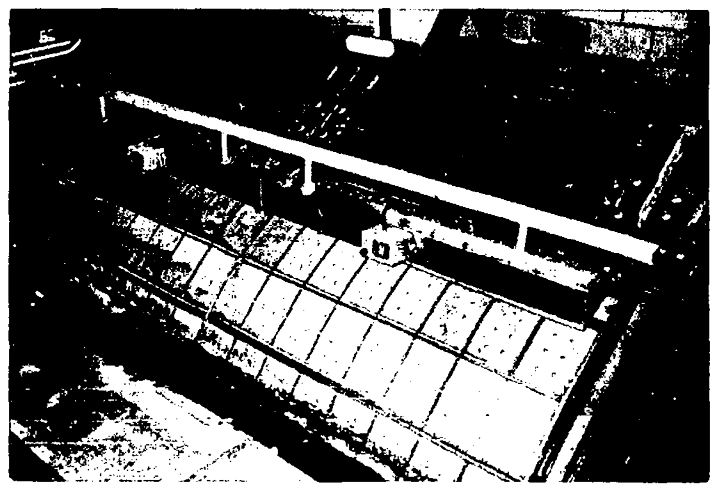


Fig. 39 Ultra-violet light radiation to control bacterial growth.

Micro-strainers were develloped thirty years ago as treatment preceding slow sand filters, to increase their capacity and/or length of filterrun. In this respect micro-strainers compete with rapid or primary filters, giving a solution much cheaper in cost of construction and operation, while also space requirements and head-losses are appreciably reduced. On the other hand, however, their action is entirely mechanical, removing particulate suspended matter such as sand, silt and algae, but there is no change in the chemical characteristics of the water. Micro-strainers are unable to remove colloïdal matter and color, ammonia and dissolved organic impurities, nor is there any marked decrease in the number of bacteria. With regard to the speed of clogging and the ensuing flow resistance, they are unfit for the treatment of heavily silted river waters.

In public water supply practice of to-day, micro-strainers are still used to lighten the load on subsequent slow filters and next to this on subsequent rapid filters, in particular when the raw water is heavily loaded by algae. These algae may also greatly disturb the process of coagulation, flocculation and sedimentation and again here preceding micro-strainers may be used to advantage. To remove the carry-over of coagulant flocs from settling tanks, micro-strainers are not well suited. When the flocstrength is large, the clogging of the fabric will be too rapid and when the floc strength is small, the efficiency too low, the coagulated matter flowing like water through even minute apertures in the fabric. In exceptional cases, the raw water is still so fair and unsullied that it can be distributed as drinking water without filtration. For safety's sake such waters must be sterilized, while micro-straining is now sufficient to keep back the little amount of debris and a sparse population of living organisms. For many industrial processes a first quality drinking water is not required and there micro-straining can be used even when the raw water contains color and colloïds. The same applies when micro-strainers are used to provide primary filtration prior to infiltration through natural sand formations in the process of artificial recharge.

Also in the field of sewage and industrial waste treatment, micro-straining may be of great value. With regard to the rapid clogging of the extremely fine openings, however, it may only be used as a primary treatment when the water to be handled is not more than lightly contaminated, as often is the case with industrial effluents. Here it may even be the sole process to which the water is subjected. With more contaminated trade wastes, micro-straining can be used for polishing the water, after a primary treatment by other means. The same holds true for the treatment of municipal sewage, where micro-strainers are used after the secondary settling tanks following trickling filters or the activated sludge process, to lower the suspended solids content and the bio-chemical oxygen demand of the effluent, in this way reducing the contamination of the receiving waters.

6. Principles of pre-coat filtration

As elaborated in the preceding sections, the efficiency of micro-straining is greatly enhanced by the building-up of a matt of retained material on the fabric. This also means, however, that especially with larger mesh-sizes and cleaner water, better results could be obtained by pre-coating the micro-fabric with inert powdered or fibrous material. In this way the actual work of clarification is done by the pre-coat, acting as a filter, while the fabric now only serves to retain and support the pre-coating materials. With moving strainers, this pre-coat would be washed away and lost after every revolution, increasing the cost of operation and interfering with the disposal of wash water. In some cases this is not objectionable, for instance not when asbestic or cellulose fibres are recovered from the wash-water and re-used, but in many other instances a more satisfactory solution is obtained with stationnary filters. These stationnary pre-coat filters operate in the same way as a conventional slow sand filter, but the filtering material is finer and the filtration rates are higher, both factors resulting in a much more rapid clogging. After reaching the maximum allowable head loss, the filter media into which is embedded the suspended matter removed during filtration is again discharged to waste and the supporting medium re-precoated to ready it for further work.

The rapid clogging of stationnary pre-coat filters could be retarted and the length of the filtering cycle greatly increased by adding during filtration a small continuous dose of filtering material (filter aid) to the raw water, in addition to the initial coating of the filter elements. With this body-feeding as it is called, the suspended solids in the raw water do not penetrate the pre-coat, reducing its porosity and permeability, but together with the filter aid they build up a cake of adequate porosity and permeability (fig. 40). The

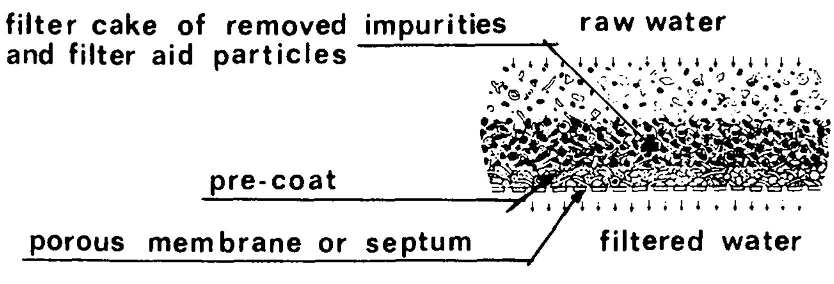


Fig. 40 Pre-coat filtration with body-feed.

increase in resistance now results from a growing cake thickness with time, again necessitating a periodic cleaning, but the lengths of filterrun are much longer, allowing the use of a more turbid raw water. The construction of a pre-coat filter with body-feed is shown schematically in fig. 41. At the beginning of the filterrun, the pre-coat is formed on a porous membrane or septum, supported in one way or another, while during filtration the cake of filter-aid particles and retained solids is built up. After reaching the maximum allowable thickness and resistance, both pre-coat as filter cake are washed away and the process begins anew.

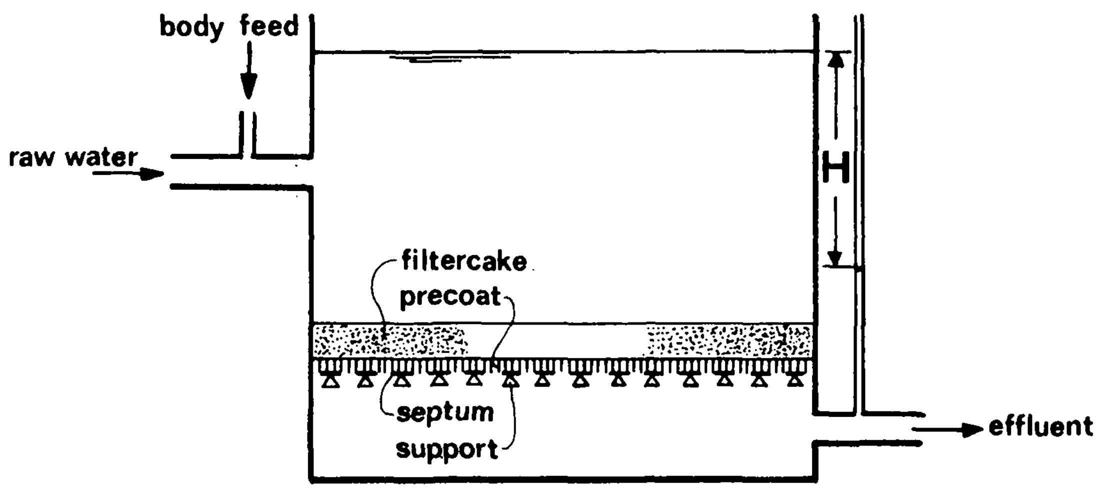


Fig. 41 Schematic diagram of pre-coat filter with body feed.

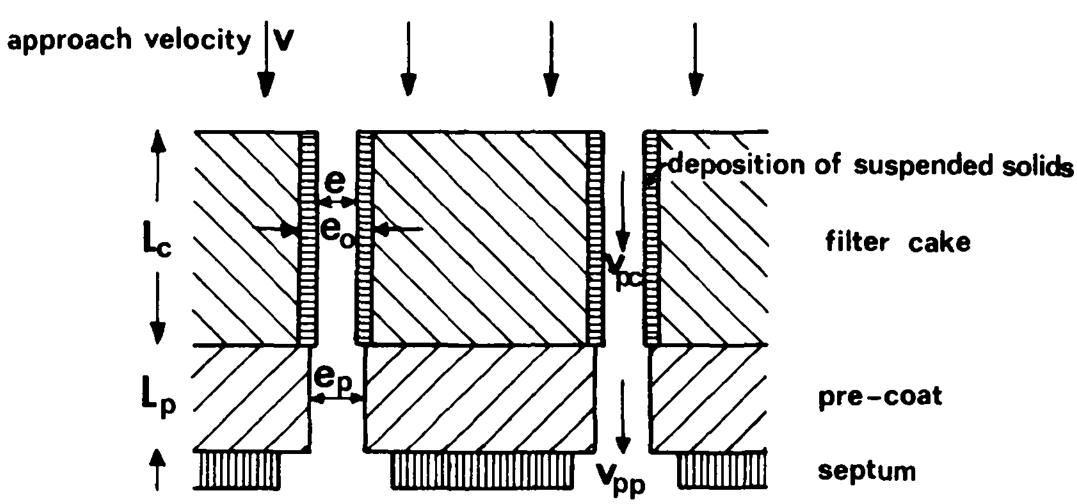


Fig. 42 Model for clogging of a pre-coat filter with body feed.

To derive a mathematical formulation of the filtration process described above, the model of fig. 42 will be considered. Three parts may be distinguished here, the septum, the pre-coat and the filter cake, each with its own resistance against the passage of water. For the septum this resistance is due to losses of friction and turbulence. The latter losses are proportional to the square of the velocity, but with the low rates and small openings commonly applied the friction losses follow the linear resistance law of laminar flow. Together

$$H_s = av + bv^2$$

with a and b depending on the construction of the septum. Next to this, the factor a will vary with the viscosity, that is with the temperature of the water. With H_s small, usually not more than 0.05 m watercolumn (including the resistance of the septum support), this variation may safely be left out of consideration.

The pre-coat is built up of filter-aid material to which sometimes other substances as for instance asbestos fibres are added to speed up its formation. In fig. 42 it is assumed that this coat has n openings of length l_p and diameter e_p per area A and that no clogging occurs. Poiseuille's law for laminar flow in capillary tubes now gives as resistance

$$H_p = \frac{32\nu}{g} l_p \frac{v_{pp}}{e_p^2}$$

with ν as kinematic viscosity of the water, g as gravity constant and v_{pp} as actual velocity of the water inside the pores. With v as approach velocity (capacity per unit area)

$$vA = v_{pp}\, n \frac{\pi}{4} e_p^2 \qquad \text{or}$$

$$H_p = \frac{128\nu}{\pi g} \frac{A}{n} l_p \frac{v}{e_p^4} = \frac{v\, l_p}{k_p}$$

with k_p as coefficient of permeability depending on the structure of the pre-coat, that is on the material applied, and on the viscosity of the water.

In case the filter cake is built up with clear water and no clogging occurs, its resistance is similar to that of the pre-coat

$$H_c = \frac{v\, l_c}{k_c}$$

The thickness l_c, however, is not constant but increases as filtration goes on. With c_a as gravimetric concentration of the filter aid material added continuously to the incoming water and $\gamma_a c_a$ as its volumetric concentration when building up the filter cake, fig. 42 gives as account for this material

$$v\gamma_a c_a tA = l_c (A - n \tfrac{\pi}{4} e_o^2) \qquad \text{or}$$

$$l_c = v\gamma_a c_a t \frac{A}{A - n \frac{\pi}{4} e_o^2} = v\gamma_a c_a t \frac{1 + \alpha}{\alpha}$$

with α as a positive number. Substituted

$$H_c = \frac{1 + \alpha}{\alpha} \; \frac{v^2 t \;\gamma_a c_a}{k_c}$$

When filtering turbid water, clogging of the filter cake will occur, reducing the diameter of its pores from e_o to e and increasing the resistance to

$$H'_c = H_c \left(\frac{e_o}{e}\right)^4$$

With c_s as gravimetric and $\gamma_s c_s$ as volumetric concentration of suspended matter retained by the filter cake

$$v\gamma_s c_s \; t\, A = l_c \; n \tfrac{\pi}{4} (e_o^2 - e^2)$$

from which follows

$$\frac{e^2}{e_o^2} = 1 - \frac{v \;\gamma_s c_s \;tA}{l_c \; n \frac{\pi}{4} e_o^2}$$

or with the value of l_c calculated above

$$\frac{e^2}{e_o^2} = 1 - \frac{A - n\frac{\pi}{4}e_o^2}{n\frac{\pi}{4}e_o^2}\,\frac{\gamma_s\, c_s}{\gamma_a\, c_a} = 1 - \alpha\,\frac{\gamma_s\, c_s}{\gamma_a\, c_a}$$

Substituted

$$H_c' = \frac{H_c}{\left(1 - \alpha\,\frac{\gamma_s\, c_s}{\gamma_a\, c_a}\right)^2}$$

and with the value of H_c

$$H_c' = \frac{1+\alpha}{\alpha}\,\frac{v^2 t}{k_c}\,\frac{\gamma_a\, c_a}{\left(1 - \alpha\,\frac{\gamma_s\, c_s}{\gamma_a\, c_a}\right)^2}$$

For a better understanding of this formula, the ratio

$$n = \frac{\gamma_a\, c_a}{\gamma_s\, c_s}$$

may be introduced. Substituted

$$H_c' = \frac{1+\alpha}{\alpha}\,\frac{v^2 t}{k_c}\,\gamma_s c_s\,\frac{n^3}{(n-\alpha)^2}$$

That is to say a resistance which increases linearly with time and which is proportional to the first power of the suspended solids content c_s and to the second power of the filtration rate v. The influence of the ratio n between body feed and suspended solids content is more difficult to evaluate. Differentiation of H_c' to n yields

$$\frac{\delta H_c'}{\delta n} = \frac{1+\alpha}{\alpha}\,\frac{v^2 t}{k_c}\,\gamma_s c_s\,\left\{\frac{3n^2}{(n-\alpha)^2} - \frac{2n^3}{(n-\alpha)^3}\right\}$$

$$\frac{\delta H_c'}{\delta n} = H\,\frac{n - 3\alpha}{n(n-\alpha)}$$

According to this formula , an increasing body feed will lower the resistance as long as

$$\alpha < n < 3\alpha$$

which operating conditions should therefore be preferred when possible.

With the many unknown factors in the meanwhile, the calculation given above is of theoretical value only. In practice another approach is commonly used, writing the flow resistances as

$$H_p = \frac{C_a v}{K_p}, \qquad H'_c = \frac{c_a v^2 t}{K_c}$$

with C_a as the mass of the filter aid in the pre-coat in grams/m^2, c_a as the body feed rate in grams/m^3 and K_p and K_c in gram/m^2/sec as constants to be determined by experiment. The constant K_p only depends on the pre-coat material, having for instance values of 15 - 50 gram/m^2/sec for diatomaceous earth , depending on the grade applied. Next to this, the value of K_c varies with the amount and nature of the suspended solids to be removed. With diatomite filtration of aerated, iron containing water, this relation will roughly equal

$$K_c = (0.001 - 0.003)\left(\frac{c_a}{c_s}\right)^{1.8-2.2}$$

with the constants depending on the grade of diatomaceous earth to be applied. In this case the resistance of the filter cake equals

$$H'_c = (300 - 1000)\,\frac{c_s^{\,1.8-2.2}}{c_a^{\,0.8-1.2}}\,v^2 t$$

that is to say a smaller resistance as the body feed rate is higher!

7. Construction, operation and application of diatomite filters

In pre-coat filtration, the essential part is the filter element, consisting of a supporting member over which the septum is stretched. The actual work of clarification, however, is carried out by deposits of filtering material on this septum, sub-divided in the pre-coat formed prior to filtration and the filter cake build up during filtration (fig. 41). As filtration goes on, the thickness and flow resistance of the filter cake increase continuously untill the maximum allowable head loss is reached. Pre-coat and filter cake with the retained solids from the raw water are then sloughed off and the septum cleaned by back-washing.

Pre-coat filtration may be carried out with various materials, such as Perlite, cellulose powders, cellulose and asbestos fibres, etc, but in the field of public water supplies diatomaceous earth is almost used exclusively.
This natural material consists of the silicious remains of diatoms (fig. 43) and can be found in many places all over the world. After

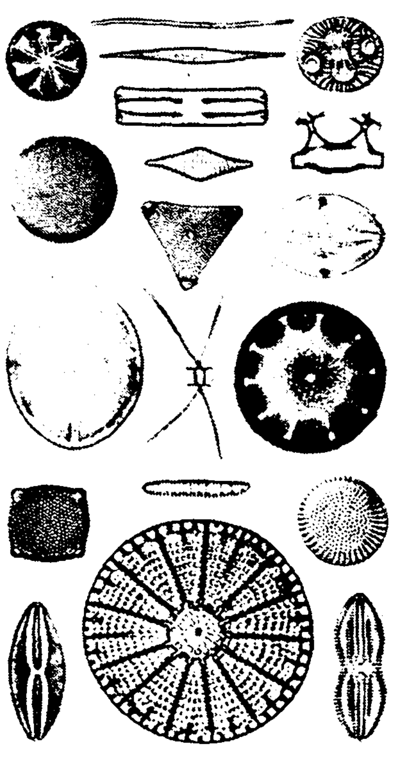


Fig. 43 Silicious skeletons in diatomeceous earth.

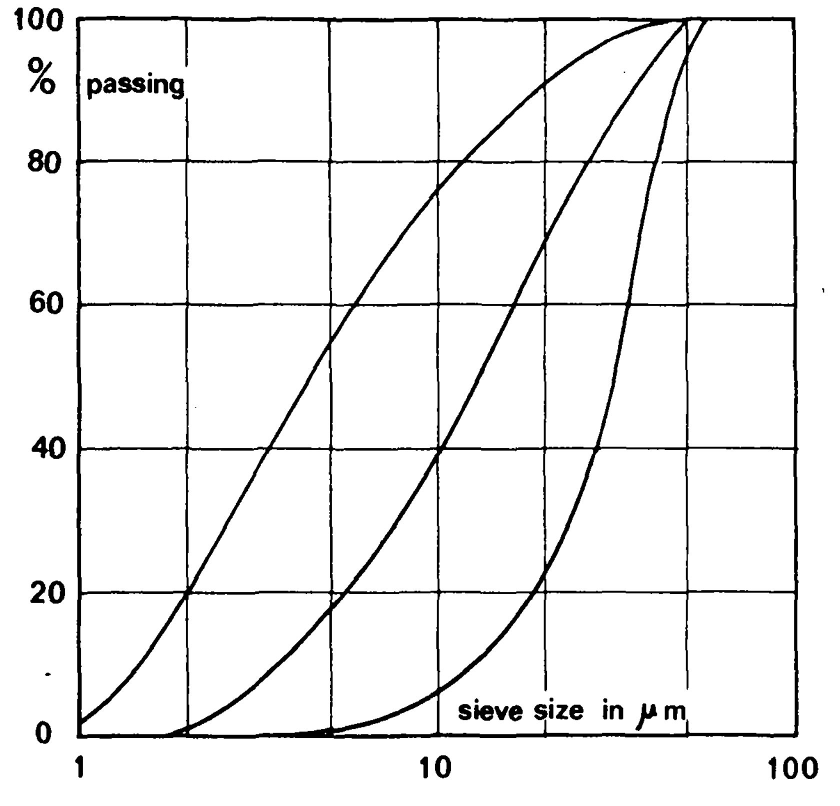


Fig. 44 Diatomite grades.

excavation it is dried, grinded, sometimes calcined and then separated by screening into the various diatomite grades commercially available (fig. 44). Which grade must be used in a particular case, depends on local circumstances, on the amount and nature of the suspended impurities to be removed and on the desired clarity of the effluent. With a fine grade an exceptional clear water can be obtained, but with coarser grains the length of filterrun will be larger, also allowing the use of a more turbid raw water. Due to its composition of about 90% SiO_2, 4% Al_2O_3 and other oxydes for the remaining part, diatomite is chemically inert. The true mass density amounts to 2600 kg/m^3, but with a pore space of for instance 93%, the apparent mass density is only 180 kg/m^3.

The septum for supporting the diatomite can be made in different ways, from finaly woven wire cloth or synthetic fabrics, from closely spaced wire wrappings and from porous materials such as sintered metal or aluminium oxyde grains. Under all circumstances it must be resistant against corrosion and not liable to fouling, requirements which are best fullfilled by stainless steel fabrics of parallel wires with long narrow openings and a width of 125 μm or less. These openings are still so large that in many cases all the diatomite is able to pass, requiring bridging (fig. 45) to build up the pre-coat. This

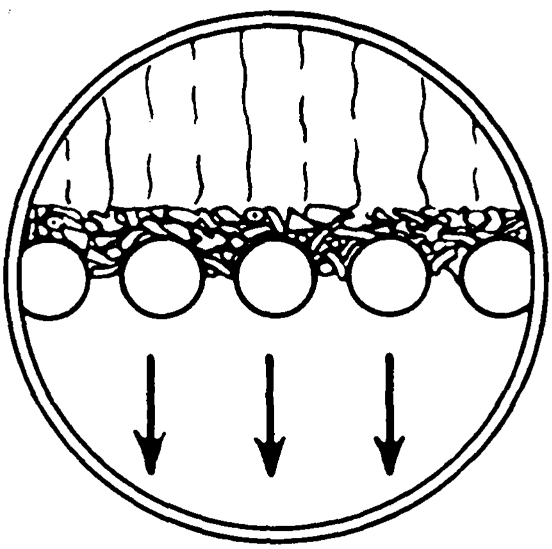


Fig. 45 Bridging to built up the pre-coat.

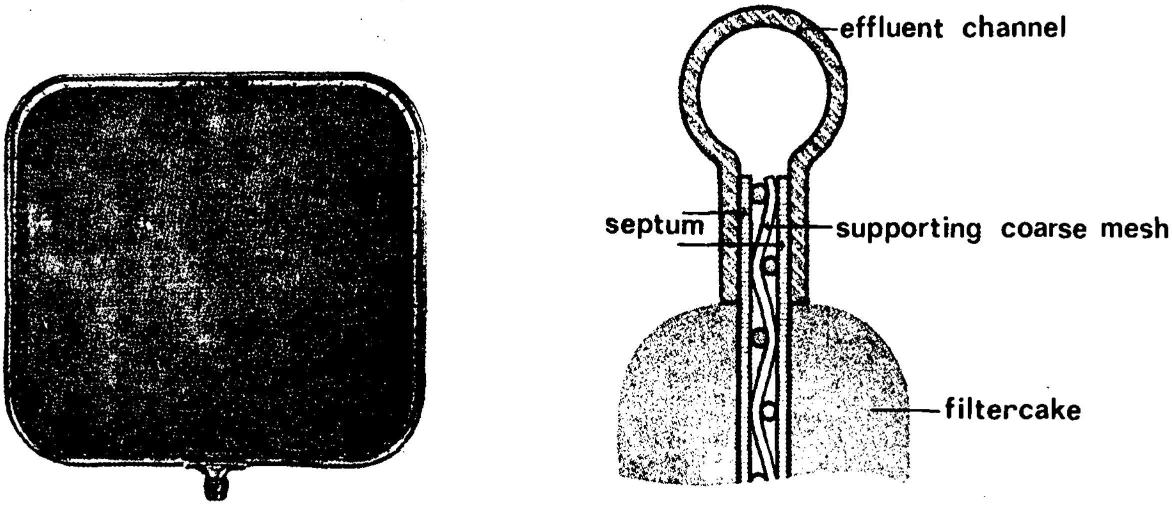


Fig. 46 Filtering leaf.

process takes much time, but it can be accelerated by the addition of coarser diatomite grains or asbestos fibres to the pre-coat material. Commonly the pre-coat contains 500-700 grams of diatomite per m^2 of septum, giving a thickness of about 3 mm, a coefficient of permeability roughly equal to $(0.1)10^{-3}$ m/sec and with a normal filter rate of $(1)10^{-3}$ m/sec, a flow resistance of a few cm's water column only.

The member supporting the septum must be rigid and sufficiently strong to prevent disturbing the filter cake when during filtration its weight and flow resistance increases. The actual construction of the supporting member depends on the shape of the filter elements to be applied. Leaves as shown in fig. 46 are rarely used in public water supply practice and here the cylinders of fig. 47 are more

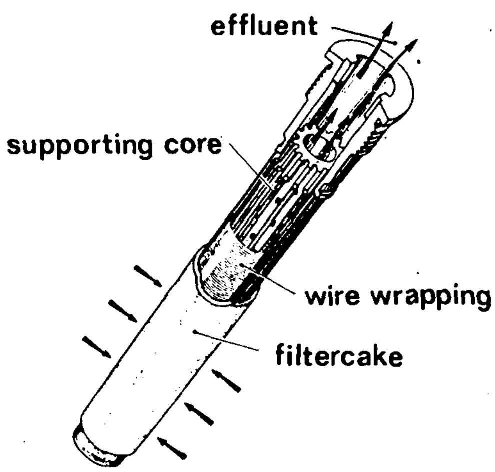


Fig. 47 Filtering candle.

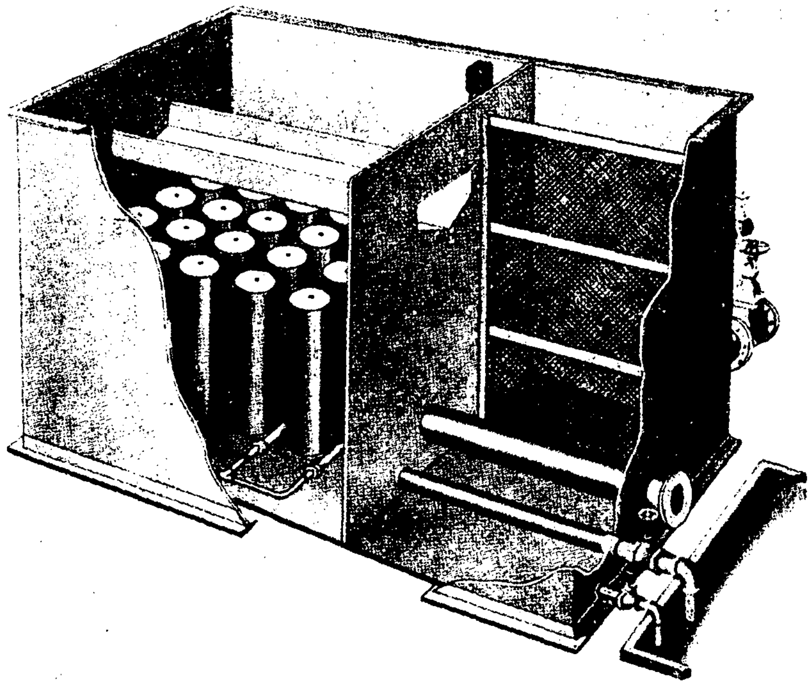
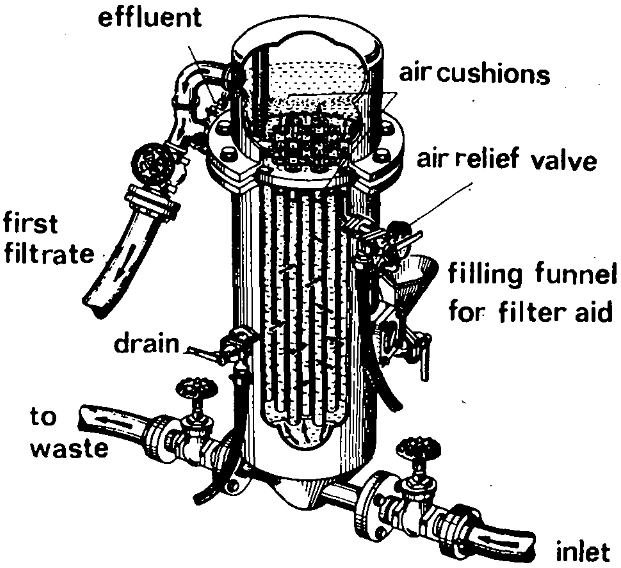


Fig. 48 Gravity filters operated by suction.

popular. Essentially they consist of a hollow, fluted and perforated core, often made of ceramics, wound with stainless steel wire as septum, providing continuous openings of any desired width. Next to the requirements of structural strength and corrosion resistance, the core must have good drainage characteristics, keeping the hydraulic resistance including the septum below 0.05 m.

With gravity filtration, the filtering elements are contained in a box, made of concrete or steel. With regard to the high losses of head accompanying continued filtration, such open filters are often operated by suction. If this is indeed necessary, a better solution can be obtained by accommodating the filtering elements in a watertight steel shell and forcing the water through under pressure (fig. 49). With regard to the built-up of the filter cake to a thickness of 10-40 mm and to provide easy access for the raw water, without excessive velocities of flow which might endanger the cake by erosion, the filtering elements must be set at adequate distances, with openings in-between of at least 50-100 mm. This still allows a large filtering area in a limited volume, giving even with small vessels and low rates of flow a high capacity and an enormous saving in weight and cost compared to ordinary rapid filters. The installation in the meanwhile



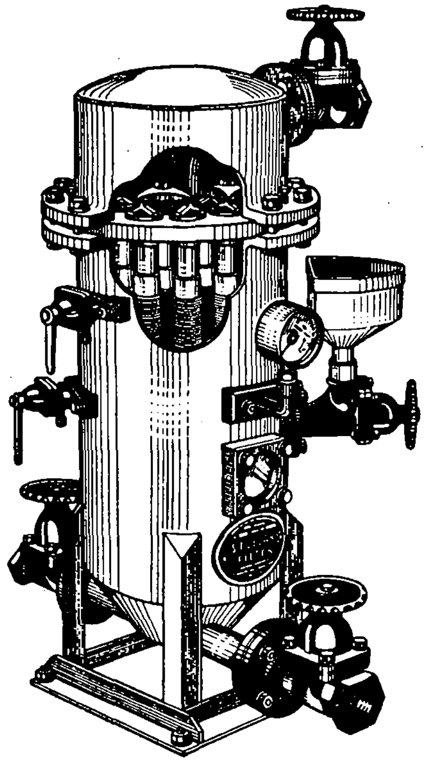


Fig. 49 Pressure filters.

is not complete with the diatomite filter alone, various accessories are still needed, in particular those to provide the body feed for building up the filter cake and pre-coat. Schematically the full installation is shown in fig. 50, while fig. 51 presents the filter aid feeder system in greater detail.

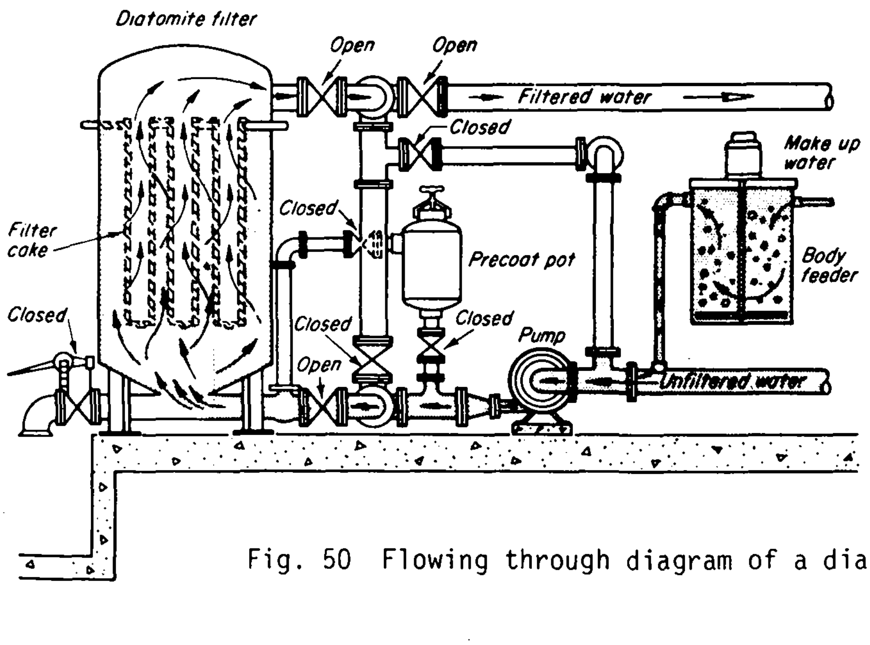


Fig. 50 Flowing through diagram of a diatomite filter.

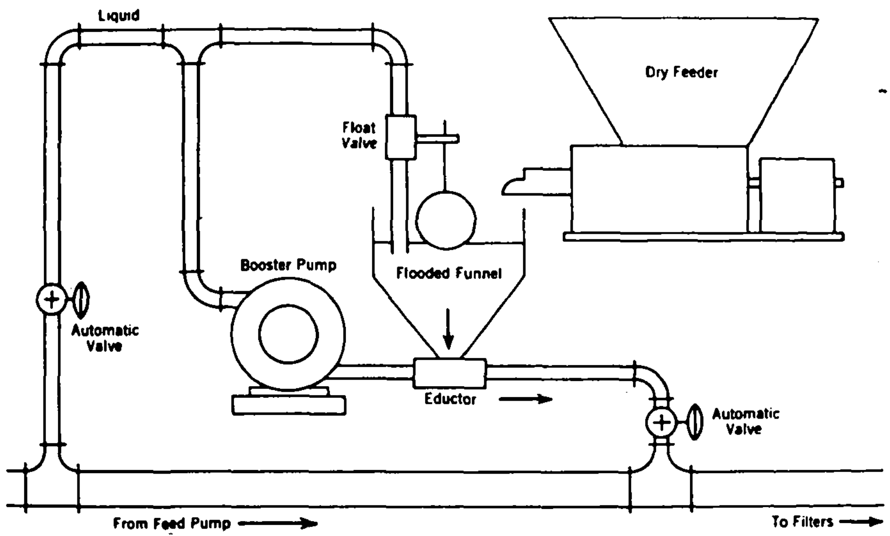


Fig. 51 Diagram of a filter aid feeder system.

Operation of a diatomite filter starts by applying the pre-coat. In a pressure vessel with detachable lid, the amount of diatomite necessary to prevent fouling of the septum,about 500-700 grams per m^2 is inserted and mixed with water to a slurry. After closing the lid and adjusting the various valves,filtered water from the pressure pump (fig. 50) flushes this slurry to the filter. In first instance a major part of the diatomite will pass the septum openings, leaving the filter through the discharge valve and flowing back to the suction side of the pump and from there again to the filter. After some time of re-circulation, however, the pre-coat will have been formed to the required extend, that is to say able to produce a clear effluent. The various valves are now reset, allowing raw water to be pumped directly to the filter and from there to the filtered water conduit. During this process of filtration, additional diatomite in amounts of 10-20, in exceptional cases 50-100 grams/m^3 is added as a slurry to the raw water, preventing a clogging of the pre-coat and together with the retained suspended solids building up a rigid, porous but retentive filter cake of ever increasing thickness and flow resistance. After reaching the maximum allowable head loss the filtration process is stopped, pre-coat and filter cake drop to the bottom of the filter shell and the septa with supporting members are cleaned by back-washing and by spraying with powerful water jets. In drinking water practice the spent material is usually discharged to waste, but in other applications it is re-used, up till 5 or 10 times, giving an appreciable saving in the cost of operation. For the purification of of extreme clarity, but it also means a nearly 100% removal of coli-bacteria, having dimensions of 1-5 μm, and a water safe in bacteriological respect. Moreover, these results are obtained immediately after filtration starts, without the need of a breaking-in period, while the regular disposal of the filtering materials prevents a persistant clogging of the filterbed. Indeed, chemical and biological actions are absent, but taste and odor producing substances may still be removed by adding powdered activated carbon to the body feed. Colloïdal particles with sizes of 0.001-1 μm are only partly removed by diatomite filtration. If this results in a too high color of the effluent, improvement may be obtained by providing the diatomite particles with an electro-positive coating of aluminium or ferric hydroxyde.

Diatomaceous earth filtration has already been applied for many decades in the food industry, for the clarification of sugar and fruit juices, wine, vinegar, beer, etc. For similar purposes it is used in the chemical, petro-chemical and pharmaceutical industry while an interesting application concerns the de-oiling of high pressure boiler feed water. In the field of drinking water supply, diatomite filtration was first used during the second world war to provide the American soldiers with a reliable water, in particular free from Entamoeba histolytica cysts which cannot be inactivated by chlorination. There they also offered the adventage of a large capacity in a small volume, especially when higher filtration rates of $(2 - 5)10^{-3}$ m/sec are applied. Today a small number of diatomite filters can be found in municipal plants. From the foregoing description it will be clear that in technical respect they are excellently suited for the sole treatment of aerated, iron and manganese containing groundwaters or rather clear surface waters and for the final treatment of more turbid water after chemical coagulation or lime-soda ash softening. Although not strictly neccessary, the effluent of diatomite filters treating surface water is often subjected to safety chlorination. With the extreme clarity of the effluent, the dose required is commonly low, 0.1-0.3 ppm. With regard to the cost of filtration, a subdivision must be made between those of construction and operation. Diatomite filters are low in first cost, about half that of a rapid filtration plant, but drinking water, normal filtration rates (capacity per unit area of septum) are (0.7 - 2)10^{-3} m/sec. The resistance increases linearly with the amount vt of water filtered and for the same rate will be lower as the body feed rate is higher (fig. 52). With pressure filters maximum allowable head losses vary between 5 and 15 m watercolumn, giving lengths of filterrun of 10-50 hours with ultimate cake thicknesses of 10-40 mm. Notwithstanding all precautions, including air-wash, fouling of the septa is difficult to prevent and a reduction of the effective open area by 5% after 100 cycles of operation is commonly allowed. Sometimes better results can be obtained with expendable septa, but still they must be constructed and operated in such a way, that renewal within one year of continuous operation is not required.

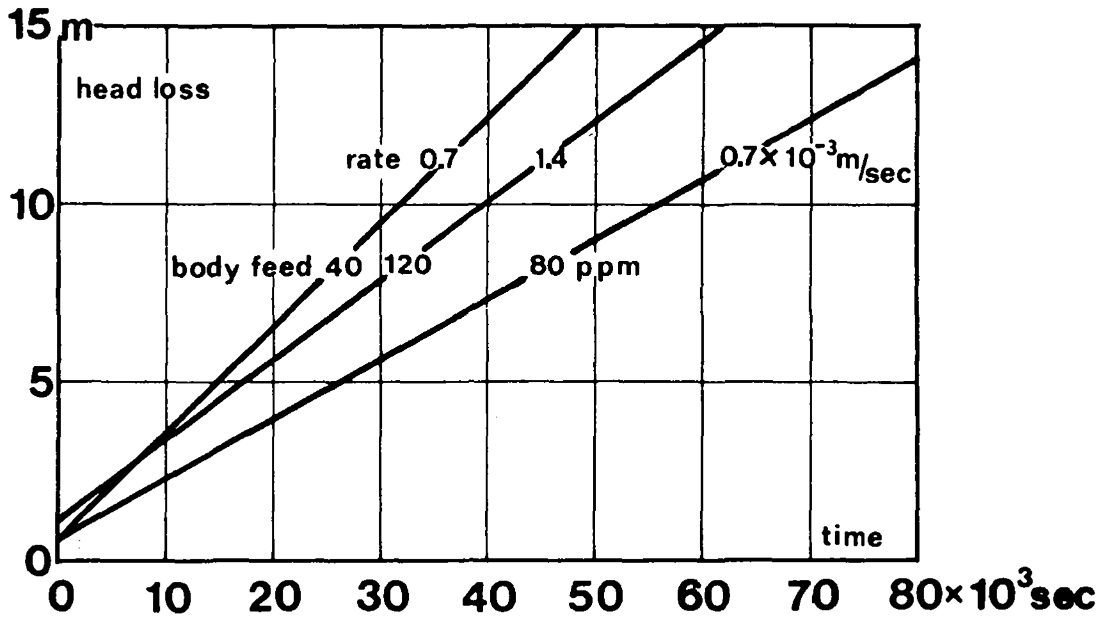


Fig. 52 Diatomite filtration of groundwater containing 7-8 ppm Fe.

Diatomite filtration is a purely physical process, in principle only removing particulate matter from the raw water. With the small grain sizes(fig. 44) and keeping in mind the ratio between pore and grain diameters (fig. 26), they are very effective in this respect, removing suspended particles with sizes larger than 0.1-1 µm, depending on the diatomite grade applied. This not only results in an effluent the operating costs are much higher. All-together, a diatomite filtration plant is more expensive to run than a plant containing rapid filters, limiting their use to small capacity supplies, for instance less than 0.05-0.1 m^3/sec, to emergency and stand-by services and to periods of peak consumption. As mentioned before, diatomite filters are excellently suited for those part-time duties, the effluent satisfying all quality requirements as soon as the pre-coat is formed. They can also be used to advantage where space is at a premium. In the field of water treatment, however, their greatest application is still the purification of water for swimmingpools, giving a visibility much better than that of ordinary good quality drinking water. With higher filtration rates, (1.5-2)10^{-3} m/sec and re-using the filter aid 5-10 times, the costs are appreciably reduced, while the saving in weight and space is here often of great importance.

In diatomite filtration only a thin layer of pre-coat and filter cake, with an initial thickness of not more than 2 mm, separates the filtered from the raw water. This layer moreover is easily damaged, by a short interruption of pumping for instance and many water engineers therefore hesitate to apply this process for public supplies.

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