15 Sedimentation

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15.1 Introduction

Sedimentation is the settling and removal of sus-pended particles that takes place when water stands still in, or flows slowly through a basin. Due to the low velocity of flow, turbulence will generally be absent or negligible, and particles having a mass density (specific weight) higher than that of the water will be allowed to settle. These particles will ultimately be deposited on the bottom of the tank forming a sludge layer. The water reaching the tank outlet will be in a clarified condition.
Sedimentation takes place in any basin. Storage basins, through which the water flows very slowly, are particularly effective but not always available. In water treatment plants, settling tanks specially designed for sedimentation are widely used. The most common design provides for the water flowing horizontally through the tank but there are also designs for vertical[1] or radial flow. For small water treatment plants, horizontal-flow, rectangular tanks generally are both simple to construct and adequate.
The efficiency of the settling process will be much reduced if there is turbulence or cross-circulation in the tank. To avoid this, the raw water should enter the settling tank through a separate inlet structure. Here the water must be divided evenly over the full width and depth of the tank. Similarly, at the end of the tank an outlet structure is required to collect the clarified water evenly. The settled-out material will form a sludge layer on the bottom of the tank. Settling tanks need to be cleaned out regularly. The sludge can be drained off or removed in another way. For manual cleaning (e.g. scraping), the tank must first be drained.

15.2 Settling tank design

The efficiency of a settling tank in the removal of suspended particles can be determined using as a basis the settling velocity (s) of a particle that in the detention time (T) will just traverse the full depth (H) of the tank. Using these notations (see fig. 15.1), the following equations are applicable:

$$s_0 = \frac{H}{T}; \quad T = \frac{BLH}{Q}, \text{ so that } s_0 = \frac{Q}{BL} \; (m^3/m^2.h = m/h)$$

s_0 = settling velocity (m/h)
T = detention time (h)
Q = flow rate (m^3/h)
H = depth of tank (m)
B = width of tank (m)
L = length (m)

1 The operational requirements of vertical-flow floc blanket-type settling (formerly known as sludge blanket) tanks are so strict that they are generally not suitable for small water treatment plants.

Assuming an even distribution of all suspended particles in the water over the full depth of the tank (by way of an ideal inlet structure), particles having a settling velocity (s) higher than s_0 will be completely removed. And particles that settle slower than s_0 will be removed for a proportional part, $s:s_0$.
This analysis shows that the settling efficiency basically only depends on the ratio between the influent flow rate and the surface area of the tank. This is called the *surface loading*. It is independent of the depth of the tank. In principle, there is no difference in settling efficiency between a shallow and a deep tank.

The settling efficiency of a tank may therefore be greatly improved by the installation of an extra bottom as indicated in figure 15.1. The effective surface area would be greatly increased and the surface loading would be much lower.

The design of a settling tank should properly be based on an analysis of the settling velocities of the settleable particles in the raw water.

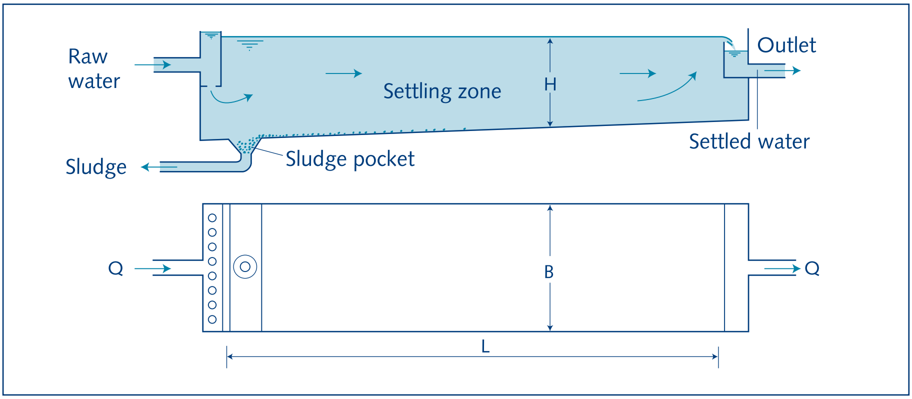

Fig. 15.1. Settling tank with extra bottom

Where sedimentation is used without pre-treatment, (this is called *plain sedimentation*) for the clari-fication of river water, the surface loading generally should be in the range from 0.1-1 m/hour. For settling tanks receiving water that has been treated by chemical coagulation and flocculation, a higher loading is possible, somewhere between 1 and 3 m/hour. In both cases, the lower the surface loading the better the clarification of the water: the settled water will have less turbidity.

The above considerations ignore the effects of tur-bulence, short-circuiting and bottom scour caused by inlet and outlet design and wind. Weirs and orifices should be arranged to reduce jetting at the inlet, and upward flow at the outlet. Windbreaks of about 2 m high can reduce wind disturbance. To keep these effects to a minimum, the tank should not be too shallow, at least 2 m deep or more, and the ratio between length and width should be between 3 and 8.

The horizontal velocity of flow, computed as $v = Q/BH$, will then be in the 4-36 m/hour range. A tank 2 m deep or more could accommodate mechanical sludge re-moval equipment but small installations are better cleaned manually. This is done at intervals varying from one to several weeks. The depth of the tank should be adequate to accommodate the sludge accumulating at the bottom between the cleanings.

For a further elaboration of settling tank design, take the example of a town with a future population of 10,000 inhabitants, requiring an average of 40 litres/day per person. Assuming a maximum daily de-mand of 1.2 times the average demand, the design capacity should be:

$$Q = 10{,}000 \times \frac{4}{1000} \times 1.2 = 480 \text{ m}^3/\text{day} = 20 \text{ m}^3/\text{hour}$$

If the raw water source is turbid river water, it may be subjected to plain sedimentation as a first treat-ment. Two settling tanks should be built. If the second tank serves only as a reserve for when the first tank is out of operation, then each of the two settling tanks has to be designed to take the full design flow ($Q = 20$ m^3/hour). An alternative is to provide two tanks of 10 m^3/hour capacity each. This would give a saving in construction costs. With one of these tanks out of operation for cleaning, the other tank has to be overloaded for the duration of the cleaning operation. In many situations this can be quite acceptable. Whether this approach may be followed must be determined in each individual case.

If experience with other installations using the same water source indicates that a surface loading of 0.5 m/hour gives satisfactory results, the sizing of the tank for a design capacity of 20 m^3/hour would be as follows:

$$\frac{Q}{BL} = \frac{20}{BL} = 0.5, \text{ so that } BL = 40 \text{ m}^2$$

The tank dimensions could be, for instance: $B = 3$ m, $L = 14$ m.

With a depth of 2 m the tank would have about 0.5 m available for filling up with sludge deposits before cleaning is needed. The horizontal flow velocity would be:

$$v = \frac{Q}{BH} = \frac{20}{(3).(1.5)} = 4.44 \text{ m/hour}$$

This is well within the design limits quoted above. Assuming that during periods of high turbidity the river water contains a suspended load of 120 mg/l which is to be reduced to 10 mg/l by sedimentation, then 110 grams of silt will be retained from every cubic metre of water clarified. With a surface loading of 0.5 m/hour this means an average sludge accumulation of 55 gram/m^2 per hour; that is for sludge having a dry matter content of 3%, an amount of 55: 0.03 = 1830 cm^3/m^2 per hour = 1.83 mm/hour. At the inlet end of the tank the deposits will accumulate faster, probably about 4 mm/hour, so that for an allowable accumulation of 0.5 m an interval of 125 hours or 5 days between cleanings is to be expected. When the periods of high turbidity are infrequent and of short duration, this is certainly acceptable.

15.3 Construction

Settling tanks with vertical walls are normally built of masonry or concrete; dug settling basins mostly have sloping banks of compacted ground with a protective lining, if necessary.

Medium-and large-sized settling tanks generally have a rectangular plan and cross-section. To facilitate sludge removal it is convenient to have the tank bottom slope lightly towards the inlet end of the tank where the sludge pocket is situated.

As described in section 15.1, a settling tank should have a separate inlet arrangement ensuring an even distribution of the water over the full width and depth of the tank. Many designs can be used; figure 15.2 shows a few examples. The arrangement shown to the left consists of a channel over the full width of the tank with a large number of small openings in the bottom through which the water enters the settling zone.
For a uniform distribution of the influent these openings should be spaced close to each other, less than 0.5 m apart, and their diameter should not be too small (e.g. 3-5 cm) otherwise they may clog up. The channel should be generously sized, with a cross-sectional area of at least twice the combined area of the openings. A settling tank as elaborated in the example given earlier, with a capacity of 20 m^3/h and a width of 3 m, would have in the inlet channel about 6 holes of 4 cm diameter. The inlet channel itself would be about 0.4 m deep and 0.3 m wide.

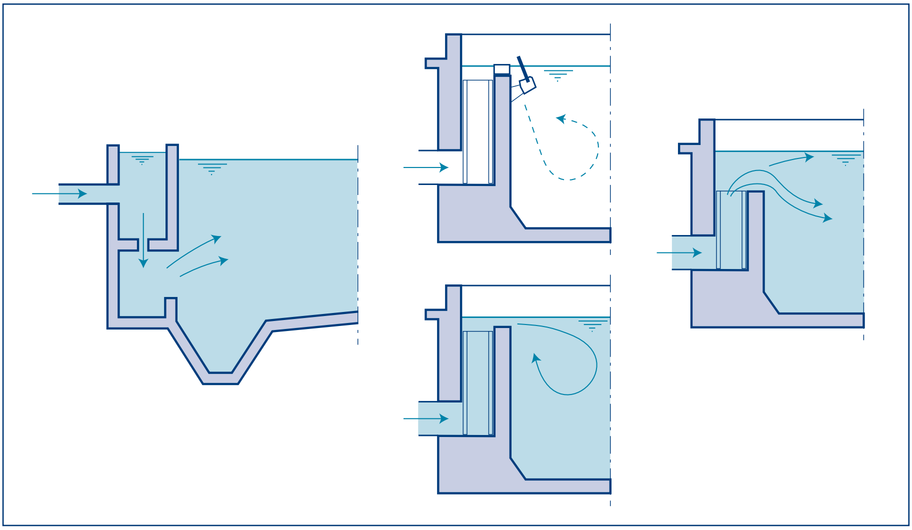

Fig. 15.2. Inlet arrangements

Frequently, the effluent water leaves the tank over weirs. Sometimes one weir is adequate but to prevent the settled material from being picked up again, the draw-off of the water should always be gentle and more weirs may have to be provided (combined length nB). The following formula can be used for computing the total weir length required:

$$nB = \frac{Q}{5\ Hs_0}$$

In the example of the preceding section:

$$(n)(3) = \frac{20}{(5)(1.5)(0.5)} \text{ or } n = 2$$

Outlet arrangements using one and more overflow weirs are shown in figure 15.3.

When low weir overflow rates are used, the precise horizontal positioning of the weir crest is of importance. In the above example the S m^3/hour overflow rate would give an overflow height Δ of only 8 mm. A slight deviation of the weir crest from the horizontal would then already cause a very uneven withdrawal of the settled water. To avoid this as much as possible, the weir crest may be made of a special metal strip fastened with bolts to the con-crete weir wall. The top of such a strip is not straight; it has triangular notches at intervals (Fig. 15.4). Another solution is shown in figure 15.3 to the left. Openings in the settling tank wall are used which should be of a smaller diameter than for the similar inlet construction. For Q = 20 m^3/hour, 6 openings of 2.5 cm diameter should be adequate. The suspended matter content of the effluent water normally being low, the danger of clogging the holes is small and cleaning should not be needed frequently.

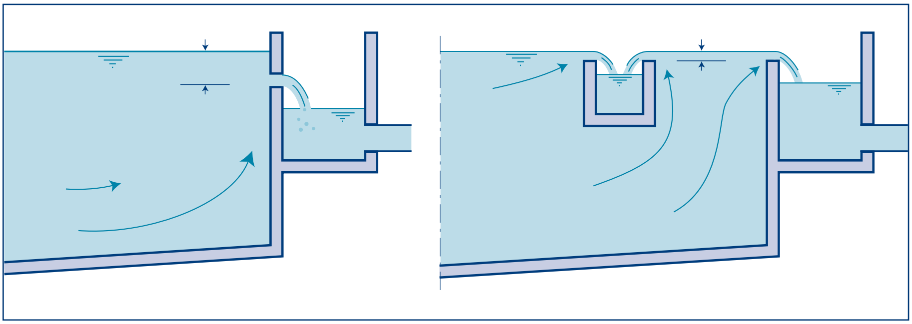

Fig. 15.3. Outlet arrangements

Earlier it was mentioned that small settling tanks may also be constructed simply as a basin with vertical walls of wooden sheet piling or similar material (Fig. 15.5), or with sloping walls (Fig.15.6). In the latter case only half the wetted slope length should be taken into account when computing the effective surface area and, thus, the surface loading of the settling basin. In both cases the basin should be constructed on raised ground to prevent flooding during wet periods.

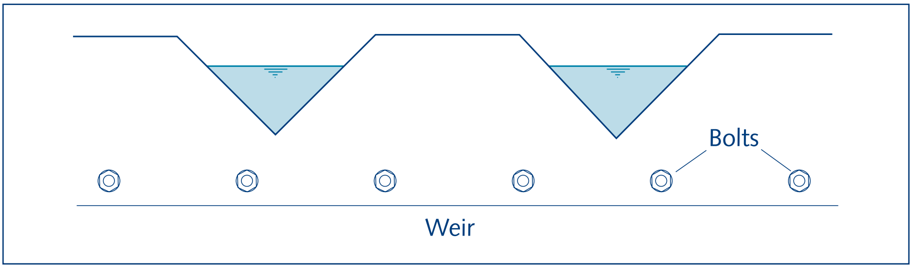

Fig. 15.4. Notched overflow weir

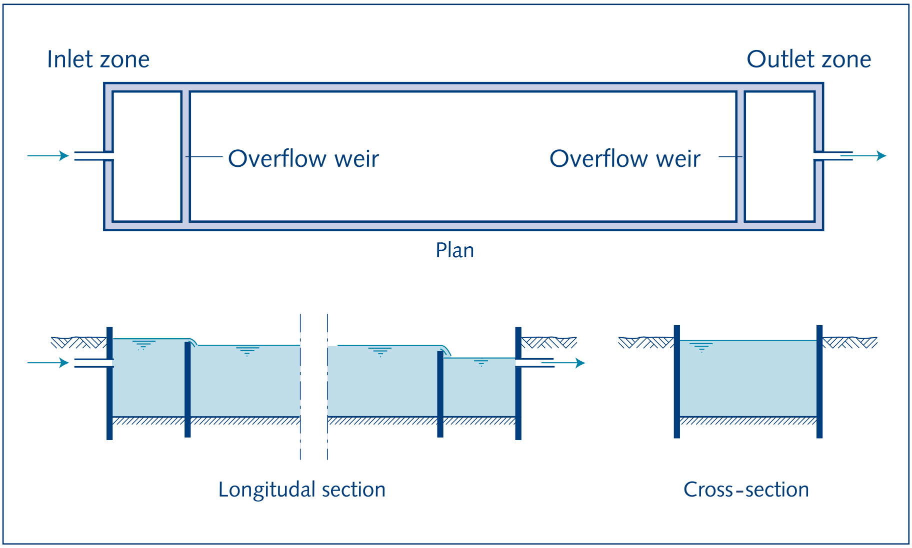

Fig. 15.5. Settling basin constructed with wooden sheet piles

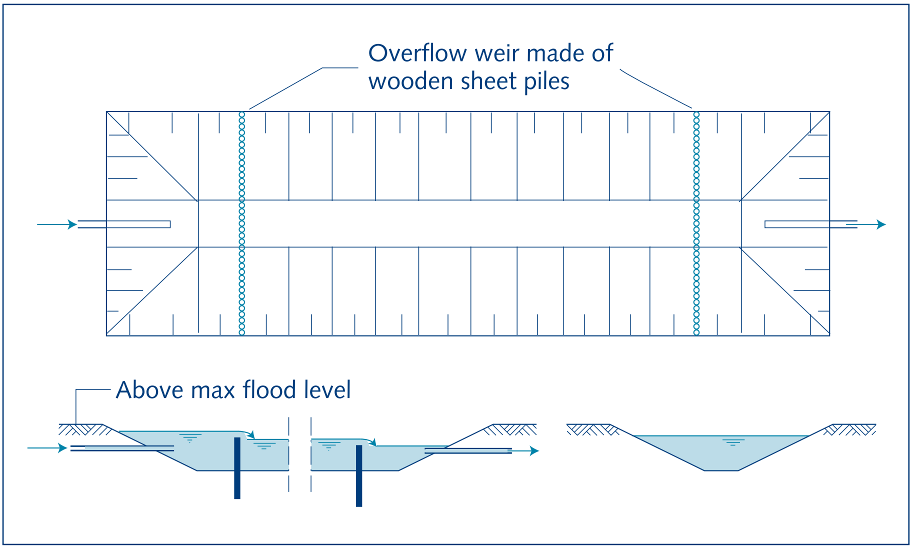

Fig. 15.6. Dug basin as settling tank

15.4.Tilted plate and tube settlers

The improvement in settling efficiency that can be obtained by the installation of one extra bottom (tray) in a settling tank can be greatly increased by using more trays as shown in figure 15.7. The space between such trays being small, it is not possible to remove the sludge deposits manually with scrapers. Hydraulic cleaning by jet washing would be feasible but a better solution is the use of self-cleaning plates. This is achieved by setting the plates steeply at an angle of 40-600 to the horizontal. The most suitable angle depends on the characteristics of the sludge, which will vary for different types of raw water. Such installations are called tilted plate settling tanks. This type of tank is shown schematically in figure 15.8. Figure 15.9 shows a cross-section.

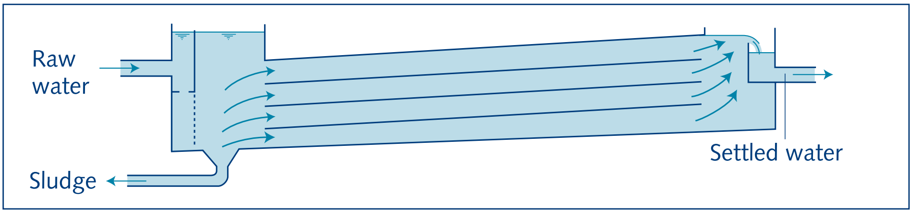

Fig. 15.7. Multiple-tray settling tank

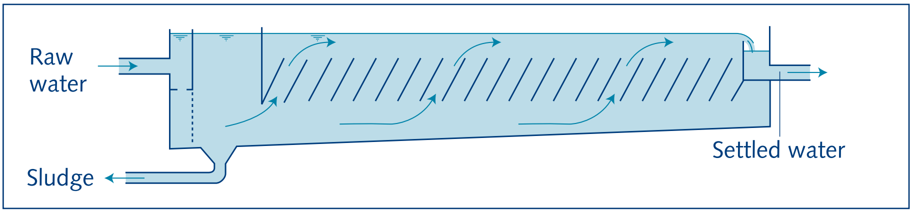

Fig. 15.8. Tilted plate settling tank

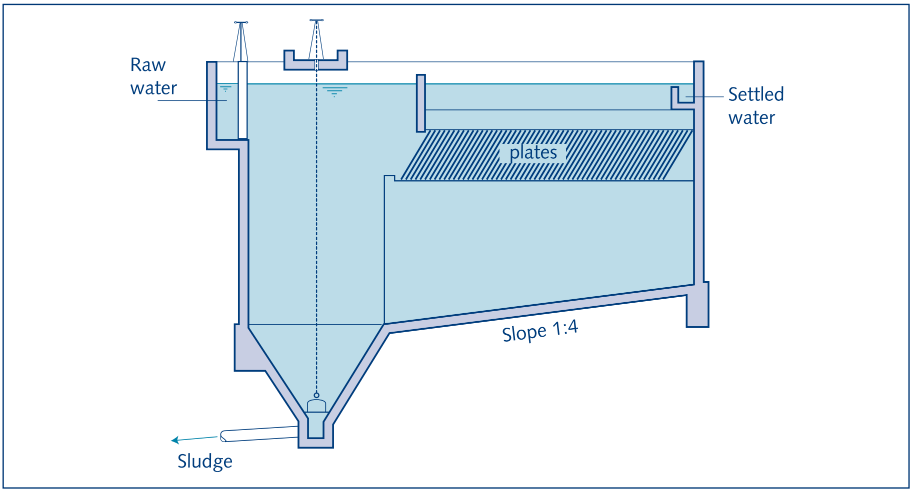

Fig. 15.9. Settling tank with tilted plates

For large tanks, quite sophisticated systems of trays or plates, have been devised but in small installations flat or corrugated plates and upward flow of the water are frequently the most suitable. For any clarification duty, tilted plate settling tanks have the advantage of packing a large capacity in a small volume. The effective surface being large, the surface loading will be low, and the settling efficiency, therefore, high. The surface loading may be computed as:

$$s = \frac{Q}{nA}$$

s $= $ surface loading ($m^3/m^2.h$)

Q $= $ rate of flow (m^3/h)

A $= $ bottom area of the tank (m^2)

n $= $ multiplication factor depending on the type and position of the tilted plates.

Water enters at the bottom of the settling tank, flows upwards, passes the tilted plates, and is collected in troughs (Fig. 15.10). As the water flows upwards past the plates the settleable particles fall to the plates. When they strike it they slide downwards, eventually falling to the area beneath the plates. An individual particle might enter the plate channels several times before it agglomerates and gathers sufficient weight to eventually settle to the tank floor.

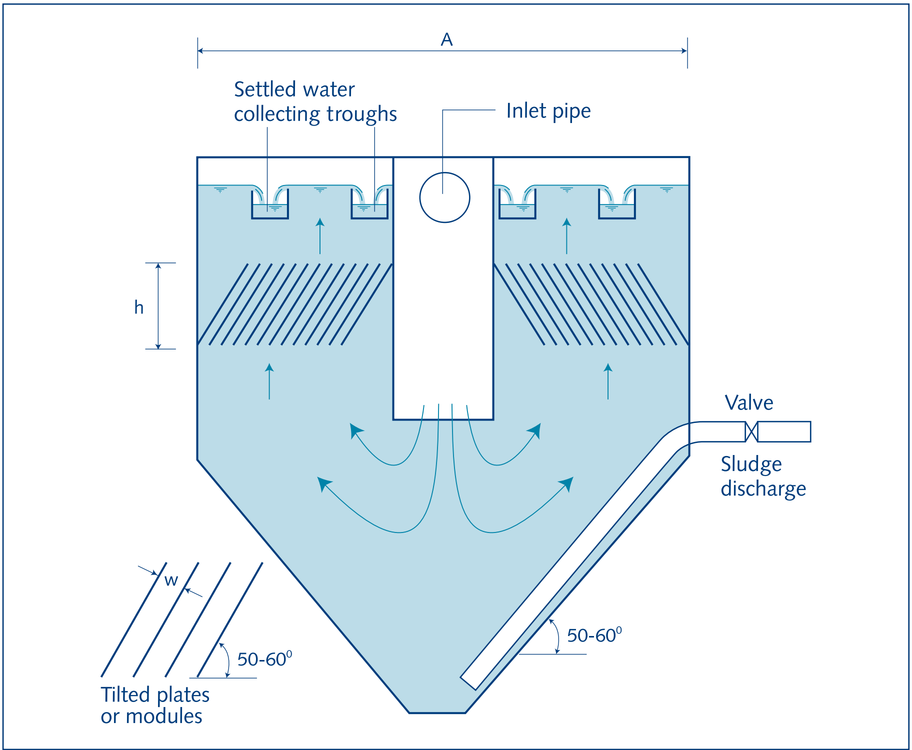

Fig. 15.10. Tilted plate settling tank design parameters

Assuming h = 1.5 m, w = 0.05 m, $\alpha = 55°$ and plates with a thickness of 6 mm, we find n = 16! One should keep in mind that the sludge deposits per unit bottom area will also be 16 times greater, for the same influent flow rate. Manual sludge removal will probably be impractical. In a tank having a square plan, rotating sludge scrapers might be used. Another possibility is the use of hopper-bottomed tanks with the walls sloping at about 50° to the horizontal. The depth of such a tank will be considerable and the costs of construction are likely to be much greater than for flat-bottomed tanks. Sludge discharge is carried out through the draining of water from the hopper-bottom section of the tank (this is called *bleeding*).

Instead of tilted plates, closely packed tubes may be used. These can easily be made of PVC pipes, usually of 3-5 cm internal diameter and sloping about 60° to the horizontal. For large installations commercially available tube models can have merit. An example is shown in figure 15.11. There are many other designs that may give an equally good settling efficiency.

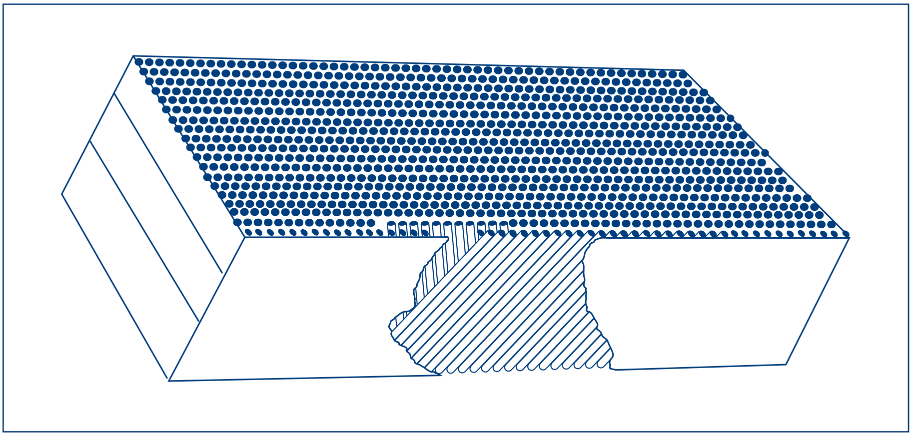

Fig. 15.11. Module for tube settler

In a 5 cm diameter tube, the farthest distance any particle must settle is from the top of the tube to the bottom. If the particle's settling rate is 2.5 cm/minute it will take only two minutes for the particle to reach the bottom. In contrast, if the same particle were to settle in a 3 m deep tank, it would take 120 minutes (2 hours) for it to fall to the tank bottom. Tube modules commonly are approxi-mately 76 cm wide, 3 m long and 54 cm deep. Because the tubes are at an angle of 60° the effective tube length is 61 cm.

Tube modules can be constructed from flat sheets of ABS plastic with the passageways formed by slabs of PVC. The passageways are slanted in a criss-cross pattern for structural strength so that the module needs to be supported only at its ends. Being of plastic these modules can be easily trimmed to fit the available space in a settling tank.

The effective settling surface is very great and, thus, the “surface loading” (overflow rate) very low. To illustrate this: a flow rate of 2 m^3/hour through a settling basin of 0.1 m^2 surface represents a surface loading of 20 m^3/m^2.hour. If twenty rows of tubes are used, the surface loading will be reduced to 1 m^3/m^2.hour. The detention time of the water in each tube will be just a few minutes.

The possibility of increasing the efficiency of a tank through the installation of tilted plates or tubes may be used with great advantage for raising the capacity of existing settling tanks. Where the available tank depth is small, less than 2 m, the installation of the tilted plates or tubes is likely to meet with problems. In deeper tanks they can be very advantageous.

In considering the expansion of existing facilities by the addition of tilted plates or tubes, it is important to remember that more sludge will be generated and so additional removal facilities may be required. Inlet and outlet pipe sizes and weir capacity should also be checked to see if they could carry the increased loading.

15.4 Dissolved air flotation (DAF)

Introduction

Since the 1970's there has been a growing interest in flotation of particles from water as opposed to traditional forms of settlement. For more than a century the separation and collection of useful minerals by flotation with coarse air bubbles, called *dispersed* air flotation, had been usual in the mineral processing and metallurgical industries, but it could not be usefully applied to the fine and light particles found in water purification. Development of very fine bubbles from air in solution, called *dissolved air flotation* (DAF) made it possible to collect and remove fine light particles, such as flocs containing colour, or algae, from water intended for drinking water supply.

In most cases the fine particles and colour need a coagulation and flocculation step, with the addition of appropriate chemicals such as aluminium or ferric salts (see chapter 14). This immediately raises a problem for small communities, but if chemical dosing is already established or envisaged for processes requiring flocculation (floc blanket, sedimentation, direct filtration), then it may be useful to consider dissolved air flotation as an alternative process. In cases where significantly high algal concentrations have occurred, DAF has been applied to remove the algae without the need for prior chemical flocculation.

Basic technology

The basic technology of DAF is illustrated in figure 15.12. The main flow of water to be treated is from left to right and comprises chemical addition and mixing, flocculation, injection of water saturated with air under pressure, nozzles for the release of pressure, a flotation tank with froth removal, and clarified water to rapid filtration. A recycle flow of clarified water has to be pumped to a saturator vessel, which contains an inert packing supplied with compressed air. This allows air to dissolve in the recycle water, which is then fed to the air release nozzles at the entrance to the flotation tank in the main flow.

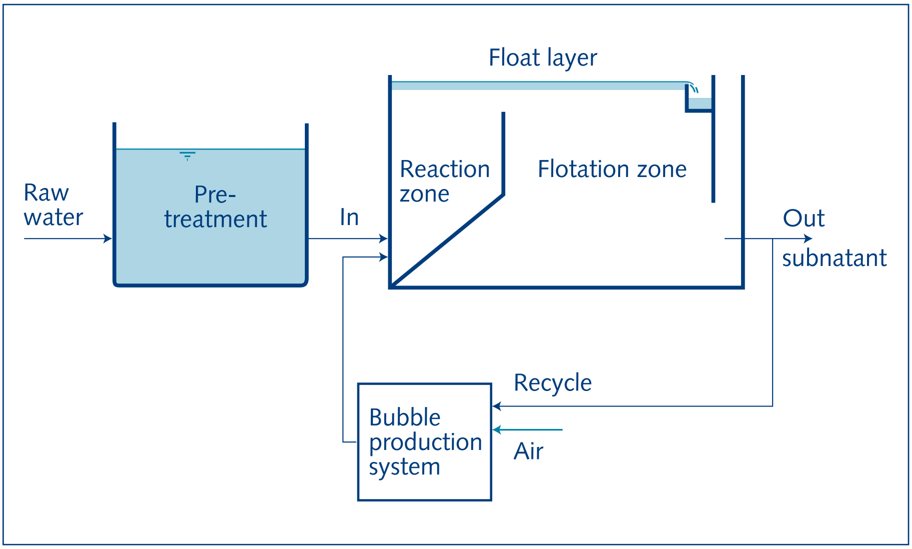

Fig. 15.12. Dissolved Air Flotation schematic diagram
Source: Van Duuren, F.A., 1997

Characteristic values for the process (after coagulation-flocculation) are:

Air injection	708 air per m^3 of raw water
Operating pressure	350-420 kPa (3.5-4.2 bar)
Water recycle	7-8% of flow (limits 6-10% to avoid inadequate mixing, or excessive floc break-up)
Flotation tank loading	$12m^3/m^2.h$ (surface loading rate, as in settling tank design).

Application

Because there are several technical steps necessary in DAF, it would not normally be advisable to use the process in small community water supplies. The technical steps are: coagulation and flocculation, dissolved air pressure release nozzles, removal of froth sludge by mechanical surface scraping devices, recycle pumping, saturator vessel operation and air compressor. The air compressor can be simplified as a venturi tube eductor on the recycle pressure line, which will suck in air due to the pressure drop at the venturi throat, open to the atmosphere. However, if responsible maintenance (a qualified technician) is available, it is possible to install package plants, delivered as a whole unit, which will sit on a concrete pad, with appropriate inflow and outflow pipe, and sludge drainage connections. A reliable electricity supply is essential.

The principal application for DAF in small community water supplies is in the flotation of algae, particularly blue-green algae, which have a natural tendency to float anyway, but which can give rise to filtration problems if not reduced beforehand. DAF has the advantage that it responds quickly to switch-on, and does not suffer when not operating in idle periods (although it must be kept clean when not working). Consequently, if algal blooms start to occur in the source (raw) water, a DAF unit can be switched on to avoid trouble with the filtration stage during the period of algal infestation, and it can be switched off (and bypassed) when it is over.

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