Chapter 4

Separation of Suspended Solids by Sedimentation Processes

4.1 SETTLING OF DISCRETE PARTICLES IN AN IDEAL FLOW TANK

In an horizontal flow settling tank, the suspension to be clarified flows horizontally through the tank and the suspension settles out on the floor of the unit. The classical theory of Hazen (1904) and Camp (1946) is based upon the concept of an ideal horizontal flow settling zone free from inlet and outlet disturbances in which particles settle freely at their terminal settling velocities as in quiescent conditions. The flow is steady and the velocity of the fluid is everywhere uniform. It is assumed that initially the particles are uniformly distributed over the cross-section of the flow and particles that settle out are not re-entrained. Flocculation, turbulence and other effects are absent.

Referring to Fig 4.1 it can be seen that a particle follows a path determined by the vector sum of its settling velocity v_t and the horizontal flow velocity v_h. Particles entering the tank at water surface level and having $v_t = v_0$, where $v_0/v_h = H/L$, will just reach the tank floor in the tank flow-through time. Particles having a settling velocity greater than v_0 will also reach floor level, while those particles having settling velocities less than v_0 will be removed from the fluid in the ratio v_t/v_0.

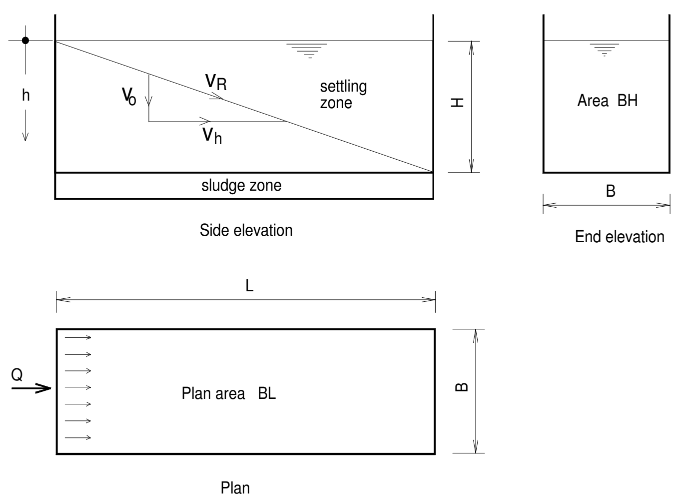

Fig 4.1 **Ideal horizontal flow clarifier**

Thus all particles having a settling velocity equal to or greater than v_0 will be completely removed, where v_0 is defined by

$$v_0 = v_h \frac{H}{L} = \frac{Q}{BH}\frac{H}{L} = \frac{Q}{BL} \qquad (4.1)$$

Q/BL = Q/A is the 'surface loading' or 'overflow rate'. Thus, particles having a settling velocity equal to or greater than the surface loading, are removed.

The overall removal of particles from a discrete suspension in an horizontal flow sedimentation basin can be estimated from the suspension settling velocity distribution curve, as illustrated in Fig 4.2. In addition to the removal of particles with settling velocities greater than or equal to v_0, there is a partial removal of particles with settling velocity less than v_0. Referring to Fig 4.2, a proportion v_t/v_0 of the fraction of particles dp, which have a settling velocity v_t, is removed. Hence the total removal may be written as

$$R = (1 - p_0) + \frac{1}{v_0}\int_0^{P_0} v_t \, dp \qquad (4.2)$$

The integral is given by the area between the settling curve and the vertical axis and hence the removal ratio for any given value of v_0 (i.e. Q/A) can be computed from the cumulative curve.

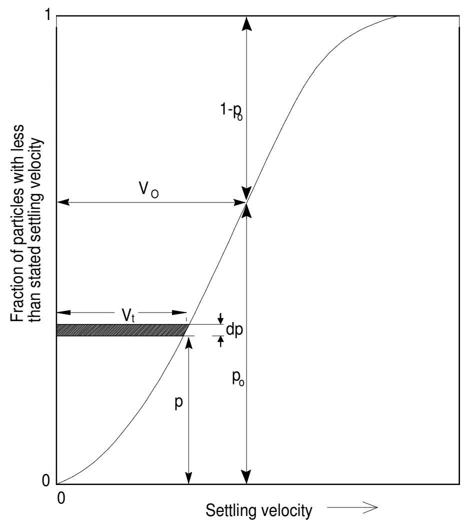

Fig 4.2 **Particle settling velocity distribution**

From a theoretical viewpoint, therefore, the removal of discrete particles in horizontal flow settling tanks depends only on surface loading and hence is independent of tank depth and hydraulic retention time. However, in reality the reduction in depth results in a concomitant increase in the horizontal velocity resulting in increased turbulence and bottom scour. These effects are discussed later in this chapter.

The above reasoning may also be applied to an ideal horizontal flow circular settling tank, where, as in the case of the ideal rectangular tank, it is assumed that the horizontal (radial) velocity is uniform over the tank depth. The radial velocity at radius r is v_r = Q/2prH; a particle with settling velocity v0 follows a trajectory defined by

$$\frac{dh}{dr} = \frac{v_0}{v} = \frac{2\pi r H v_0}{Q} \qquad (4.3)$$

Integrating this expression between the boundary values of r1 and r2, the radii defining the sedimentation zone of the tank:

$$H = \frac{\pi H v_0}{Q}\left(r_2{}^2 - r_1{}^2\right) = \frac{H v_0 A}{Q}$$

or

$$v_0 = \frac{Q}{A}$$

confirming that surface loading is also the key design parameter for circular tanks.

For upward flow sedimentation tanks, it is quite clear that particles having settling velocities less than v_0 will move upwards with the flow and hence are not removed. Thus, referring to the settling velocity distribution shown in Fig 4.2, the theoretical fractional removal of this suspension in a vertical flow settling tank at a surface loading v_0 would be $(1-P_0)$, where P_0 is the fractional mass of particles with settling velocities less than v_0.

4.2 RESIDENCE TIME DISTRIBUTION

In the ideal settling tank, the inflow is assumed to be evenly distributed over the cross-section of the tank and the flow is assumed to advance as piston or plug flow to the outlet. Each element of the fluid remains in the tank for a period equal to the theoretical detention time T. however, in an actual tank, the flow pattern is complex because of inlet and outlet disturbances, and because of density, convection and wind-induced currents and dead spaces. Consequently, the hydraulic retention time, instead of being a single value for the flow regime, typically varies over a wide range.

The distribution may be measured by adding a tracer to the inlet flow, as a pulse, and measuring the concentration in the outlet as a function of time. The results of such flow tests are usually plotted as dimensionless values c/c0 versus t/T, where c is the effluent tracer concentration at any time t and c0 is the concentration that would be obtained if the quantity of tracer injected was diluted up to the tank volume. Examples of plug flow, practical tank flow and perfectly mixed flow are shown in Fig 4.3.

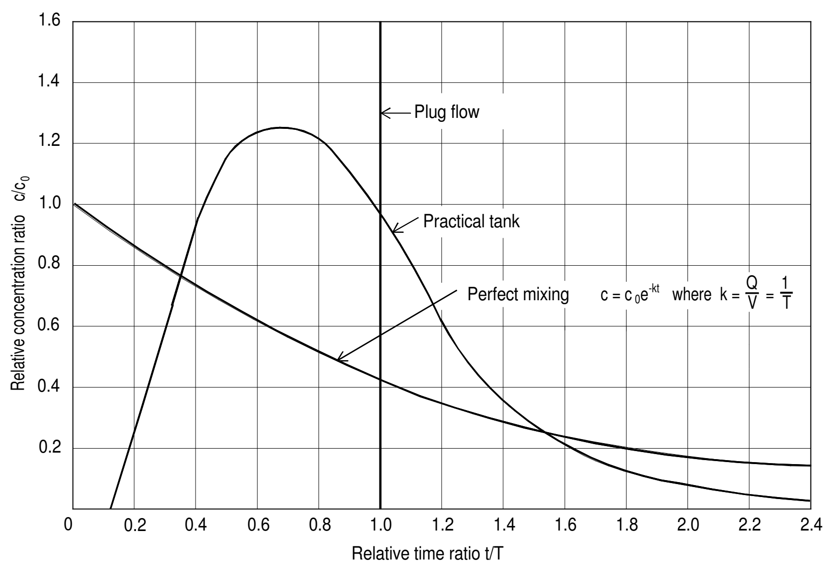

Fig 4.3 Residence time distribution in settling tanks

It is common to use measures of the central tendency and dispersion of the distribution curve to give qualitative information on the flow pattern in a tank. The mean retention time, t_{mean}, is defined by the position of the centre of gravity (first moment) of the distribution curve. The modal tome, t_{mode}, is the time of flow of the maximum concentration, and the median time, t_{median}, is the time taken for 50% of the added tracer to reach the outlet. Mean, mode and median are all measures of central tendency. If there is no short circuiting their values coincide and the distribution curve is symmetrical, the spread being caused by diffusion alone. The degree of short circuiting may be gauged by the value of

$$\frac{\text{mean} - \text{mode}}{\text{mean}} \quad \text{or} \quad \frac{\text{mean} - \text{median}}{\text{mean}}$$

The presence of dead spaces is indicated by the difference between t_{mean} and the theoretical detention time T. In practice, $t_{mean} < T$.

The variability of exposure to sedimentation is indicated by the spread of the curve. A measure of this is the ratio of t_{10}/t_{90}, where t_{10} and t_{90} are the times for 10% and 90% of the tracer, respectively, to reach the outlet.

4.3 INFLUENCE OF TURBULENCE

Flow in sedimentation tanks is nearly always turbulent with a consequent reduction in sedimentation due to the random convective movement of particles. The Reynolds number, Re, provides an index of flow conditions:

$$R_e = \frac{v_h R}{\nu} \tag{4.4}$$

where R is the hydraulic radius and ν is the kinematic viscosity. If $R_e <$ ca. 600, flow is laminar; if $R_e >$ ca. 2000, flow is turbulent. R_e values for rectangular and circular horizontal flow sedimentation tanks may be expressed in terms of flow Q and tank dimensions as follows:

rectangular tanks:
$$R_e = \frac{\frac{Q}{BH} \cdot \frac{BH}{B+2H}}{\nu} = \frac{Q}{\nu(B+2H)} \tag{4.5}$$

circular tanks:
$$R_e = \frac{\frac{Q}{2\pi rH} \cdot \frac{2\pi rH}{2\pi r}}{\nu} = \frac{Q}{2\pi r\nu} \tag{4.6}$$

Thus, to reduce Re for rectangular tanks, the width and/or depth has to be increased. In circular tanks, the flow regime is fixed and R_e decreases with distance from the centre. The adverse influence of turbulence on sedimentation, based on analyses by Dobbins (1944) and Camp (1946), is shown in Fig 4.4. The extent of reduction of the idealized removal ratio, v_t/v_0, is seen to be a function of v_t/v_h, the ratio of particle settling velocity to horizontal flow velocity.

4.4 SCOURING OF DEPOSITED PARTICLES

In sedimentation tanks the horizontal flow velocity should be kept below the scour threshold level. For light flocculent particles, such as metal hydroxide flocs and activated sludge, the critical scour velocity (Ingersoll et al., 1956) is defined by the following relationship:

$$\left(\frac{\tau}{\rho}\right)^{0.5} \geq v_t \tag{4.7}$$

where τ is the shear stress at the sludge liquid interface and ρ is the supernatant liquid density. $(\tau/\rho)^{0.5}$ is sometimes called the shear velocity, denoted as v*.

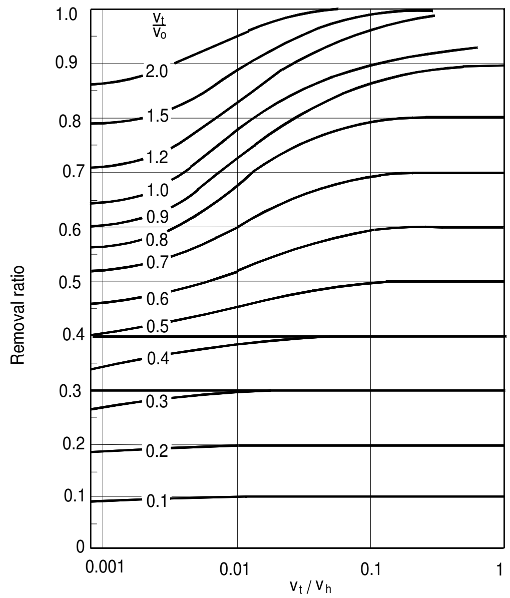

Fig 4.4 Influence of turbulence on the removal ratio in the Settling of discrete suspension (Reprinted from Camp, 1946, by permission of ASCE)

Considering the flow regime in a rectangular sedimentation basin to be steady open channel flow, the following relationships can be applied to the flow:

$$S_f = \frac{f v_h^{\,2}}{8gR_h} \qquad \text{and} \qquad \tau = \rho g R_h S_f$$

where S_f is the friction slope, v_h is the horizontal velocity = Q/BH, f is the friction factor and R_h is the hydraulic radius = BH/(B + 2H). It follows from the foregoing flow relationships that

$$\frac{\tau}{\rho} = \frac{f}{8} v_h^{\,2} \tag{4.8}$$

Combining equations (4.7) and (4.8), the criterion for the prevention of scour may be expressed as follows:

$$v_t \geq v_h \left(\frac{f}{8}\right)^{0.5}$$

i.e.

$$\frac{Q}{BL} \geq \frac{Q}{BH}\left(\frac{f}{8}\right)^{0.5}$$

or

$$\frac{H}{L} \geq \left(\frac{f}{8}\right)^{0.5}$$

hence

$$\frac{L}{H} \leq \left(\frac{8}{f}\right)^{0.5}$$

If f is assumed to have a typical value of 0.024, then the foregoing L/H criterion becomes

$$\frac{L}{H} \leq 18.0 \qquad (4.9)$$

To provide a margin of safety in design and to allow for a possible non-uniform velocity distribution, it is advisable to limit the L/H ratio to about 10.

For heavier and more dense particles such as grit, the horizontal velocity v_{sc} required to initiate scour of deposited particles is given by the following correlation (Camp, 1946):

$$v_{sc} = \sqrt{\frac{8\beta}{f} g\left(S_g - 1\right)d} \qquad (4.10)$$

where β has a value of about 0.04 for rounded granular material and a value of about 0.06 for non-uniform sticky and flocculent material. For example, the scouring velocity for a grit particle of size 0.2mm, S_g 2.5, is estimated from equation (4.10), taking β = 0.06 and f = 0.025, to be 0.23 m s^{-1}.

4.5 FLOW STABILITY

Hydrodynamic stability is important in sedimentation in order to reduce the effects on velocity distribution of disturbing influences such as wind, density currents, etc. The flow Froude number Fr can be used as an index of stability:

$$F_r = \frac{{v_h}^2}{gR_h} \qquad (4.11)$$

which, in terms of flow Q and tank dimensions, is expressed as follows:

rectangular tank: $$F_r = \frac{Q^2}{B^2H^2} \cdot \frac{1}{g} \cdot \frac{B+2H}{BH} = \frac{Q^2\left(B+2H\right)}{gB^3H^3} \qquad (4.12)$$

circular tanks: $$F_r = \frac{Q^2}{\left(2\pi r H\right)^2} \cdot \frac{1}{g} \cdot \frac{2\pi r}{2\pi r H} = \frac{Q^2}{4\pi^2 r^2 H^3 g} \qquad (4.13)$$

Thus, flow stability in rectangular tanks is improved by reducing the cross-sectional area and increasing the length, which is in conflict with the requirements for the reduction in turbulence and the prevention of bottom scour. While there appears to be no generally agreed minimum value for F_r for rectangular tanks, it is considered desirable to have a value in excess of 10^{-5}. In circular tanks, F_r decreases with distance from the tank centre, resulting in poor stability in the outer perimeter zone.

4.6 SEDIMENTATION IN WASTEWATER TREATMENT

Sedimentation is a very widely used solids/liquid separation process in wastewater treatment. It is used in the preliminary stage of treatment for the selective separation of grit, in primary treatment for the separation of settleable solids from raw wastewaters, and in secondary treatment for the separation of biological sludges and chemical precipitates.

4.6.1 Grit separation

The inert fraction of settleable solids in municipal wastewater consisting of ashes, clinker, sand particles etc. is termed grit. It has a higher density than the organic fraction (Sg > 2), settles rapidly and, if not removed in the preliminary stage of treatment, would make subsequent sludge handling more difficult. Grit removal processes are typically designed to remove grit particles of diameter ≥0.2mm and Sg of 2.6, which have a settling velocity (Fig 2.2) of about 20 mm s^{-1}. Grit separation may be carried out in channel-type grit chambers or in vortex-based circular tanks.

In channel-type grit separators the settlement of putrescible organic solids is prevented by maintaining a scour velocity of about 0.3 m s^{-1}. Combining this value with a particle settling velocity of 20 mm s^{-1} results in a required L/H ratio ≥ 15. The horizontal velocity is maintained at a constant value, independent of flow rate, by an appropriate flow control device at the channel outlet end. Proportional flow (Sutro) weirs (Fig 4.5a) are used with channels of rectangular cross-section, while rectangular critical-depth flumes are used with channels of parabolic cross-section (Fig 4.5b). These devices, which can also be used for flow measurement, are illustrated in Fig 4.5.

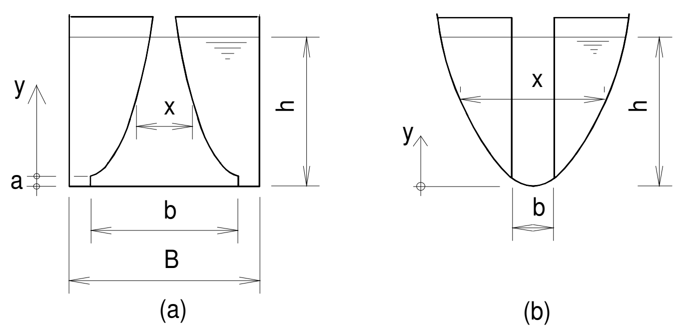

Fig 4.5 Grit channel flow control devices
(a) Proportional flow Sutro) weir at outlet end of rectangular channel
(b) Critical-depth flume at outlet end of a parabolic channel

The width x of a Sutro weir decrease with height y (Fig 4.5a) according to the relationship:

$$\frac{x}{b} = 1 - \frac{2}{\pi}\tan^{-1}\left(\frac{y}{a}\right)^{0.5} \qquad (4.14)$$

The discharge through a Sutro weir can be expressed in terms of the head h as follows:

$$Q = C_d b(2ga)^{0.5}\left(h - \frac{a}{3}\right) \qquad (4.15)$$

Since a is typically small relative to h, Q may be considered to vary linearly with h. Hence the horizontal velocity in the upstream rectangular channel (i.e. Q/BH) is effectively constant. For a sharp-edged plate weir the discharge coefficient is approximately 0.6. The lower limit value for a and x may be taken as 5mm.

The geometrical profile for a parabolic channel (Fig 4.5b) conforms to the correlation $x^2 = ky$, and its sectional area $A = 2/3\ xy = 2/3\ k^{0.5}y^{1.5}$. The discharge through a rectangular critical-depth flume may be expressed in terms of the upstream head h as follows:

$$Q = C_d \frac{2}{3}\left(\frac{2}{3}g\right)^{0.5} bh^{1.5} \tag{4.16}$$

Assuming the discharge coefficient Cd has a unit value (rounded upstream converging section), the foregoing expression may be simplified to

$$Q = 1.7\ bh^{1.5} \tag{4.17}$$

Thus the horizontal velocity Q/A through a parabolic channel controlled by a rectangular critical depth outlet flume is

$$\frac{Q}{A} = \left(1.7bh^{1.5}\right) / \left(\frac{2}{3}k^{0.5}h^{1.5}\right) = \text{constant} \tag{4.18}$$

Vortex type grit removal units – insert section

The quantity of grit in municipal wastewater varies over a wide range (25-250 mg l-1), being greater in discharges from combined sewer systems than in separate systems and also varying with season and rainfall.

4.6.2 Primary sedimentation

The term 'primary sedimentation' in wastewater treatment technology refers to separation of settleable solids from raw wastewater. The settleable solids in municipal sewage and many industrial wastewaters include flocculent and discrete-settling particles. Primary sedimentation tanks at municipal works have to cope with considerable variations in loading, especially when serving combined sewer systems. Primary tanks are typically designed for a maximum hydraulic loading of 3 DWF (dry weather flow). Because of the flocculent nature of the raw sewage solids, solids removal is influenced by retention time as well as by surface loading. The influence of retention time on solids removal is shown in Fig 4.6.

The design surface loading for primary sedimentation tanks is typically in the range 1-2 m h^{-1} at maximum flow. However, it would appear that significantly higher overflow rates could be used without incurring an excessive penalty in performance, as may be seen from the pilot plant data summarized in Table 4.1. The liquid depth is usually in the range 3-4 m, giving retention times in the range 2-4 hours at maximum flow.

Table 4.1 Primary sedimentation pilot plant test results

Surface loading rate ($m^3m^{-2}d^{-1}$)	Mean influent solids (mg l^{-1})	Solids removal efficiency (%)
25	411	49
50	402	43
100	355	36
150	365	34

Source: Tebbutt and Christoulas (1975)

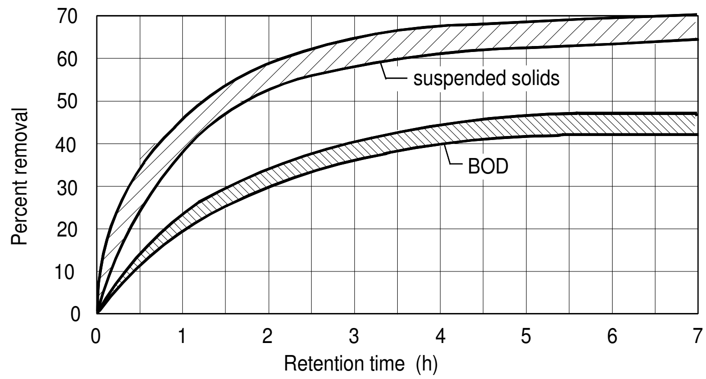

Fig 4.6 Expected solids and BOD removal in primary sedimentation
Source: Fair et al (1968) (Reprinted by permission of John Wiley, Inc.)

The expected performance of primary sedimentation tanks according to the German design guidelines (ATV-DVWK, 2000) are outlined in Table 4.2.

Table 4.2

Performance indicator	Retention time in primary clarifier	
	0.5 to 1.0 h	1.5 to 2 h
Suspended solids % removal	50	64
BOD_5 % removal	25	33

At small plants, upward flow tanks of hopper-bottom construction, as shown in Fig 4.7, are used. Tanks of this type allow sludge thickening and removal without the aid of mechanical equipment. Because of their shape, however, they are expensive to construct.

Circular tanks with rotary bridge scrapers, as shown in Fig 4.8, are commonly used in wastewater treatment systems. The scraper blades move the settled solids (sludge) towards a central hopper, from which it is withdrawn. The side wall depth is typically in the range 1.5-3.0m; floor slopes are usually about 1:10. Tanks up to 60m in diameter have been constructed. The central inflow is directed upwards towards the free surface and its kinetic energy is dissipated within a cylindrical diffusion box or stilling chamber. The diameter of the latter is usually in the range 12-20% of the tank diameter and typically extends to about mid-tank depth.

Rectangular tanks are also used for primary sedimentation, but usually only at large works. The length:width ratio is typically 3-4:1, while the length:depth ratio should preferably not exceed 10 (to avoid scouring effects). Travelling bridge scrapers or submerged flight-type scrapers are used to move settled sludge to hoppers at the inlet end of rectangular tanks. An example of the latter type is shown in Fig 4.9. The floor slope (towards the inlet end) is generally about 1:100.

Primary sludge is preferably withdrawn continuously from sedimentation tanks so as to avoid septicity and the generation of biogas, which reduces the effective density of the settled solids and thus disrupts sedimentation. The primary sludge generated in municipal wastewater treatment plants has good thickening characteristics and can be concentrated in primary tank hoppers to a solids concentration of 4-6%.

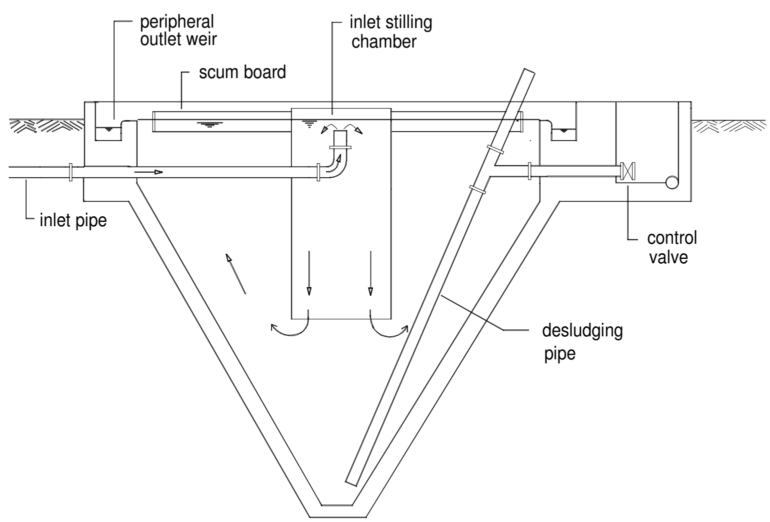

Fig 4.7 Upward flow clarifier

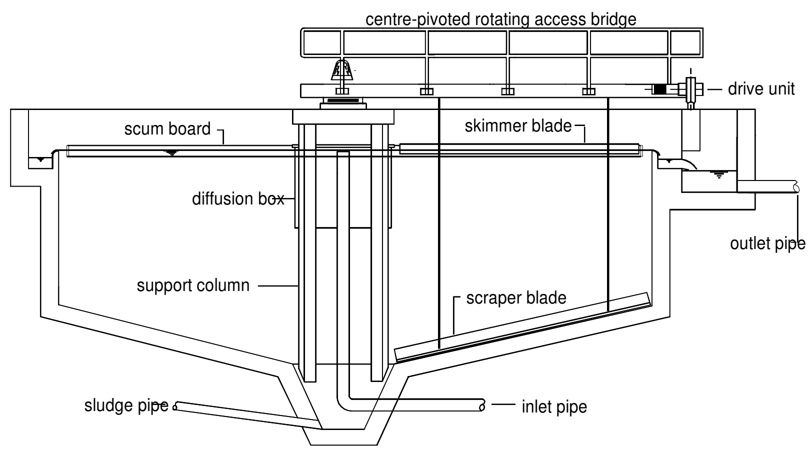

Fig 4.8 Circular clarifier with rotary bridge scraper

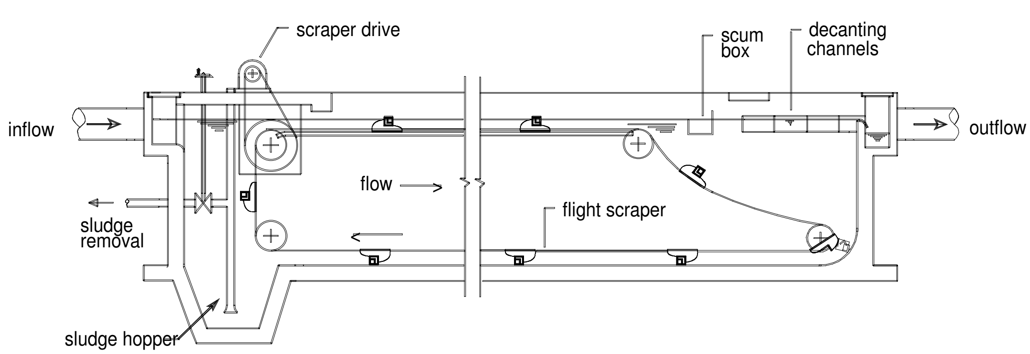

Fig 4.9 Rectangular primary clarifier fitted with flight scraper

4.6.3 Secondary sedimentation

The term ‘secondary sedimentation’ refers to sedimentation processes following biological (activated sludge, biofiltration) or physicochemical secondary treatment processes. The performance target for secondary treatment tanks is generally expressed in terms of a limiting effluent suspended solids concentration, typically ≤ 35 mg l^{-1}. The sedimentation basins used in secondary sedimentation applications are generally the same as described for primary sedimentation.

Factors other than those already described require to be taken into account in the design of secondary sedimentation basins associated with activated sludge processes. These include the basin solids flux capacity and the sludge recycle rate. The activated sludge concentration in aeration basins is typically in the range 2000-4000 mg l^{-1} and thus is significantly higher than other suspensions encountered in wastewater treatment. A design procedure for sizing activated sludge secondary sedimentation basins is presented in Chapter 12. The design surface loading is typically in the range 0.8-1.2 m h^{-1}.

The concentration of suspended solids in biofilter effluents (humus sludge) is characteristically variable but is usually less than 100 mg l^{-1}. Secondary sedimentation basins following biofilters are generally sized on the basis of a design surface loading in the range 1-1.5 m h^{-1}.

The metal hydroxide/organic sludges resulting from physicochemical treatment are flocculent in nature and have settling characteristics generally similar to flocculent activated sludge. They can be separated by conventional sedimentation processes at surface loading rates in the range 0.8-1.2 m h^{-1}.

4.7 SEDIMENTATION IN WATER TREATMENT

Plain sedimentation of source water is rarely necessary in the production of potable water since the settleable solids concentrations in surface waters used as drinking water sources are invariably low. However, where river sources intermittently carry large silt loads under flood conditions, it may be necessary to have either intermediate reservoir storage or primary sedimentation as part of the overall treatment system.

Natural surface waters generally contain suspended and colloidal material resulting in colour and turbidity levels not acceptable in drinking water. Dissolved/suspended colour and colloidal solids are converted to a settleable form by the process of chemical coagulation, as described in Chapter 3. The resultant flocculent suspensions are clarified by sedimentation and filtration processes.

Sedimentation tanks of the kind already described for wastewaters may be used for this purpose. However, upward flow fluidized beds or so-called ‘sludge blanket’ clarifiers are more usually used for this application.

4.7.1 Sludge-blanket clarifiers

Sludge-blanket clarifiers combine the roles of flocculation and floc separation. As shown in Figs 4.10 and 4.11, the sludge blanket, which has zone settling characteristics, is fluidized by the upward-flowing water. Sludge is withdrawn from the top of the blanket, where there is a well-defined interface between the blanket and the supernatant water. Inflow is at floor level, where there is considerable turbulence and mixing which helps to promote flocculation. The earlier form of sludge-blanket clarifier was of the hopper-bottom type, as shown in Fig 4.10. The hopper-bottom shape has the advantages of a simple inlet system and of providing a simple means of concentrating settled sludge to permit easy removal. Hopper-bottom tanks are, however, expensive to construct; they are limited in plan area if excessive depth is to be avoided, requiring multiple units in large plants. The hydraulic characteristics of these units are also unfavourable owing to the non-uniformity of the divergent upward flow, which causes short-circuiting and blanket instability. For these reasons the surface loading of hopper-bottom sludge –blanket clarifiers is usually limited to 1-1.5 m h^{-1}. This range can be extended upwards by the use of energy-dissipating inlet devices.

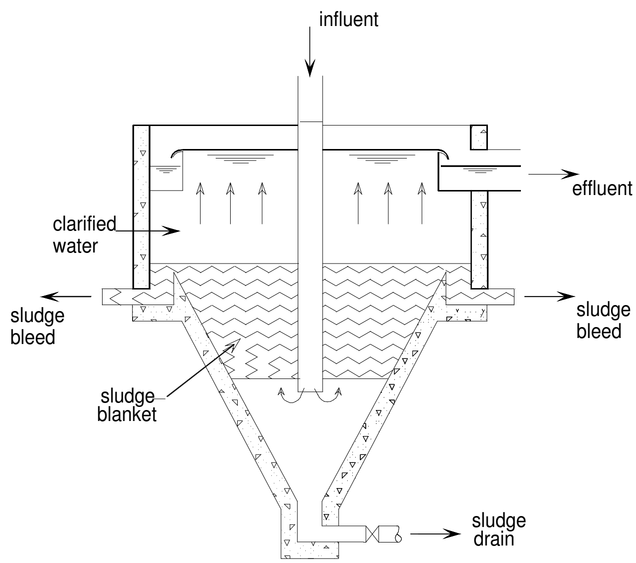

Fig 4.10 **Hopper-bottom sludge-blanket clarifier**

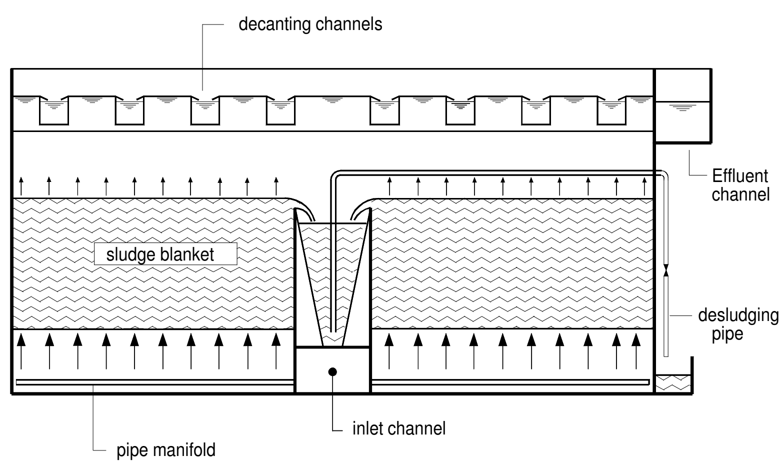

Fig 4.11 **Flat-bottom sludge-blanket clarifier**

The flat-bottom type of sludge blanket clarifier typically has a pipe manifold inlet system, located near floor level and discharging downward. In some designs the flow is pulsed to improve flocculation and prevent the deposition of solids on the tank floor. Clarified effluent is withdrawn by a system of decanting channels at the tank surface, as shown in Fig 4.11. To avoid disturbance of the blanket surface by the upward flow, the spacing of decanting channels is usually not greater than twice the distance between the top of the blanket and the water surface. The design surface loading for flat-bottom sludge-blanket clarifiers is normally in the range 2-4 m h^{-1}, depending on the settling characteristics of the floc.

4.8 INCLINED PLATE AND TUBE SETTLERS

The introduction of inclined plates or tubes into the settling zone of a sedimentation unit, as shown in Fig 4.12, effectively reduces its surface loading rate. Each pair of plates or individual tube acts as a sedimentation unit.

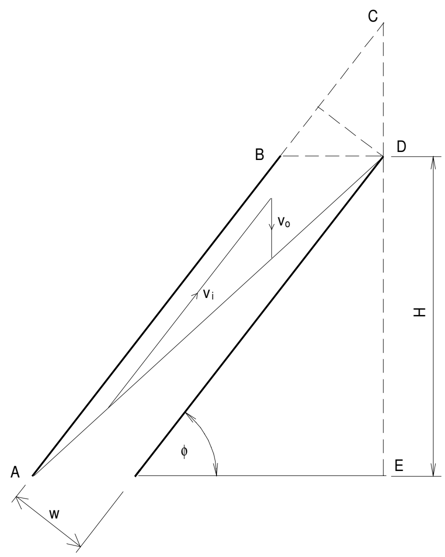

Fig 4.12 **Inclined plate pair**

Referring to Fig 4.12, it will be clear that particles having a settling velocity v_0 will be deposited on the plate surface and hence be removed, i.e. v_0 is the effective surface loading rate. In travelling from A to D, a particle with settling velocity v_0 settles through the vertical distance CD. From geometrical considerations, the following relationships are derived:

$$\frac{CD}{v_0} = \frac{AC}{v_1}$$

$$v_0 = v_1\left(\frac{CD}{AC}\right) = v_1\left[\frac{w/\cos\varphi}{(H + w/\cos\phi)/\sin\phi}\right]$$

also

$$v_1 \sin\phi = \frac{Q}{A}$$

where Q is the total flow and A is the tank plan area. Hence

$$v_0 = \frac{Q}{A}\left[\frac{w/\sin\phi}{H/\tan\phi + w/\sin\phi}\right] = \frac{AF}{AE} \qquad (4.19)$$

Taking, as an example, w = 0.3m, H = 2m, and $\phi = 60^o$, then

$$v_0 = \frac{Q}{A} \cdot \frac{0.3}{(2x0.5) + 0.3} = 0.23\frac{Q}{A}$$

Thus, the introduction of the plate system as dimensioned has the effect of reducing the surface loading to 23% of the plain tank value. The inter-plate flow is usually laminar, i.e. the inter-plate Reynolds number $v_i w/2\nu < 500$, where ν is the kinematic viscosity.

The practical effect of introducing an inclined plate system into a fluidized bed clarifier of the type illustrated in Fig 4.11 is demonstrated by the results plotted in Fig 4.13. These results were obtained in an upflow fluidized bed clarifier at Dublin City Council's water treatment plant at Ballymore Eustace; the water being treated was an alum-coagulated impounded surface water. The plotted surface loading values are the maximum upflow velocities that could be achieved while maintaining a stable sludge blanket, with and without inclined plates. The plate spacing was 300 mm and the plate length was 2.9m. The results show that the process capacity of the clarifier system was more than doubled by the introduction of an inclined plate system.

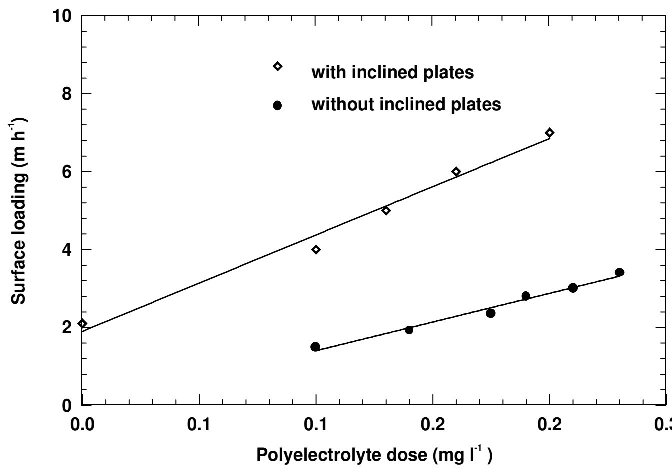

Fig 4.13 **Performance of upward flow fluidized bed clarifier pilot plant with and without an inclined plate system** (Purcell, 1983)

4.9 SOME ASPECTS OF HYDRAULIC DESIGN

4.9.1 Inlet systems

While inlet systems vary, depending on the type and shape of the sedimentation tank, the common design goal for all types of inlet system is the achievement of a uniform distribution of flow over the cross-section. In plain sedimentation processes there is the additional requirement of dissipation of kinetic energy of the incoming fluid so as not to disrupt the sedimentation process, while in fluidized bed processes there is the rather different additional requirement to inhibit settling of particles on the tank floor.

In rectangular tanks, as illustrated in Fig 4.9, the incoming flow is distributed over the tank width by a transverse manifold-type feed channel with multiple uniformly spaced discharge slots or orifices. The kinetic energy from these individual discharges may be locally dissipated by an appropriate baffle system.

In upward flow tanks (Fig 4.7) and circular tanks (Fig 4.8), central stilling chambers are used to contain the turbulent mixing generated by the inflow which, as illustrated in these diagrams, is directed upwards towards the free surface. The stilling chamber diameter is typically within the range 12-20% of the tank diameter (Christie and Harbinson, 1978; Metcalf and Eddy, 1991) and extends from above the water surface to about mid-depth. If the stilling chamber extends too low into the tank, then the inflow may disrupt the settled solids in the central sludge hopper.

In sludge-blanket clarifiers (Figs 4.10 and 4.11), the inflow is directed downward towards the tank floor, thus preventing deposition of solids on the floor. In rectangular flat-bottom tanks pipe manifold distribution systems are used, with downward discharging jets distributing the flow over the tank floor area. This uniformity of distribution, coupled with a corresponding uniformity of collection by closely spaced surface decanting channels, is essential to the maintenance of a stable sludge blanket in this type of clarifier system.

4.9.2 Outlet systems

It is the universal practice to take the outflow from the top of the tank. Weir outlets provide such a simple direct control of tank level that they are almost universally used. In horizontal flow plain sedimentation single-sided launders or channels are to be preferred to double-sided launders (Ekama and Marais 1986) since it has been found that the approach flow patterns associated with double-sided launders are more likely to cause erosion of settled solids leading to solids loss. Kawamura and Lang (1986) found little difference in clarified effluent quality in a side-by-side comparison of equally loaded twin rectangular tanks, one of which was fitted with in-tank launders to give a weir loading of 10.4 m^3m^{-1} h^{-1}, while the second tank had a simple transverse weir resulting in a weir loading of 259.2 m^3 $m^{-1}h^{-1}$. Thus, there seems little empirical justification for the common design practice of providing in-tank decanting channels to limit weir loading to a maximum value of about 10 m^3 h^{-1} per m length of weir.

To avoid disturbance of the blanket surface in upward flow sludge blanket clarifiers, the spacing of decanting channels (see Fig 4.11) is usually not greater than twice the distance between the top of the blanket and the water surface.

Plain weirs are unsatisfactory at low overflow rates because small variations in level result in an uneven draw-off. For this reason it is general practice to use notched weir plates on sedimentation tank outlets (Fig 4.14). Circular tanks are typically fitted with a peripheral outlet weir and rectangular tanks with a transverse outlet weir.

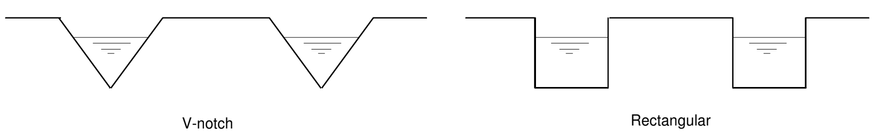

Fig 4.14 Notched outlet weirs

Flow in sedimentation tank decanting channels is hydraulically designated as steady gradually varied flow; typically, such channels have a uniform lateral inflow over the channel length and have a free overfall at the outlet end, as illustrated in Fig 4.15.

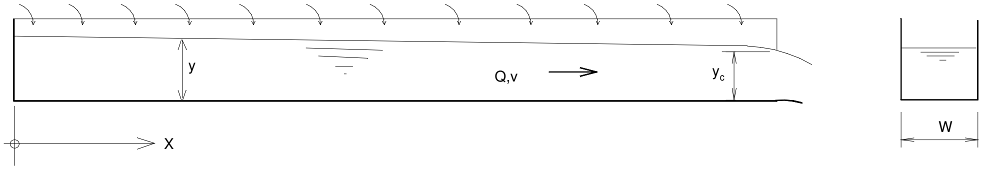

Fig 4.15 Gradually varied flow in clarifier decanting channel

The water surface slope (dy/dx) in such a channel is described by the equation:

$$\frac{dy}{dx} = \frac{S_0 S_f - \frac{Q}{gA^2}\frac{dQ}{dx}}{1 - \frac{WQ^2}{gA^3}} \qquad (4.20)$$

where S_0 is the channel bottom slope, S_f is the friction slope = $fv^2/8gR_h$, Q is the flow, $dQ/dx = q_L$ is the lateral inflow per unit length, W is the channel width, and A is the cross-sectional area.

Equation (4.20) can be solved numerically, starting from a defined depth at the outlet end. With a free discharge, as illustrated in Fig 4.15, the outlet end depth is the critical depth y_c, defined by the relationship:

$$\frac{WQ^2}{gA^3} = 1 \qquad (4.21)$$

REFERENCES

Camp, T. R. (1946) *Trans. ASCE*, 111, 895-936

Christie, I. F. and Harbinson, R.W. (1978) *Proc. ICE*, Part 2, 65, 71-84.

Dobbins, W. E. (1944) *Trans ASCE*, 109, No. 2218, 629-656.

Ekama, G. A. and Marais, G.v.R. (1986), *Wat. Poll. Cont*., 85, No. 1, 101-113.

Fair, G. M., Geyer, J. C. and Okun, D. A. (1968) Water and Wastewater Engineering, John Wiley & Sons Inc., New York

Hazen A. (1904) *Trans. ASCE*, 53, Paper No. 980, 45-88.

Ingersoll, A. C., McKee, J. E. and Brooks, N. H. (1956) *Trans. Am. Soc. Civil Eng*., 1176.

Kawamura, S. and Lang, J. (1986) J. WPCF, 58, No. 12,1124-1128.

Metcalf and Eddy Inc. (1991) Wastewater Engineering: Treatment, Disposal, and Re-use, 3rd. edition, McGraw Hill Publishing co., Ltd., New York.

Purcell, P. J. (1983) High-rate sludge blanket clarification, Thesis presented for the MEngSc degree, Dept. of Civil Engineering, University College Dublin.

Quek, K. H., Bliss, P. J. and Ball, J. E. (1992) *Inst. Eng. Aust*., CE34, No. 1, 37-56.

Tebbutt, T. H. and Christoulas, D. G. (1975) Water Res., 9, 347-356.

Related reading

Hudson, H. E. Jr. (1981) Water Clarification Processes, Practical Design and Evaluation, Van Nostrand Rheinhold, New York.