**TABLE 11.2** Microstrainer Design Parameters[a]

| Item | Typical value |
|---|---|
| Screen mesh | 20–25 $\mu$m |
| Submergence | 75% of height (66% of area) |
| Hydraulic loading | 12–24 $m^3/m^2/h$ (300–600 $gal/ft^2/d$) of submerged drum surface area |
| Headloss through screen ($h_L$) | 7.5–15 cm (3–6 in.) |
| Max. $h_L$ | 30–45 cm[b] (12–18 in.) |
| Peripheral drum speed | 4.5 m/min at 7.5-cm $h_L$ (15 ft/min at 3-in. $h_L$)<br>40–45 m/min at 15-cm $h_L$ (130–150 ft/min at 6-in. $h_L$) |
| Typical drum diameter | 3m (10 ft) |
| Washwater flow | 2% of throughput at 345 $kN/m^2$ (50 psi)<br>5% of throughput at 100 $kN/m^2$ (14.5 psi) |

[a]After USEPA (1975).
[b]Typical designs provide an overflow to bypass part of the flow when $h_L$ exceeds 15–20 cm (6–8 in.).

degree of clogging, and time, and inversely proportional to the surface area ($A$) of the strainer. These parameters are incorporated into a first-order relation:

$$\frac{dh_L}{dt} = k\frac{Q}{A}h_L \tag{11.4}$$

where
$k$ is a characteristic loss coefficient

The above equation integrates to:

$$h_L = h_0 e^{k\frac{Q}{A}t} \tag{11.5}$$

where
$h_0$ is the headloss of the clean strainer

The loss coefficient for the strainer should be experimentally determined.

Typical design parameters for solids removal from secondary effluents are given in Table 11.2. The USEPA (1975) surveyed a number of microstrainers treating secondary effluent with solids concentrations in the range of 6–65 mg/L and found average removals from 43 to 85%. Microstrainers also find application in the treatment of stormwater runoff and the polishing of effluents (removal of algae) from stabilization pond systems.

## 11.2 SEDIMENTATION

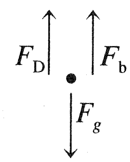
Sedimentation is the physical separation of suspended material from a water by the action of gravity. It is a common operation for water treatment and found in almost all wastewater treatment plants. It is less costly than many other treatment operations.

### 11.2.1 Particle Settling Velocity

Before a basin to settle particles is designed, the settling velocities of the particles must be known. The physical characteristics of a particle determine its settling velocity.

Consider a particle falling in a body of fluid with the following assumptions:

1. The particle is discrete and its size and shape do not change.
2. Infinite size vessel.
3. Viscous fluid.
4. Single particle.
5. Quiescent fluid.




The forces acting on the particle are the effective gravitational force, $F_1$, and the drag force, $F_D$, caused by fluid resistance. The effective gravitational force (downward) is the difference between the gravitational force, $F_g$, and the buoyant force, $F_b$.

$$F_n = F_g - F_b = (\rho_p - \rho) g V_p \tag{11.6}$$

where
- $F_n$ is the net downward force
- $\rho_p$ is the density of the particle
- $\rho$ is the density of water
- $V_p$ is the volume of the particle

The drag force ($F_D$) can be found from dimensional analysis to be

$$F_D = \tfrac{1}{2} \rho C_D A_p v^2 \tag{11.7}$$

where
- $C_D$ is the drag coefficient
- $A_p$ is the cross-sectional area of the particle
- $v$ is the settling velocity of the particle

The force balance applying to the particle while it is accelerating is

$$m\vec{a} = \vec{F}_g + \vec{F}_b + \vec{F}_D$$

where
- $a$ is the rate of acceleration of the particle
- $m$ is the mass of the particle
- the arrows represent vector quantities

Removing the vector notation from this equation and substituting for the forces in the vertical direction results in the governing differential equation.

$$-\rho_p V_p a = -\rho_p V_p \frac{dv}{dt} = -(\rho_p - \rho) g V_p + \tfrac{1}{2} \rho C_D A_p v^2$$

The settling velocity increases in a very short time from 0 to a constant ultimate settling velocity (see Problem 8). Taking a force balance after the ultimate settling velocity has been reached ($a = 0$):

$$(\rho_p - \rho) g V_p = \tfrac{1}{2} \rho C_D A_p v^2 \tag{11.8}$$

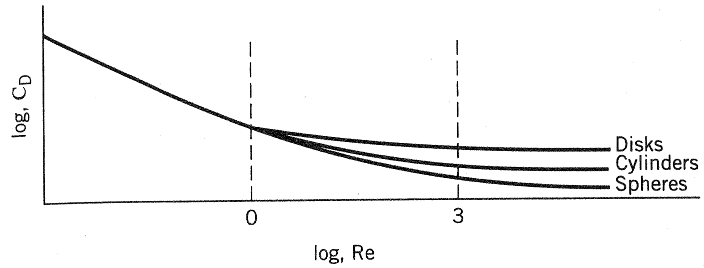



***Figure 11.5*** Variation of $C_D$ with particle geometry.

Solving for $v$:

$$v = \sqrt{\frac{2g}{C_D}\frac{V_p}{A_p}\left(\frac{\rho_p - \rho}{\rho}\right)} \tag{11.9}$$

For a spherical particle of diameter, $d$,

$$v = \sqrt{\frac{4}{3}\frac{gd}{C_D}\left(\frac{\rho_p - \rho}{\rho}\right)} \tag{11.10}$$

The drag coefficient, $C_D$, is not constant but varies with the Reynold's number, Re, and with the shape of the particle.

$$\text{Re} = \frac{\rho v d}{\mu} \tag{11.11}$$

For spherical particles the following relations apply:

$$\text{Re} < 1: \qquad C_D = \frac{24}{\text{Re}} \tag{11.12a}$$

Re <1 is the laminar range also known as the Stoke's range. The next range is the transition between laminar and turbulent settling.

$$1 < \text{Re} < 10^3: \qquad C_D = \frac{24}{\text{Re}} + \frac{3}{\text{Re}^{0.5}} + 0.34 \qquad \text{or} \qquad C_D = \frac{18.5}{\text{Re}^{0.6}} \tag{11.12b}$$

For fully developed turbulent settling:

$$\text{Re} > 10^3: \qquad C_D = 0.34 \text{ to } 0.40 \tag{11.12c}$$

$C_D$ varies with the effective resistance area per unit volume of the particle as shown in Fig. 11.5.

## 11.3 TYPE I SEDIMENTATION

The design of an ideal settling basin is based on the removal of all particles that have a settling velocity greater than a specified settling velocity. The work of Hazen (1904) and Camp (1945) provides the basis of sedimentation theory and basin design. Type I sedimentation refers to discrete particle settling.

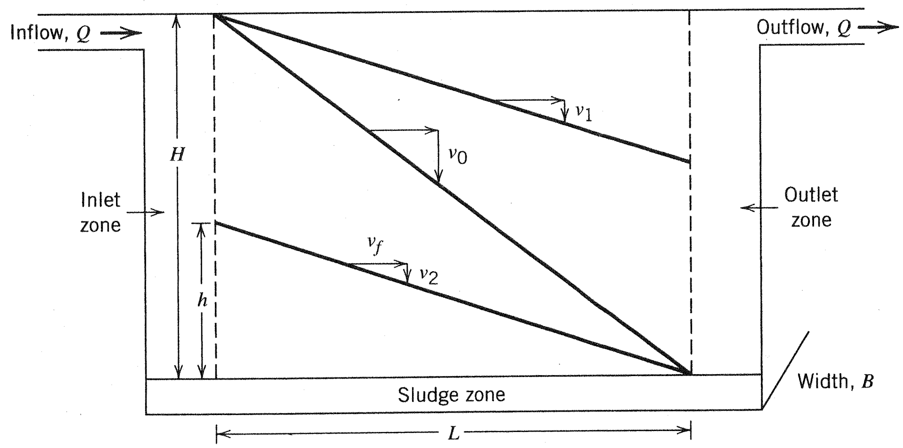



***Figure 11.6*** An ideal horizontal flow sedimentation basin.

### 11.3.1 Theory

A definition sketch for an ideal, horizontal flow, rectangular section basin is given in Fig. 11.6. In Fig. 11.6, $H$ is the effective depth of the settling zone and $v_f$ is the longitudinal velocity of the water. The width of the basin is $B$. The settling velocities $v_1$ and $v_0$ apply to two different particles entering at the top of the basin. The settling velocity $v_2$ applies to a particle entering the settling zone at a height, $h$, above the sludge zone.

There will be dissipation of energy (turbulence) near the entrance as the flow profile through the basin is established. There is assumed to be no settling in the inlet zone. A similar phenomenon occurs at the exit side as the flow streamlines turn upward, and no settling is assumed to occur in the outlet zone. Sludge accumulates in the sludge zone, which is not part of the effective settling zone.

Other important assumptions are as follows:

1. There is uniform dispersion of water and suspended particles in the inlet zone. Therefore, the suspended solids concentration is the same at all depths in the inlet zone.
2. Continuous flow at a constant rate (steady flow) exists.
3. Once a particle enters the sludge zone, it remains there (i.e., there is no resuspension of settled particles).
4. The flow-through period is equal to the detention time, i.e., there is no dead space or short circuiting in the volume above the sludge zone.
5. PF conditions exist.
6. Settling is ideal discrete particle sedimentation.
7. Particles move forward with the same velocity as the liquid.
8. There is no liquid movement in the sludge zone.

The design volume must be related to the influent flow rate and the particle settling velocity. The particle that takes the longest time to remove will be one that enters at the top of the effective settling zone. The design settling velocity is $v_0$, which is the settling velocity of the particle that settles through the total effective depth of the tank in the theoretical detention time. The flow-through velocity is $v_f$.

$$t_d = \frac{V}{Q} \tag{11.13}$$

$$v_f = \frac{Q}{BH} \tag{11.14}$$

Because the particle must travel the length and depth of the basin in the time $t_d$,

$$v_0 t_d = H \tag{11.15a}$$

$$v_f t_d = L \tag{11.15b}$$

Substituting Eq. (11.15b) into Eq. (11.15a) and using Eq. (11.14):

$$\frac{L}{v_f} = \frac{H}{v_0} \qquad \text{and} \qquad v_0 = v_f \frac{H}{L} \qquad \text{or} \qquad v_0 = \frac{QH}{BHL} = \frac{Q}{BL} \tag{11.16}$$

The surface overflow rate, $Q/A_s$ ($A_s$ is the surface area of the basin) is defined by

$$v_0 = \frac{Q}{BL} = \frac{Q}{A_s} \tag{11.17}$$

This proves that the sedimentation basin design is independent of the depth and depends only on the surface overflow or loading rate ($Q/A_s$) for particles with a specified settling velocity $v_0$. From this it also follows that the sedimentation efficiency is also theoretically independent of detention time in the basin. This fact is not a mathematical curiosity. Consider a basin with the flow at the bottom of the basin and uniformly distributed introduced across the plan (surface) area, resulting in an upflow velocity $v_0$. Any particle with a settling velocity greater than $v_0$ will be removed (settled) after being introduced into the basin regardless of the residence time of the water in the basin. Likewise, any particle with a settling velocity less than $v_0$ will eventually exit with the effluent overflowing from the basin.

Horizontal (or radial) flow and upflow are the two possible operational modes of a settling basin. In either case all particles with settling velocities greater than $v_0$ will be removed. In the horizontal flow mode some particles with settling velocities less than $v_0$ will also be removed if they enter the basin at a depth less than $H$. Assumption 1 above is critical to analysis of the total removal. Assume that a particle with settling velocity, $v$, which is less than $v_0$, will travel a vertical distance $h$ in the time $t_d$.

$$h = v t_d \tag{11.18}$$

All particles with settling velocity $v$ that enter at a depth $h$ or lower will be removed. The criterion for removal of particles with this settling velocity is

$$\frac{h}{H} \leq \frac{v}{v_0} \tag{11.19}$$

Because all particles with settling velocity $v$ are uniformly distributed throughout the inlet depth (assumption 1), the fractional removal, $r$, of particles with this settling velocity is

$$r = \frac{h}{H} = \frac{v}{v_0} \tag{11.20}$$

The settling velocity distribution for a suspension can be determined from a column settling test as described in Section 11.4. The results of the test provide data to construct

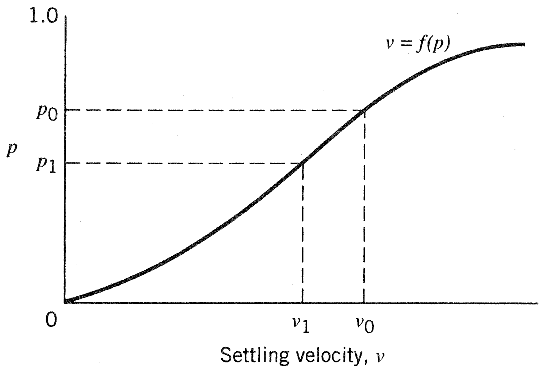



***Figure 11.7*** Settling velocity curve for a suspension, where $p$ is the weight fraction of particles with settling velocity less than stated velocity.

a plot as shown in Fig. 11.7, which is a cumulative settling velocity frequency distribution.

The total removal $R$ (fraction by weight) of all particles is the sum of the fractional removals of each fraction of particles. Applying Eq. (11.20) to each fraction $\Delta p$,

$$R = 1 - p_0 + \sum r_i$$

$$R = 1 - p_0 + \frac{v_0 + v_1}{2v_0}(p_0 - p_1) + \frac{v_1 + v_2}{2v_0}(p_1 - p_2) + \cdots + \frac{v_i + v_{i+1}}{2v_0}(p_i - p_{i+1}) + \cdots \quad (11.21a)$$

which in the limit is

$$R = (1 - p_0) + \frac{1}{v_0}\int_0^{p_0} v\,dp \quad (11.21b)$$

where

$p_0$ is the fraction of particles by weight with a settling velocity equal to or less than $v_0$

A polynomial fit can be applied to data used to construct the curve in Fig. 11.7 to use Eq. (11.21b).

For circular tanks with an inlet in the middle and radial flow under the same assumptions as above, it can also be shown that

$$v_0 = \frac{Q}{A_s} \qquad r = \frac{h}{H} = \frac{v}{v_0}$$

and the same overall removal expressions (Eqs. 11.21a and 11.21b) apply (see Problem 5).

Equation (11.17) also holds for vertical flow tanks. Particles with a settling velocity less than the upflow velocity are entrained in the upward flow and washed out of the system.

## 11.3.2 Overall Removals and Overflow Rates

When the settling velocity distribution of the suspension is known (such as given in Fig. 11.7), it is a routine matter to construct a plot of overall removals versus the

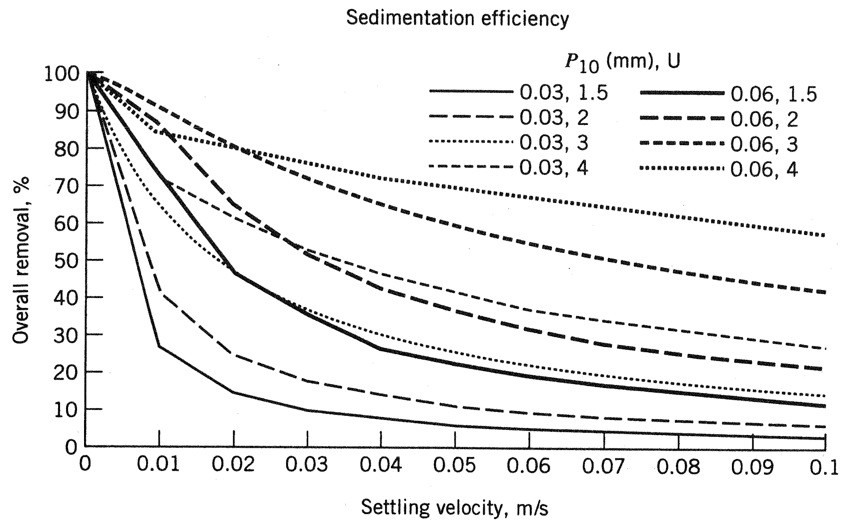



***Figure 11.8*** Overall removal as a function of surface overflow rate. See text for definition of terms.

surface overflow rate using Eqs. (11.21a) or (11.21b). Figure 11.8 shows typical curves. The general shape of these curves suggests that a parametric equation can be used to describe them. A solids size distribution is characterized by the effective size, $P_{10}$ (the size through which 10% by weight of the particles pass) and the uniformity coefficient, $U$ (the ratio of sizes through which 60 and 10% by weight of the particles, respectively, pass). These parameters and size distributions are discussed in Section 14.2.1. Bhargava and Rajagopal (1989) have determined a correlation that readily provides the overall removal curve when the particle size distribution and specific gravity (s.g.) of a suspension are known. The correlation should be used for suspensions that contain a large quantity of inorganic matter such as surface waters or raw sewage.

The equation for removal determined by Bhargava and Rajagopal (1989) for a temperature of 30°C and particle s.g. of 2.65 is

$$\frac{1}{R} = \left[\frac{U}{177.88 + 44.71U}\right] + \left\{\frac{1}{\exp\left[(3.186 \times 10^{-3}U + 2.036)\ln P_{10} + \exp\left(\dfrac{\ln U + 4.627}{2.809}\right)\right]}\right\} v_0 \qquad (11.22)$$

where

$P_{10}$ is expressed in mm

$v_0$ is expressed in $m^3/m^2/s$

For temperatures different from 30°C or particle s.g.s different from 2.65, the settling velocity in the second term of Eq. (11.22) must be adjusted to the equivalent settling velocity at 30°C and s.g. of 2.65 using Eq. (11.10). For a particle with a terminal settling velocity in the Stoke's range, the temperature and s.g. correction factors ($C_T$ and $C_{sg}$, respectively) to multiply $v_0$ are

$$C_T = \frac{\nu_T}{\nu_{30}} \qquad (11.23a) \qquad\qquad C_{sg} = \frac{2.65 - 1}{S_s - 1} \qquad (11.23b)$$

where

$S_s$ is the s.g. of the suspension

$\nu$ is kinematic viscosity

The s.g. of inorganic particles is near 2.65 and the s.g.s of organic particles are normally in the range of 1.001–1.01. For mixed suspensions, the overall removal will be the weighted sum of the removals of each fraction. Under conditions of a typical settling test it may be assumed that the organic matter removal is about 5%. The TSS–VSS analysis may be used to discriminate between inorganic and organic matter. When inorganic solids predominate, Eq. (11.10) can be used to calculate the nominal diameters of the fractions from the settling velocity distribution.

---

**■ Example 11.1 Overall Removal in a Sedimentation Basin**

What is the overall removal of a sewage suspension that contains 76% inorganic matter with a s.g. of 2.557 and 24% organic matter with a s.g. of 1.096 at a temperature of 28.5°C and surface overflow rate of 0.008 $m^3/m^2/s$? The $P_{10}$ and $U$ values at 30°C for the inorganic particles were found to be 0.057 5 mm and 1.315, respectively, based on a settling test.

From Eq. (11.22), the overall removal of inorganic solids with specified $P_{10}$ and $U$ values and a s.g. of 2.65 is calculated to be 69.3%. For the mixed suspension, the overall solids removal at 30°C and $v_0 = 0.008$ $m^3/m^2/s$ will be

$$R = 69.3 \times 0.76 + 5.0 \times 0.24 = 53.9\%$$

The removal of the inorganic solids ($R_I$) is 52.7%.

Correcting the overflow rate to 28.5°C and a s.g. of 2.557,

$$R_{28.5,2.557} = \frac{1}{5.556 \times 10^{-3} + \left(\dfrac{0.828 \times 10^{-6}}{0.800 \times 10^{-6}}\right)\left(\dfrac{2.65 - 1}{2.557 - 1}\right)\left(\dfrac{0.008}{0.902\ 5}\right)} = 65.5$$

$$R_I = 65.5 \times 0.76 = 49.8\%$$

and the overall removal for the suspension is 51.0%.

---

Theory dictates that increasing the surface area of a settling basin will improve its performance. Lamella and tube clarifiers, discussed in Section 11.4.2, exploit the concepts from the basic theory resulting in the design of clarifiers with very high loading rates.

## 11.4 TYPE II SEDIMENTATION

Under quiescent conditions suspended particles in many waters exhibit a natural tendency to agglomerate or the addition of chemical agents promotes this tendency. This phenomenon is known as flocculent or type II sedimentation. Analysis of type II sedimentation proceeds from the principles of type I sedimentation.

As particles settle and coalesce with other particles, the sizes of particles and their settling velocities will increase. The trajectory traced by a settling particle will be

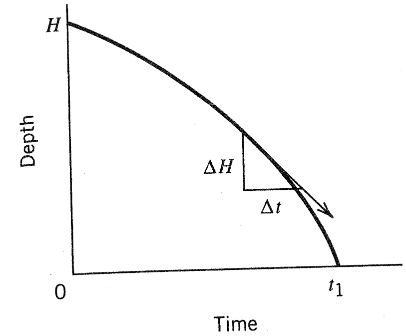



***Figure 11.9*** Settling trajectory in type II sedimentation.

curvilinear (Fig. 11.9) because of the increase in its settling velocity as other particles attach to it.

The instantaneous settling velocity is the tangent to the curve. The average settling velocity for the particle in Fig. 11.9 is

$$\bar{v} = \frac{H}{t_1} \tag{11.24}$$

The average settling velocity distribution for the suspension is continually changing with time as shown in Fig. 11.10.

To design a basin for flocculent settling, the average settling velocity distribution variation with time must be found to calculate the total removal as time (or volume of the basin) increases. At some point an incremental increase in the volume of the basin (which increases the detention time) will not produce a significant increase in the amount of solids removed. There is no theoretical means of predicting the amount of flocculation and settling velocity distribution variation for a suspension. A laboratory analysis as described in the following section is required.

### *Laboratory Determination of Settling Velocity Distribution*

The water to be analyzed must have the same coagulants and other agents added that will be applied in the field situation. The suspension is mixed and added to a column that has approximately the same depth as the anticipated settling basin. Because type II sedimentation is time–depth dependent, more representative settling curves are obtained when the column depth is near the prototype basin depth.

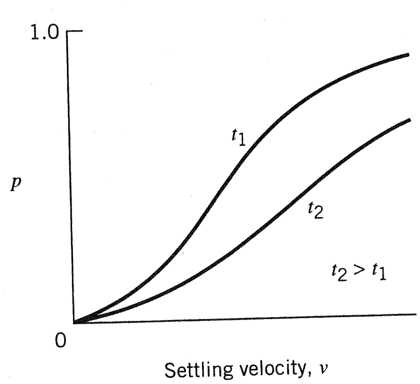



***Figure 11.10*** Settling velocity distribution at various times.

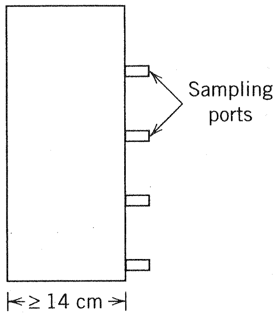



*Figure 11.11* Settling column.

The column (Fig. 11.11) is normally made of clear plastic so that one may visually observe the process. Sampling ports are uniformly spaced along the length of the column. The bottom port will provide samples that are indicative of the compaction of the settled sludge. The effective settling depth is the depth above the bottom port. The column internal diameter should be at least 14 cm (5.5 in.) to avoid bridging of the suspension and other wall effects. After the initial sample is taken, samples are taken from each port at uniform time intervals, noting the time and port number.

The volume of samples removed from the column causes the water surface elevation to descend, which should be accounted for in processing the data.

### 11.4.1 Type II Sedimentation Data Analysis

The analysis of data gathered as outlined in the previous section is best presented by example. The object of the analysis is to obtain a plot of the settling trajectories for various fractions of the suspended solids. Then the total removals at any time may be estimated. For a column with a total depth of 240 cm (7.87 ft) and sampling ports spaced at 60-cm (1.97-ft) intervals, the data in Table 11.3 have been obtained. The effective depth of the sedimentation basin under consideration is 1.8 m (5.91 ft). The initial concentration of SS was 430 mg/L.

**TABLE 11.3** Raw Data

| | Concentration, mg/L | | |
|---|---|---|---|
| Time min | 60 cm (1.97 ft) | 120 cm (3.94 ft) | 180 cm (5.91 ft) |
| 5 | 357 | 387 | 396 |
| 10 | 310 | 346 | 366 |
| 20 | 252 | 299 | 316 |
| 30 | 198 | 254 | 288 |
| 40 | 163 | 230 | 252 |
| 50 | 144 | 196 | 232 |
| 60 | 116 | 179 | 204 |
| 75 | 108 | 143 | 181 |

**TABLE 11.4** Percentage Solids Removed

| Time min | Solids removed, % 60 cm (1.97 ft) | 120 cm (3.94 ft) | 180 cm (5.91 ft) |
|---|---|---|---|
| 5 | 17.0 | 10.0 | 7.9 |
| 10 | 28.0 | 19.5 | 14.9 |
| 20 | 41.5 | 30.5 | 26.0 |
| 30 | 54.0 | 40.9 | 33.0 |
| 40 | 62.0 | 46.5 | 41.4 |
| 50 | 66.5 | 54.4 | 46.0 |
| 60 | 73.0 | 58.6 | 52.5 |
| 75 | 75.0 | 66.7 | 57.9 |

The first step is to convert the concentrations to percentage removals at each depth.

$$r_{ti,d}(\%) = \frac{C_0 - C_{ti,d}}{C_0} \times 100$$

where
the subscript ti,d indicates time i and depth, d, respectively

The results of these calculations are listed in Table 11.4.

The desired plot of the settling trajectories of various fractions of the suspended solids (see Fig. 11.13) can be obtained by constructing a depth–time plot with percentage solids removed as a parameter. An intermediate step improves the interpolation that is required (Ramalho, 1977).

A plot of the percentage solids removed at each depth versus time is constructed using the data in Table 11.4. This is done in Fig. 11.12. A smooth curve is drawn between the data points for each depth.

Figure 11.12 can be used to easily estimate the time required to attain a specified removal at a given depth. The times to attain a given removal at each depth are found by extending a horizontal line from the removal to the curves and dropping vertical

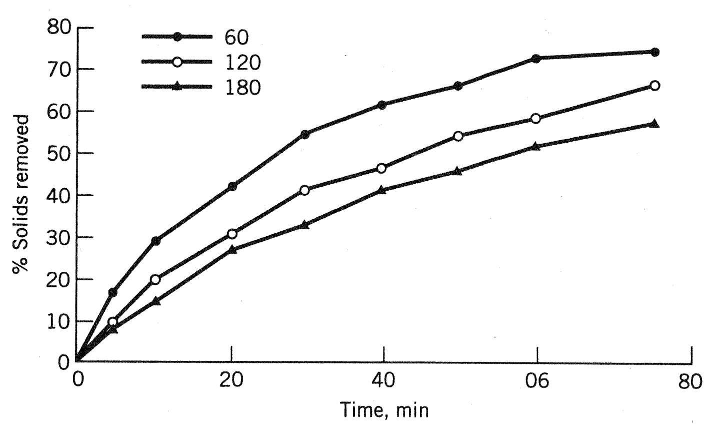



***Figure 11.12*** Percentage SS removed at each depth.

**TABLE 11.5** Interpolated Percentage Solids Removed

| % SS removed | t, min 60 cm (1.97 ft) | 120 cm (3.94 ft) | 180 cm (5.91 ft) |
|---|---|---|---|
| 5 | 1.2 | 2.5 | 3.7 |
| 10 | 2.5 | 5.0 | 6.5 |
| 20 | 6.7 | 11.0 | 14.5 |
| 30 | 11.7 | 19.0 | 25.0 |
| 40 | 18.0 | 30.0 | 39.0 |
| 50 | 27.0 | 44.0 | 56.5 |
| 60 | 38.5 | 61.5 | 77.5 |
| 70 | 55.0 | 87.5 | — |
| 75 | 75.0 | — | — |

lines at the intersections. These times are then tabulated for the removals at each depth (Table 11.5).

Using the data in Table 11.5, isoconcentration (or isoremoval) lines are now constructed on a depth versus time plot (Fig. 11.13).

The curves on Fig. 11.13 trace the settling trajectories of particulate fractions of the suspension. The actual particle makeup of each fraction is continuously changing as the particles coalesce and these curves represent the gross phenomena.

Now for any time, a *p–v* plot similar to Fig. 11.7 can be made and the overall removal for the suspension can be determined in the same manner outlined for discrete particle settling. The data from Fig. 11.13 can be used directly to estimate the total removal. To find the total removal at any chosen time, a vertical line is projected upward. It is most convenient to choose times that are at the end of an isoconcentration line. (Why?) When a time of 39 min is chosen for the data, it is seen that 40% of the particles are completely removed; i.e., 40% of the particles had an average settling

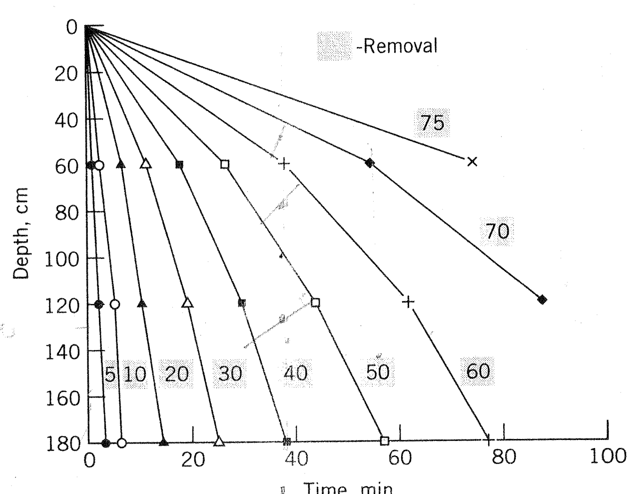



***Figure 11.13*** Isoconcentration curves.

velocity greater than or equal to

$$\frac{180 \text{ cm}}{39 \text{ min}} = 4.62 \text{ cm/min} \qquad \left(\frac{5.9 \text{ ft}}{39 \text{ min}} = 0.152 \text{ ft/min}\right)$$

in the first 39 min of settling.

The other fractions are partially removed. To estimate the removal of these fractions, median lines should be drawn between the isoconcentration curves. Where possible the median lines should be located based on the average depth between two isoconcentration lines at a given time. For example, at a time of 57 min the 50 and 60% isoconcentration lines intersect the vertical at depths of 180 and 106 cm (5.90 and 3.48 ft), respectively. The median line between these isoconcentration lines should pass through the point (57 min, 143 cm) (57 min, 4.69 ft).

The fractional removal of the 40–50% fraction (10% of the particles) is calculated by reading the depth at the intersection of the vertical and the median isoconcentration lines for this fraction (130 cm; 4.27 ft). This is the average depth that this fraction reached in 39 min. In a manner analogous to the discrete particle settling analysis, the average settling velocity of a fraction compared to the design settling velocity will dictate the percentage removal of the fraction.

$$\frac{v_i}{v_0} = \frac{\dfrac{d_i}{t_d}}{\dfrac{D}{t_d}} = \frac{d_i}{D} \tag{11.25}$$

where

$d_i$ is average depth reached by the $i$th fraction in the time $t_d$
$D$ is the total effective settling depth
$v_i$ is the average settling velocity of the $i$th fraction

The fractional removal, $r_i$, of the fraction, $\Delta p_i$, is

$$r_i = \frac{d_i}{D} \Delta p_i \tag{11.26}$$

The fractional removals for the data above in a time of 39 min are

$$\frac{130}{180}(50 - 40) = 7.2$$

$$\frac{78}{180}(60 - 50) = 4.3$$

$$\frac{48}{180}(70 - 60) = 2.7$$

$$\frac{30}{180}(75 - 70) = 0.8$$

The same fractional removals would be obtained using U.S. units.

The removal of the fraction between 75 and 100% is small and is ignored. The upper value of 100% removal would probably exist only at the surface plane of the volume. A lower upper limit would be dictated by the presence of colloidal particles that are practically unsettleable.

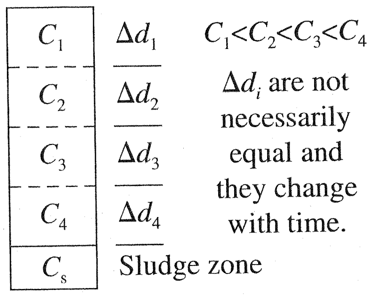



***Figure 11.14*** Solids concentrations in the column.

The total removal, $R$, is in this case

$$R = r_0 + \sum r_i = 40 + 7.2 + 4.3 + 2.7 + 0.8 = 55.0\% \tag{11.27}$$

This procedure is repeated for different times and the overall removals at each selected time are tabulated to construct a graph as shown later in Fig. 11.15.

## *Alternative Method for Calculating Total Removal*

As noted, the method just given is analogous to the procedure for discrete particle settling. An equivalent method to find the total removal is to examine the amount of removal in each section of the column (Fig. 11.14) and sum them.

The initial suspended mass, $M_0$, in the column is

$$M_0 = C_0 A D \tag{11.28}$$

where
- $A$ is the cross-sectional area of the column
- $C_0$ is the initial concentration of suspended particles

Referring to Fig. 11.14, the suspended mass, $M_t$, at any time is

$$\begin{aligned} M_t &= C_1V_1 + C_2V_2 + C_3V_3 + C_4V_4 \\ &= A(C_1\Delta d_1 + C_2\Delta d_2 + C_3\Delta d_3 + C_4\Delta d_4) \end{aligned} \tag{11.29}$$

The percentage removal on any isoconcentration line in Fig. 11.13 is

$$r_i = \frac{C_0 - C_i}{C_0} \times 100$$

A vertical line projected upward from a chosen time in Fig. 11.13 intersects isoconcentration lines that determine each column section length, $\Delta d_i$. The average removal in a section of column is the average of the isoconcentrations that define $\Delta d_i$. Assuming the section numbering to start from the lower depth, the total removal $R$ is

$$R = \frac{\Delta d_1}{D}\left(\frac{r_1 + r_2}{2}\right) + \frac{\Delta d_2}{D}\left(\frac{r_2 + r_3}{2}\right) + \cdots \frac{\Delta d_i}{D}\left(\frac{r_i + r_{i+1}}{2}\right) + \cdots \tag{11.30}$$

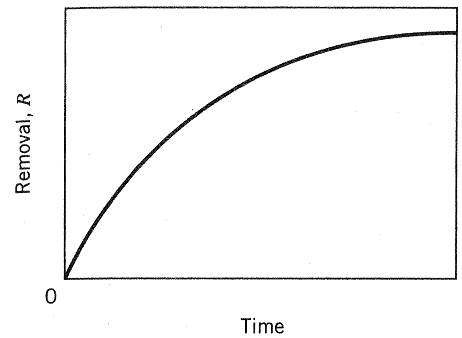



***Figure 11.15*** Overall removal versus detention time.

Equation (11.30) is applied for different times and the results for overall removal at various times can be tabulated and plotted as shown in Fig. 11.15.

### *Sizing the Basin*

A graph of total removal ($R$) versus time will provide a design curve. A typical graph is shown in Fig. 11.15. The removal curve will eventually become nearly horizontal as time increases. The design point is in the region where the marginal increase in removal is less than the marginal increase in time, which is equivalent to the size of the clarifier. Costs of the clarifier compared to costs associated with other solids removal processes will determine the optimum design point.

The design detention time from the laboratory column is equivalent to a design settling velocity of $D/t_d$, which is also equal to the design surface loading rate ($Q/A_s$). To translate the laboratory data to the field, where nonideal flow conditions exist and sedimentation does not occur under completely quiescent conditions, safety factors of 1.25–1.75 are applied to the detention time and the surface overflow rate.

1. Multiply the design $t_d$ based on the column performance by 1.25–1.75.
2. Divide the design $Q/A_s$ based on the column performance by 1.25–1.75.

Application of the safety factors will result in an increase in the surface area of the settling basin.

## 11.4.2 Tube and Lamella Clarifiers

Theoretically the efficiency of a clarifier is independent of depth as discussed previously. Fundamentally, this is because the liquid upflow velocity in the basin must be less than the velocity of the slowest settling particle that is to be removed. The pioneer environmental engineer, T. R. Camp (1945), attempted to exploit this concept by inserting a number of subfloors into horizontal sedimentation basins to increase the surface area. Practically, sludge removal (each floor would need a scraping device) was a problem and the idea became dormant until the 1960s, with the development of tube or lamella settlers, which are an interesting and effective application of the concept.

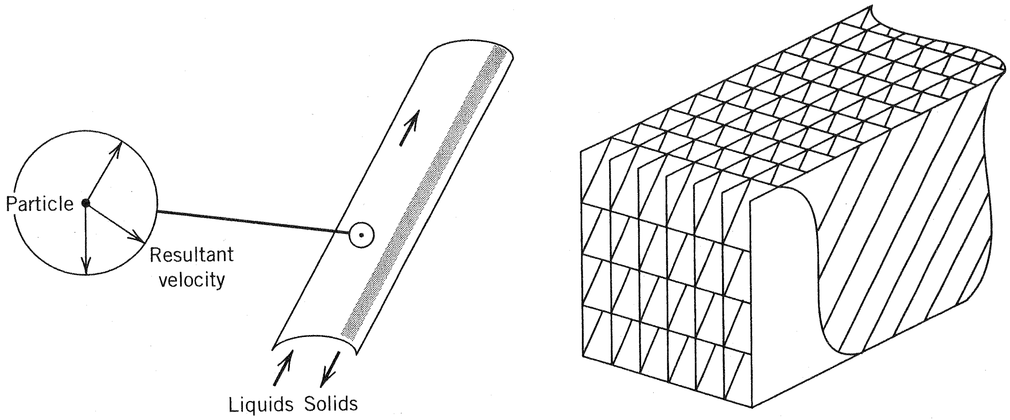



*Figure 11.16* Tube clarifier.

Tube settlers are plastic (PVC) modules with uniformly spaced, inclined channels (Fig. 11.16). Lamella settlers have uniformly spaced, inclined panels (Fig. 11.17). Lamella settlers can be made with plastic, rawhide, or other available resilient materials. Both types of clarifiers solve the problem of sludge removal. The resultant velocity on a particle from the upward flow of water and the vertically downward settling velocity of the particle directs the particle to the bottom wall of a tube or toward the lamella. The particle then slides down the surface and exits at the bottom to be collected in a sludge chamber (see the insert in Fig. 11.16).

The theory of these clarifiers is discussed by Yao (1970) but is beyond the scope of this textbook. The increased surface area available in tube or lamella clarifiers allows surface loading rates based on plan area that are two or more times higher than loading rates for conventional clarifiers with the same or better performance (Richter, 1987). The Reynold's number in the tubes or between lamella plates is very low compared to the Reynold's number for a conventional clarifier, which will be in the turbulent range. Turbulence effectively decreases the settling velocity of particles and causes resuspension of settled particles by scour. Existing clarifiers can be upgraded

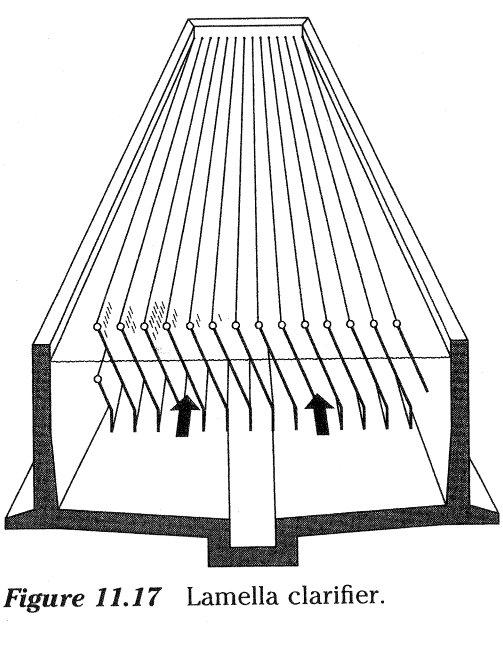


*Figure 11.17* Lamella clarifier.

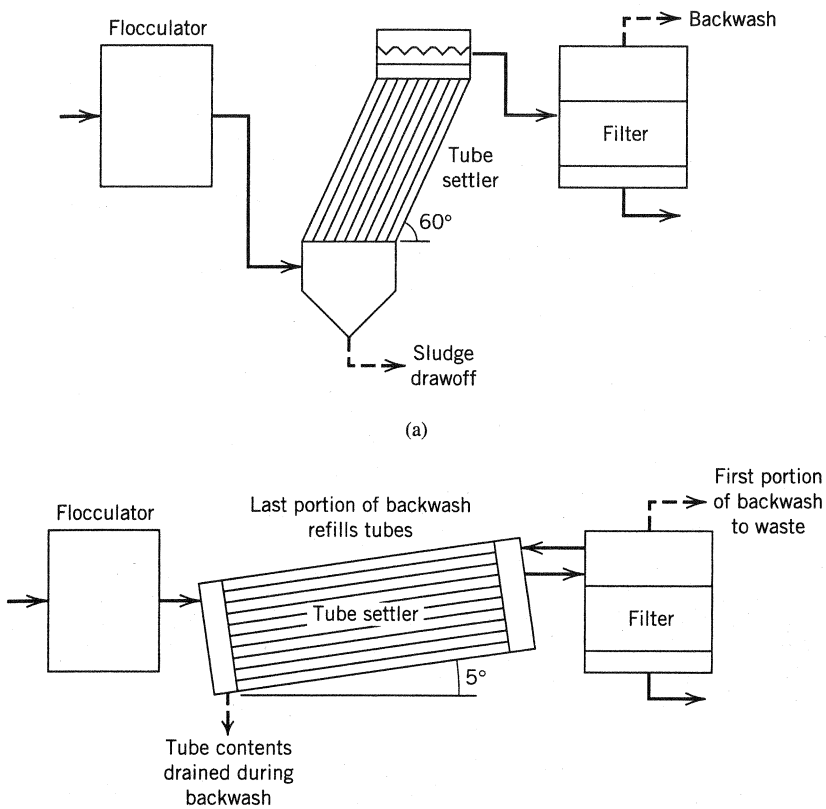



***Figure 11.18*** Tube settler configurations. (a) Large angle (60°); (b) small angle (5°).

to higher loading rates by the installation of a tube module or lamella. Lamellae can be installed in a concentric array in circular clarifiers. Both tubes and lamellae can be installed in horizontal flow sedimentation basins and upflow solids contact clarifiers. There are a number of companies that supply ready-made tube modules.

The settlers are installed in two different ways. Steeply inclined settlers at angles of 60° or more are stand alone units that are essentially self-cleansing (Fig. 11.18a). From time to time a high-pressure wash may need to be applied to the module to remove particles and biological growth that have accumulated on the settler walls.

The other alternative is to reduce the angle of inclination to a small value, but the angle must be high enough that solids are not washed out with the clarified effluent (Fig. 11.18b). Angles as small as 5° have been used. Solids will not travel significantly downward in a clarifier at this angle of inclination. But these clarifiers are designed in conjunction with rapid sand filters (Chapter 14) such that the discharge of water from backwashing the filter is directed through the clarifier to clean the accumulated solids from the walls. Normal backwash quantities and velocities are adequate to scour the tubes. The first portion of the backwash is not directed into the tube settler because it is laden with solids removed from the filter; the intermediate portion is used to wash the tube module; and the latter portion of the backwash water is used to fill the tube settler.

Conventional inlets can be used for lamella clarifiers when they are configured as indicated in Fig. 11.17. However, in tube settlers, front end inlets produce flow patterns that do not distribute the flow uniformly to the clarifier module (Fig. 11.19). If lamellae are oriented perpendicular to the inlet flow the same problem will occur.

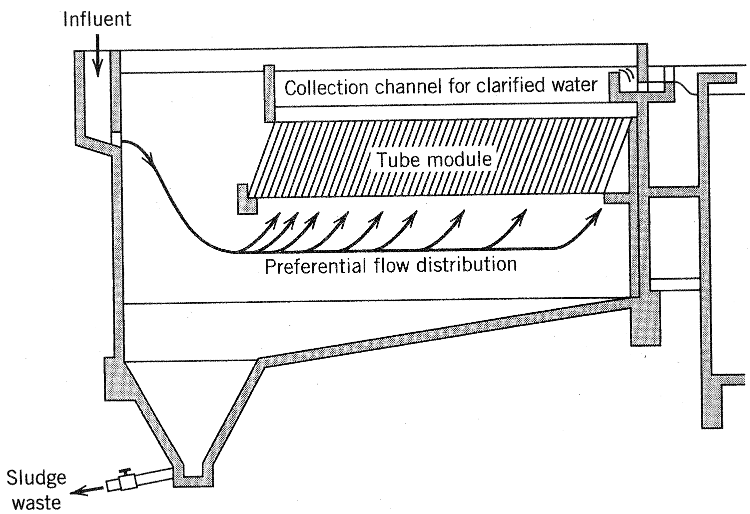



*Figure 11.19* Flow distribution from a conventional inlet in a tube settler. From Di Bernardo, L. and ABES, ed. (1993).

An improved inlet arrangement used in Brazil is shown in Fig. 11.20 (Di Bernardo, 1993). Flow is evenly discharged along the length of the clarifier by a series of ports. Sludge is withdrawn through tubes (minimum 38-mm or 1.5-in. diameter) equidistantly spaced along the bottom of the clarifier. Di Bernardo (1993) has also given other sludge removal designs for tube settlers.

## 11.5 TYPE III SEDIMENTATION: ZONE SETTLING

When solids concentrations become high, forces between the particles become significant and settling is hindered by the additional resistance to movement provided by

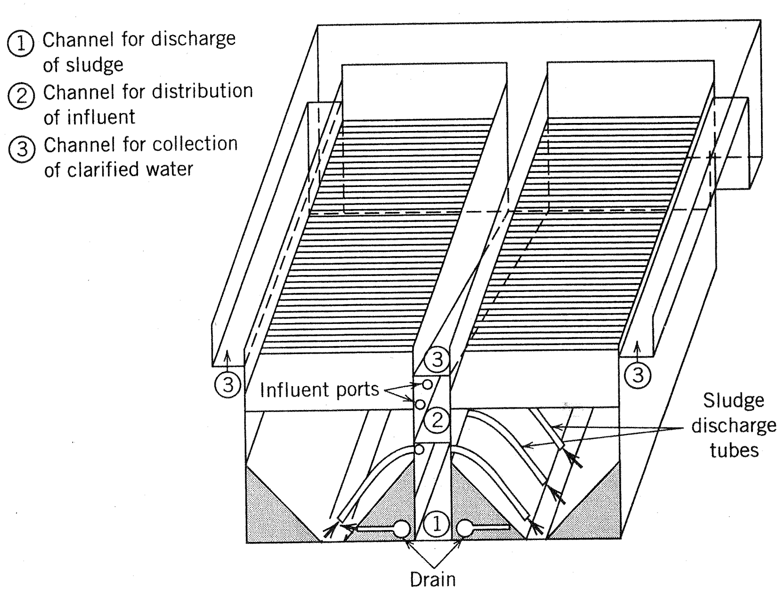



*Figure 11.20* Influent distribution for tube settler. From Di Bernardo, L. and ABES, ed. (1993).

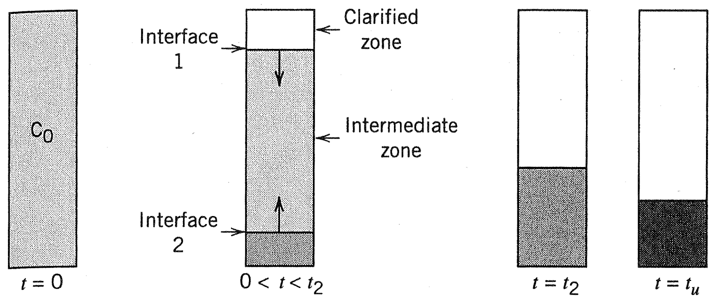



***Figure 11.21*** Progression of zone sedimentation.

other particles. The suspension tends to settle *en masse.* A clarified zone exists in the upper portion of the clarifier, followed by a zone in which the suspension is moving down and concentrating toward a bottom layer, where slowly compacting sludge exists. This behavior is exhibited by flocculent suspensions with solids concentrations above 500 mg/L. Effluents from biological treatment units such as activated sludge processes or trickling filters exhibit zone settling. Properly designed and operated zone clarifiers can produce a clarified effluent with a very low concentration of suspended solids. The only further treatment that normally may be applied to the clarified effluent from a settler receiving effluent from a biological treatment process is disinfection.

The progression of zone sedimentation in a batch process is shown in Fig. 11.21. Starting from a uniform concentration of $C_0$, the formation of two interfaces becomes apparent as the suspension settles. A relatively clear water exists above the top interface and a concentrated sludge exists below the bottom interface. These two interfaces are propagated downward and upward, respectively, as time goes on. At time $t_2$ the interfaces meet. After this time, compaction of the sludge occurs relatively slowly until its ultimate compaction limit is reached.

The data for zone sedimentation are gathered from a laboratory column settling test using a graduated cylinder. A 1- or 2-L cylinder may be used, although the latter is recommended to prevent bridging and minimize wall effects. To further prevent these two phenomena from influencing the results, gentle stirring at around 1 rpm is recommended. The initial concentration of the suspension is measured and the height of the top interface is monitored with time.

Continuous flow clarifiers that receive these suspensions are normally designed to accomplish sedimentation and some thickening of the settled sludge. These clarifiers are usually circular in plan, which facilitates collection of sludge by scrapers that travel around the floor of the clarifier. A schematic of a clarifier, identifying the zones that will occur in a clarifier receiving a continuous flow, is shown in Fig. 11.22 and schematic views of a circular secondary clarifier are shown in Fig. 11.23. In a continuous flow situation with relatively constant influent flow and solids concentration, a dynamic steady state condition is established as solids continuously move through the sludge blanket into the concentrated sludge zone. In zone I there is an upflow velocity caused by fluid movement; in zone II, it is assumed that the net movement of fluid is downward.

It is the settling velocity of the **suspension** that governs the design of the clarifier in zone I (the sludge blanket zone). The drawoff of the thickened sludge at the bottom of the tank influences settling in zone I. However, the thickening of sludge that occurs in zone II (concentrated sludge zone) is not influenced by the drawoff of clarified effluent at the top of the tank. The detention time of sludge in this zone is independent of the detention time in zone I.

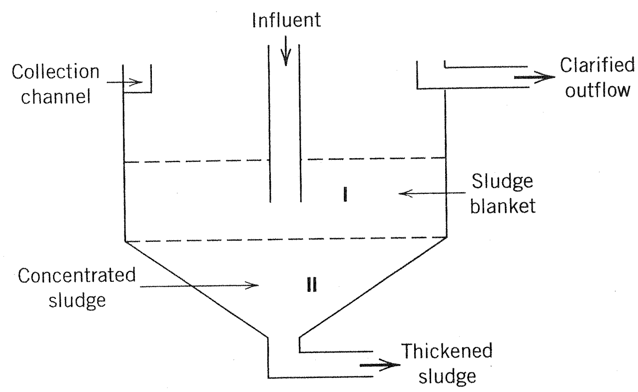



***Figure 11.22*** Clarifier for zone sedimentation.

## 11.5.1 Analysis of Zone Sedimentation

The governing principle for design of a sedimentation basin is $v_s \leq Q/A_s$. The limiting settling velocity of the suspension will be the design settling velocity, $v_s$, which will dictate the surface area of the basin. A plot of the top interface height with time will result in a curve as shown in Fig. 11.24.

The slope of the line at any time gives the settling velocity of the interface. Because compaction occurs in zone II the problem is to find the limiting settling velocity of the suspension before compaction begins. Kynch (1952) analyzed type III behavior. The important assumption in his analysis was that settling velocity of a layer is solely a function of the concentration of solids in the layer. The solids flux, $N$ (mass/area/time), is a function of concentration and velocity.

$$v = v(C) \qquad N = vC$$

Examine the elemental volume of liquid in the cylinder in Fig. 11.25. If $N$ is taken to be positive downward then

$$N = -Cv \tag{11.31}$$

because $v$ will be a negative number.

The mass balance for the volume is

$$\text{IN} - \text{OUT} + \text{GENERATION} = \text{ACCUMULATION}$$

$$\left(N + \frac{\partial N}{\partial y}dy\right)A - NA + 0 = \frac{\partial C}{\partial t}A\,dy \tag{11.32}$$

where

$y$ is the vertical distance

There is no production or destruction of solids. Expanding and simplifying the above equation, the governing differential equation is

$$\frac{\partial N}{\partial y} = \frac{\partial C}{\partial t} \tag{11.33}$$

Differentiating Eq. (11.31) with respect to depth,

$$\frac{\partial N}{\partial y} = -\frac{\partial (Cv)}{\partial y} = -v\frac{\partial C}{\partial y} - C\frac{\partial v}{\partial y} \tag{11.34}$$

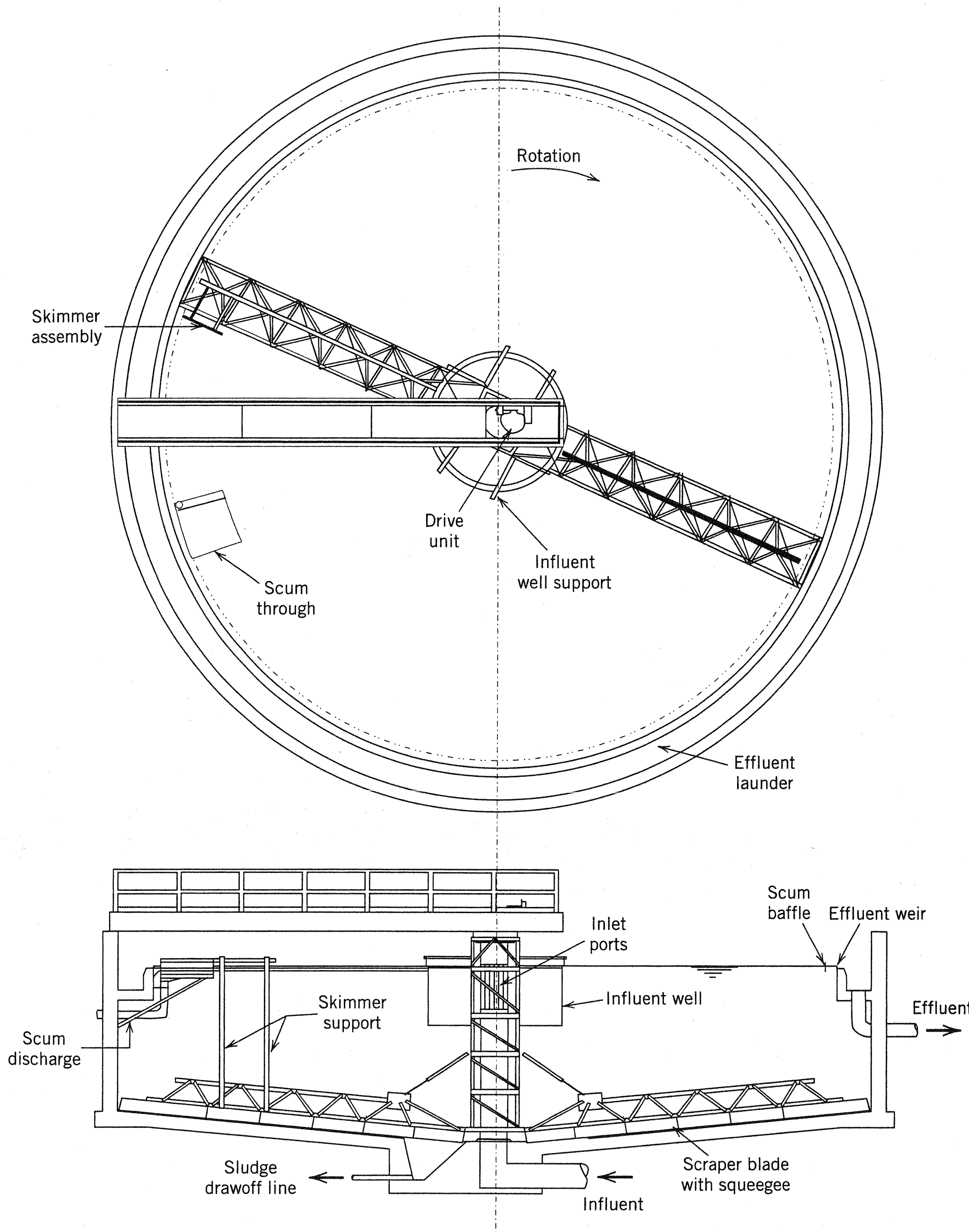



***Figure 11.23*** Final clarifier for an activated sludge process. Courtesy of Envirex.

Because $v = v(C)$, $N$ is only a function of concentration and, using the chain rule,

$$\frac{\partial N}{\partial y} = \frac{\partial N}{\partial C}\frac{\partial C}{\partial y} \qquad (11.35a) \qquad \text{also,} \qquad \frac{\partial v}{\partial y} = \frac{\partial v}{\partial C}\frac{\partial C}{\partial y} \qquad (11.35b)$$

From Eq. (11.33),

$$\frac{\partial C}{\partial t} - \frac{\partial N}{\partial y} = 0 \quad \text{or using Eq. (11.35a)} \quad \frac{\partial C}{\partial t} - \frac{\partial N}{\partial C}\frac{\partial C}{\partial y} = 0 \qquad (11.36)$$

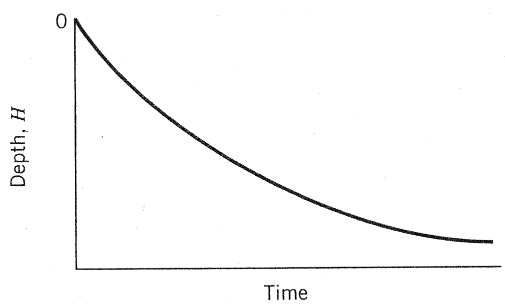



***Figure 11.24*** Interface settling rate in type III sedimentation.

Using Eq. (11.34),

$$\frac{\partial C}{\partial t} + v\frac{\partial C}{\partial y} + C\frac{\partial v}{\partial y} = 0$$

Substituting Eq. (11.35b) into this equation,

$$\frac{\partial C}{\partial t} + v\frac{\partial C}{\partial y} + C\frac{\partial v}{\partial C}\frac{\partial C}{\partial y} = 0 \qquad \text{and} \qquad \frac{\partial C}{\partial t} + \left(v + C\frac{\partial v}{\partial C}\right)\frac{\partial C}{\partial y} = 0 \quad (11.37)$$

The expression in parentheses in Eq. (11.37) can be obtained by differentiating Eq. (11.31) with respect to $C$.

$$\frac{\partial N}{\partial C} = -\frac{\partial (Cv)}{\partial C} = -v - C\frac{\partial v}{\partial C} = -V \quad (11.38)$$

Equation (11.38) is the definition of $V$, which is discussed later. From Eqs. (11.36) and (11.38) it is also found that

$$\frac{\partial C}{\partial t} - \frac{\partial N}{\partial C}\frac{\partial C}{\partial y} = 0 = \frac{\partial C}{\partial t} + V\frac{\partial C}{\partial y} \quad (11.39)$$

From this development, it is seen that $V$ is only a function of concentration because $v = f_1(C)$ and $\partial v/\partial C = f_2(C)$.

The interface between the clarified water and suspended solids zones is moving down at a velocity, $v = -dy/dt$. What is $V$?

Considering Eq. (11.39) and letting $V = K_1$, a constant, implies that concentration $C$ is constant. Therefore,

$$C(y, t) = K_2 \qquad \text{if } V = K_1$$

Taking the differential of $C$,

$$dC = \frac{\partial C}{\partial y}dy + \frac{\partial C}{\partial t}dt = 0 \text{ along } C = K_2 \quad (11.40)$$

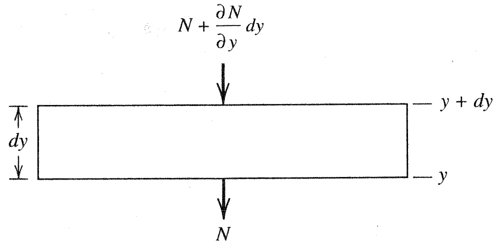



***Figure 11.25*** Elemental volume of liquid in the cylinder.

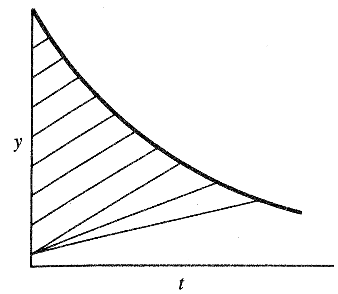



***Figure 11.26*** Rate at which layers of different concentration are propagated to the surface.

This results in

$$\frac{\partial C}{\partial y}\frac{dy}{dt} + \frac{\partial C}{\partial t} = 0 \tag{11.41}$$

When Eq. (11.41) is compared with Eq. (11.39) it is seen that $V = dy/dt$ along lines of constant $C$. Elementary calculus shows this to be true for Eq. (11.39). These lines are described by

$$\int_{y_0}^{y} dy = V \int_0^t dt$$

which integrates to

$$y = y_0 + Vt \tag{11.42}$$

Equation (11.42) holds along each line where $C$ is a constant. On a plot they appear as shown in Fig. 11.26. The lines have a constant slope $V$ and terminate when they meet the interface curve. The lines actually describe the rate at which a layer of a given concentration $C$ is propagated to the surface of the suspension. Because the initial concentration of the suspension was uniform, $V$ is constant for all lines (i.e., they are parallel) until the upper and lower interfaces meet and more highly concentrated layers are propagated to the surface. To design a clarifier properly, the velocity of the slowest moving layer in the clarifier zone (as opposed to the sludge thickening zone) must be assessed. Solving Eqs. (11.39) and (11.42) together gives the interface point.

Practically it is difficult to apply the preceding theory to continuous flow situations and real sludges exhibit behavior that deviates to some extent from the ideal Kynch curve (Vesilind and Jones, 1990). The design of a clarifier is based on the tangent to the settling curve determined at the point where the critical (rate controlling) concentration reaches the surface. Dick and Ewing (1967) have given a discussion of zone sedimentation theory as applied to biological suspensions. Dick and co-workers have developed the theory most commonly applied to continuous flow type III sedimentation.

### 11.5.2 Design of a Basin for Type III Sedimentation

The data from the progression of batch sedimentation depicted in Figs. 11.21 and 11.24 must be translated to the continuous flow situation. The clarifier must be designed with the minimum surface area to allow for maximum thickening and settling of the suspension. The analysis and design technique for type III continuous flow clarifiers

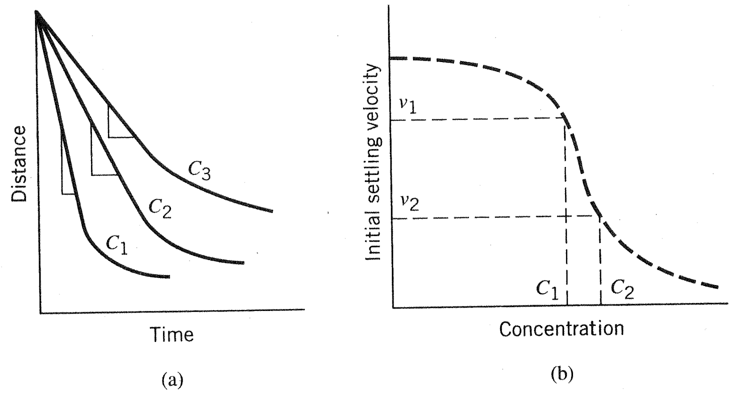



***Figure 11.27*** Preliminary plots for gravity flux determination.

was developed by Dick (1970). The analysis applies to zone I, where the limiting conditions for clarification exist.

## *Gravity Flux*

Solids in the clarified zone are transported downward by gravity and by bulk transport because of solids removal by the underflow. The hindered settling velocity, $v_h$, of a suspension is the settling velocity of the interface during the initial straight line portion of the interface settling curve. The gravity flux, $N_g$, is

$$N_g = C_i v_h \tag{11.43}$$

where

$C_i$ is the initial concentration of the suspension

Different dilutions of the suspension are settled in the laboratory to obtain data for a plot as shown in Fig. 11.27a. The variation of the initial or hindered settling with concentration can be plotted as shown in Fig. 11.27b.

From the curve in Fig. 11.27b, the gravity mass flux can be calculated using Eq. (11.43). The curve can be plotted as shown in Fig. 11.28. At low concentrations of the suspension, the flux is low because of the small amounts of solids; at high solids concentrations the flux is low because of extremely hindered settling.

Vesilind (1979) observed that a general equation with two adjustable parameters, $a$ and $b$, of the form

$$v_h = ae^{-bC} \tag{11.44a}$$

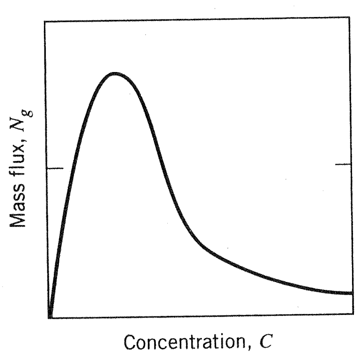



***Figure 11.28*** Mass flux resulting from gravity.

could be used to describe the settling velocity of a suspension at any concentration, $C$. Regression analysis can be used to establish the values of $a$ and $b$. After an extensive investigation of many nonchemically amended activated sludges, Wahlberg and Keinath (1988) found the following regression equation:

$$v_h = [25.3 - 0.061(\text{SVI})]e^{[-0.426+(0.003\,84)(\text{SVI})-(0.000\,054\,3)(\text{SVI})^2]C} \tag{11.44b}$$

where

SVI is the stirred sludge volume index measured in a 1-L cylinder as prescribed by *Standard Methods* (1992). SVI is discussed in Section 17.6.1.

35 mL/g $\leq$ SVI $\leq$ 220 mL/g

The relation in Eq. (11.44b) does not necessarily hold for plants where chemical addition for phosphorus removal or improved settling is practiced or for other sludges. But for appropriate sludges the equation can be readily used to establish the gravity flux curve. Keinath (1990) has used this relation to develop design charts.

Wilson and Lee (1982) have used the following equation to describe initial settling velocities of the suspension.

$$v_h = aC^b \tag{11.44c}$$

Experimental verification of Eqs. (11.44a)–(11.44c) should be performed. A variety of mathematical formulations of the settling velocity for the suspension have been reported. Once the appropriate equation is established it can be substituted into the flux equations to facilitate a mathematical analysis. Otherwise the discrete $v_h$ data can be used to construct the $N_g$ curve and perform the design as outlined next.

### *Underflow Flux*

In addition to gravity induced settling, the underflow withdrawal increases the downward movement of solids. The underflow velocity, $U_b$, is

$$U_b = \frac{Q_u}{A_s} \tag{11.45}$$

where

$Q_u$ is the volumetric underflow flow rate

$A_s$ is the surface area of the clarifier

The underflow flux in the clarified zone is

$$N_u = CU_b \tag{11.46}$$

where

$N_u$ is the underflow flux

Note that the underflow flux varies with the local concentration.

### *Total Flux*

The total flux, $N$, is the sum of the above two fluxes and is plotted in Fig. 11.29 along with its component fluxes.

$$N = N_g + N_u = Cv_h + CU_b \tag{11.47}$$

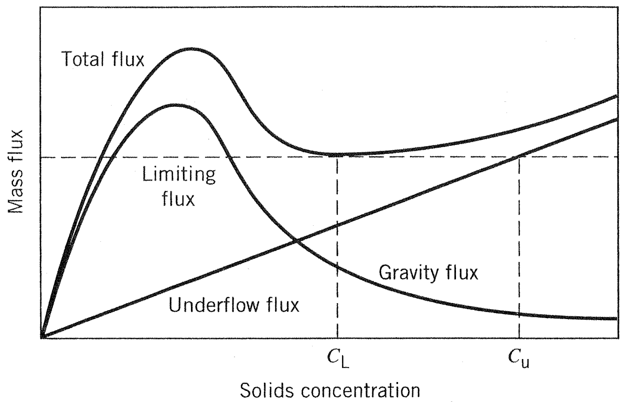



***Figure 11.29*** Total mass flux as a function of concentration.

The solids transmitting capacity of each layer varies with the concentration of the layer. The overall flux usually has a maximum and minimum, as shown in Fig. 11.29, and the minimum point on the curve determines the design area of the clarifier. The minimum solids handling capacity of the suspension is the limiting flux, $N_L$. The concentration at which the limiting flux occurs is $C_L$. The clarifier must be able to handle the mass flow rate of solids coming into it; therefore,

$$A_s N_L \geq Q C_0 \tag{11.48}$$

where
- $Q$ is the influent volumetric flow rate
- $C_0$ is the concentration of solids in the influent

The design area of the clarifier is then

$$A_s \geq \frac{Q C_0}{N_L} \tag{11.49}$$

Effectively the area of the clarifier is being sized to maintain the upward velocity (flux) of water at or less than the minimum settling velocity that will occur in the solids suspension as the concentration in the suspension increases to higher values. It is assumed that the influent jet is dispersed and distributed over the whole plan area of the clarifier.

At the withdrawal point for the thickened solids at the bottom of the clarifier, there is no gravity flux. All solids are removed by bulk flow ($U_b$).

$$C_u Q_u = Q C_0 = A_s N_L \tag{11.50}$$

By extending the limiting flux line to its intersection with the underflow flux line and dropping a perpendicular, the concentration of solids in the underflow, $C_u$, can be determined. Choosing a lower value for $U_b(Q_u)$ will result in a higher value of $C_u$ and the total flux curve will shift down. Of course, the value of $C_u$ must be physically attainable. The values of $U_b$ for biological sludges are typically in the range of 25–50 cm/h (0.84–1.7 ft/h).

There is no upward flux of water in the lower compaction zone section in the clarifier. The influent jet is dissipated and distributed above the bottom layers of the compaction zone. Theoretically, the compaction zone can be designed to hold the sludge as long as desired. However, on a practical basis, biological sludges cannot be held in the clarifier for excessive time or denitrification and anaerobic decomposition become significant. Any oxygen present in the influent to the clarifier will be rapidly consumed in the clarifier. Denitrification occurs if a significant amount of nitrate is present, with the production of nitrogen gas. In time the microorganisms will acclimate to the anaerobic conditions and begin to anaerobically metabolize residual substrate present in the wastewater and, also, the microorganisms will begin to digest themselves. Anaerobic decay results in the production of gases such as methane that have low solubilities. Carbon dioxide will also be produced in significant quantities. Gases will leave solution and form bubbles, which will attach to some settled particles and buoy them up and out of the clarifier, significantly deteriorating the quality of the clarified effluent.

### *Alternate Method for Determining the Minimum Area of a Clarifier for Type III Sedimentation*

There is a more convenient method to size the area of a clarifier considering different underflow rates. The underflow concentration, $C_u$, is chosen based on the most reasonable lab or field data. There are five unknowns needed to solve for the minimum area of the clarifier: $N_L$, $C_L$, $v_L$ (the velocity of the suspension resulting from gravity at the limiting flux), $U_b$, and $C_u$.

At the limiting flux in the clarifier, the following relations hold:

$$N_L = C_L v_L + C_L U_b = N_{gL} + C_L U_b \tag{11.51}$$

where

$N_{gL}$ is the gravity flux component of the limiting flux

In the underflow:

$$N_L = C_u U_b \tag{11.52}$$

For a given set of conditions, $N_L$ is constant; Eq. (11.51) can be rearranged to

$$N_{gL} = N_L - C_L U_b \tag{11.53}$$

If $C_u$ and $U_b$ are chosen, Eqs. (11.52) and (11.53) along with the gravity flux curve define the system.

The procedure to use the equations is as follows. Consider the equation,

$$N_g = N_L - C U_b \tag{11.54}$$

which is similar to Eq. (11.53). The following conditions apply.

$$C = 0 \qquad N_g = N_L$$
$$C = C_u \qquad N_g = 0$$

Also consider the equation

$$N_g = C v_L \tag{11.55}$$

If $v_L$ is constant, this curve produces a straight line beginning at the origin. At the limiting flux, $C = C_L$ and $N_{gL} = C_L v_L$. Equations (11.54) and (11.55) and the $N_g$ curve must intersect at the same point. The point of intersection occurs after the rise

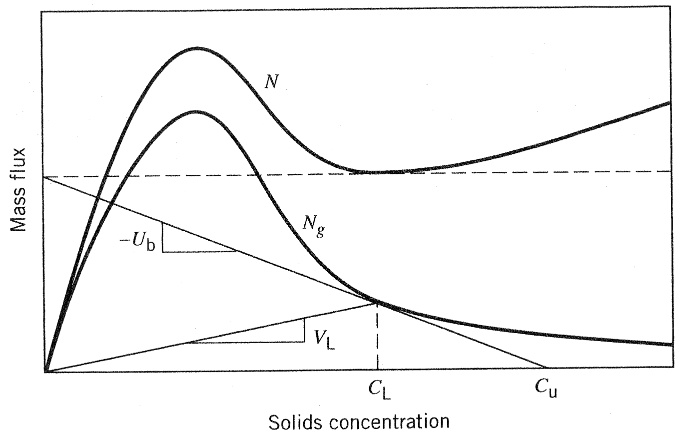



***Figure 11.30*** Alternate method for determining $N_L$.

in the $N_g$ curve. The situation is shown in Fig. 11.30. The total flux curve ($N$) is also drawn for reference. It is not necessary to draw the line for Eq. (11.55). Once $U_b$ is selected, which determines the slope of Eq. (11.54), the line is simply moved to its point of tangency with the $N_g$ curve and $N_L$ and $C_u$ are read at the intersection of the line with the $y$ and $x$ axes, respectively.

This method conveniently allows the designer to evaluate the effects of different underflow velocities on the design. Different values of $U_b$ should be checked, corresponding to the expected maximum and minimum concentrations of thickened sludge in the underflow. In a biological treatment unit the concentration of sludge in the return line to the biological unit in turn influences the volume of flow that is recycled and consequently the influent volumetric flow rate to the clarifier.

### ■ Example 11.2 Design of a Clarifier Using the Solids Flux Method

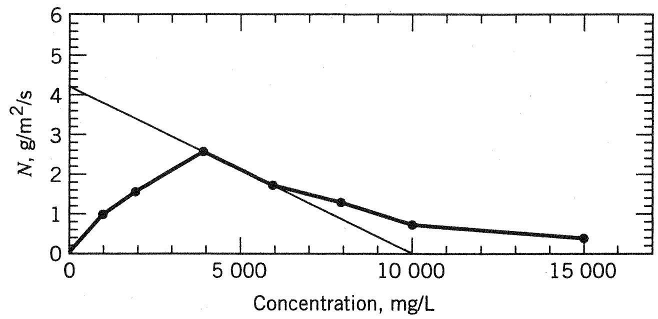
The data for initial hindered settling velocities of a suspension are given in the following table. Size the area of a clarifier to handle a flow of 2 300 m³/d containing a TSS concentration of 2 100 mg/L. The minimum underflow SS concentration is 10 000 mg/L. What is the rate of underflow?

| $C$, mg/L | 1 000 | 2 000 | 4 000 | 6 000 | 8 000 | 10 000 | 15 000 |
|---|---|---|---|---|---|---|---|
| $v_h$, m/h | 3.74 | 2.82 | 2.26 | 1.04 | 0.49 | 0.25 | 0.072 |

The $N_g$ values were obtained from $N_g = Cv_h$ and are given in the following table.

| $C$, mg/L | 1 000 | 2 000 | 4 000 | 6 000 | 8 000 | 10 000 | 15 000 |
|---|---|---|---|---|---|---|---|
| $N_g$, g/m²/s | 1.04 | 1.57 | 2.51 | 1.73 | 1.09 | 0.69 | 0.30 |

They were plotted in the following figure.




A tangent was drawn to the $N_g$ curve passing intercepting the $x$ axis at the desired underflow concentration, 10 000 mg/L. From the intersection of the tangent line and the $y$ axis, the limiting flux is 4.2 g/m²/s.

From Eq. (11.49) the minimum surface area of the clarifier is

$$A_s = \frac{QC_0}{N_L} = \frac{(2\,300 \text{ m}^3/\text{d})(2\,100 \text{ mg/L})}{4.2 \text{ g/m}^2/\text{s}}\left(\frac{1}{86\,400 \text{ s}}\right)\left(\frac{1\,000 \text{ L}}{1 \text{ m}^3}\right)\left(\frac{1 \text{ g}}{1\,000 \text{ mg}}\right) = 13.3 \text{ m}^2$$

$$QC_0 = Q_u C_u$$

$$Q_u = \frac{C_0}{C_u} Q = \frac{2\,100 \text{ mg/L}}{10\,000 \text{ mg/L}}(2\,300 \text{ m}^3/\text{d}) = 483 \text{ m}^3/\text{d}$$

## 11.6 WEIR–LAUNDER DESIGN

Rising water in a sedimentation basin flows over a weir into a channel or launder that conveys the collected water to the exit channel or pipe. Weirs are located as far as possible from the basin inlet. Weir loading rates are specified to prevent strong updrafts that would carry solids out of the basin. Weir loading rates are specified later on for clarifiers in water and wastewater treatment. As the depth of the basin increases higher weir loading rates have less influence on the performance of the clarifier.

For basins that are not covered, the weir is frequently of the V-notch type (Fig. 11.31) to minimize wind effects. Also, straight edge weirs that are not perfectly level do not have uniform flow over the entire weir. This will cause uneven flow patterns in the sedimentation basin and deterioration of its performance. Submerged orifices are also sometimes designed to discharge effluent from a clarifier.

V-Notch weirs must have a depth that permits discharge of the peak flow through the basin. Spacing of the notches is in the range of 150–300 mm (6–12 in.) center to center.

The discharge through a V-notch weir is given by (Vennard and Street, 1982),

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan\frac{\theta}{2} H_w^{5/2} \qquad (11.56)$$

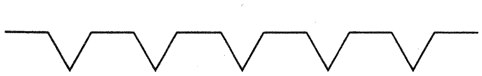


***Figure 11.31*** V-notch weir.

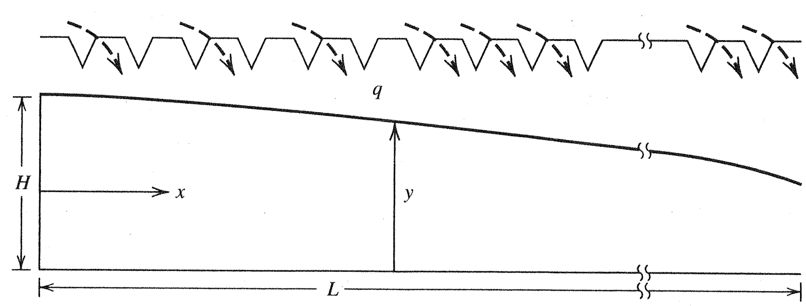



***Figure 11.32*** Definition sketch for flow in a launder.

where

$\theta$ is the angle of the V-notch
$C_d$ is the discharge coefficient
$H_w$ is the depth of water over the weir

The discharge coefficient is around 0.62. The depth–width relation for a V-notch (triangular) weir is

$$\frac{w}{2H_w} = \tan\frac{\theta}{2} \qquad (11.57)$$

where

$w$ is the width of the weir at any height $H_w$

Flow in the launder is spatially varied flow; refer to the definition sketch in Fig. 11.32. These flumes are usually built with no slope. The momentum principle is used for analysis.

The following symbols are used in the development:

$A$ cross sectional area
$b$ launder width
$F$ force
$g$ acceleration of gravity
$H$ water depth at upstream end of launder
$L$ launder length
$P$ pressure
$q$ discharge per unit length
$Q$ total volumetric discharge rate
$x$ distance from upstream end of launder
$y$ depth of flow in the launder at any $x$
$v$ flow velocity
$\rho$ density of water

Friction ($F_f$ in Fig. 11.33) will be ignored in the development of the governing equation. It will be accounted for after the equation is derived. This approach is an approximate but reasonable solution of the problem. Steady state conditions are also assumed. Benefield et al. (1984) give a program to perform the numerical integration of the flow equations for this lateral spillway channel and arrive at a more exact solution.

In a circular basin, the flow into the launder is uniform around the circumference. Furthermore, the flow in the launder is symmetrical for the two halves of the launder. At the point opposite from the launder discharge outlet, the flow splits and travels in opposite directions. Therefore the development is for half of the channel with a length of $L/2$. To develop the equation governing flow in either half of the channel, examine an elemental volume (Fig. 11.33) indicating parameters to be used in a momentum analysis.

... Skip to p. 323

Equation (11.64) is used to determine the value of $y$ at any $x$. Equation (11.68) is applied to find the energy slope at the midpoint of the flume, $S_{MP}$, and at the end (depth of flow is $y_c$), $S_c$. The energy slope at the beginning of the flume is 0. The headloss ($h_L$) in each half of the flume is estimated from

$$h_L = \left(\frac{0 + S_{MP}}{2}\right)\frac{L}{4} + \left(\frac{S_{MP} + S_c}{2}\right)\frac{L}{4} \tag{11.69}$$

The headloss is added to the value of $H$ determined from Eq. (11.64). The depth of the launder is increased by 5–10 cm (2–4 in.) to ensure free fall from the weir. Freeboard depth is also added to prevent splashing and to account for nonuniform flow variation over the weir because of the wind or flow distribution in the clarifier and uncertainty in the estimation of the flow. A freeboard of 10 to 20% of the depth calculated from the headloss and $H$ is used. The total depth of the launder is the sum of freeboard, $H$, headloss, and free fall allowance.

## 11.7 CLARIFIER DESIGN FOR WATER AND WASTEWATER TREATMENT

The design ranges for clarification basins used in water treatment plant are highly variable depending on the quality of raw water and type of floc formed, which is dependent on the coagulant used and the operation of the flocculation process (see Chapter 13). There are also a variety of designs used ranging from rectangular horizontal flow basins to circular and lamella clarifiers. Solids contact clarifiers incorporate coagulation, flocculation, and clarification into a single unit and are usually circular. Ranges for design variables for different configurations are given in Table 11.6. Handbooks or other references should be consulted to find the narrower ranges for different

**TABLE 11.6** Clarifiers in Water Treatment[a]

| Item | Value |
|---|---|
| *Rectangular and Circular Clarifiers* | |
| Depth, m (ft) | 2.4–4.9 (8–16) |
| Overflow rate, $m^3/m^2/d$ ($gal/ft^2/d$) | 20–70 (490–1 720) |
| Weir loading rate, $m^3/m/d$ (gal/ft/d) | Less than 1 250 (100 000) |
| Maximum length of rectangular basin, m (ft) | 70–75 (230–250) |
| Circular basin maximum diameter, m (ft) | 38 (125) |
| *Upflow Solids Contact Clarifiers* | |
| Depth, m (ft) | 2.5–3 (8–10) |
| Overflow rate, $m^3/m^2/d$ ($gal/ft^2/d$) | 24–550 (590–13 500) |
| *Inclined Tube or Lamella Clarifiers* | |
| Inclined length, m (ft) | 1–2 (3.3–6.6) |
| Angle of inclination (°) | 7–60 |
| Tube diameter or plate spacing, cm (in) | Near 5 (2) |
| Overflow rates based on plan area, $m^3/m^2/d$ ($gal/ft^2/d$) | 2–8 times rate for conventional clarifiers, 88–178 (2 160–4 370) |
| Depth, m (ft) | 6–7 (20–23) |

[a]Compiled from Gregory and Zabel (1990), ASCE and AWWA (1990), AWWA (1971), Culp and Culp (1974).

**TABLE 11.7** Clarifiers in Wastewater Treatment[a]

| Item | Value |
|---|---|
| *Primary Clarifiers*[b] | |
| Overflow rate, $m^3/m^2/d$ ($gal/ft^2/d$) | |
| For average dry weather flow rate | 32–49 (785–1 200) |
| For peak flow condition | 49–122 (1 200–3 000) |
| Sidewater depth, m (ft) | 2.1–5 (6.9–16.4) |
| Weir loading rate[c], $m^3/m/d$ (gal/ft/d) | 125–500 (10 000–40 000) |
| *Secondary Clarifiers* | |
| Overflow rate[d], $m^3/m^2/d$ ($gal/ft^2/d$) | |
| For average dry weather flow rate | 16–29 (393–712) |
| For peak flow condition | 41–65 (1 006–1 595) |
| Sidewater depth, m (ft) | 3.0–5.5 (9.8–18) |
| Floor slope | Nearly flat to 1:12 |
| Maximum diameter, m (ft) | 46 (150) |

[a]From WEF and ASCE (1992), *Design of Municipal Wastewater Treatment Plants,* vol. 1, WEF, © WEF 1992.
[b]Criteria are based on the maximum ranges specified by a number of firms and agencies reported in WEF and ASCE (1992).
[c]Generally for average flow conditions.
[d]For circular or rectangular tanks.

types of sediment and coagulants. The majority of suspended solids removal will occur in the sedimentation basin in a water treatment process.

Dissolved air flotation is also used for clarification of flocculated waters in water treatment plants. See Chapter 19 for a description of this process.

Clarifiers used in wastewater treatment may be rectangular, square, or circular. Circular clarifiers are most commonly used for both primary and secondary clarifiers. Design guidelines for primary and secondary clarifiers are given in Table 11.7. Variation among design codes is large. Primary clarifiers are designed more conservatively if sedimentation is the only treatment and if activated sludge is being returned to the primary clarifiers. Rectangular tanks are generally designed with the same criteria as circular tanks. Length to width ratios employed for design of rectangular primary clarifiers range from 3:1 to 5:1, although values for existing tanks range from 1.5:1 to 15:1 (WEF and ASCE, 1992).

Properly designed and operated primary clarifiers should remove 50–65% of the influent TSS.

### ■ Example 11.3a Clarifier Design

Determine the number of clarifiers and surface area of a primary clarifier system for a wastewater treatment plant. Use circular clarifiers that meet criteria given in Table 11.7. The minimum, average, and maximum flows have been determined to be 0.174, 0.347, and 0.868 $m^3/s$ (6.18, 12.4, and 30.9 $ft^3/s$), respectively. These flows correspond to 15 000, 30 000, and 75 000 $m^3/d$ (3.96, 7.93, and 19.8 Mgal/d), respectively.

A minimum of two sedimentation basins should be installed. For two basins, with one basin out of service, the maximum flow into the basin will be 0.868 $m^3/s$. At low flow conditions, only one basin will be in service. The required surface area of the basins at various flow conditions was calculated using the criteria in Table 11.7. The results are given in the following table.

| Flow $m^3/d$ | Number of basins | Surface overflow rate $m^3/m^2/d$ | Surface area $m^2$ |
|---|---|---|---|
| 30 000 | 2 | 32 | 469 |
| 30 000 | 2 | 49 | 306 |
| 75 000 | 2 | 49 | 765 |
| 75 000 | 2 | 122 | 307 |
| 75 000 | 1 | 122 | 615 |

From the results in this table it appears that a surface area of 450 $m^2$ for each clarifier will suffice. The overflow rates for various flow conditions for the design surface area are given in the following table.

| Flow, $m^3/m^2/d$ | Number of basins | $Q/A$, $m^3/m^2/d$ |
|---|---|---|
| 15 000 | 1 | 33.3 |
| 30 000 | 2 | 33.3 |
| 75 000 | 2 | 83.3 |
| 75 000 | 1 | 166.6 |

The peak flow condition with only one basin in service is above the recommended value but this condition is rare. Furthermore, in this system activated sludge–secondary clarification will follow primary treatment and provide further opportunity to remove the suspended solids.

The radius of each clarifier will be

$$\pi r^2 = A \qquad r = \sqrt{A/\pi} = \sqrt{(450\ \text{m}^2)/\pi} = 12.0\ \text{m}\ (39.4\ \text{ft})$$

---

### ■ Example 11.3b Weir-Launder Design

Design the overflow weirs and collection launders for the clarifiers in Example 11.3a. The launders will be made from smooth concrete; take $n$ to be 0.014.

The weir length around the perimeter of a basin is

$$L = 2\pi r = 2\pi(12.0\ \text{m}) = 75.4\ \text{m}$$

The weir loading rates at peak flow with one and both basins in service are

$$\text{Both basins: } q = Q/L = (75\,000\ \text{m}^3/\text{d})/2(75.4\ \text{m}) = 497\ \text{m}^3/\text{m/d} = 0.005\,75\ \text{m}^3/\text{m/s}$$

$$\text{Single basin: } q = (75\,000\ \text{m}^3/\text{d})/(75.4\ \text{m}) = 995\ \text{m}^3/\text{m/d} = 0.011\,5\ \text{m}^3/\text{m/s}$$

The weir loading rate is satisfactory at average flow, which is less than the peak flow with two basins in service.

The weir and collection launder must be designed for the peak flow condition with only one basin in service. A V-notch weir will be used with an angle of 90° and spacing of 25.0 cm center to center (c/c). The number of V-notches is

$$n = (75.4\ \text{m})/(0.250\ \text{m}) = 301.6$$

Use 302 notches.

The flow per notch, $q_n$ is

$$q_n = Q/n = (0.868\ \text{m}^3/\text{s})/302 = 0.002\,87\ \text{m}^3/\text{s}\ (0.101\ \text{ft}^3/\text{s})$$

For a discharge coefficient of 0.62 and peak flow through one basin, the depth of water over the crest of a V-notch is (Eq. 11.56)

$$H_w = \left(\frac{15Q}{8C_d\sqrt{2g}\tan\frac{\theta}{2}}\right)^{2/5} = \left[\frac{15(0.002\,87\ \text{m}^3/\text{s})}{8(0.62)\sqrt{2(9.81\ \text{m/s}^2)}}\right]^{2/5} = 0.082\,6\ \text{m} = 8.26\ \text{cm}\ (3.25\ \text{in.})$$

The elevation of water over the crest of the weir should be increased above this value by a safety factor. A safety factor of 15% will be used. The total depth of the weir is

$$H_w = 1.15(8.26\ \text{cm}) = 9.50\ \text{cm}\ (3.74\ \text{in.})$$

The width of a V-notch at the top is

$$w = 2H_w = 19.0\ \text{cm}\ (7.48\ \text{in.})$$

The launder width is somewhat arbitrary; after a few trial calculations a launder width of 0.70 m (2.30 ft) was chosen. The peak flow with only one basin in service is used. To establish the depth of flow in the launder Eq. (11.66) is used to find the critical depth at the discharge point from the launder.

$$y_c = \left[\frac{(qL)^2}{4b^2g}\right]^{1/3} = \left\{\frac{[(0.011\,5\ \text{m}^3/\text{m/s})(75.4\ \text{m})]^2}{4(0.70\ \text{m})^2(9.81\ \text{m/s}^2)}\right\}^{1/3} = 0.339\ \text{m}\ (1.11\ \text{ft})$$

Now apply Eq. (11.64) to find the upstream depth.

$$H = \left(y^2 + \frac{2q^2x^2}{gb^2y}\right)^{0.5} = \left[(0.339\ \text{m})^2 + \frac{2(0.011\,5\ \text{m}^3/\text{m/s})^2\left(\frac{75.4\ \text{m}}{2}\right)^2}{(9.81\ \text{m/s}^2)(0.70\ \text{m})^2(0.339\ \text{m})}\right]^{0.5}$$

$$= 0.588\ \text{m}\ (1.93\ \text{ft})$$

The headlosses are found by applying Eq. (11.67) at the midpoint ($x = L/4$) found from Eq. (11.63).

$$H^2 = y^2 + \frac{2q^2x^2}{gb^2y}$$

$$(0.588\ \text{m})^2 = y_{mp}^2 + \frac{2(0.011\,5\ \text{m}^3/\text{m/d})^2\left(\frac{75.4\ \text{m}}{4}\right)^2}{(9.81\ \text{m/s}^2)(0.70\ \text{m})^2 y_{mp}} \Rightarrow y_{mp} = 0.56\ \text{m}\ (1.84\ \text{ft})$$

$$S_{MP} = \frac{\left(q\frac{L}{4}\right)^2 n^2}{(by_{mp})^{10/3}}(b + 2y_{mp})^{4/3}$$

$$= \frac{\left[(0.011\,5\ \text{m}^3/\text{m/d})\left(\frac{75.4\ \text{m}}{4}\right)(0.014)\right]^2}{[(0.70\ \text{m})(0.56\ \text{m})]^{10/3}}[0.70\ \text{m} + 2(0.56\ \text{m})]^{4/3} = 0.000\,464$$

At the discharge end of the launder,

$$S_c = \frac{\left[(0.011\,5\ \text{m}^3/\text{m/d})\left(\frac{75.4\ \text{m}}{2}\right)(0.014)\right]^2}{[(0.70\ \text{m})(0.339\ \text{m})]^{10/3}}[0.70\ \text{m} + 2(0.339\ \text{m})]^{4/3} = 0.006\,83$$

The total headloss is approximately

$$h_L = \left(\frac{0 + S_{MP}}{2}\right)\frac{L}{4} + \left(\frac{S_{MP} + S_c}{2}\right)\frac{L}{4} = \left(\frac{0.000\,464}{2}\right)\left(\frac{75.4\ \text{m}}{4}\right)$$

$$+ \left(\frac{0.000\,464 + 0.006\,83}{2}\right)\left(\frac{75.4\ \text{m}}{4}\right)$$

$$= 0.073\ \text{m}\ (0.24\ \text{ft})$$

The total depth of flow in the basin is 0.588 m + 0.073 m = 0.661 m (2.16 ft)

Using a factor of 10% for freeboard and adding 5 cm to ensure free fall, the total depth provided in the launder is

$$H_t = 1.10(0.661\ \text{m}) + 0.05\ \text{m} = 0.777\ \text{m, say } 0.78\ \text{m}\ (2.56\ \text{ft})$$

## *Depth in Sedimentation Basins*

Because sedimentation is theoretically independent of depth, the question may be asked why any significant depth at all is provided. All models assume that the full cross-sectional or surface area of the clarifier is utilized equally by the incoming flow and that suspensions are uniformly distributed. Nonideal flow patterns that result in dead volume, in particular, reduce the effective area as well as the effective volume of the clarifier. Providing more depth allows flow patterns to develop at the expense of the additional volume but with improvement in performance. Also, providing more depth minimizes scour of settled solids. Figure 11.34 shows a circular clarifier design in which the location of the effluent weir promotes a flow pattern that utilizes more of the volume of the clarifier.

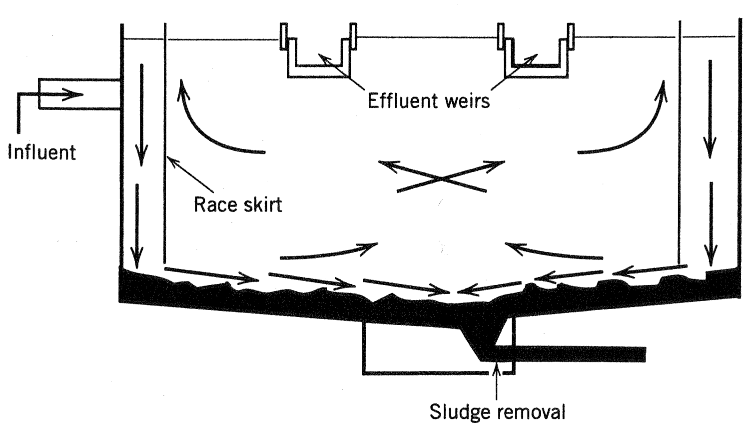



***Figure 11.34*** Locating the collection weirs near the center of a circular clarifier improves the hydraulic performance of the clarifier. Courtesy of Lakeside Equipment Corp.