

# *Introduction to Wastewater Treatment Processes*

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## Pretreatment and Primary Treatment

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### 1. Introduction

Selection of a wastewater treatment process or sequence of processes depends on a number of factors, i.e., (1) characteristics of the wastewater, e.g., BOD, % of suspended solids, pH, presence of toxic materials; (2) required effluent quality; (3) cost and availability of land, e.g., certain biological processes (stabilization ponds) are only economically feasible if low cost land is available; and (4) consideration of a possible future upgrading of water quality standards, necessitating design of a more sophisticated type of treatment for future use.

Pretreatment of wastewater implies removal of suspended solids or con-

ditioning of wastewater for discharge into either a receiving body of water or a secondary treatment facility through neutralization and/or equalization. Types of primary treatment discussed in this chapter are (1) screening, (2) sedimentation, (3) flotation, and (4) neutralization and equalization.

### 2. Screening

Screening is employed for removal of suspended solids of various sizes. Screen openings range in size depending on their purpose, and cleaning of screens is done either manually or mechanically. Screenings are disposed of by burial, incineration, or anaerobic digestion. Screens are classified as fine and coarse.

Fine screens have openings of 3/16 in. or smaller. They are usually made of steel mesh or perforated steel plates and sometimes used instead of sedimentation tanks. However, whereas they remove from 5 to 25% of suspended solids, 40–60% is removed by sedimentation. For this reason, and also because clogging is frequently a problem, use of fine screens is not very common.

Coarse screens have openings ranging from 1.5 to 3.0 in. They are used as protecting devices so that large suspended solids do not damage pumps and other equipment.

Sometimes shredders are used instead of coarse screens. These devices tear down suspended solids, which are then removed by sedimentation.

### 3. Sedimentation

#### 3.1. INTRODUCTION

Sedimentation is utilized in wastewater treatment to separate suspended solids from wastewaters. Removal by sedimentation is based on the difference in specific gravity between solid particles and the bulk of the liquid, which results in settling of suspended solids.

In some cases, sedimentation is the only treatment to which the wastewater is subjected. Sedimentation is also utilized in one or more steps of a treatment sequence. In a typical activated sludge plant, sedimentation is utilized in three of the treatment steps: (1) in grit chambers, in which inorganic matter (e.g., sand) is removed from the wastewater; (2) in the primary clarifier, which precedes the biological reactor, solids (organic and others) are separated; and (3) in the secondary clarifier, which follows the biological reactor, the biological sludge is separated from the treated effluent.

#### 3.2. TYPES OF SETTLING

Three types of settling are recognized depending on the nature of solids present in the suspension.

1. *Discrete settling.* Particles being settled keep their individuality, i.e., they do not coalesce with other particles. Thus, the physical properties of the particles (size, shape, specific gravity) are unchanged during the process. The settling of sand particles in grit chambers is a typical example of discrete settling.

2. *Flocculent settling.* Agglomeration of the settling particles is accompanied by changes in density and settling velocity. The sedimentation occurring in primary clarifiers is an example.

3. *Zone settling.* Particles form a lattice (or blanket) which settles as a mass exhibiting a distinct interface with the liquid phase. Examples include sedimentation of activated sludge in secondary clarifiers and that of alum flocs in water treatment processes.

### 3.3. THEORY OF DISCRETE SETTLING

The fundamental relationship for settling of discrete particles is Newton's law, which is based on the assumption that particles are spherical with a uniform diameter. When a particle settles, it accelerates until the forces promoting settling, i.e., the particle effective weight, are balanced by the drag or frictional resistance of the liquid. When this equality is achieved, the particle reaches a constant settling velocity called the terminal or settling velocity of the particle.\*

Consider the particle in Fig. 3.1, which has reached its terminal velocity, and write the appropriate force balance. The force promoting sedimentation, i.e., the effective weight of the particle, is the difference between its actual weight and the hydrostatic lift:

$$F_s = v\rho_s g - v\rho_L g = (\rho_s - \rho_L)gv \quad (3.1)$$

where  $F_s$  is the particle effective weight;  $\rho_s$  the particle density;  $\rho_L$  the liquid density;  $g$  the acceleration of gravity; and  $v$  the particle volume,  $\frac{1}{6}\pi d^3$ , where  $d$  is the diameter of the spherical particle.

The drag force impeding sedimentation is

$$F_D = C_D A(\rho_L V^2/2) \quad (3.2)$$

where  $F_D$  is the drag force;  $C_D$  the drag coefficient;  $A$  the projected area of the particle,  $A = \frac{1}{4}\pi d^2$ ; and  $V$  the relative velocity between particle and fluid.

For the condition defining the terminal velocity, equate Eqs. (3.1) and (3.2).

$$(\rho_s - \rho_L)gv = C_D A(\rho_L V_s^2/2)$$

where

$$V = V_s = \text{settling velocity}$$

\* This results from force = (mass) (acceleration). Thus zero acceleration corresponds to a net force of zero, i.e., a perfect balance of forces.

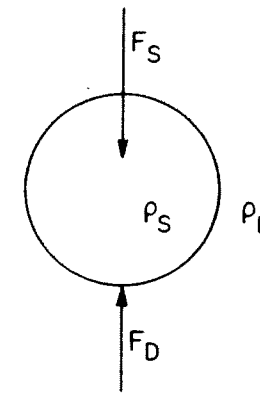


Fig. 3.1. Discrete settling of a particle.

Substituting  $v = \frac{1}{6}\pi d^3$ ,  $A = \frac{1}{4}\pi d^2$ , and solving for the terminal velocity,  $V_s$  [Eq. (3.3)]:

$$V_s = \left[ \frac{4}{3}(g/C_D) \frac{\rho_s - \rho_L}{\rho_L} \cdot d \right]^{1/2} \quad (3.3)$$

which is Newton's law.

For spherical particles, the drag coefficient  $C_D$  is related to the Reynolds number  $N_R$  defined in Eq. (3.4).

$$N_R = dV_s\rho_L/\mu_L \quad (3.4)$$

where  $d$  is the diameter of sphere,  $V_s$  the terminal velocity (settling velocity), and  $\rho_L$  and  $\mu_L$  the density and viscosity of liquid. This relationship is shown in Fig. 3.2.\*

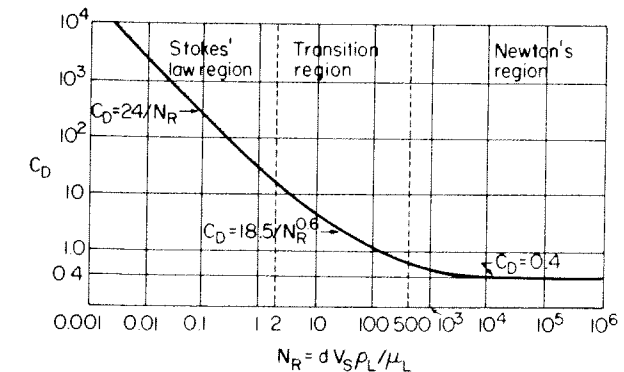


Fig. 3.2. Correlation for drag coefficient for spherical particles.

\* For nonspherical particles Fig. 3.2 is plotted as a family of curves, each curve corresponding to specified value of a parameter defined as sphericity [sphericity,  $\psi$  = (surface area of a sphere having same volume as particle)/(surface area of particle)]. See Waddell [8].

In general, the drag coefficient  $C_D$  is approximated by

$$C_D = b/N_R^n \quad (3.5)$$

where coefficients  $b$  and  $n$  for the different regions of Fig. 3.2 are indicated in Table 3.1. The approximate relationship between  $C_D$  and  $N_R$  given by Eq.

**TABLE 3.1**  
**Drag Coefficient  $C_D$**

Region	$b$	$n$	$C_D = b/N_R^n$
Stokes' law $N_R < 2$	24	1.0	$C_D = 24/N_R$
Transition $2 < N_R < 500$	18.5	0.6	$C_D = 18.5/N_R^{0.6}$
Newton's $N_R > 500$	0.4	0.0	$C_D = 0.4$

(3.5) is written in logarithmic form for the three regions indicated in Fig. 3.2.

Stokes' region:  $C_D = 24/N_R$   
 $\therefore \log C_D = -\log N_R + \log 24$

Transition region:  $C_D = 18.5/N_R^{0.6}$   
 $\therefore \log C_D = -0.6 \log N_R + \log 18.5$

Newton's region:  $C_D = 0.4$   
 $\therefore \log C_D = \log 0.4 = 0.0 \log N_R + \log 0.4$

Thus the curve in Fig. 3.2 is approximately replaced by three segments of straight line with slopes of, respectively,  $-1$ ,  $-0.6$ , and  $0.0$ .

Many sedimentation problems in wastewater treatment occur in Stokes' region. Substituting  $C_D = 24/N_R = 24\mu_L/dV_S\rho_L$  in Eq. (3.3) and simplifying, Stokes' law is obtained.

$$V_S = 1/18 \cdot \frac{\rho_S - \rho_L}{\mu_L} \cdot gd^2 \quad (3.6)$$

For a specific problem in Stokes' region ( $\rho_S$ ,  $\rho_L$ , and  $\mu_L$  fixed) Eq. (3.6) is written as [Eq. (3.7)]

$$V_S = Kd^2 \quad (3.7)$$

(where  $K$  is a constant), which in logarithmic form becomes [Eq. (3.8)]

$$\log V_S = 2 \log d + \log K = 2 \log d + C \quad (C \text{ is a constant}) \quad (3.8)$$

Thus, a logarithmic plot of  $V_S$  vs.  $d$  yields a straight line of slope equal to 2.0 for Stokes' region.

For a specific problem in Newton's region, since  $C_D$  is a constant ( $C_D = 0.4$ ), Eq. (3.3) yields Eq. (3.9).

$$V_S = K'd^{1/2} \quad (3.9)$$

(where  $K'$  = a constant) which in logarithmic form becomes Eq. (3.10).

$$\log V_S = \frac{1}{2} \log d + \log K' = \frac{1}{2} \log d + C' \quad (3.10)$$

where  $C'$  is a constant. Thus, a logarithmic plot of  $V_S$  vs.  $d$  yields a straight line of slope equal to  $\frac{1}{2}$  for Newton's region.

For the transition region,  $C_D = 18.5/N_R^{0.6} = (18.5\mu_L^{0.6})/(d^{0.6}V_S^{0.6}\rho_L^{0.6})$ . Substituting this value in Eq. (3.3) and simplifying, one obtains Eq. (3.11).

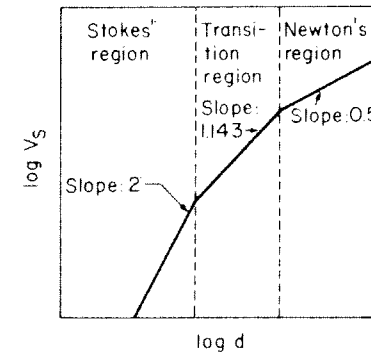
$$V_S = [(4g/55.5)(\rho_L^{0.6}/\mu_L^{0.6})(\rho_S - \rho_L)/\rho_L]^{1/4} d^{1.143} = K''d^{1.143} \quad (3.11)$$

where  $K''$  is a constant. In logarithmic form this becomes [Eq. (3.12)]

$$\log V_S = 1.143 \log d + \log K'' = 1.143 \log d + C'' \quad (3.12)$$

where  $C''$  is a constant. Thus, a logarithmic plot of  $V_S$  vs.  $d$  yields a straight line of slope 1.143 for the transition region. The logarithmic plot of  $V_S$  vs.  $d$  for the three regions is shown in Fig. 3.3. Even in the case of grit chambers, the theory just outlined suffers from two serious limitations: (1) grit particles are seldom spherical, and (2) grit particles do not have uniform density.

A graph corresponding to Eq. (3.3) is plotted in Fig. 3.4, giving the relationship between particle diameter and velocity  $V_S$ . Particles of specific gravities 1.001, 1.01, and 2.65 are considered in plotting Fig. 3.4. Value 2.65 corresponds to the specific gravity of typical sand. The liquid used is water at temperatures indicated, corresponding to respective values of  $\rho_L$ ,  $\mu_L$ . Values of  $C_D$  are obtained from Fig. 3.2 by a trial and error procedure: (1) for specified particle



**Fig. 3.3. Logarithmic plot of  $V_S$  vs.  $d$ .**

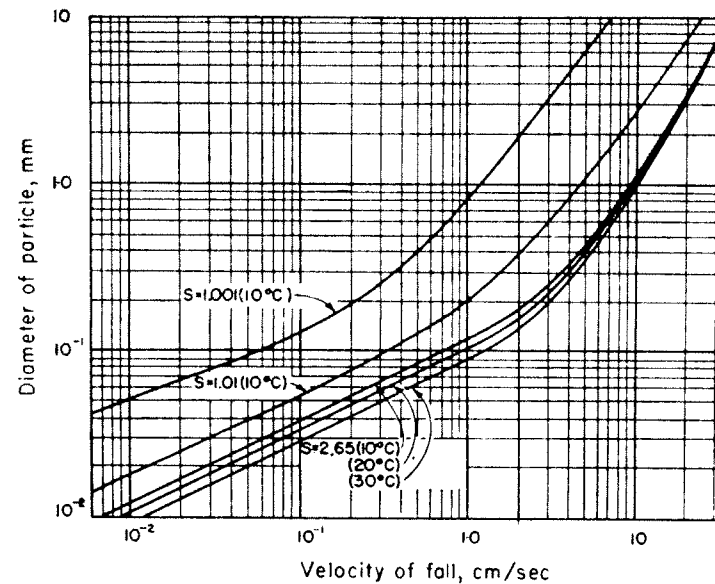


Fig. 3.4. Relation between settling velocity and particle diameter [4].

diameter and temperature ( $\rho_L$  and  $\mu_L$  fixed) assume a settling velocity  $V_S$ ; (2) Calculate  $N_R$  based upon this assumed velocity; (3) From Fig. 3.2 read  $C_D$ ; and (4) From Eq. 3.3 recalculate  $V_S$ . If it agrees with the value assumed in (1) calculations are consistent. Otherwise, iteration is continued until agreement is obtained.

Figure 3.4 is constructed in this manner. Since construction is based upon the actual curve of  $C_D$  vs.  $N_R$  (i.e., Fig. 3.2), the lines in Fig. 3.4 exhibit some curvature, by contrast with the three straight line segments in Fig. 3.3 [construction of which is based on the approximate relationships given by Eqs. (3.8), (3.10), and (3.12)]. As an approximation, however, curves in Fig. 3.4 are replaceable by three straight line segments.

### 3.4. THE IDEAL SEDIMENTATION TANK CONCEPT

This concept, developed by Hazen [5] and Camp [1], is the basis for arriving at relationships utilized in the design of sedimentation tanks. The model chosen for a sedimentation tank consists of four zones (Figs. 3.5 and 3.6).

1. Inlet zone. Here the flow becomes quiescent. It is assumed that at the limit of this zone (i.e., along vertical line  $x\tau$ ) particles are uniformly distributed across the influent cross section.

2. Sedimentation zone. A particle is assumed to be removed from suspension once it hits the bottom of this zone (horizontal line  $\tau y$ ).

### 3. Sedimentation

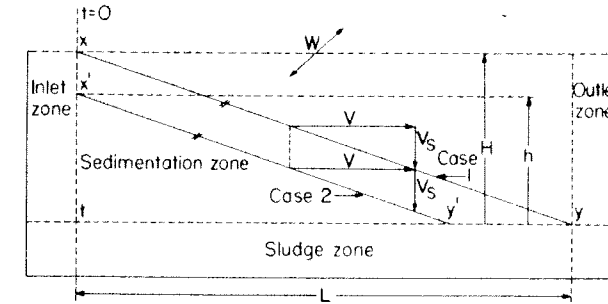


Fig. 3.5. Model of a sedimentation tank with discrete settling particles (Cases 1 and 2).

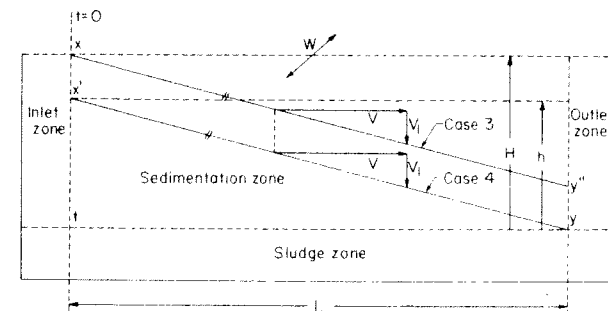


Fig. 3.6. Model of a sedimentation tank with discrete settling particles (Cases 3 and 4).

3. Outlet zone. Wastewater is collected here prior to transfer to the next treatment.

4. Sludge zone. This zone is provided for sludge removal.

Settling paths of particles entering the sedimentation zone at points  $x$  and  $x'$  for discrete settling are indicated by lines  $xy$  and  $x'y'$  in Fig. 3.5 and lines  $xy''$  and  $x'y'$  in Fig. 3.6. These settling paths are the net result of two velocity vector components:

1. Flow-through velocity  $V$  [Eq. (3.13)]

$$V = Q/A' = Q/WH \quad (3.13)$$

where  $V$  is the flow-through velocity (ft/sec);  $Q$  the flow rate (ft<sup>3</sup>/sec);  $A'$  the vertical cross-sectional area of sedimentation zone, namely  $A' = WH$  (ft<sup>2</sup>) (refer to Fig. 3.7);  $W$  the width of sedimentation zone (ft); and  $H$  the depth of sedimentation zone (ft).

2. Settling velocity, indicated by either vectors  $V_S$  or  $V'_1$  in Figs. 3.5 and 3.6, respectively.

For discrete settling the settling velocity is constant for any specific settling

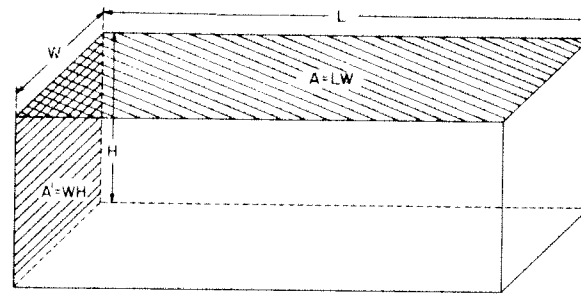


Fig. 3.7. Geometry of the sedimentation zone.

path, i.e.,  $V_s$  and  $V_1$  do not vary along their respective paths. This is due to the fact that a discrete particle is unhindered by neighboring ones (no coalescence), so it settles with a uniform velocity, read from Fig. 3.4, as a function of the particle diameter.

For flocculent settling the situation is different. Figure 3.8 illustrates a typical sedimentation path of flocculent settling (Section 3.5). As coalescence

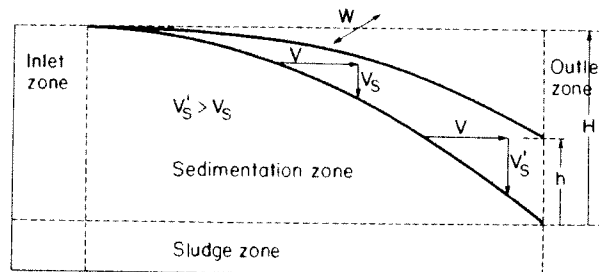


Fig. 3.8. Model of a sedimentation tank for flocculent settling.

with neighboring particles takes place, the effective diameter of the particle increases, and thus its settling velocity  $V_s$  also increases. The net result is that settling paths are curved in contrast with straight line paths for discrete settling.

Consider the following cases with reference to Figs. 3.5 and 3.6, keeping in mind that a particle is assumed to be removed from the suspension once it hits the bottom of the sedimentation zone.

*Case 1* (Fig. 3.5). A particle which at time zero ( $t = 0$ ) is located at point  $x$  and possesses a settling velocity  $V_s$  (and diameter  $d_s$  read from Fig. 3.4). This particle is removed, since it touches the bottom of the sedimentation zone at  $y$  (path  $xy$ ).

*Case 2* (Fig. 3.5). A particle which at  $t = 0$  is located at  $x'$  on the same vertical line as  $x$  but below  $x$ , and has a settling velocity  $V_s$  (or greater than  $V_s$ ). This particle is also removed since it hits the bottom of the sedimentation zone

to the left of point  $y$  (at  $y'$ ). If the settling velocity is greater than  $V_s$  the particle touches the bottom of the sedimentation zone to the left of  $y'$ . Figure 3.6 portrays the case of particles with a diameter  $d_1$  ( $d_1 < d_s$ ), possessing a settling velocity  $V_1$  ( $V_1 < V_s$ ). (Settling velocity  $V_1$  is read from Fig. 3.4 for  $d = d_1$ .) These particles are shown separately in Fig. 3.6 simply to avoid overcrowding of the diagram. In fact they are together in the slurry with particles of settling velocity  $V_s$ .

*Case 3* (Fig. 3.6). A particle which at  $t = 0$  is located at  $x$  and has a settling velocity  $V_1$  (where  $V_1 < V_s$ ). This particle is not removed since it does not reach the bottom of the sedimentation zone (i.e., line  $ty$ ) in time for removal (sedimentation path  $xy''$ ).

*Case 4* (Fig. 3.6). A particle with settling velocity  $V_1$  (where  $V_1 < V_s$ ) and situated at  $x'$  at  $t = 0$ . This particle is removed (sedimentation path  $x'y$ ). Consider now the settling velocity  $V_s$ . From Eq. (3.13)

$$Q = VA' = VWH \quad (3.14)$$

From consideration of similar triangles in Fig. 3.5

$$V = V_s(L/H) \quad (3.15)$$

Substitution of Eq. (3.15) in Eq. (3.14) and simplification leads to

$$Q = V_s LW = V_s A \quad (3.16)$$

where  $A = LW =$  horizontal cross-sectional area of the sedimentation zone ( $\text{ft}^2$ ) (Fig. 3.7).

From Eq. (3.16)

$$V_s = Q/LW = Q/A \quad (3.17)$$

From Eq. (3.17) it follows that the settling efficiency is a function of the horizontal cross-sectional area, rather than of the depth  $H$ . Thus, in principle, it is advisable to utilize sedimentation tanks of high surface area  $A$  and low depths. The only reasons for using a reasonable depth are (1) to satisfy depth requirements in order to provide for mechanical rakes utilized for removal of settled sludge, and (2) the horizontal component of velocity (flow-through velocity  $V$ ) must be kept within certain limits to prevent scouring the particles which have settled.

From Eq. (3.13) it follows that  $H$  should not be too low since  $V$  would rise above the scour velocity. The subject of scour velocity is discussed in this section. Scour occurs when flow-through velocity  $V$  is sufficient to suspend previously settled particles. Scour is not usually a problem in large settling tanks, but it can be an important factor in grit chambers and narrow channels.

The two fundamental premises of the ideal tank concept are (1) uniform distribution of particles across the influent cross section (i.e., along vertical

line  $xt$ ) and (2) assumption that a particle is considered removed when it reaches the bottom of the sedimentation zone (i.e., horizontal line  $ty$ ).

Keeping in mind these two premises, two corollaries follow: (1) All particles with a settling velocity equal to or greater than  $V_s$  are removed; and (2) all particles with a settling velocity less than  $V_s$  (such as  $V_1$  in Fig. 3.6) are removed in a proportion given by the ratio  $V_1/V_s$ .

From geometric consideration of the particle path (Figs. 3.5 and 3.6) touching point  $y$  at the bottom of the sedimentation tank (particles with settling velocity  $V_s$  in Fig. 3.5 following path  $xy$ , and particles with settling velocity  $V_1$  in Fig. 3.6 following path  $x'y$ ), one writes [Eq. (3.18)]

$$V_1/V_s = h/H \quad (3.18)$$

For example, if  $H = 100$  in. and  $h = 75$  in., then

$$V_1/V_s = 75/100 = 0.75$$

or 75% of the particles with a settling velocity  $V_1$  are removed, i.e., those which at time  $t = 0$  are at a height  $x'$  or below. The remaining particles with settling velocity  $V_1$ , i.e., those situated between  $x$  and  $x'$  at time  $t = 0$ , are *not* removed.

The overflow rate defined as

$$Q/A = \text{ft}^3/(\text{ft}^2)(\text{hr}) = \text{ft}/\text{hr}$$

is defined as the settling velocity  $V_s$  of a particle that settles through a distance exactly equal to the effective depth of the tank during the theoretical detention period. This results from the definition of detention period:

$$t = \text{detention period} = (\text{volume of tank})/Q = HA/Q \quad (A = LW) \quad (3.19)$$

From Eq. (3.19) it follows that a settling velocity  $V_s$  defined as  $V_s = H/t$  is equivalent to the overflow rate, since

$$V_s = H/t = H/(HA/Q) = Q/A$$

which is Eq. (3.17).

The *scour velocity*  $V_c$  is the value of the flow-through velocity  $V$  [Eq. (3.20)],

$$V = V_c = Q/A' = Q/WH \quad (3.20)$$

for which "previously settled" particles are scoured away.\*

\* The words "previously settled" are placed in quotes because a particle which is scoured away never actually settles. Mentally one separates the processes of settling and scouring and imagines that a particle settles and subsequently is scoured away. This reasoning is compatible with the hypothetical resolution of the velocity trajectory into vectors  $V$  (flow-through velocity) and  $V_s$  (settling velocity).

The scour velocity is estimated by the following empirical equation [Eq. (3.21)] [1]:

$$V_c = [8\beta g d(s-1)/f]^{1/2} \quad (3.21)$$

where  $V_c$  is the velocity of scour (mm/sec); i.e., flow-through velocity required to scour all particles of diameter  $d$  or smaller;  $\beta$  the constant (0.04 for ungranular sand, 0.06 for nonuniform sticky material);  $f$  the Weisbach-D'Arcy friction factor (0.03 for concrete);  $g$  the acceleration of gravity (mm/sec<sup>2</sup>) (normal: 9800 mm/sec<sup>2</sup>);  $d$  the particle diameter (mm) (particles with diameter  $d$  or less than  $d$  are scoured away); and  $s$  the specific gravity of particle.

### Example 3.1

Consider a suspension of sand ( $s = 2.65$ ) in water at 20°C with a uniform particle size ( $d = 0.07$  mm). Flow is 1.0 Mgal/day.

1. Calculate the settling tank surface (horizontal cross section) for obtaining removal of 70% of the particles.

2. Suppose that instead of a uniform particle diameter, there is, besides particles of  $d = 0.07$  mm, another set with a uniformly larger diameter, which are completely removed in the settling tank designed for 70% removal of the particles with  $d = 0.07$  mm. Determine what is the minimum particle diameter for total removal.

3. For case (2), determine the flow-through velocity  $V_c$  so that all particles of lower settling velocity than those completely removed are scoured away. What combination of length, width, and depth for the settling tank meets these requirements?

SOLUTION: Part 1

Step 1. From Fig. 3.4 [for  $d = 0.07$  mm and  $s = 2.65$  (at 20°C)], read

$$V_s = 0.45 \text{ cm/sec}$$

or

$$V_s = 0.45 \text{ cm/sec} \times \text{ft}/30.48 \text{ cm} \times 3600 \text{ sec/hr}$$

$$V_s = 53.1 \text{ ft/hr} = 53.1 \text{ ft}^3/(\text{ft}^2)(\text{hr})$$

The overflow rate in gal/(day)(ft<sup>2</sup>) is

$$53.1 \text{ ft}^3/(\text{ft}^2)(\text{hr}) \times 7.48 \text{ gal/ft}^3 \times 24 \text{ hr/day}$$

$$\therefore V_s = 9533 \text{ gal}/(\text{day})(\text{ft}^2)$$

Step 2. The horizontal cross-sectional area is (for 100% removal)

$$A = Q/V_s = \frac{1,000,000 \text{ gal/day}}{9533 \text{ gal}/(\text{day})(\text{ft}^2)} = 105 \text{ ft}^2$$

With this area, 100% removal is obtained. For 70% removal the residence time (and thus the cross-sectional area) is reduced by 30%. The cross-sectional area is

$$A = 105 \times 0.7 = 73.5 \text{ ft}^2$$

Under these circumstances, the settling velocity for 100% removal is

$$1,000,000/73.5 = 13,605 \text{ gal}/(\text{day})(\text{ft}^2)$$

*Note:* Check on percent removal. Since the settling velocity is 9533 gal/(day)(ft<sup>2</sup>), fixed by the particle diameter as determined from Fig. 3.4, percentage removal is 9533/13,605 = 0.70 (70%). Referring to Fig. 3.6 the particles removed are those which at the end of inlet zone, along vertical line *xt*, are already at distance *h* (or less than *h*) from the bottom of the sedimentation zone, where [from Eq. (3.18)]

$$V_s = 13,605 \text{ gal}/(\text{day})(\text{ft}^2) \quad (100\% \text{ removal})$$

$$V_1 = 9533 \text{ gal}/(\text{day})(\text{ft}^2) \quad (70\% \text{ removal})$$

$$\therefore h/H = 0.70$$

**SOLUTION: Part 2** If there is a distribution of particle diameters instead of uniform diameter *d* = 0.07 mm, one reads from Fig. 3.4 the diameter, which is larger than 0.07 mm, for which the settling velocity corresponds to 13,605 gal/(day)(ft<sup>2</sup>). Therefore, the abscissa in Fig. 3.4 is

$$(13,605/9533) \times 0.45 \text{ cm/sec} = 0.45/0.7 = 0.642 \text{ cm/sec}$$

From Fig. 3.4 [for  $V_s = 0.642 \text{ cm/sec}$  and  $s = 2.65$  ( $t = 20^\circ\text{C}$ )], read  $d = 0.085 \text{ mm}$  (100% removal). If distribution of particle diameter in the influent is known, one can calculate the % removal corresponding to each group of particles for a given diameter (Example 3.2).

**SOLUTION: Part 3** The scour velocity to sweep all particles of lower settling velocity than those to be completely removed is calculated from Eq. (3.21).

$$V_c = [8 \times 0.04 \times 9800 \times 0.07(2.65 - 1)/0.03]^{1/2} = 110 \text{ mm/sec}$$

Assuming that the sand contains only two particle sizes, e.g., 0.07 and 0.085 mm, the scour velocity  $V_c = 110.0 \text{ mm/sec}$  sweeps away all particles of  $d = 0.07 \text{ mm}$ , leaving behind those of  $d = 0.085 \text{ mm}$ . The value of  $V_c$  in practical units is

$$V_c = 110 \text{ mm/sec} \times \text{ft}/304.8 \text{ mm} = 0.36 \text{ ft/sec}$$

The vertical cross-sectional area is calculated from Eq. (3.13).

$$A' = \frac{1,000,000 \text{ gal/day} \times \text{day}/86,400 \text{ sec} \times \text{ft}^3/7.48 \text{ gal}}{0.36 \text{ ft/sec}} = 4.3 \text{ ft}^2$$

Any practical combination of length, width, and depth is used to satisfy the requirements  $A = 73.5 \text{ ft}^2 = LW$  and  $A' = 4.3 \text{ ft}^2 = WH$ .

### Example 3.2

Suppose that for Example 3.1 instead of a uniform particle size, there is a distribution of diameters. Assume the same specific gravity and temperature as in Example 3.1, i.e.,  $s = 2.65$  and  $t = 20^\circ\text{C}$ . Assume that for each 100 lb of grit the following distribution of particle sizes applies (see tabulation below)

(1) Group no.	(2) lb of each particle size	(3) Particle size, <i>d</i> (mm)
1	50	0.085
2	20	0.070
3	20	0.060
4	10	0.050
	100	

The settling velocities for each group of particles are read from Fig. 3.4. This is indicated in column (4) of Table 3.2. Percent removals are then calculated [column (5)] and expressed as fractions of unity in column (6). The

**TABLE 3.2**  
Calculations for Example 3.2

(1) Group no.	(2) lb of each particle size	(3) Particle size, <i>d</i> (mm)	(4) $V_s$ (Fig. 3.4)	(5) % removal = $\frac{(4)}{0.642} \times 100$	(6) Fraction removed = (5) ÷ 100	(7) lb removed (7) = (2) × (6)
1	50	0.085	0.642	(0.642/0.642)100 = 100 <sup>a</sup>	1.00	50.0
2	20	0.070	0.450	(0.45/0.642)100 = 70 <sup>a</sup>	0.70	14.0
3	20	0.060	0.350	(0.35/0.642)100 = 54.5	0.545	10.9
4	10	0.050	0.220	(0.22/0.642)100 = 34.3	0.343	3.43
	100					78.3

<sup>a</sup> Already calculated in Example 3.1.

weight of sand removed by settling for each group of particles is computed in column (7). Therefore 78.3% of the weight of the original particles is removed by settling.



In this example, if the vertical cross section  $A'$  is taken equal to  $4.3 \text{ ft}^2$  (value calculated in Example 3.1), all particles of  $d = 0.07 \text{ mm}$  and smaller are scoured away (groups 2, 3, and 4). Therefore, the net removal is of only 50 lb per 100 lb of total grit, i.e., the particles with  $d = 0.085$  (group 1). This indicates a net removal by weight of 50%.

If  $A'$  is taken larger than  $4.3 \text{ ft}^2$  the net removal is greater, since there is less scouring. Examining calculations for Example 3.1, it follows that if  $A'$  is taken as  $8.6 \text{ ft}^2$  (twice  $4.3 \text{ ft}^2$ ) the value of  $V_c$  is  $0.18 \text{ ft/sec}$  (instead of  $0.36 \text{ ft/sec}$ ). This corresponds to

$$V_c = 55 \text{ mm/sec} \quad (\text{instead of } 110.0 \text{ mm/sec})$$

Since from Eq. (3.21),  $V_c$  is proportional to  $d^{1/2}$ , it follows that  $d$  is  $0.07/4 = 0.0175 \text{ mm}$ , so as to yield  $V_c = 55 \text{ mm/sec}$  (half of  $110.0 \text{ mm/sec}$ ). Therefore, only particles with  $d = 0.0175 \text{ mm}$  or smaller are removed by scouring. Since for the given distribution the smallest particle diameter is  $0.05 \text{ mm}$ , there is no removal by scouring. Consequently, the net removal is 78.3 lb from every 100 lb of sand, or 78.3% by weight.

If this value is adopted,

$$A = 73.5 \text{ ft}^2 = LW$$

$$A' = 8.6 \text{ ft}^2 = WH$$

Selecting  $H = 4 \text{ ft}$ , then

$$W = 8.6/4 = 2.15 \text{ ft}$$

$$L = 73.5/2.15 = 34.2 \text{ ft}$$

This indicates specification of a narrow settling channel 34.2 ft long, 2.15 ft wide, and 4 ft deep.

### 3.5. FLOCCULENT SETTLING

Flocculent settling takes place when settling velocity of the particles increases due to coalescence with other particles. A diagram of flocculent sedimentation profiles is shown in Fig. 3.8. The settling paths of the particles are curves, rather than straight lines as for discrete settling.

Design criteria for systems exhibiting flocculent settling are established by a laboratory settling analysis. A typical laboratory settling column is shown in Fig. 3.9.

Concentration of suspended solids is kept uniform throughout the column at the beginning of the test by means of a portable stirrer. The depth of the column is approximately the same as that of the settling tank to be designed. Temperature is kept constant during the test. A practical design of a settling column (Fig. 3.9) is 8 ft deep, with sampling ports at depths of 2, 4, 6, and 8 ft.

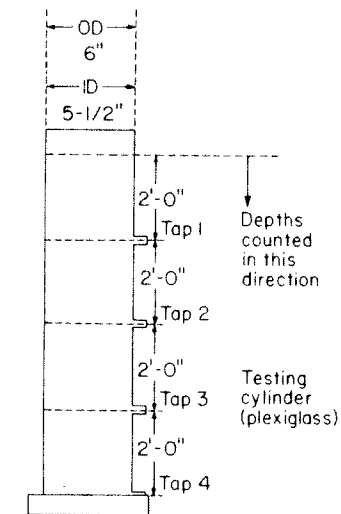


Fig. 3.9. Laboratory settling column.

Data taken at 2-, 4-, and 6-ft depths are utilized to determine settling velocity and detention time relationships. Data from the 8-ft port are used for sludge concentration and compaction determinations.

*Step 1.* Fill the column with wastewater, maintaining a uniform concentration of suspended solids throughout. A portable stirrer is used for this purpose.

*Step 2.* Remove the stirrer from the cylinder. At each sampling port, the concentrations of suspended solids are measured at predetermined times.

Examples 3.3 and 3.4 illustrate the procedure from data treatment to the design of a clarifier.

#### Example 3.3

The suspension being tested has an initial suspended solids concentration of  $430 \text{ mg/liter}$  ( $SS_0$ ). The suspended solids (hence abbreviated as SS) concentrations in Table 3.3 are measured at the indicated times at the 2-, 4-, and 6-ft sampling ports. Perform treatment of the data arriving at curves for (a) % SS removal vs. detention time (min), (b) % SS removal vs. overflow rate [ $\text{gal}/(\text{day})(\text{ft}^2)$ ], and (c) % SS remaining (fraction of particles with less than stated velocity) vs. settling velocity (ft/hr).

*Step 1.* Calculate fraction of solids remaining in suspension for each sample [Eq. (3.22)].

$$x = SS/SS_0$$

or in %

$$y = SS/SS_0 \times 100 \quad (3.22)$$

**TABLE 3.3**  
Laboratory Sedimentation Data (Example 3.3)

Time (min)	SS concentrations at indicated depths		
	2 ft (Tap 1)	4 ft (Tap 2)	6 ft (Tap 3)
5	356.9	387.0	395.6
10	309.6	346.2	365.5
20	251.6	298.9	316.1
30	197.8	253.7	288.1
40	163.4	230.1	251.6
50	144.1	195.7	232.2
60	116.1	178.5	204.3
75	107.5	143.2	180.6

Then calculate for each sample the fraction of solids removed

$$1 - x$$

or in %

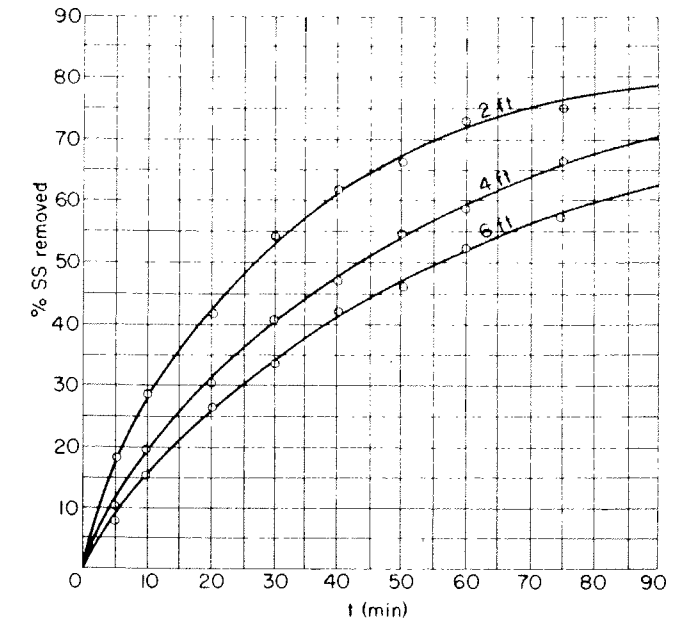
$$z = 100 - y \quad (3.23)$$

A sample of these calculations (for a 2-ft depth) is shown in Table 3.4. Similar calculations are performed for 4- and 6-ft depths.

**TABLE 3.4**  
Calculation of Fraction of Solids Remaining and Removed for a 2-ft Depth

(1) Time (min)	(2) SS remaining (mg/liter) (Table 3.3)	(3) Solids remaining (%) $y = (SS/SS_0) \times 100$	(4) Solids removed (%) $z = 100 - y$
5	356.9	83.0	17.0
10	309.6	72.0	28.0
20	251.6	58.5	41.5
30	197.8	46.0	54.0
40	163.4	38.0	62.0
50	144.1	33.5	66.5
60	116.1	27.0	73.0
75	107.5	25.0	75.0

*Step 2.* In order to smooth the experimental data construct a graph of % SS removed vs. time. This plot is shown in Fig. 3.10 for the 2-, 4-, and 6-ft depths.



**Fig. 3.10.** Suspended solids (% SS) removed vs. time.

*Step 3.* From Fig. 3.10 construct a settling profile graph (Fig. 3.11). This is done by reading from the smoothed curves of Fig. 3.10 the abscissas ( $t$ , min) corresponding to selected values of % SS removed (e.g., 5, 10, 20, ..., 70, 75%) for each one of the three depths. These values are tabulated (Table 3.5) and utilized for construction of Fig. 3.11.

*Step 4.* Calculate % removal of SS and overflow rate [gal/(day)(ft<sup>2</sup>)]. Before the procedure described here is fully understood, some preliminary considerations must be made. An effective settling velocity  $V_s$  is defined as the effective depth (6 ft in this example) divided by the time (detention time,  $t$ ) required for a given particle to travel this distance, i.e. [Eq. (3.24)].

$$V_s = H/t \quad (3.24)$$

If a suspension contains particles with different settling velocities, the efficiency of removal by sedimentation is obtained by performing a settling column test as just described. Let SS be the concentration of solids remaining for one specific sample and time. Thus

$$x_0 = SS/SS_0 = \text{fraction of solids remaining}$$

and

$$1 - x_0 = 1 - SS/SS_0 = y_0 = \text{fraction of solids removed}$$

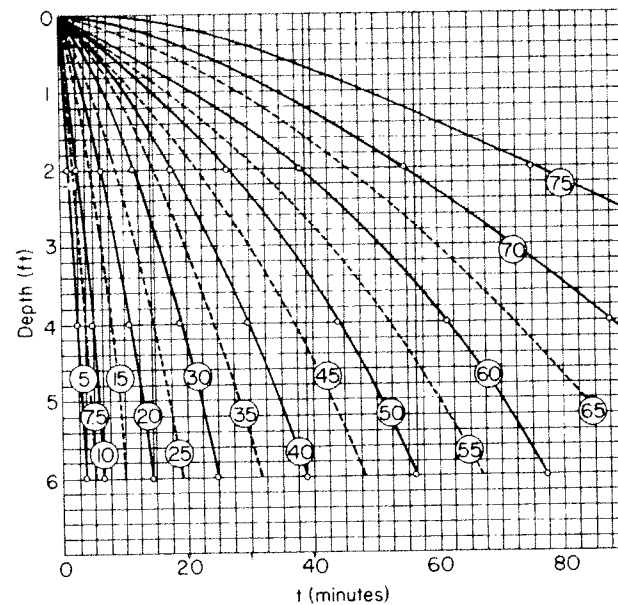


Fig. 3.11. Settling profile. Encircled numbers are % SS removed.

TABLE 3.5  
Values for Plotting Fig. 3.11

% SS removed	t (min)		
	2 ft	4 ft	6 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	—	—

Particles with a settling velocity  $V_S$  or higher (where  $V_S = H/t$ ) are completely removed. Particles with a lower settling velocity  $V_1$  ( $V_1 < V_S$ ) are removed at a ratio given by Eq. (3.18).

A typical graph like the one in Fig. 3.12 is plotted by analysis of data obtained with the sedimentation column. The details for construction of such

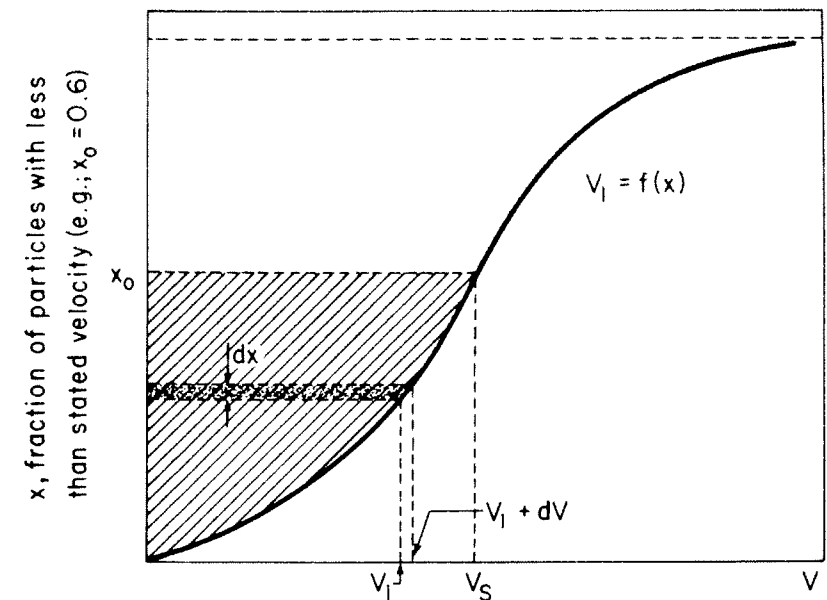


Fig. 3.12. Determination of overall removal.

a graph from experimental data are discussed later in Step 4(d). However, for understanding the calculation procedure described next, it is convenient to assume that this graph is already available.

Figure 3.12 is a plot of the fraction of particles with less than the stated velocity vs. the settling velocity in question. Notice that the fraction of particles with less than the stated velocity (if  $V_S = H/t$ ) corresponds to the fraction of particles *not* completely removed. For example, if 40% of the particles in a specific case are completely removed, then  $x_0 = 0.6$  is the ordinate corresponding to the settling velocity  $V_S = H/t$ .

For particles with settling velocities between  $V_1$  and  $V_1 + dV$  (where  $V_1 < V_S$ ), the fraction removed is  $V_1/V_S$ . Similarly, for particles with settling velocities between  $V_1 + dV$  and  $V_1 + 2dV$ , the fraction removed is  $(V_1 + dV)/V_S$ .

The overall removal of suspended solids is

$$\text{Overall removal} = (1 - x_0) + \int_0^{x_0} (V_1/V_S) dx \quad (3.25)$$

In Eq. (3.25)  $V_1$  is a variable ( $0 \leq V_1 \leq V_S$ ) with  $V_1 = f(x)$  portrayed by the curve in Fig. 3.12. Term  $(1 - x_0)$  is the fraction completely removed, corresponding to particles with velocities  $\geq V_S$ . The second term in Eq. (3.25), i.e.,

$$\int_0^{x_0} (V_1/V_S) dx = 1/V_S \int_0^{x_0} V_1 dx$$

which is the fraction of removal corresponding to particles with velocities less than  $V_s$  (calculated by graphical integration as indicated by the hatched area in Fig. 3.12). The differential area of width  $dx$ , indicated in Fig. 3.12, corresponds to particles with settling velocities between  $V_1$  and  $V_1 + dV$ .

Combining Eqs. (3.18) and (3.25) the final expression for the overall removal is obtained [Eq. (3.26)].

$$\begin{aligned} \text{Overall removal} &= (1 - x_0) + \int_0^{x_0} (V_1/V_s) dx = (1 - x_0) + (1/V_s) \int_0^{x_0} V_1 dx \\ &= (1 - x_0) + \int_0^{x_0} (h/H) dx \end{aligned} \quad (3.26)$$

After these considerations return to discussion of Step 4.

*Step 4(a).* From Fig. 3.11 for a depth of 6 ft read the values  $t$  (min) corresponding to 5, 10, 20, 30, 40, 50, and 60% removal, and calculate the corresponding settling velocities  $V_s$  (ft/hr). These values are tabulated in Table 3.6.

**TABLE 3.6**  
Settling Velocities ( $H = 6$  ft)

Constant % removal	$t$ (min) $H = 6$ ft	Settling velocity (ft/hr) $V_s = H/t = 6/(t/60) = 360/t$
5	3.7	97.2
10	6.5	55.2
20	14.5	24.8
30	25.0	14.4
40	39.0	9.2
50	56.5	6.35
60	77.5	4.64

*Step 4(b).* Calculate % removal of SS. Calculations for % removal of SS and overflow rate for a 25-min settling time (fourth entries in Table 3.6) are illustrated next. Similar calculations are also performed for the other settling times listed in the second column of Table 3.6. For  $t = 25$  min for the settling depth  $H = 6$  ft, 30% of the suspended solids are completely removed. Consider next the particles in each additional 10% range. Start with those in the range 30–40% removal in Fig. 3.11. Particles in this range are removed in the proportion  $V_1/V_s$  or in the proportion of average settled depth ( $h_1$ ) to the total settling depth ( $H$ ). The average settled depth ( $h_1$ ) is estimated by drawing (by interpolation) a curve corresponding to 35% constant removal in Fig. 3.11, and reading from it the depth  $h_1$  corresponding to  $t = 25$  min. Therefore for this first interval, the % solids removal is  $(h_1/H) \times 10 = (4.2/6) \times 10 = 7.0\%$ .

In a similar manner for succeeding 10% intervals, the curves for constant % removal of 45, 55, 65, and 75% are drawn and the average settled depths of 2.4, 1.4, 0.84, and 0.28 ft are read for  $t = 25$  min. The calculations for  $t = 25$  min are indicated below.

Settling velocity:  $V_s = H/t = 6.0/(25/60) = 14.4$  ft/hr

Percent solids removal (for  $t = 25$  min)

100% removal (at 30%)	30.00%
1st interval (35%): $(4.2/6.0) \times 10 =$	7.00%
2nd interval (45%): $(2.4/6.0) \times 10 =$	4.00%
3rd interval (55%): $(1.4/6.0) \times 10 =$	2.33%
4th interval (65%): $(0.84/6.0) \times 10 =$	1.40%
5th interval (75%): $(0.28/6.0) \times 10 =$	0.46%
	45.19%
Total removed after 25 min	45.2%

Beyond the fifth interval the % removals are negligible, so calculations are stopped at that point. In general, if 10% intervals are selected, the total % removal is given by

$$\text{Total \% removal} = X_{\text{total}} + (h_1/H) \times 10 + (h_2/H) \times 10 + (h_3/H) \times 10 + \dots \quad (3.27)$$

Equation (3.27) is simply an approximation of Eq. (3.26).  $(1 - x_0)$  corresponds to  $X_{\text{total}}$  and the integral  $\int_0^{x_0} (h/H) dx$  is replaced by a finite summation of terms.

$$\sum (h_{\text{ave}}/H) \times \Delta x$$

where  $h_{\text{ave}}$  is the average settled depth for each selected interval. The  $\Delta x$ 's in this example are selected arbitrarily as a 10% range. The smaller the  $\Delta x$  selected, the closer the approximation between the finite summation and the integral.

Similar calculations are performed for the other residence times listed in the second column of Table 3.6. The final results are summarized in Table 3.7.

**TABLE 3.7**  
SS (%) Removed vs. Detention Time

$t = (25 \text{ ft})^2 / (V_s)$	(1) $t$ (min)	(2) % SS removal
	3.7	13.4
	6.5	20.1
	14.5	33.9
	25.0	45.2
	39.0	55.0
	56.5	64.3
	77.5	71.1

From Table 3.7 a graph of % SS removed vs. detention time is prepared (Fig. 3.13).

Step 4(c). Prepare a plot of % SS removed vs. overflow rate. Calculations needed to prepare this plot are presented in Table 3.8.

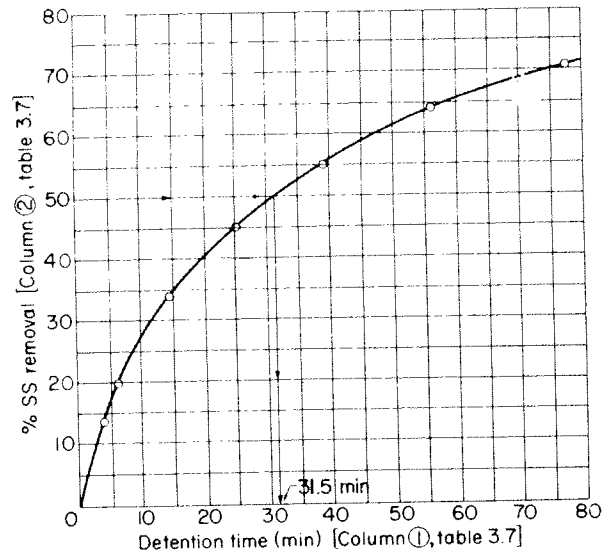


Fig. 3.13. Suspended solids removal (% SS) vs. detention time.

TABLE 3.8  
SS (%) Removed vs. Overflow Rate

(1) $t$ (min)	(2) Settling velocity, $V_s$ (ft/hr) (Table 3.6)	(3) Overflow rate $\left[ \frac{\text{gal}}{(\text{day})(\text{ft}^2)} \right]$ $V_s \left[ \frac{\text{ft}}{\text{hr}} \times \frac{24 \text{ hr}}{\text{day}} \times \frac{7.48 \text{ gal}}{\text{ft}^3} \right] =$ $V_s \times 24 \times 7.48 = 179.5 V_s$	(4) % SS removal [column (2) of Table 3.7]
3.7	97.2	17,450	13.4
6.5	55.2	9,908	20.1
14.5	24.8	4,452	33.9
25.0	14.4	2,585	45.2
39.0	9.2	1,651	55.0
56.5	6.35	1,140	64.3
77.5	4.64	833	71.1

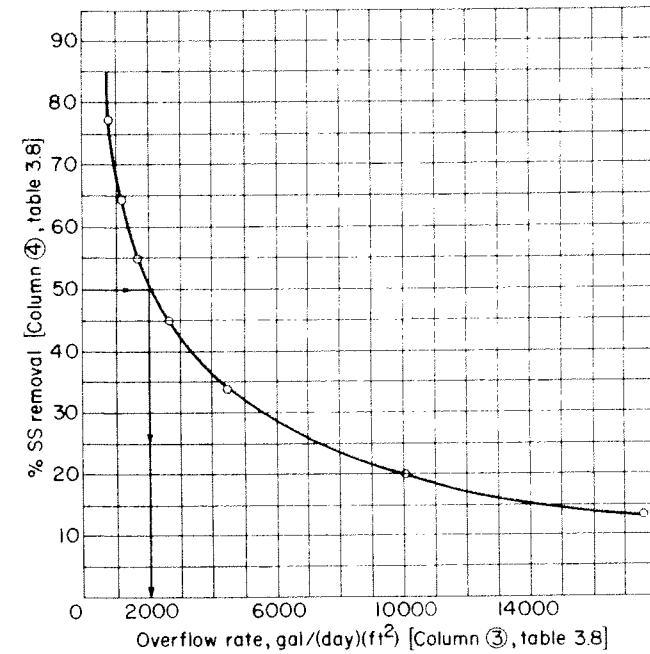


Fig. 3.14. Suspended solids removal (% SS) vs. overflow rate.

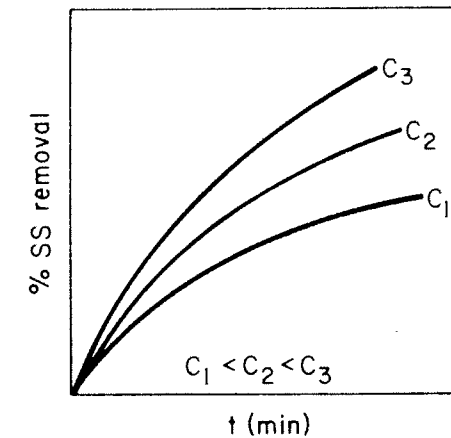


Fig. 3.15. Suspended solids removal (% SS) vs. detention time for different initial SS concentrations.

The plot of % SS removed vs. overflow rate is presented in Fig. 3.14. All calculations are performed for an initial suspended solids concentration  $SS_0$  of 430 mg/liter. If similar calculations are performed for other values of these concentrations ( $C_1, C_2, C_3, \dots$ ), the data plotted in Figs. 3.13 and 3.14 yield families of curves, as indicated in Figs. 3.15 and 3.16.

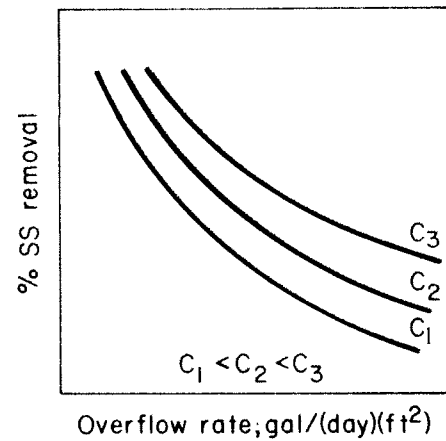


Fig. 3.16. Suspended solids removal (% SS) vs. overflow rate for different initial SS concentrations.

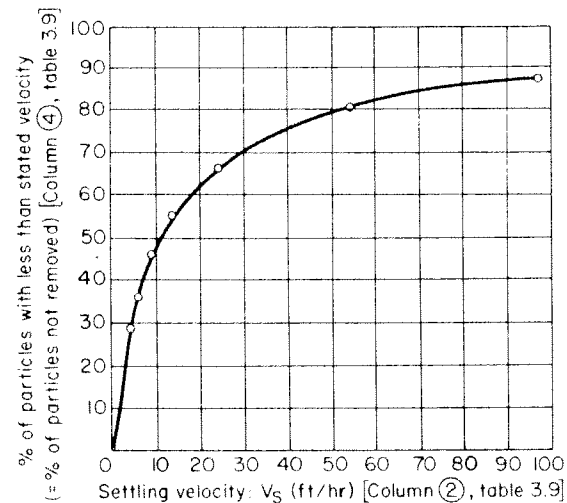


Fig. 3.17. Percentage of particles with less than stated velocity vs. settling velocity.

Step 4(d). Prepare a plot of percentage of particles with less than stated velocity (percentage not removed) vs. settling velocity (ft/hr). Calculations needed to prepare Fig. 3.17 are presented in Table 3.9. Figure 3.17 is not required for the design calculations; it is shown because a typical graph of this type was utilized in developing Eq. (3.26). The plot is presented in Fig. 3.17.

TABLE 3.9  
Percentage of Particles with Less Than Stated Velocity vs. Settling Velocity

(1) $t$ (min)	(2) $V_s$ (ft/hr) (Table 3.6)	(3) % SS removal (Table 3.7)	(4) Percentage not removed: $100 - (\% \text{ SS removal})$
3.7	97.2	13.4	86.6
6.5	55.2	20.1	79.9
14.5	24.8	33.9	66.1
25.0	14.4	45.2	54.8
39.0	9.2	55.0	45.0
56.5	6.35	64.3	35.7
77.5	4.64	71.1	28.9

3.5.1. Design Calculations from Laboratory Data

For purposes of scale-up, the fact that the efficiency of the process in an actual settling tank is reduced owing to the effect of parameters such as turbulence, short circuiting, and interference of the inlet and outlet must be taken into account. The net effect of these factors results in a decrease of the overflow rate and an increase in the detention time over values derived from the laboratory analysis. For design purposes, it is customary to divide the overflow rate obtained from the laboratory analysis by a factor ranging between 1.25 and 1.75, and to multiply the detention time by a factor in the same range [3]. Table 3.10 presents some commonly used design values.

TABLE 3.10  
Design Values (Primary Clarifiers)

Depth: 7-12 ft
Detention time: 1-2 hr
Flow-through velocity, $V = 1-5$ ft/min
Overflow rate: 900-1200 gal/(day)(ft <sup>2</sup> )
Efficiencies
SS removal: 40-60%
BOD removal: 30-50%

Design procedure of a primary clarifier is illustrated by Example 3.4.

#### Example 3.4

It is determined by field observation that a raw wastewater has an average of 430 mg/liter suspended solids at a flow of 1.0 Mgal/day. Data shown in Table 3.3 are obtained from laboratory settling tests.

1. Design a settling tank of circular cross section, i.e., calculate its diameter and effective depth, to remove 50% of the suspended solids at the flow rate of 1.0 Mgal/day.

2. For the tank designed in (1), what is the removal if flow is doubled to 2.0 Mgal/day?

3. For the flow of 1.0 Mgal/day, calculate the daily accumulation of sludge in lb/day and the average pumping rate in gal/min. Sludge concentration is estimated as 1.5% solids from tests made with samples withdrawn from Tap 4 of the laboratory settling column (Fig. 3.9). A plot of % solids for the compacted sludge vs. settling time is constructed from data obtained from samples withdrawn from Tap 4 (Fig. 3.9). A typical plot of this type is shown in Fig. 3.18.

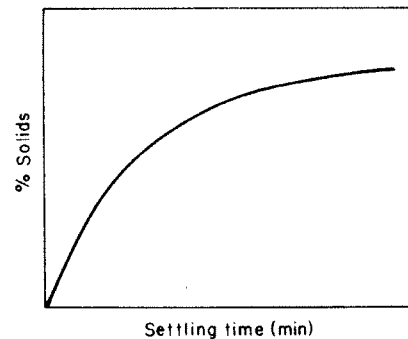


Fig. 3.18. Typical plot of % solids in the sludge vs. detention time.

SOLUTION: Part 1

Step 1. Determine the material balance for SS (see Fig. 3.19).

Influent: 430 mg/liter

Removal:  $(0.50)(430) = 215$  mg/liter

Effluent:  $430 - 215 = 215$  mg/liter

Step 2. Determine the overflow rate. From Fig. 3.14 read overflow rate corresponding to a 50% removal,  $2000 \text{ gal}/(\text{day})(\text{ft}^2)$ . Using a 1.75 scale-up factor, take a design overflow rate of  $2000/1.75 = 1143 \text{ gal}/(\text{day})(\text{ft}^2)$ .

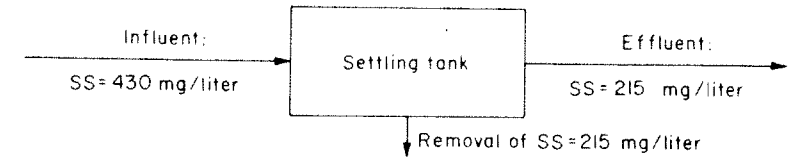


Fig. 3.19. Material balance for primary clarifier (Example 3.4).

Step 3. Determine the detention time. From Fig. 3.13 read detention time corresponding to a 50% removal value,  $t = 31.5$  min.\* Using a 1.75 scale-up factor, take  $t = (31.5)(1.75) = 55.1$  min or  $t = 55.1/60 = 0.92$  hr.

Step 4. Calculate required horizontal cross section of clarifier and its diameter. Horizontal cross section of clarifier is

$$A = \frac{1 \times 10^6 \text{ gal/day}}{1143 \text{ gal}/(\text{day})(\text{ft}^2)} = 874.9 \text{ ft}^2$$

and diameter is

$$D = (4A/\pi)^{1/2} = (874.9/0.785)^{1/2} = 33.4 \text{ ft}$$

Step 5. Calculate effective depth of the clarifier.

$$H = \text{volume}/A = Qt/A$$

$$H = \frac{1 \times 10^6 \text{ gal/day} \times \text{ft}^3/7.48 \text{ gal} \times 0.92/24 \text{ day}}{874.9 \text{ ft}^2} = 5.9 \text{ ft}$$

SOLUTION: Part 2 Removal for a flow rate of 2 MGD with the same clarifier

This amounts to doubling the design overflow rate, i.e.,

$$\text{New design overflow rate} = (2)(1143) = 2286 \text{ gal}/(\text{day})(\text{ft}^2)$$

From Fig. 3.14 this corresponds to a removal of 47.5% of the suspended solids.

SOLUTION: Part 3 Daily accumulation of sludge and average pumping rate for flow of 1.0 MGD

Step 1. Determine the daily accumulation of sludge.

$$\text{Removal of SS} = 215 \text{ mg/liter} \rightarrow 215 \times 10^{-6} \text{ lb SS/lb liquor}$$

Therefore, the daily accumulation of sludge in lb/day is

$$\begin{aligned} &1 \times 10^6 \text{ gal liquor/day} \times 8.34 \text{ lb liquor/gal liquor} \times 215 \times 10^{-6} \text{ lb SS/lb liquor} \\ &= 1793 \text{ lb SS/day} \end{aligned}$$

\* From this value of the residence time ( $t = 31.5$  min), the % solids in the sludge is estimated as 1.5% from a curve of the type in Fig. 3.19.

Step 2. Calculate the average pumping rate. Notice that 1.5% solids corresponds to 1.5 g SS/100 g of liquor = 15 g SS/1000 g of liquor  $\approx$  15 g SS/liter = 15,000 mg/liter = ppm =  $15,000 \times 10^{-6}$  lb SS/lb liquor =  $15,000 \times 8.34 \times 10^{-6}$  lb SS/gal liquor. Since accumulation is 1793 lb SS/day, pumping rate in gal/day is

$$\frac{1793 \text{ lb SS/day}}{(15,000 \times 8.34 \times 10^{-6}) \text{ lb SS/gal liquor}} = 0.0143 \times 10^6 \text{ gal/day}$$

or

$$(0.0143 \times 10^6)/(24 \times 60) = 9.93 \text{ gal/min}$$

Since pumping rate is low, intermittent pumping is used.

### 3.6. ZONE SETTLING

Zone settling occurs in clarifiers of activated or chemically coagulated sludge when the concentration exceeds 500 mg/liter. The sludge blanket exhibits several distinct zones. Each zone is characterized by a specific sludge concentration and settling velocity. Consider what happens when a suspension which initially has a uniform sludge concentration  $C_0$  (mg/liter) is placed in a settling cylinder (Fig. 3.20).

Sludge begins to settle out and an interface (interface 1) is established between the surface of the blanket of settling sludge and the clarified liquid above. The zone below the clarified liquid is called the interfacial zone. Concentration of the sludge in this zone is uniform, and it settles as a blanket with a constant velocity ( $V_s$ ). Simultaneously with formation of interface 1

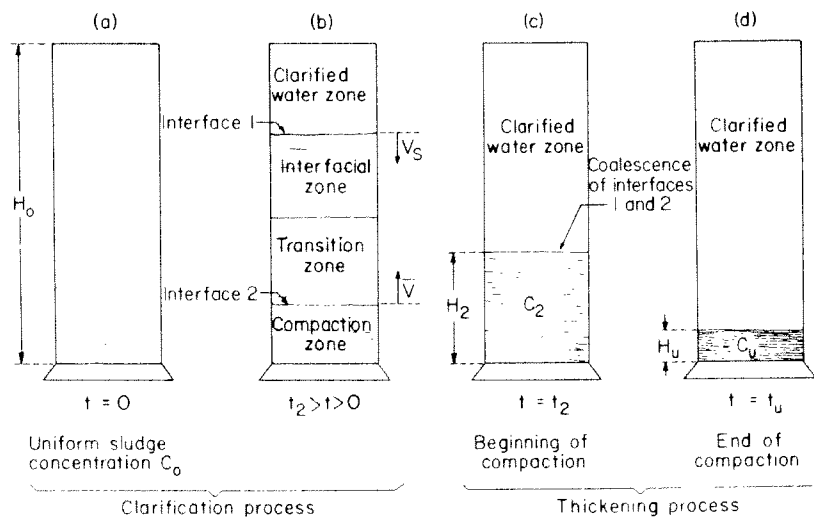


Fig. 3.20. Zone settling.

and interfacial zone, compaction of suspended solids starts at the bottom of the cylinder (compaction zone). In this zone concentration of SS is also uniform, and the interface bordering this zone (interface 2) rises in the cylinder with a constant velocity ( $\bar{V}$ ).

Between the interfacial and compaction zones there is a transition zone. There, the velocity of settling solids decreases owing to increase of viscosity and density of the suspension. In this same zone, sludge changes gradually in concentration from that of the interfacial zone to that of the compaction zone.

Consider interfaces 1 and 2 in Fig. 3.20(b). Interface 1 moves downward with a constant velocity  $V_s$ , whereas interface 2 moves upward with a constant velocity  $\bar{V}$ . Eventually, interfacial and compaction zones meet, at which time ( $t_2$ ) the transition zone fades away [Fig. 3.20(c)]. At this time, the settled sludge exhibits a uniform concentration  $C_2$ , which is termed the critical concentration. Compaction starts and the sludge begins to thicken, eventually reaching an ultimate concentration  $C_u$  [Fig. 3.20(d)]. Sedimentation velocity at time  $t_2$  corresponds to a value  $V_2$ , which is given by the slope of the tangent to the settling curve at  $C_2$ , as indicated in Fig. 3.21 where  $V_2 < V_s$ .

Procedure for designing clarifiers operating under conditions of zone settling:

1. Calculate the minimum surface area required to allow for clarification of sludge.
2. Calculate the minimum surface area required to provide for thickening of sludge to the desired underflow concentration.
3. Take the larger of these two areas as the design area for the clarifier.

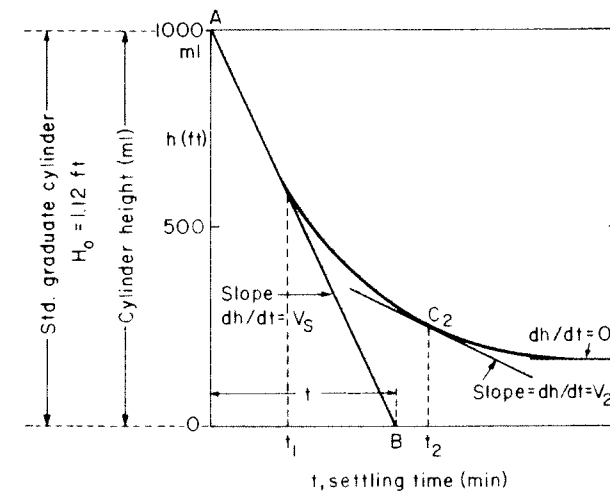


Fig. 3.21. Sludge settling curve.



### 3.6.1. Laboratory Measurements

To obtain the parameters necessary for design of the clarifier, a settling test for the sludge is performed in the laboratory using a 1000-ml graduated cylinder (a standard graduate cylinder has a height of 1.12 ft). The cylinder is filled with the slurry to be studied. At the beginning of the experiment ( $t = 0$ ), slurry concentration is uniform throughout the cylinder. Height of interface 1 is recorded at selected time intervals. This yields the type of settling curve shown in Fig. 3.21.

It is important to stir the suspension at a rate of about 5 rph. This stirring simulates the action of the mechanical rakes utilized in sludge removal and prevents stratification of the sludge.

Figure 3.21 shows that from the start of the experiment up to a time  $t_1$ , interface 1 falls with a constant velocity  $V_S$  given by the slope of the tangent, which essentially coincides with the settling curve from  $t = 0$  to  $t = t_1$ . After  $t = t_1$  this velocity decreases appreciably. At time  $t = t_2$ , the velocity is  $V_2$  given by the slope of the tangent at  $C_2$ . At  $t_2$  compaction starts and the velocity is further reduced until it becomes essentially zero, the tangent being parallel to the abscissa.

Zone settling velocity (ZSV) corresponds to the velocity at which the suspension settles prior to reaching the critical concentration  $C_2$ , and is given by the slope of the tangent AB in Fig. 3.21 [Eq. (3.28)].

$$V_S = \overline{OA}/\overline{OB} = H_0/t = 1.12 \text{ ft/min} \quad (3.28)$$

### 3.6.2. Determination of Minimum Surface Area Required to Allow Clarification of the Sludge

Minimum surface area  $A_c$  required for clarification depends on velocity  $V_S$  at which the suspension settles before reaching the interfacial critical concentration  $C_2$ . Under continuous flow conditions, velocity of the liquor over the overflow weir cannot exceed  $V_S$  if clarification is to take place. This follows directly from the basic concept of the ideal sedimentation tank [Section 3.4, Eqs. (3.17) and (3.19)].

Therefore,

$$A_c = Q/V_S \quad (3.29)$$

where  $Q$  is the flow rate ( $\text{ft}^3/\text{min}$ );  $V_S$  the settling velocity ( $\text{ft}/\text{min}$ ); and  $A_c$  the minimum surface area required for clarification ( $\text{ft}^2$ ).

The value of the zone settling velocity  $V_S$  is determined from Fig. 3.21 and Eq. (3.28). Value of  $t$  is read directly from the abscissa of Fig. 3.21 (point B).  $V_S$  is then calculated from Eq. (3.28) and  $A_c$  obtained from Eq. (3.29).

### 3.6.3. Determination of Minimum Surface Area Required for Thickening of the Sludge

Consider settling of a sludge under zone settling conditions in a cylinder (Fig. 3.20). At start of the experiment, let  $C_0$  be the uniform sludge concentration throughout the cylinder. Total weight of solids in the cylinder is  $C_0 A H_0$ , where  $A$  is the cross-sectional area of the cylinder. Let  $t_2$  be the time counted from the beginning of the experiment, when interfacial and compaction zones merge together [Fig. 3.20(c)]. Let  $C_2$  be the critical concentration which is uniform throughout this sludge zone formed by the merging of interfacial and compaction zones.

A graphical procedure has been proposed [7] for determining  $t_2$ . Consider the settling curve in Fig. 3.22. Draw two tangents (AB and CD) to the two branches of the curve. Tangent AB corresponds to the constant velocity

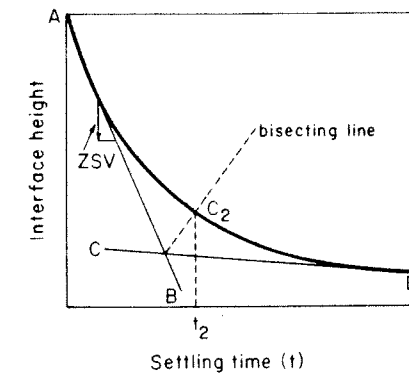


Fig. 3.22. Determination of  $t_2$ .

of settling  $V_S$  for the interfacial zone (zone settling velocity, ZSV), and tangent CD corresponds to the settling velocity for the compacted sludge. Point  $C_2$  (corresponding to time  $t_2$ ) is obtained by bisecting the angle formed by tangents AB and CD. The abscissa of the point where the bisecting line cuts the settling curve corresponds to the desired value of  $t_2$ .

Consider now the thickening process.

1. Start of thickening [Fig. 3.20(c)]. Time,  $t_2$ ; concentration of SS in sludge zone,  $C_2$ ; height of sludge zone,  $H_2$ .
2. End of thickening [Fig. 3.20(d)]. The compacted sludge reaches desired underflow concentration  $C_u$ . The time at which this occurs is designated as  $t_u$ . Height of the sludge zone is  $H_u$ .

Consider separately the sludge zone at the start and end of thickening (Fig. 3.23). Since the total mass of sludge in the cylinder is constant, the

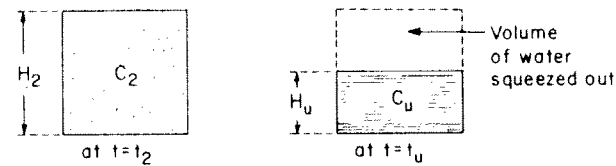


Fig. 3.23. Thickening process.

following material balance equation [Eq. (3.30)] is written, neglecting the amount of suspended solids in the clarified water zone.

$$C_0 A H_0 = C_2 A H_2 = C_u A H_u \quad (3.30)$$

or

$$C_0 H_0 = C_2 H_2 = C_u H_u \quad (3.31)$$

Consider Fig. 3.23. The volume of water which is squeezed out and discharged over the overflow weir is calculated from Eq. (3.32).

$$V = A(H_2 - H_u) \quad (3.32)$$

The time interval required to discharge this volume of water is  $t_u - t_2$ . Average rate of flow  $Q'$  ( $\text{ft}^3/\text{min}$ ) over the weir is [Eq. 3.33]

$$Q' = V/(t_u - t_2) = A(H_2 - H_u)/(t_u - t_2) \quad (3.33)$$

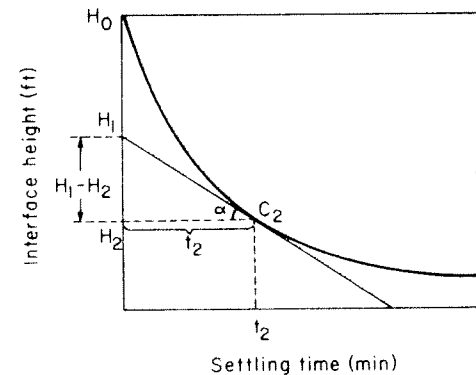
Solving for  $t_u - t_2$ ,

$$t_u - t_2 = A(H_2 - H_u)/Q' \quad (3.34)$$

Consider now the settling curve and determine graphically the settling velocity  $V_2$  at time  $t_2$  (tangent at point  $C_2$ ). This is shown in Fig. 3.24.

Let  $H_1$  be the intercept of this tangent. Settling velocity at  $t_2$  is shown in Eq. (3.35).

$$V_2 = \tan \alpha = (H_1 - H_2)/t_2 \quad (3.35)$$

Fig. 3.24. Determination of velocity  $V_2$ .

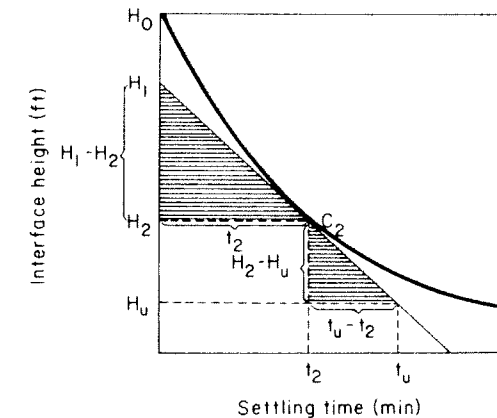
Under continuous flow conditions the velocity of the liquor over the weir cannot be greater than  $V_2$  if thickening is to take place. Therefore, the flow rate  $Q'$  at time  $t_2$  when thickening starts is

$$Q' = AV_2 = A[(H_1 - H_2)/t_2] = \text{ft}^3/\text{min} \quad (3.36)$$

Substitution of  $Q'$  given by Eq. (3.36) in Eq. (3.34) yields after simplification and rearrangement Eq. (3.37).

$$(H_2 - H_u)/(t_u - t_2) = (H_1 - H_2)/t_2 \quad (3.37)$$

This equation is the basis for the graphical procedure for determination of  $t_u$  illustrated by Fig. 3.25.

Fig. 3.25. Determination of  $t_u$ .

To summarize the steps in the graphical procedure for determination of  $t_u$ :

1. Draw the tangent to the settling curve at  $C_2$ .
2. From material balance [Eq. (3.31)]

$$H_u = H_0 C_0 / C_u \quad (3.38)$$

Calculate  $H_u$  from Eq. (3.38).

3. Mark distance  $H_u$  on the ordinate axis of Fig. 3.25. Draw the horizontal dotted line from  $H_u$  until its intersection with the tangent to  $C_2$ . The abscissa of this intersection is the value of  $t_u$ . This can be seen by inspection of Eq. (3.37) and consideration of the two cross-hatched similar triangles in Fig. 3.25.

Minimum surface area required for thickening ( $A_t$ ) is obtained from the following considerations. Average rate at which the layer of concentration  $C_u$  forms (in  $\text{lb}/\text{min}$ ) is

$$C_u H_u A_t / t_u \quad (3.39)$$

Since from Eq. (3.31)  $C_u H_u = C_0 H_0$ , then Eq. (3.39) is rewritten as

$$C_0 H_0 A_t / t_u \quad (3.40)$$

Under conditions of continuous flow and steady state, the rate at which the layer of concentration  $C_u$  is formed must equal that at which suspended solids enter in the influent ( $QC_0$ ). Therefore

$$QC_0 = C_0 H_0 A_t / t_u \quad (3.41)$$

Solving for  $A_t$ ,

$$A_t = Qt_u / H_0 \quad (3.42)$$

where  $H_0 = 1.12$  ft.

The design procedure for clarifiers under zone settling conditions is illustrated by Example 3.5.

### Example 3.5

Design a secondary settling tank to produce an underflow concentration of 10,900 mg/liter from an influent containing 2510 mg/liter of suspended solids. Wastewater flow is 1.2 MGD. Calculate the clarifier area required.

The data tabulated below are obtained in a laboratory test of the slurry.

$t$ (min)	Interface height $H$ (m)
0	1000
1	850
2	725
3	600
5	450
8	350
12	280
16	240
20	220
25	210

### SOLUTION

*Step 1.* Settling curve is plotted from available data (Fig. 3.26).

*Step 2.* Minimum surface area required for clarification ( $A_c$ ) is determined.

1. Draw tangent AB. Read  $t = 7.5$  min. Then  $V_s = H_0/t = 1.12/7.5 = 0.149$  ft/min.

2. Area required for clarification:

$$A_c = \frac{1,200,000 \text{ gal/day} \times \text{ft}^3/7.48 \text{ gal} \times \text{day}/1440 \text{ min}}{0.149 \text{ ft/min}} = 748 \text{ ft}^2$$

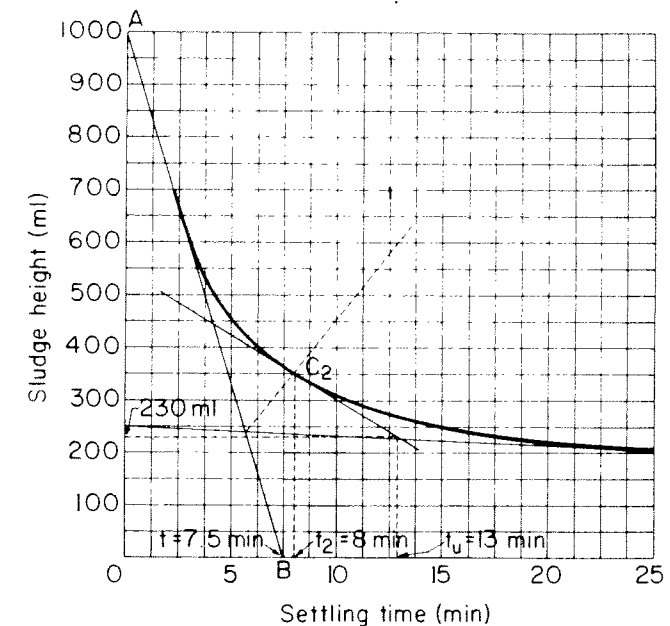


Fig. 3.26. Graph for Example 3.5.

*Step 3.* Minimum surface area required for thickening ( $A_t$ ) is calculated.

1. Determine  $t_2$  by the graphical procedure studied. From Fig. 3.26 read  $t_2 = 8.0$  min.

2. Determine time  $t_u$ . Draw the tangent to the settling curve at  $C_2$  and calculate  $H_u$  from Eq. (3.28).

$$H_u = (1000 \times 2510) / 10,900 = 230 \text{ ml}$$

3. Determine  $t_u$  by the graphical procedure described. From Fig. (3.26) read  $t_u = 13$  min.

4. Calculate  $A_t$  from Eq. (3.42).

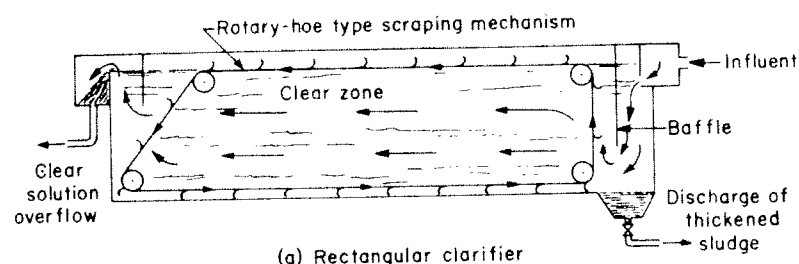
$$A_t = \frac{1,200,000 \text{ gal/day} \times \text{ft}^3/7.48 \text{ gal} \times \text{day}/1440 \text{ min} \times 13 \text{ min}}{1.12 \text{ ft}} = 1293 \text{ ft}^2$$

*Step 4.* Take  $A = A_t = 1293 \text{ ft}^2$ . Required diameter for a sedimentation tank of circular cross section is

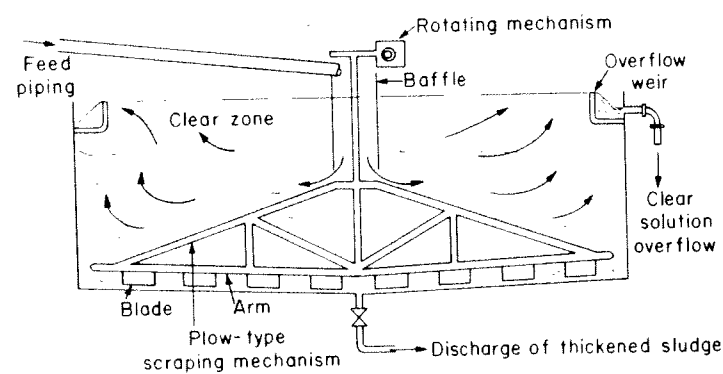
$$d = (4A/\pi)^{1/2} = (1293/0.785)^{1/2} = 40.6 \text{ ft}$$

### 3.7. TYPES OF CLARIFIERS

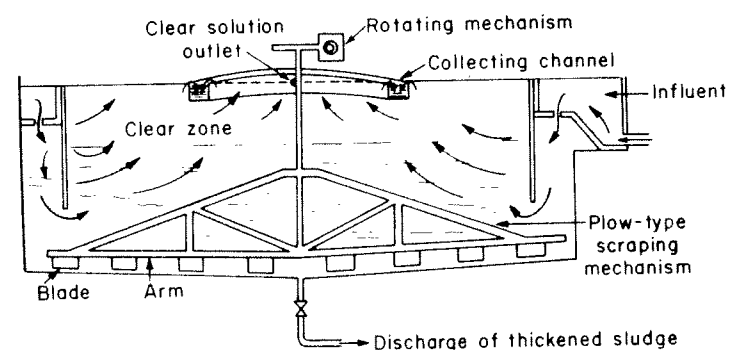
Clarifiers are classified according to geometry of their horizontal cross section as (1) rectangular and (2) circular: (a) center and (b) peripheral feed. Sketches of typical clarifiers are shown in Figs. 3.27(a), (b), and (c).



(a) Rectangular clarifier



(b) Circular clarifier (center feed)



(c) Circular clarifier (peripheral feed)

Fig. 3.27 Types of clarifiers (a), (b), and (c).

## 1. Rectangular clarifier [Fig. 3.27(a)]

In the type shown in Fig. 3.27(a), scraped sludge is moved toward the inlet end of the tank. Some other designs move sludge toward the effluent end of the tank.

Scraping mechanism shown is of rotary-hoe type, consisting of a series of short scrapers mounted on an endless chain, which make contact with the bottom of the tank. It moves slowly at speeds of approximately 1 ft/min.

## 2a. Circular clarifier with center feed [Fig. 3.27(b)]

Feed is at the center and clear solution overflows to a collecting channel at the periphery. The bottom of the clarifier has a minimum slope of 1 in./ft. Scraping mechanism is of plow type to overcome inertia and prevent adherence of sludge to the bottom of the tank.

## 2b. Circular clarifier with peripheral feed [Fig. 3.27(c)]

The feed is at the periphery and the clear solution overflows to a collecting channel at the center. The other details are similar to those for the type shown in Fig. 3.27(b).

The inlet section should be carefully designed for a uniform flow distribution across the width and depth of the tank. Similarly, the outlet section should be designed to collect the effluent uniformly.

A good design of inlet and outlet sections reduces possibilities of flow short circuiting, which lead to poor performance of the clarifier. Proper positioning of weirs and baffles, as indicated in Fig. 3.27, prevents short circuiting.

## 4. Flotation

## 4.1. INTRODUCTION

Flotation is a process for separating low density solids or liquid particles from a liquid phase. Separation is brought about by introduction of gas (usually air) bubbles into the liquid phase. The liquid phase is pressurized to an operating pressure ranging from 30 to 60 psia (2–4 atm) in presence of sufficient air to promote saturation of air in the water. Then, this air-saturated liquid is depressurized to atmospheric pressure by passage through a pressure-reducing valve. Minute air bubbles are released from the solution because of depressurization. Suspended solids or liquid particles, e.g., oil, are floated by these minute air bubbles, causing them to rise to the surface of the tank. Concentrated suspended solids are skimmed off by mechanical means from the tank surface. Clarified liquor is withdrawn near the bottom, and part of it may be recycled [Fig. 3.28(b)]. A flotation system without recycle is shown diagrammatically in Fig. 3.28(a).

In the field of wastewater treatment, flotation is used for the following purposes: (1) separation of greases, oils, fibers, and other low density solids from wastewaters; (2) thickening of the sludge from the activated sludge

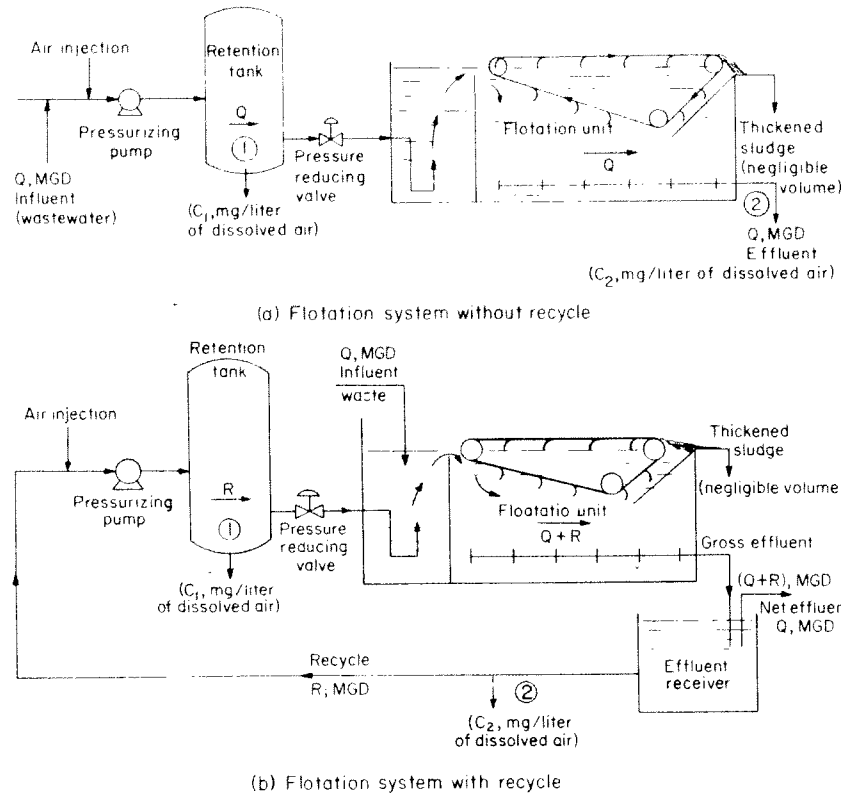


Fig. 3.28. Flotation systems (a) and (b).

process; and (3) thickening of flocculated chemical sludges resulting from chemical coagulation treatment. Superior effluent quality, i.e., effluent containing lower percentage of suspended solids, and economy in power are achieved by flotation systems with recycle.

Basic components of a flotation system are (1) pressurizing pump; (2) air injection facilities; (3) retention tank (to provide air-liquid contact); (4) pressure-reducing valve; and (5) flotation tank.

## 4.2. EVALUATION OF FLOTATION VARIABLES FOR PROCESS DESIGN

### 4.2.1. Parameter $A/S$

For design of flotation systems, a fundamental parameter commonly utilized is a dimensionless air to solids ratio ( $A/S$ ) defined by Eq. (3.43).

$$A/S = \frac{\text{lb/day of air released by depressurization}}{\text{lb/day of solids in the influent}} \quad (3.43)$$

This parameter is estimated from studies with a laboratory-scale flotation cell of pilot-plant data. The value of  $A$  is obtained from determinations of dissolved air (mg/liter) at sampling locations indicated as (1) and (2) in Fig. 3.28(a) and (b). Thus [Eq. (3.44)]

$$A = A_1 - A_2 \quad (3.44)$$

where  $A$  is the lb/day of air released by depressurization;  $A_1$  the lb/day of dissolved air at (1) [Fig. 3.28(a) and (b)]; and  $A_2$  the lb/day of dissolved air at (2) [Fig. 3.28(a) and (b)].

For flotation systems without recycle [Eq. (3.45)],\*

$$A_1 = Q \frac{\text{Mgal liquor}}{\text{day}} \times C_1 \frac{\text{lb air}}{\text{Mlb liquor}} \times 8.34 \frac{\text{lb liquor}}{\text{gal liquor}} = 8.34QC_1 \quad (\text{lb air/day}) \quad (3.45)$$

Similarly [Eq. (3.46)]

$$A_2 = 8.34QC_2 \quad (\text{lb air/day}) \quad (3.46)$$

Therefore

$$A = A_1 - A_2 = 8.34Q(C_1 - C_2) \quad (\text{lb/day of air released}) \quad (3.47)$$

For flotation systems with recycle the corresponding equation is

$$A = A_1 - A_2 = 8.34R(C_1 - C_2) \quad (\text{lb/day of air released}) \quad (3.48)$$

If  $S_i$  is the concentration of suspended solids (mg/liter) in the influent, the value of  $S$  [denominator of Eq. (3.43)] is

$$S = Q \frac{\text{Mgal liquor}}{\text{day}} \times \frac{8.34 \text{ lb liquor}}{\text{gal liquor}} \times S_i \frac{\text{lb SS}}{\text{Mlb liquor}} = 8.34QS_i \quad (\text{lb SS/day}) \quad (3.49)$$

Substitution of Eqs. (3.47) [or Eq. (3.48)] and (3.49) in Eq. (3.43) leads to Flotation systems without recycle

$$A/S = (C_1 - C_2)/S_i \quad (3.50)$$

Flotation systems with recycle

$$A/S = (R/Q)(C_1 - C_2)/S_i \quad (3.51)$$

### 4.2.2. Correlation of Flotation Variables to Parameter $A/S$

By use of a laboratory flotation cell or pilot-plant data, it is possible to correlate mg/liter of suspended solids contained in liquid effluent to parameter  $A/S$ . A typical correlation curve for a wastewater has the shape indicated in Fig. 3.29.

\* Notice that  $C_1$  mg of air/liter of liquor =  $C_1$  lb of air/Mlb of liquor.

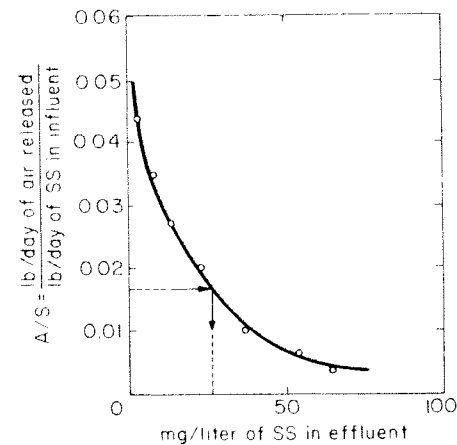


Fig. 3.29. Typical correlation of parameter  $A/S$  vs. concentration of SS in effluent.

For a given influent, the lb/day of suspended solids (term  $S$  in ratio  $A/S$ ) is fixed. Ratio  $A/S$  increases by operating at higher air rates, which results in increase of air released ( $A$ ). From Fig. 3.29 it follows that a higher quality effluent is obtained.

Graphs like Fig. 3.29 permit selection of the  $A/S$  ratio for a required degree of effluent clarification. These curves indicate that increasing the  $A/S$  ratio beyond an optimum value does *not* result in substantial reduction in effluent suspended solids.

Judicious selection of the  $A/S$  ratio involves an economical balance between equipment and maintenance costs and desired effluent quality. Typical range of  $A/S$  ratios for thickening of sludges in wastewater treatment is 0.005–0.060.

#### 4.2.3. Alternative Expressions for Parameter $A/S$ [2]

Consider Eq. (3.46) for  $A_2$ , where  $C_2$  is the solubility of air in water in mg/liter. Frequently, the solubility of air is expressed in terms of  $\text{cm}^3$  of air/liter of water. It is assumed that conditions at (2) [Figs. 28(a) and (b)] are atmospheric pressure and ambient temperature. Solubility of air in water in  $\text{cm}^3$  of air/liter of water [hence denoted as  $S_a$ ] is presented in Table 3.11 for atmospheric pressure at several temperatures.

This concentration of dissolved air in  $\text{cm}^3$  of air/liter of water is converted to the value  $C_2$  (mg air/liter of water) [Eq. (3.52)].

$$S_a \text{ cm}^3 \text{ air/liter water} \times \rho_a \text{ mg air/cm}^3 \text{ air} = C_2 \quad (\text{mg air/liter water}) \quad (3.52)$$

TABLE 3.11  
Solubility of Air in Water at Atmospheric Pressure at Several Temperatures [6]

Temperature ( $^{\circ}\text{C}$ )	$S_a$ ( $\text{cm}^3/\text{liter}$ )
0	29.2
10	22.8
20	18.7
30	15.7

Utilize for the density of air  $\rho_a$  a mean value of  $1.2 \text{ mg/cm}^3$ . (This corresponds to the value at 1 atm and  $20^{\circ}\text{C}$ .) Then [Eq. (3.53)]

$$C_2 = 1.2S_a \quad (3.53)$$

Consider now Eq. (3.45), where  $C_1$  is the solubility of air in mg/liter. The  $\text{cm}^3$  of air/liter of water at the retention tank [point (1)] is shown in Eq. (3.54).

$$\varphi S_a \quad (\text{cm}^3 \text{ air/liter water}) \quad (3.54)$$

where  $\varphi$  is the ratio of solubility of air in water at the pressure in the retention tank [at (1)] to the solubility at atmospheric pressure [at (2) in Figs. 28(a) and (b)] [Eq. (3.55)].

$$\varphi = C_1/C_2 \quad (\varphi > 1.0) \quad (3.55)$$

It is found experimentally that for a specific design of the retention tank, and within ordinary pressure ranges utilized in flotation operations, ratio  $\varphi$  is proportional to the pressure

$$\varphi = fP \quad (3.56)$$

where  $f$  is the proportionality factor and  $P$  the pressure in retention tank in atmospheres.

At  $20^{\circ}\text{C}$  for pressure range 30–60 psia utilized in most flotation systems and for baffled retention tanks, the value of  $f$  in Eq. (3.56) is approximately 0.5. Consequently, values of  $\varphi$  vary from

$$30 \text{ psia} (= 30/14.7 = 2.04 \text{ atm}) = 0.5 \times 2.04 = 1.02$$

$$60 \text{ psia} (= 60/14.7 = 4.08 \text{ atm}) = 0.5 \times 4.08 = 2.04$$

Therefore for the pressure range from 30 to 60 psia at  $20^{\circ}\text{C}$ , solubility of air in baffled retention tanks varies from 1.02 to 2.04 times its saturation value at  $20^{\circ}\text{C}$  and 1 atm. Since from Eq. (3.55)

$$C_1 = \varphi C_2 \quad (3.57)$$

substitution of  $\varphi$  and  $C_2$  by their values given by Eqs. (3.56) and (3.53) yields

$$C_1 = fP(1.2S_a) \quad (3.58)$$

Substitution of values of  $C_1$  and  $C_2$  given, respectively, by Eqs. (3.58) and (3.53) in Eqs. (3.50) and (3.51) leads to

Flotation systems without recycle

$$A/S = 1.2S_a(fP - 1)/S_i \quad (3.59)$$

Flotation systems with recycle

$$A/S = (R/Q)1.2S_a(fP - 1)/S_i \quad (3.60)$$

### 4.3. DESIGN PROCEDURE FOR FLOTATION UNITS WITHOUT AND WITH RECYCLE

#### 4.3.1. Flotation Systems without Recycle

From Eq. (3.59) it follows that if a ratio  $A/S$  is selected,  $f$  being fixed for a selected type of retention tank and  $S_i$  from characteristics of the influent, this amounts to specification of operating pressure  $P$ , which is calculated from Eq. (3.59) as

$$P = (1/f)[(A/S)S_i/1.2S_a + 1] \quad (3.61)$$

Design of flotation systems without recycle involves calculating the required operating pressure [Eq. (3.61)] and determining the cross-sectional area of the flotation unit. This area is calculated from a selected value of the overflow rate, usually a value between 2 and 4 gal/(min)(ft<sup>2</sup>) (Example 3.6).

#### Example 3.6

Laboratory flotation tests for a given wastewater indicate optimum air/solids ratio ( $A/S$ ) as 0.04 lb air/lb solids. Flow of wastewater is 1.0 MGD and it contains 250 mg/liter of suspended solids. Laboratory flotation tests (without recycle) indicate for a ratio  $A/S = 0.04$  an optimum effluent containing 25 mg/liter of suspended solids. Take  $f = 0.50$  for retention tank and an operating temperature of 20°C. Design a flotation system without recycle for this service.

#### SOLUTION

*Step 1.* Select  $A/S = 0.04$ , as stated.

*Step 2.* Calculate  $P$  from Eq. (3.61).

$$P = (1/0.5)(0.04 \times 250/1.2 \times 18.7 + 1) = 2.9 \text{ atm}$$

*Step 3.* Select an overflow rate,

$$OR = 3 \text{ gal}/(\text{min})(\text{ft}^2)$$

*Step 4.* Calculate required surface area,

$$Q = 1.0 \text{ MGD}$$

or

$$Q = 1,000,000 \text{ gal/day} \times \text{day}/24 \text{ hr} \times \text{hr}/60 \text{ min} = 695 \text{ gal/min}$$

$$\text{Surface area (ft}^2\text{)} = Q/OR = \frac{695 \text{ gal/min}}{3.0 \text{ gal}/(\text{min})(\text{ft}^2)} = 232 \text{ ft}^2$$

#### 4.3.2. Flotation Systems with Recycle

It follows from Eq. (3.60) that for a specific application (i.e., for fixed values of  $f$  and  $S_i$ ) one must specify not only the  $A/S$  ratio but also the recycle before the operating pressure  $P$  becomes fixed. The usual procedure is to specify an operating pressure  $P$  and an  $A/S$  ratio and calculate the required recycle from Eq. (3.60), which solved for  $R$  yields Eq. (3.62).

$$R = (A/S)QS_i/1.2S_a(fP - 1) \quad (3.62)$$

The design procedure for flotation systems with recycle is illustrated by Example 3.7.

#### Example 3.7

For the application in Example 3.6 design a flotation system with recycle, taking an operating pressure of 2.9 atm.

#### SOLUTION

*Step 1.* Select  $A/S = 0.04$ , as stated.

*Step 2.* Calculate  $R$  from Eq. (3.62).

$$R = (0.04)(1.0)(250)/1.2 \times 18.7(0.5 \times 2.9 - 1) = 0.99 \text{ MGD} \approx 1.0 \text{ MGD}$$

This means that recycle ratio  $R/Q$  is approximately unity.

*Step 3.* Select an overflow rate

$$OR = 3 \text{ gal}/(\text{min})(\text{ft}^2)$$

*Step 4.* Required surface area is defined as

$$Q + R \approx 2.0 \text{ MGD}$$

or

$$2,000,000 \text{ gal/day} \times \text{day}/24 \text{ hr} \times \text{hr}/60 \text{ min} = Q + R = 1390 \text{ gal/min}$$

$$\text{Surface area (ft}^2\text{)} = (Q + R)/OR = 1390/3.0 = 464 \text{ ft}^2$$

Surface area is twice as large as for the unit without recycle. However, an effluent of superior quality (i.e., SS < 25 mg/liter) is obtained.

## 5. Neutralization (and Equalization)

### 5.1. NEUTRALIZATION IN THE FIELD OF WASTEWATER TREATMENT

Neutralization treatment is often utilized in the following cases arising in wastewater treatment:

1. Prior to discharge of the wastewater into a receiving water. The justification for neutralization is that aquatic life is sensitive to pH variations beyond a narrow range around pH 7.

2. Prior to discharge of industrial wastewaters to the municipal sewer system. Specification of the pH of industrial discharges into municipal sewer systems is frequently made. It is more economical to neutralize industrial wastewater streams prior to the discharge into the municipal sewer, rather than attempting to perform neutralization of the larger volume of combined domestic and industrial sewage.

3. Prior to chemical or biological treatment. For biological treatment, pH of the system is maintained within the range 6.5–8.5 to ensure optimum biological activity. The biological process itself provides a neutralization and buffer capacity as a result of production of  $\text{CO}_2$ , which forms carbonates and bicarbonates in solution. The degree of preneutralization required for biological treatment depends on two factors: (1) the alkalinity or acidity present in the wastewater and (2) the mg/liter BOD to be removed in the biological treatment. The latter is related to the production of  $\text{CO}_2$ , which may provide for partial neutralization of alkaline wastes.

### 5.2. METHODS FOR NEUTRALIZATION OF WASTEWATERS

Methods for neutralization of wastewaters include (1) equalization, which consists of mixing acidic and alkaline waste streams available in the plant and (2) direct pH control methods, which consist of addition of acids (or bases) for neutralization of alkaline (or acidic) wastewater streams.

### 5.3. EQUALIZATION

When utilized for purpose of neutralization, equalization involves mixing wastewater streams of acidic and alkaline nature in an equalization basin. Equalization is often used for purposes other than neutralization such as (1) to smooth out individual wastewater stream flow variations, so that a composite stream of relatively constant flow rate is fed to the treatment plant; and (2) to smooth out variations in influent BOD to the treatment facilities. Constant and variable level equalization basins are utilized.

1. *Constant level equalization basins.* This arrangement is illustrated in Fig. 3.30. The level in the equalization basin is held constant. Therefore as

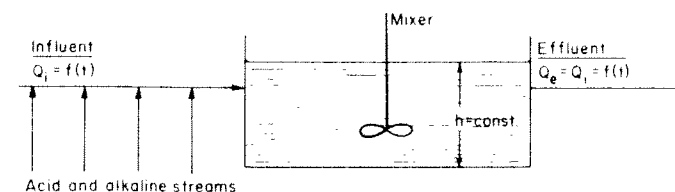


Fig. 3.30. Constant level equalization basin.

the rate of flow of influent varies, that of effluent is equally affected. Consequently, this is not a technique of flow equalization, simply a method of neutralization. If fluctuations in the flow rate are too great, the effluent from the constant level equalization basin is fed to another equalization basin having as objective flow equalization.

2. *Variable level equalization basins.* In this method of equalization, the effluent is taken out at a constant rate, and since the flow rate of influent varies with time, the level of the equalization basin is variable. This method is also utilized for the purpose of flow equalization, as well as providing for neutralization. A diagram of a variable level equalization basin is shown in Fig. 3.31.

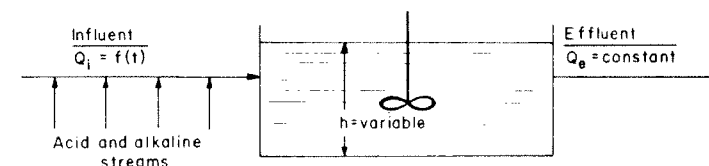


Fig. 3.31. Variable level equalization basin.

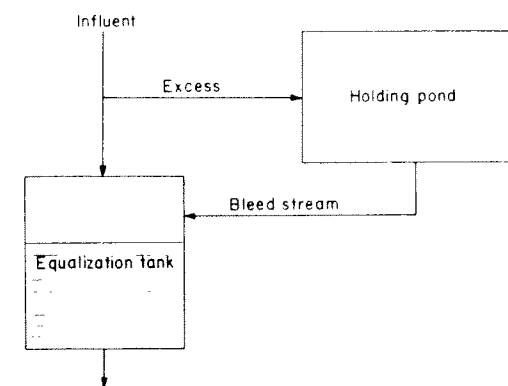


Fig. 3.32. "Holding pond" method of equalization.



Another method of equalization consists of diverting the "excess" of the incoming stream to a holding pond, from which a bleed stream is fed to the equalization tank. This method is not used for neutralization purposes, but only for equalization of BOD content or flow rate. This is illustrated by Fig. 3.32.

#### 5.4. DIRECT pH CONTROL METHODS: NEUTRALIZATION OF ACIDIC WASTES BY DIRECT pH CONTROL METHODS

The following methods of direct neutralization of acidic wastes are the most commonly employed: (1) limestone beds, (2) slurried lime neutralization, (3) caustic soda (NaOH) neutralization, (4) sodium carbonate neutralization, and (5) ammonia neutralization.

A few specific comments about these methods are as follows:

Slurried lime neutralization is the most common method and is discussed in Section 5.6. Design of limestone beds is described in Section 5.5.

Caustic soda (NaOH) is more expensive than lime. It offers an advantage with respect to uniformity of the reagent, ease of storage and feeding, rapid reaction rate, and the fact that the end products of neutralization (sodium salts) are soluble.

Sodium carbonate ( $\text{Na}_2\text{CO}_3$ ) is not as reactive as caustic soda and presents frothing problems owing to release of carbon dioxide.

Ammonia ( $\text{NH}_4\text{OH}$ ) presents the disadvantage of being a contaminant; consequently its use may be ruled out by pollution control standards.

Factors guiding selection of a neutralization reagent are (1) purchase cost, (2) neutralization capacity, (3) reaction rate, and (4) storage and disposal of neutralization products.

### 5.5. LIMESTONE BEDS

#### 5.5.1. Types of Equipment

Both upflow and downflow types of limestone beds are employed. For wastewaters containing  $\text{H}_2\text{SO}_4$ , limestone beds should not be used if concentration of  $\text{H}_2\text{SO}_4$  exceeds 0.6%. The reason for this limitation is that the limestone becomes covered with an insoluble coat of  $\text{CaSO}_4$ , rendering it ineffective. In addition, evolution of  $\text{CO}_2$  causes frothing problems.

Upflow type arrangement is preferable to downflow type since in upflow units,  $\text{CaSO}_4$  tends to be flushed out before precipitation on the limestone. Also, escape of  $\text{CO}_2$  generated by the neutralization reaction is easier in upflow type units. For these reasons, maximum hydraulic rate for downflow systems is limited to approximately 50 gal/(hr)(ft<sup>2</sup>).

Presence of metallic ions (e.g.,  $\text{Al}^{3+}$ ,  $\text{Fe}^{3+}$ ) in the wastewater reduces

effectiveness of the limestone bed owing to coating of limestone with precipitated hydroxides. Finally, if dilution of the acid in the wastewater is increased, higher residence times are required for neutralization.

#### 5.5.2. Design Procedure for Limestone Beds

In this section, the laboratory procedure recommended by Eckenfelder and Ford [3] for obtaining the basic design data is described. A numerical example is presented to illustrate design of an actual limestone column. A model of a laboratory limestone neutralization column is shown in Fig. 3.33. Bench-scale columns operate with heights of limestone of 1.0–5 ft, which is the actual range of heights for plant-scale units. Column diameter is approximately 6 in., and rates of flow [gal/(hr)(ft<sup>2</sup>)] are comparable to those for plant operation.

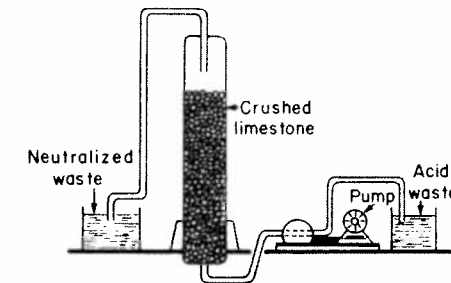


Fig. 3.33. Laboratory model of limestone neutralization column.

*Step 1.* Fill neutralization columns with limestone (after washing and screening) to depths of 1, 2, 3, 4, and 5 ft (5 columns).

*Step 2.* Adjust upward flow rate of acid wastewater for each column. Flow rates varying from 50 to 1000 gal/(hr)(ft<sup>2</sup>) are used.

*Step 3.* Check effluent pH from each column at each flow rate utilized until it is stabilized.

*Step 4.* After each run replace limestone used in the columns.

*Step 5.* Plot terminal pH as a function of rate of flow [gal/(hr)(ft<sup>2</sup>)] for each depth of limestone. A typical plot of this type is shown in Fig. 3.34.

Purposes of the design procedure are (1) to select the most economical height of column for a specified pH of the effluent. This is the column height corresponding to a maximum allowable flow rate, expressed in terms of volume of limestone utilized, i.e., gal of liquor/(hr)(ft<sup>3</sup> of limestone); and (2) to calculate annual requirement of limestone under these conditions, which corresponds to a minimum requirement of limestone.

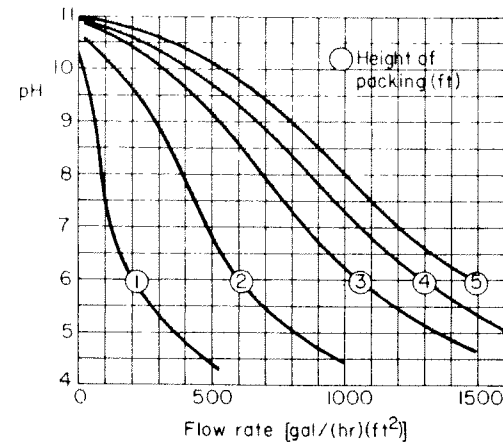


Fig. 3.34. Limestone neutralization data.

**Example 3.8**

It is desired to neutralize a wastewater acid stream containing 0.1 N HCl to a pH of 7.0 through a limestone bed. Laboratory tests with the wastewater yield data plotted in Fig. 3.34. Design a neutralization system for 100 gal/min (6000 gal/hr) of wastewater and estimate annual limestone requirement for the most economical operation.

**SOLUTION**

*Step 1.* For pH 7 read from Fig. 3.34 rates of flow [gal/(hr)(ft<sup>2</sup>)] corresponding to each column depth. Then calculate cross-sectional area, volume of limestone bed required, and flow rate in gal/(hr)(ft<sup>3</sup> of limestone bed) (Table 3.12).

**TABLE 3.12**  
Calculations for Example 3.8

(1) Depth (ft)	(2) Flow rate [gal/(hr)(ft <sup>2</sup> ) [From Fig. 3.34 for pH 7]	(3) Cross section (3) = $\frac{6000 \text{ gal/hr}}{(2)}$	(4) Volume of limestone (ft <sup>3</sup> ) (4) = (1) × (3)	(5) Flow rate [gal/(hr)(ft <sup>3</sup> )] (5) = $\frac{6000 \text{ gal/hr}}{(4)}$
1.0	118	51	51	118
2.0	492	12.2	24.4	246
3.0	845	7.1	21.3	282
4.0	1047	5.73	22.9	262
5.0	1200	5.0	25.0	240

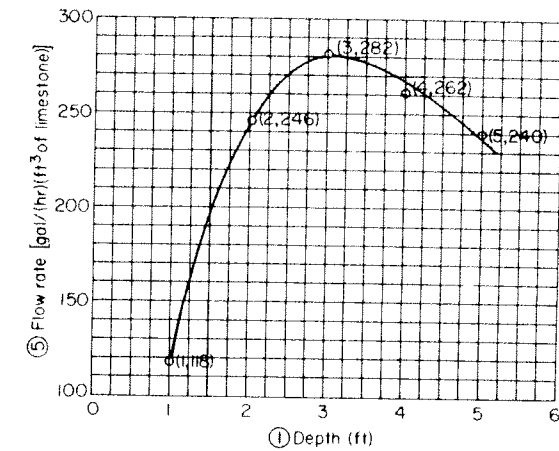


Fig. 3.35. Determination of optimum bed depth.

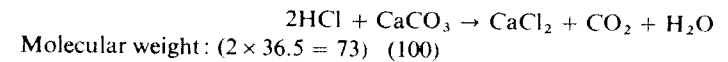
*Step 2.* Plot flow rates [gal/(hr)(ft<sup>3</sup>)] from column (5) of Table 3.12 vs. depths [column (1)]. This plot is shown in Fig. 3.35 and indicates that a 3-ft limestone bed is the most economical, corresponding to a maximum on the curve. The cross-sectional area in this case (see Table 3.12 for a depth of 3.0 ft) is 7.1 ft<sup>2</sup>, corresponding to a diameter of 3.0 ft.

*Step 3.* Calculate the lb of acid to be neutralized per day (acid contained in the 6000 gal/hr of the 0.1 N solution of HCl). Since this is a dilute acid solution, calculation is based on the density of water, taken as 8.34 lb/gal. A 0.1 N solution of HCl contains 3.65 g/liter of HCl, or approximately 3.65 lb of acid per 1000 lb of solution. Therefore

$$\text{Mass flow rate: } 6000 \text{ gal/hr} \times 24 \text{ hr/day} \times 8.34 \text{ lb/gal} = 1.2 \text{ Mlb/day}$$

$$\text{Acid content: } 3.65 \times (1,200,000/1000) = 4380 \text{ lb/day}$$

*Step 4.* Calculate limestone (CaCO<sub>3</sub>) required. The neutralization equation is



Therefore limestone required is

$$4380 \times 100/73 = 6000 \text{ lb/day of limestone}$$

This is the theoretical amount of limestone assuming 100% reactivity. For design purposes, assume an 80% reactivity. Limestone required is then

$$6000/0.8 = 7500 \text{ lb/day or } 2.738 \times 10^6 \text{ lb/year}$$

## 5.6. SLURRIED LIME TREATMENT

### 5.6.1. Equipment for Slurried Lime Systems

Slurried lime is the most commonly used reagent for neutralization of acid wastewaters, the low cost of lime being the main reason for its widespread utilization. A flowsheet of a two-stage slurried lime neutralization system is shown in Fig. 3.36.

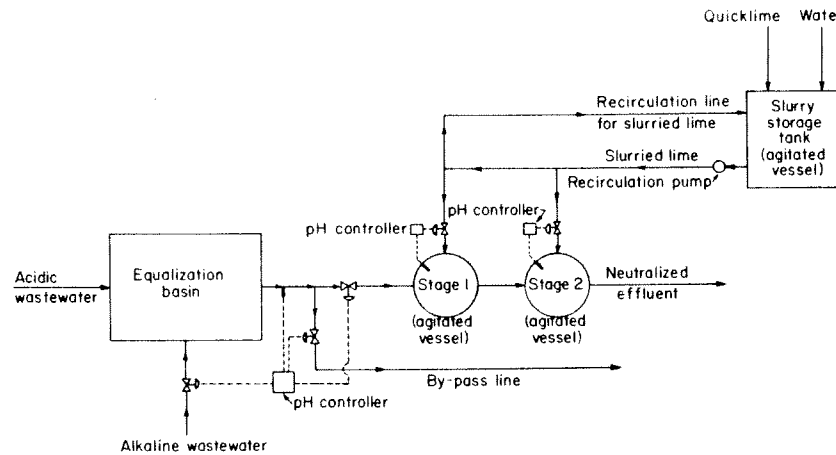


Fig. 3.36. Flow diagram of a two-stage slurried lime neutralization system.

Stepwise addition of lime is recommended. For highly acidic wastes a minimum of two stages is desirable, the first (bulk neutralization) to raise pH to a value of 3.0–3.5, and the second (fine tuning) to adjust pH to desired effluent value. Sometimes a third stage is desirable.

Automatic control of this process is not simple because the relationship between pH and amount of lime added is highly nonlinear, particularly in the vicinity of the neutralization point (pH 7). This is appreciated by inspection of a typical neutralization curve of an industrial wastewater, shown in Fig. 3.37.

In the vicinity of the neutralization point, the pH becomes exceedingly sensitive to small additions of lime, varying in an ordinary operation of slurried lime systems at a rate as fast as one pH unit per minute. Also, fluctuation in flow rate of influent complicates operation of the process. Use of an equalization tank is indicated to dampen fluctuations, as shown in Fig. 3.36. A relatively small amount of reagent is thoroughly mixed with a large liquid volume in a short time interval. Mechanical mixers are provided for this purpose.

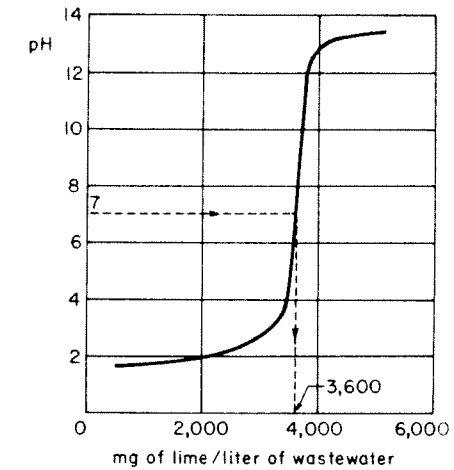


Fig. 3.37. Typical neutralization curve for an industrial wastewater.

### 5.6.2. Design Procedure for Slurried Lime Neutralization Systems

The procedure recommended by Eckenfelder and Ford [3] is summarized in this section and illustrated by a numerical example. Basic information required is (1) neutralization curve for the wastewater (see Fig. 3.37), and (2) power consumption data, i.e., a curve of level of agitation vs. detention time for a desired terminal pH (see Fig. 3.38). This basic information is obtained by simple laboratory procedures [3].

Purposes of the design procedure illustrated by Example 3.9 are (1) to select the number of stages of neutralization and to size neutralization reactors, and (2) to select appropriate mixing equipment.

#### Example 3.9

200 gal/min of an acidic industrial wastewater are neutralized to pH 7.0. From laboratory tests, the neutralization curve is plotted (Fig. 3.37). Also, a curve of level of agitation vs. detention time is obtained for this neutralization (Fig. 3.38). Design a lime slurry neutralization system.

#### SOLUTION

*Step 1.* From the neutralization curve in Fig. 3.37, lime slurry requirement for neutralizing the wastewater to a pH of 7 is read as 3600 mg/liter.

*Step 2.* Calculate lime slurry requirement for 200 gal/min wastewater flow. Since

$$3600 \text{ mg lime/liter waste} = 3600 \times 10^{-6} \text{ lb lime/lb waste}$$

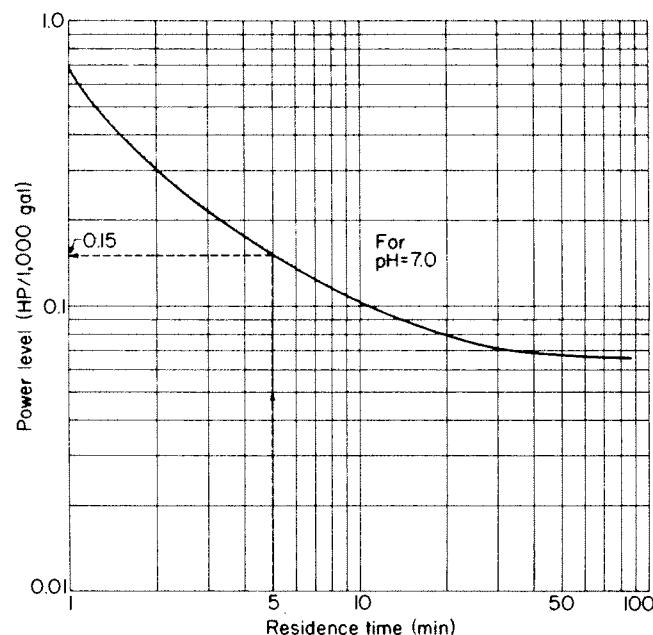


Fig. 3.38. Level of agitation vs. detention time.

then

$$200 \text{ gal waste/min} \times 60 \text{ min/hr} \times 24 \text{ hr/day} \times 8.34 \text{ lb waste/gal waste} \\ \times 3600 \times 10^{-6} \text{ lb lime/lb waste} = 8647 \text{ lb lime/day}$$

*Step 3.* Neutralize acid in two steps as indicated by flow diagram in Fig. 3.36, the first stage for bulk neutralization, and the second for fine tuning.

*Step 4.* Select a detention time and size the reactors.

$$\text{Volume of reactor (gal)} = Q \text{ (gal/min)} \times t \text{ (min)} \quad (3.63)$$

Selection of the optimum detention time is arrived at by an economical balance.

1. Assume a residence time (usually a value between 5 and 10 min).
2. Size the reactor [Eq. (3.63)].
3. From Fig. 3.38 for the assumed residence time, determine power level required.
4. Select mixers (as shown in Step 5) from knowledge of power level [item (3)].

5. Estimate total costs (capital and operating) corresponding to this assumed residence time. Main items in the column of capital costs are the reactors themselves (and auxiliary equipment) and the mixers. Energy requirement is the variable item among operating costs.

6. Repeating steps (1)–(5), a curve of total cost per year vs. a series of selected residence times is plotted. This curve passes through a minimum which corresponds to the optimum detention time. Capital costs are expressed on a yearly basis by estimating equipment life and utilizing the current value for interest rate.

Assume for Example 3.9 that optimum residence time is estimated by this procedure as 5 min. Then

$$\text{Volume of each reactor (gal)} = 200 \text{ gal/min} \times 5 \text{ min} = 1000 \text{ gal}$$

or

$$1000 \text{ gal} \times \text{ft}^3/7.48 \text{ gal} = 134 \text{ ft}^3$$

Selecting a reactor depth of 5 ft, required cross-sectional area is

$$\text{Area} = 134 \text{ ft}^3/5 \text{ ft} = 26.8 \text{ ft}^2$$

corresponding to a diameter of 5.84 ft.

*Step 5.* Select mixers. From Fig. 3.38, power level required for 5-min detention time is

$$0.15 \text{ HP/1000 gal}$$

Since each tank has a volume of 1000 gal, specify one 0.15-HP mixer for each tank.

### 5.7. NEUTRALIZATION OF ALKALINE WASTES

In principle, any strong acid can be used to neutralize alkaline wastewaters. Cost considerations limit choice to  $\text{H}_2\text{SO}_4$  (the most common) and  $\text{HCl}$ . Reaction rates are essentially instantaneous. The basic design procedure for alkaline wastes is similar to that for acidic wastes described in Section 5.6.

Flue gases containing 14% or more of  $\text{CO}_2$  are used for neutralization of alkaline wastewaters. When bubbled through the wastewater the  $\text{CO}_2$  forms carbonic acid, which reacts with the base. Reaction rate is slow but sufficient if pH need not be adjusted below 7 or 8. Either bubbling through a perforated pipe or using spray towers is satisfactory.

### Problems

I. *Sedimentation (discrete settling).* A particle size distribution is obtained from a sieve analysis of sand particles. For each weight fraction an average settling velocity is calculated. Data [6] are presented in the following tabulation.

Settling velocity (ft/min)	Weight fraction remaining
10.0	0.55
5.0	0.46
2.0	0.35
1.0	0.21
0.75	0.11
0.50	0.03

1. Prepare a plot of fraction of particles with less than stated velocity vs. settling velocity (ft/min).
2. For an overflow rate of 100,000 gal/(day)(ft<sup>2</sup>), calculate overall removal utilizing Eq. (3.26).

**II. Sedimentation (floculent settling).** A laboratory settling analysis gave the results tabulated below.

Time Time (min)	% suspended solids removed at indicated depth		
	2 ft	4 ft	6 ft
10	40	25	16
20	54	37	28
30	62	47	37
45	71	56	46
60	76	65	53

1. Perform analysis of the data and arrive at curves for % SS removal vs. detention time (min), and % SS removal vs. overflow rate [gal/(day)(ft<sup>2</sup>)].
2. If the initial concentration of the slurry is 430 ppm, design a settling tank (i.e., calculate diameter and effective depth of the tank) to remove 70% of the suspended solids for a 1 Mgal/day flow.
3. What removal is attained if flow is increased to 2 Mgal/day?
4. For the flow of 1 Mgal/day calculate daily accumulation of sludge in lb/day and average pumping rate in gal/min. Assume sludge concentration to be 1.5% solids ( $\approx 15,000$  mg/liter).

**III. Sedimentation (zone settling).** It is desired to design a secondary settling tank to produce an underflow concentration of 15,000 mg/liter from a mixed liquor solids content of 3750 mg/liter in the influent. Wastewater flow is 2.0 Mgal/day. Calculate clarifier area required. Data below are obtained in a laboratory test of the slurry.

$t$ (min)	Interface height, $H$ (ml)
0	1000
2	920
4	840
6	760
8	690
10	600
15	400
20	300
25	280
30	270

**IV. Flotation.** A pilot-plant flotation operation indicated optimum air/solid ratio to be 0.04 lb air/lb of solids.

1. If a wastewater to be treated has 250 ppm suspended solids, compute the % recycle to be pressurized to 60 psia at 20°C. Take  $f = 0.68$ .
  2. For a wastewater flow of 1.0 Mgal/day and an overflow rate of 4.0 gal/(min)(ft<sup>2</sup>) compute surface area required.
- V. Neutralization.** For the 3-ft limestone bed designed in Example 3.8 prepare a plot of volume of limestone required vs. a range of selected values of pH for the effluent (select pH = 5, 6, 7, 8, 9, and 10).

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