

# Secondary Treatment: Other Aerobic and Anaerobic Wastewater Treatment Processes

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## 1. Introduction

The basic mechanism for aerobic treatment of wastewaters is described in Chapter 5, Section 4.1.1. The conventional activated sludge process is also studied in detail in Chapter 5.

In Sections 2–4 of this chapter, several modifications of the activated sludge process are described. Other types of wastewater treatment (aerated lagoons, stabilization ponds, and trickling filters) are discussed in Sections 5–7. These are mostly aerobic processes. Anaerobic treatment of wastewaters is studied in Section 8.

For both aerobic and anaerobic processes the approach utilized for the mathematical formulation, as well as the procedure followed to obtain design data from bench scale units, are those developed by Eckenfelder and associates.

## 2. Extended Aeration (or Total Oxidation Process)

### 2.1. INTRODUCTION

This process, also referred to as total oxidation, is a modification of the activated sludge process. The fundamental idea in extended aeration as compared to the conventional activated sludge process is to minimize the amount of excess sludge. This is achieved by increasing residence time; thus the reactor volume is comparatively larger than that required in the conventional activated sludge process. As a result, essentially all degradable sludge formed is consumed by endogenous respiration. Referring to Eq. (5.68), the condition for zero net yield of sludge is

$$\Delta X_v \approx 0 \quad (6.1)$$

or

$$aS_r Q = bX_{v,a} V \quad (6.2)$$

Theoretical residence time to achieve zero net yield of MLVSS is obtained from Eq. (6.2).

$$t = V/Q = aS_r/bX_{v,a} \quad (6.3)$$

The main advantage of the extended aeration process is that sludge handling facilities are minimal compared to those required for the activated sludge process.

### 2.2. COMPARISON OF EXTENDED AERATION AND ACTIVATED SLUDGE PROCESSES

There are four basic features which distinguish extended aeration from the conventional activated sludge process:

1. Longer detention time in aerator
2. Lower organic loadings. For the extended aeration process organic loading, expressed in terms of food to microorganism ratio ( $F/M$ ) (Chapter 5, Section 5), is usually between 0.10 and 0.25, as compared to values of 0.3–0.7 for the conventional activated sludge process.
3. Higher concentration of biological solids in the aerator. These values range from 3500 to 5000 mg/liter for extended aeration, as compared to 2000–4000 mg/liter for the activated sludge process. Combination of features

considered under (2) and (3) (i.e., less food for greater microorganism population) results in starvation conditions for the microorganisms. Resulting "cannibalism" (endogenous respiration conditions) reduces concentration of MLVSS, and thus a minimization of sludge accumulation is achieved.

4. Higher consumption of oxygen in extended aeration process. For domestic wastewater treatment, Pasveer [11] reports for the extended aeration process an oxygen consumption approximately twice that for the conventional activated sludge process, namely 18 vs. 9 kwh/(capita)(year). With this comparatively small extra cost for energy, substantial savings in capital

**TABLE 6.1**  
Comparison of Conventional Activated Sludge and Extended Aeration Processes

Characteristics	Activated sludge	Extended aeration
Food to microorganism ratio [lb BOD <sub>5</sub> /(day)(lb MLVSS)]	0.3–0.7	0.10–0.25
MLVSS concentration in reactor (mg/liter)	2000–4000	3500–5000
Overall BOD <sub>5</sub> removal efficiency (% includes both soluble and suspended BOD <sub>5</sub> )	90–95	85–98
Effluent characteristics		
Soluble BOD <sub>5</sub> (mg/liter)	10–20	10–20
Total BOD <sub>5</sub> (suspended + colloidal + soluble) (mg/liter)	15–25	20–40
Suspended solids (mg/liter)	< 20	< 70
Sludge yield (lb/lb BOD <sub>5</sub> removed)	≈ 0.03	≈ 0.01
O <sub>2</sub> requirement (as % of BOD <sub>5</sub> removed)	90–95	120

expenditure may be achieved. Table 6.1 presents a comparison of the main characteristics of conventional activated sludge and extended aeration processes.

### 2.3. APPLICATION OF EXTENDED AERATION

The extended aeration process has been applied mostly in treatment of wastewaters when daily volume is less than 2000 gal/day. This includes treatment of domestic sewage for small communities, housing developments, recreational areas, and some industrial wastes. Extended aeration package units are commercially available. If well designed and operated, they should not present odor problems and thus can be located within populated areas.

### 2.4. EXTENDED AERATION UNITS

Figures 6.1 and 6.2 illustrate a conventional extended aeration unit and a variation known as the oxidation ditch. In the conventional aeration unit (Fig. 6.1), the influent passes first through a screen to remove large suspended

## 6. Secondary Treatment: Aerobic and Anaerobic Processes

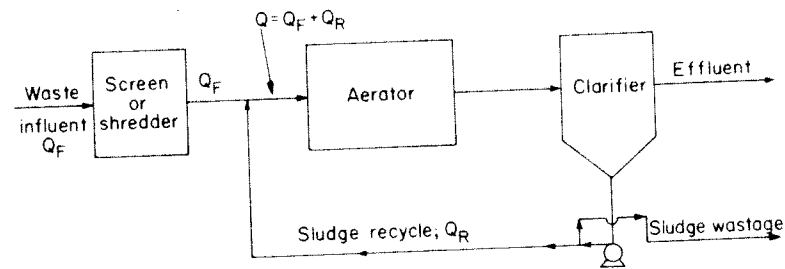


Fig. 6.1. Conventional extended aeration process.

solids, in order to protect the aerator unit from damage resulting from clogging. In some units a shredder is provided instead of a screen. The flow diagram of the conventional unit is essentially identical to that of the activated sludge process. Effluent from the clarifier may be chlorinated prior to discharge in the receiving water.

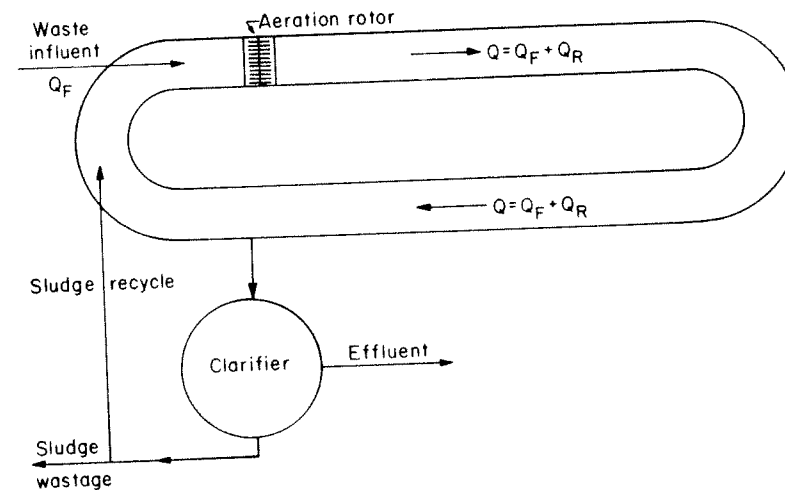


Fig. 6.2. Oxidation ditch.

Figure 6.2 shows a diagram of the oxidation ditch. An essential part of this system is an aeration ditch provided with an aeration rotor. This rotor has two functions: aeration and provision of a flow velocity to the mixed liquor in the ditch. Liquid flow velocity is of the order of 1 ft/sec. The mixture of sewage and activated sludge repeatedly passes over the aeration rotor at short intervals. A typical rotor has a diameter of approximately 30 in., revolves at about 75 rpm, has a depth of immersion of about 6 in., and an oxygenation capacity (OC) of the order of 6 lb/hr.

## 2.5. SETTLING OF SLUDGE FROM EXTENDED AERATION

Although theoretically sludge yield is nil for the extended aeration process, in practice this is not the case since part of the sludge is not biodegradable and therefore accumulates. The net sludge yield must be wasted.

Figure 5.7 (Chapter 5) reveals that for values of  $F/M$  in the range utilized for the extended aeration process (0.10–0.25), the microorganisms are driven to endogenous respiration and metabolize the cytoplasmic material of their "fellow" microorganisms. The remains of this "cannibalist feast" are non-degradable cellular shells which are relatively light compared to cytoplasmic material and settle with difficulty. Settling tanks for extended aeration systems should therefore provide longer retention time than for the conventional activated sludge process. Retention time is approximately 4 vs. 2 hr for the conventional activated sludge process.

## 2.6. NITRIFICATION IN EXTENDED AERATION

As organic loading ( $F/M$  ratio) is low in extended aeration and since a large excess of air is supplied, nitrification may occur to an appreciable extent involving the conversion of ammonia nitrogen to nitrite and nitrate. A problem related to nitrification is a drop in pH for the system due to formation of nitric acid. The pH may drop as low as 4.5, in which case the biological process may be adversely affected.

## 2.7. DESIGN CRITERIA FOR EXTENDED AERATION

### 2.7.1. Calculation of Residence Time for Extended Aeration Units

Since the detention period required for BOD removal is much shorter than that for autoxidation of sludge, aerator volume is controlled by the rate of sludge oxidation.

Formulation of design procedure for extended aeration is based on Eq. (6.2), which must be slightly modified prior to its application. The modifications are as follows: in Eq. (6.2), the left-hand member presupposes that all sludge formed ( $aS_rQ$ ) is biodegradable. Experimental data indicate that approximately 77% of the sludge produced is biodegradable, the remaining 23% comprising nonbiodegradable cellular shells. Therefore, the left-hand member of Eq. (6.2) is rewritten as Eq. (6.4).

$$a_o S_r Q = fa S_r Q \quad (6.4)$$

where  $a_o$  is the lb of biodegradable MLVSS produced/lb total BOD<sub>5</sub> removed or  $fa$ ;  $f$  the lb biodegradable MLVSS produced/lb total MLVSS produced  $\approx 0.77$ ; and  $a$  the lb total MLVSS produced/lb total BOD<sub>5</sub> removed.

In the right-hand member of Eq. (6.2), MLVSS concentration corresponds only to biodegradable sludge, i.e.,  $X_{v,a}$  to be substituted by  $fX_{v,a}$  (mg/liter of biodegradable sludge). As written in Eq. (5.68) parameter  $b$  represents the fraction of total MLVSS oxidized per day.

$$b = \text{lb MLVSS oxidized}/(\text{day})(\text{lb total MLVSS in reactor})$$

Define parameter  $b_o$  referred to lb of biodegradable sludge.

$$b_o = \text{lb MLVSS oxidized}/(\text{day})(\text{lb biodegradable MLVSS in reactor})$$

The relationship between  $b_o$  and  $b$  is

$$b_o = \text{lb MLVSS oxidized}/(\text{day})[f(\text{lb total MLVSS in reactor})] = b/f$$

If in the right-hand member of Eq. (6.2)  $b$  and  $X_{v,a}$  are substituted by  $b_o$  and  $fX_{v,a}$ , respectively,

$$b_o(fX_{v,a})V = (b/f)(fX_{v,a})V = bX_{v,a}V \quad (6.5)$$

Therefore, the right-hand member of Eq. (6.2) is left unchanged.

Consequently, the modified Eq. (6.2) is obtained by equating Eq. (6.4) to Eq. (6.5).

$$faS_r Q = a_o S_r Q = bX_{v,a}V \quad (6.6)$$

Residence time is then shown in Eq. (6.7).

$$t = V/Q = a_o S_r / bX_{v,a} = faS_r / bX_{v,a} = fa(S_o - S_e) / bX_{v,a} \quad (6.7)$$

It is convenient to write the expression for residence time in terms of BOD for the fresh feed,  $S_F$ . If in Eq. (6.7)  $(S_o - S_e)$  is substituted by the value given in Eq. (5.97), one obtains

$$t = (fa/bX_{v,a})[(S_F - S_e)/(1+r)] \quad (f \approx 0.77) \quad (6.8)$$

### 2.7.2. Expression for Recycle Ratio $r$

Consider Eq. (5.91) for recycle ratio  $r$  (letting  $X_{v,F} \approx 0$ ):

$$r = (8.34Q_F X_{v,a} - \Delta X_v) / [8.34Q_F (X_{v,u} - X_{v,a})] \quad (6.9)$$

For extended aeration, wastage  $\Delta X_v$  corresponds to nonbiodegradable cells which are approximately 23% of the sludge formed.

$$\Delta X_v = 8.34(1-f)a(S_o - S_e)Q - (\text{effluent loss}) \quad (6.10)$$

where

$$1-f \approx 1-0.77 \approx 0.23$$

Neglecting effluent loss in Eq. (6.10),\* and substituting  $\Delta X_v$  in Eq. (6.9) by its value from Eq. (6.10) yields after simplification

$$r = [Q_F X_{v,a} - (1-f)a(S_o - S_e)Q] / Q_F (X_{v,u} - X_{v,a}) \quad (6.11)$$

Substitution of  $Q$  and  $(S_o - S_e)$  in Eq. (6.11) by their values from Eqs. (5.5) and (5.97) yields after simplification

$$r = [X_{v,a} - (1-f)a(S_F - S_e)] / (X_{v,u} - X_{v,a}) \quad (6.12)$$

### 2.7.3. Expression for Reactor Volume

Substitution of the residence time  $t$  in Eq. (5.104) by the value given in Eq. (6.8) leads to Eq. (6.13).

$$V = Q_F fa(S_F - S_e) / bX_{v,a} \quad (6.13)$$

The design procedure for an extended aeration unit is illustrated by Example 6.1.

#### Example 6.1.

26,000 gal/day of an industrial wastewater are to be treated by extended aeration. Influent BOD<sub>5</sub> is  $S_F = 1200$  mg/liter, and it is desired to reduce it to a value not over 50 mg/liter in the effluent ( $S_e$ ). Take  $X_{v,a}$  and  $X_{v,u}$  as 4000 and 12,730 mg/liter, respectively. Values of design parameters  $a$ ,  $b$ ,  $a'$ , and  $b'$  have been estimated as 0.7, 0.1, 0.5, and 0.142, respectively (units: BOD<sub>5</sub>, day). Assume that 77% of the MLVSS formed is biodegradable and neglect  $X_{v,F}$ . Calculate (1) recycle ratio, (2) residence time in hr, (3) BOD<sub>5</sub> of combined feed, (4) combined feed in gal/day, (5)  $F/M$  ratio, (6) reactor volume in gal, and (7) oxygen requirements in lb/day.

#### SOLUTION

1. Recycle ratio from Eq. (6.12):

$$r = [4000 - (1-0.77)(0.7)(1200-40)] / (12,730-4000) = 0.437$$

2. Residence time from Eq. (6.8):

$$t = [(0.77 \times 0.7) / (0.1 \times 4000)] [(1200-50) / (1+0.437)] = 1.078 \text{ days (26 hr)}$$

3. BOD<sub>5</sub> of combined feed from Eq. (5.96):

$$S_o = [1200 + (0.437)(50)] / (1+0.437) = 850.3 \text{ mg/liter}$$

4. Combined feed from Eq. (5.5):

$$Q = 26,000(1+0.437) = 37,360 \text{ gal/day}$$

\* Effluent loss in extended aeration is more significant than in the conventional activated sludge process because as explained in Section 2.5, nonbiodegradable material is difficult to settle.

5.  $F/M$  ratio from Eq. (5.80):

$$F/M = 850.3 / (4000 \times 1.078) = 0.197$$

6. Reactor volume from Eq. (5.104):

$$V = 37,360 \times 1.078 = 40,290 \text{ gal}$$

7. Oxygen requirements from Eq. (5.85):

$$R, V = 0.5(850.3 - 50)(0.03736)(8.24) + (0.142)(4000)(0.04029)8.34$$

$$R, V = 124.7 + 190.9 = 315.6 \text{ lb/day.}$$

### 3. Contact Stabilization

#### 3.1. INTRODUCTION

Contact stabilization is another modification of the activated sludge process. A flow diagram for the system is shown in Fig. 6.3.

Influent wastewater is mixed with stabilized sludge, and this mixture is aerated in the initial contact tank for which detention time is only 20–40 min. During initial contact an appreciable fraction of suspended and dissolved BOD is removed by biosorption after contact with the well-aerated activated sludge. The mixed effluent from the initial contact tank flows into a clarifier. Clarified effluent is removed and underflow from the clarifier is taken to a stabilization tank, where it is aerated for a period of 1.5–5 hr.

During this stabilization period, biosorbed organics are broken down by aerobic degradation. Stabilized sludge leaving the stabilization tank is in a "starved" condition and ready to adsorb organic waste.

#### 3.2. ADVANTAGE OF CONTACT STABILIZATION VS. CONVENTIONAL ACTIVATED SLUDGE PROCESS

Since only recycled sludge is subject to lengthy aeration, this system permits appreciable reduction in aeration basin volume. This is the main advantage of contact stabilization vs. the conventional activated sludge process. For a wastewater flow  $Q_F$  ( $\text{ft}^3/\text{hr}$ ) and a sludge recycle of  $0.3Q_F$ , approximate tank volumes for the conventional activated sludge process and contact stabilization are  $11Q_F$  and  $4Q_F$ , respectively. This corresponds to a nearly threefold tank reduction. Overall removal efficiencies are usually lower than in the conventional activated sludge process, but could easily reach 85–90%  $\text{BOD}_5$  removal.

The contact stabilization process is suitable when the wastewater contains a high proportion of BOD in suspended and colloidal forms. Contact stabilization plants may operate without need of primary clarification.

### 3. Contact Stabilization

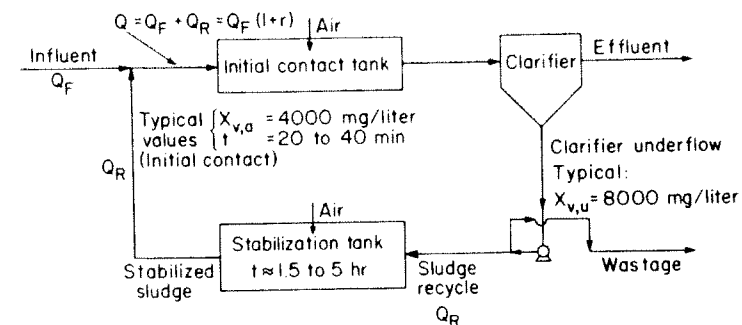


Fig. 6.3. Flow diagram of contact stabilization system.

#### 3.3. SOLUBILITY INDEX (SI) AND OVERALL EFFICIENCY

The solubility index (SI) of a wastewater is defined as

$$SI = \text{soluble BOD} / \text{total BOD} \quad (6.14)$$

where  $0 \leq SI \leq 1.0$ .

As SI approaches zero total BOD tends to be of suspended or colloidal form, and the wastewater becomes suitable for treatment by contact stabilization since most BOD can be removed within a short initial contact period. As SI approaches one, total BOD tends to be of soluble form, and the conventional activated sludge process is more efficient.

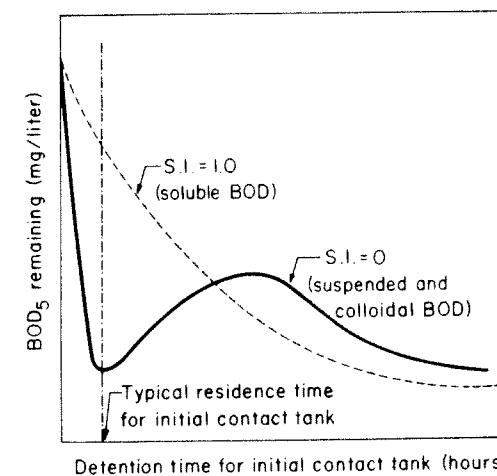


Fig. 6.4. Relationship of SI, BOD removal, and initial contact time.

The relationship of SI, BOD removal, and initial contact time is shown in Fig. 6.4. For typical residence times in the initial contact tank, considerably greater BOD reduction of effluent is achieved when the value of SI approaches unity. The rise in the curve for  $SI = 0$  following the initial drop is due to overoxidation, a concept which is discussed in Section 3.4.

### 3.4. DESIGN OF CONTACT STABILIZATION SYSTEMS

#### 3.4.1. Selection of Residence Times for Contact and Stabilization

The main objective in design of contact stabilization systems is the selection of residence times for initial contact and stabilization tanks. For a specific wastewater, laboratory tests are performed to determine the effect of a combination of various residence times for initial contact and stabilization tanks on the % BOD removal. A typical set of curves obtained for a specific wastewater is shown in Fig. 6.5. Each curve corresponds to a fixed initial contact detention time. The abscissa is stabilization time and the ordinate is the % BOD removal.

If the desired BOD removal cannot be obtained in the initial contact tank (i.e., at stabilization time = 0), stabilization time must be extended to reach the desired removal. For each contact time (that is, for each curve in Fig. 6.4) there is an optimum stabilization time as indicated, corresponding to a maximum % BOD removal. For stabilization times longer than those corre-

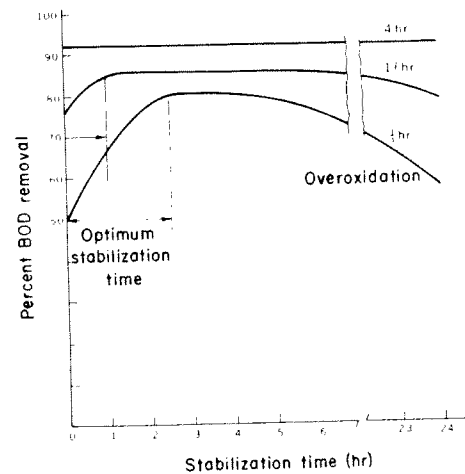


Fig. 6.5. Relationship of contact time, stabilization time, and % BOD removal [3].

sponding to these maxima, sludge degradation (endogenous phase) results in such a breaking down that sludge effluent from the stabilization tank is not adequate to perform biosorption in the initial contact. Therefore, % BOD removal drops. This is known as overoxidation.

#### 3.4.2. Determination of Recycle Ratio

The recycle ratio is determined from Eq. (5.92), neglecting wastage. Typical values of  $X_{v,a}$  and  $X_{v,u}$  are 4000 and 8000 mg/liter, respectively, in which case  $r = 0.5$ . Once the recycle ratio and detention times have been determined, sizing of contact and stabilization tanks is straightforward.

#### 3.4.3. Determination of Oxygen Requirements

Oxygen requirements are calculated from Eq. (5.64). Contact and stabilization tanks are considered separately and the corresponding values of  $a'$  and  $b'$  are used in the calculations.

## 4. Other Modifications of Conventional Activated Sludge Process: Step Aeration, Complete Mix Activated Sludge Process, and Tapered Aeration

### 4.1. STEP AERATION

Step aeration is a modification of the conventional activated sludge process in which fresh feed is introduced at several points along the aeration tank. This arrangement provides for an equalization of the  $F/M$  ratios along the tank.

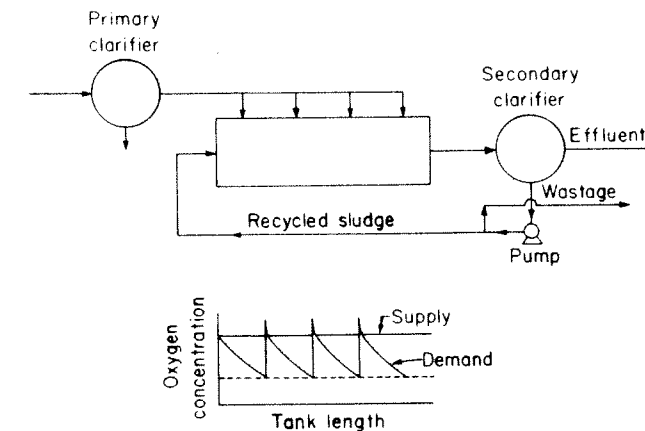


Fig. 6.6. Step aeration process [8].

The aeration tank is divided by baffles into several parallel channels. Each channel constitutes one step of the process, and the steps are linked together in series. This process, as well as oxygen supply and demand along the tank length, are illustrated by Fig. 6.6.

#### 4.2. COMPLETE MIX ACTIVATED SLUDGE PROCESS

In this modification of the activated sludge process, fresh feed and recycled sludge are combined and then introduced at several points in the aeration tank from a central channel. Aerated liquor leaves the reactor from effluent channels on both sides of the aeration tank (Fig. 6.7).

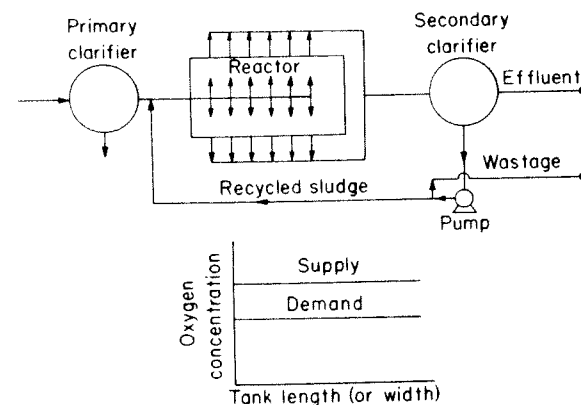


Fig. 6.7. Complete mix activated sludge process [8].

Oxygen supply and demand are uniform along the tank, as indicated by the graph accompanying Fig. 6.7. The mathematical model for the conventional activated sludge process developed in Chapter 5, Section 3.2 assumes complete mixing. If plug flow conditions are assumed, oxygen demand decreases along the length of the aeration tank, whereas the oxygen supply remains constant (Fig. 6.8).

#### 4.3. TAPERED AERATION

The purpose of tapered aeration is to match the amount of air supplied with the oxygen demand along the aeration tank. Since at the inlet oxygen demand is the highest, aerators are spaced more closely to provide a higher oxygenation rate. Spacing between aerators is increased toward the outlet as oxygen demand decreases.

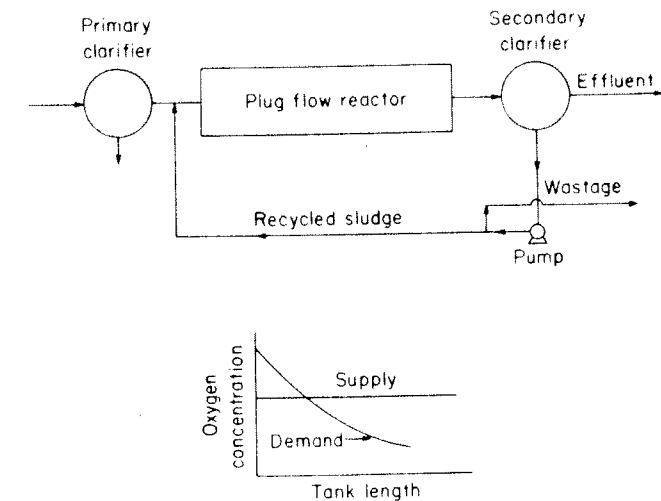


Fig. 6.8. Conventional activated sludge process with plug flow reactor [8].

## 5. Aerated Lagoons

### 5.1. INTRODUCTION

Aerated lagoons are basins having depths varying from 4 to 12 ft in which oxygenation of wastewaters is accomplished by aeration units. The fundamental difference between aerated lagoons and the activated sludge system is that recycling of the sludge is provided in the latter as a means of controlling the amount of biological sludge in the aerator. Aerated lagoons are flow-through devices, i.e., no recycle of sludge is provided. Solids concentration in the lagoon is a function of wastewater characteristics and detention time. It is usually between 80 and 200 mg/liter, i.e., much lower than that for activated sludge units (2000–4000 mg/liter).

### 5.2. MIXING REGIMES FOR AERATED LAGOONS

Turbulence level in lagoons is the basis for their classification into two types.

#### 5.2.1. Completely Mixed Lagoons

The turbulence level is sufficient to maintain solids in suspension. Detention times are usually less than 3 days, and power levels are higher than 25 HP per million gallons of basin volume. Power levels for activated sludge units are in the vicinity of 0.25 HP/1000 gal (or 250 HP per million gallons), i.e., about ten times higher than for aerated lagoons. Assumption of complete mixing, which even for activated sludge units is an idealized approximation, is

questionable for aerated lagoons. Nevertheless, this assumption is often made, leading to a simple mathematical model for the lagoon. Utilizing appropriate safety factors, this idealized approach is useful.

### 5.2.2. Facultative Lagoons

The turbulence level is insufficient to maintain all solids in suspension. Part of the solids settle to the bottom of the lagoon, where they undergo anaerobic decomposition. Detention times are usually over 6 days and power levels are 4–25 HP per million gallons of basin volume. There is a gradual buildup of residue which has to be desludged at periods of 1–10 years.

Selection of mixing regime is the result of an economic balance between power requirements (greater in completely mixed regime) and acreage cost (more acreage required for facultative lagoons). These two mixing regimes are illustrated in Fig. 6.9.

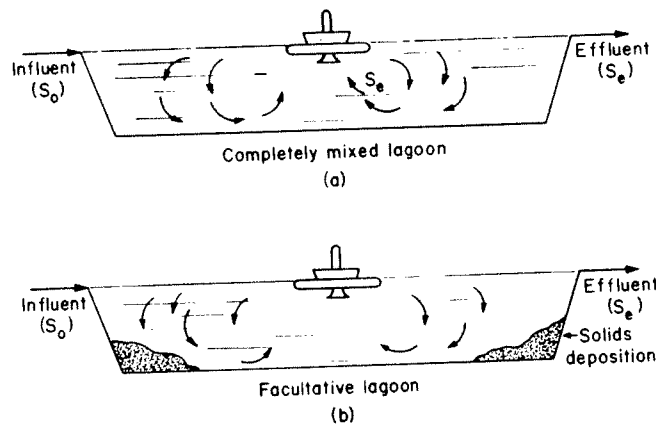


Fig. 6.9. Mixing regimes for aerated lagoons.

### 5.3. KINETICS OF BOD REMOVAL

Assuming appropriate environmental conditions (pH, presence of nutrients, etc.), the rate of BOD removal is a function of detention time, temperature, nature of wastewater, and concentration of suspended volatile solids.

Usually BOD removal rate is assumed to follow first-order kinetics, and the formulation for the continuous reactor assuming complete mixing (Chapter 5, Section 3.2) is utilized. Therefore, Eq. (5.18) is taken as the kinetic model for the lagoon. It is convenient to rewrite Eq. (5.18) in terms of the ratio  $S_e/S_o$ , i.e., the percentage of BOD remaining in the effluent. Letting  $kX_{v,a} = K$ , rearrangement of Eq. (5.18) yields

$$S_e/S_o = 1/(1 + Kt) \quad (6.15)$$

where  $S_o$  is the soluble BOD of the influent,  $S_e$  the soluble BOD of the effluent,  $K$  the removal rate constant, and

$$t \text{ (detention time)} = V/Q = AD/Q \quad (6.16)$$

In Eq. (6.16),  $V$  is the lagoon volume,  $Q$  the flow rate,  $A$  the horizontal cross-sectional area of the lagoon, and  $D$  the depth of the lagoon.

It should be emphasized that this mathematical model is based on two fundamental assumptions, those of first-order kinetics and total mixing conditions. Neither assumption is valid in all cases, nevertheless, this formulation is useful for design purposes.

Values of  $K$  can be determined from bench scale data. A reactor like the one shown in Fig. 5.10 (Chapter 5, Section 6.1) is used to simulate an aerated lagoon. The sliding baffle is removed for this simulation. The value of  $K$  is determined from linear plots similar to those in Fig. 5.5 (Chapter 5, Section 3.2) based on Eq. (5.18).  $K$  values should be corrected for the lagoon temperature (summer and winter conditions) by the procedure described in Section 5.4. Values of  $K$  can also be determined from pilot-plant data or estimated from data on operating lagoons.

### 5.4. ESTIMATE OF LAGOON TEMPERATURE ( $T_w$ )

The rate of BOD removal nearly doubles for every 10°C of temperature rise (Chapter 2, Section 7.1). Thus it is necessary to estimate lagoon temperature under average summer and winter conditions. This is done by performing a heat balance. Consider the lagoon represented by Fig. 6.10. Temperature values shown are those for the summer conditions in Example 6.2.

Let

$T_i$  = temperature of the influent

$T_w$  = lagoon (and effluent) temperature (°F)

$T_a$  = atmospheric temperature (°F)

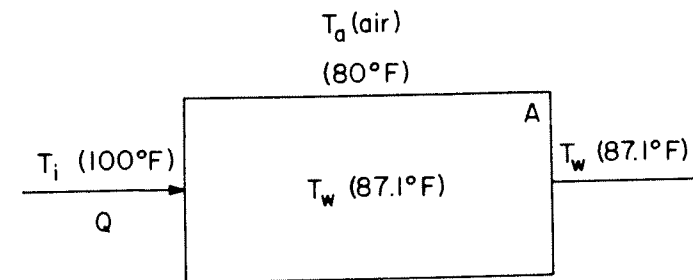


Fig. 6.10. Heat balance for aerated lagoon.



$h$  = heat transfer coefficient [BTU/(day)(ft<sup>2</sup>)(°F)] between lagoon and atmosphere (should take into account such factors as wind and humidity effects). In the absence of more accurate information a value of 100 BTU/(day)(ft<sup>2</sup>)(°F) may be utilized for estimate purposes

$Q$  = flow rate (Mgal/day)

$A$  = lagoon surface (ft<sup>2</sup>)

$C$  = specific heat of wastewater [take as 1.0 BTU/(lb)(°F)]

The enthalpy change of the influent is

$$\begin{aligned} & Q \frac{\text{Mgal}}{\text{day}} \times 8.34 \times 10^6 \frac{\text{lb liquor}}{\text{Mgal liquor}} \times C \frac{\text{BTU}}{(\text{lb liquor})(\text{°F})} \times (T_i - T_w) \text{°F} \\ & = Q(T_i - T_w) 8.34 \times 10^6 \quad (\text{BTU/day}) \end{aligned} \quad (6.17)$$

This should equal heat loss to surrounding air given by

$$h \text{ BTU}/(\text{day})(\text{ft}^2)(\text{°F}) \times A \text{ ft}^2 \times (T_w - T_a) \text{°F} = hA(T_w - T_a) \quad (\text{BTU/day}) \quad (6.18)$$

Equating Eqs. (6.17) and (6.18),

$$Q(T_i - T_w) 8.34 \times 10^6 = hA(T_w - T_a) \quad (6.19)$$

Let

$$h \times 10^{-6}/8.34 = f$$

Note: If  $h \approx 100 \text{ BTU}/(\text{day})(\text{ft}^2)(\text{°F})$ , then  $f \approx 12 \times 10^{-6}$ .

One then writes

$$Q(T_i - T_w) = fA(T_w - T_a) \quad (6.20)$$

Solving for  $T_w$ ,

$$T_w = (AfT_a + QT_i)/(Af + Q) \quad (6.21)$$

Equation (6.21) permits an estimate of lagoon temperature.

The effect of lagoon temperature ( $T_w$ ) on BOD removal rate  $K$  is given by the empirical equation

$$K_{T_w} = K_{20} \theta^{T_w - 20} \quad (6.22)$$

where  $K_{T_w}$  is the BOD removal rate at temperature  $T_w$ ,  $K_{20}$  the BOD removal rate at 20°C, and  $\theta$  the temperature coefficient 1.056 (20–30°C) and 1.135 (4–20°C).

Consider Eqs. (6.15), (6.16), (6.21), and (6.22). Substituting in Eq. (6.15) values of  $t$ ,  $K_{T_w}$ , and  $T_w$  given by Eqs. (6.16), (6.22), and (6.21), one obtains

$$S_e/S_o = 1/[1 + (AD/Q) K_{20} \theta^{(AfT_a + QT_i)/(Af + Q) - 20}] \quad (6.23)$$

Equation (6.23) permits evaluation of the effect of temperature on percentage of BOD remaining in the effluent.

### 5.5. OXYGEN REQUIREMENTS FOR AERATED LAGOONS

Oxygen requirements for aerobic oxidation processes are given by Eq. (5.64). Since the concentration of MLVSS ( $X_{r,a}$ ) is low for aerated lagoons, the term  $b'X_{r,a}V$  is usually neglected. Therefore

$$\text{lb O}_2/\text{day} \approx a'S_r Q \approx a'(\text{lb BOD removed/day}) \quad (6.24)$$

Values of  $a'$  for aerated lagoons vary from 0.9 to 1.4 depending on the nature of waste, mixing regime, and temperature.

### 5.6. SOLUBLE EFFLUENT BOD FOR AN AERATED LAGOON

The soluble effluent BOD for an aerated lagoon is calculated from Eq. (6.15). Solving for  $S_e$ ,

$$S_e = S_o/(1 + Kt) \quad (6.25)$$

This equation does not take into account BOD feedback to the lagoon due to anaerobic degradation of deposited solids. Usually a correction factor to account for this is introduced in Eq. (6.25). Since anaerobic BOD feedback is greater during the summer, two modified forms of Eq. (6.25) are recommended by Eckenfelder and Ford [5].

Summer conditions:  $S_e = 1.2S_o/(1 + Kt)$  (6.26)

Winter conditions:  $S_e = 1.05S_o/(1 + Kt)$  (6.27)

For activated sludge plants, design procedure consists of specifying a desired effluent quality ( $S_e$ ) and then calculating residence time ( $t$ ) necessary to achieve this specified quality. For aerated lagoons the design approach is normally the reverse of that for activated sludge plants. Usually when an aerated lagoon is being considered, one has available a given acreage of land presumably at a reasonably low cost. The design procedure starts from the known value of the surface area  $A$  and an assumed reasonable depth. Therefore, the volume of the lagoon and consequently residence time are fixed. Effluent quality  $S_e$  achieved for this residence time is calculated from Eq. (6.25) [or Eqs. (6.26) and (6.27)]. For the activated sludge plant it is unimportant to write Eq. (6.25), since  $S_e$  is a *primary specification*, rather than a *calculated value*.

### 5.7. MLVSS CONCENTRATION IN AERATED LAGOONS

Consider an aerated lagoon for complete mix conditions indicated by Fig. 6.11. Let  $X_{r,o}$  be the concentration of VSS in the influent (mg/liter) and  $X_{r,a}$  the concentration of MLVSS for the lagoon (same as that in effluent, mg/liter).

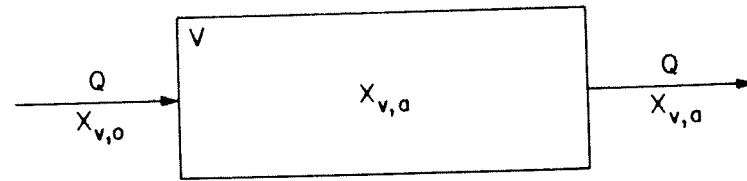


Fig. 6.11. Material balance for MLVSS.

A material balance for VSS yields

$$(\text{Solids in}) + (\text{net synthesis in lagoon}) = (\text{solids out})$$

or

$$QX_{v,o} + (aS_r Q - bX_{v,a} V) = QX_{v,a}$$

Dividing through by  $Q$ , letting  $V/Q = t =$  detention time, and solving for  $X_{v,a}$ ,

$$X_{v,a} = (X_{v,o} + aS_r)/(1 + bt) \quad (6.28)$$

For facultative lagoons,  $X_{v,a}$  is lower than the value estimated from Eq. (6.28) depending on lagoon geometry, aerator spacing, power level, and the nature of influent solids.

Remarks similar to those made in Section 5.6 concerning the difference in design approach between activated sludge plants and aerated lagoons are applicable to Eq. (6.28). For activated sludge plants, MLVSS concentration in the aerator (i.e.,  $X_{v,a}$ ) is specified by the designer. For aerated lagoons,  $X_{v,a}$  is a calculated quantity (from the specified residence time  $t$  from which  $S_e$  is calculated, and the value for  $S_r = S_o - S_e$  is thus established). Equation (6.28) is then utilized to calculate  $X_{v,a}$ . For activated sludge plants it is unimportant to write Eq. (6.28), since  $X_{v,a}$  is a primary specification rather than a calculated quantity.

### 5.8. RETENTION PERIOD REQUIRED FOR A SPECIFIED EFFLUENT SOLUBLE BOD

From the design approach in Section 5.6, retention period is usually a primary variable which is indirectly specified by the designer. Consequently, obtaining an equation for  $t$  for the aerated lagoon is less important than it was for the activated sludge plant. Nevertheless, an equation for residence time is derived which may be utilized to evaluate residence times for a specified value of  $S_e$  for several possible selected values of lagoon depth. Take Eq. (5.18) and substitute  $X_{v,a}$  by its value given in Eq. (6.28). Simplifying and solving for residence time  $t$ , one obtains

$$t = S_r / [(X_{v,o} + aS_r)kS_e - bS_r] \quad (6.29)$$

where  $S_r = S_o - S_e$ . If the concentration of VSS in the influent ( $X_{v,o}$ ) is negligible, Eq. (6.29) simplifies to yield

$$t = 1/(akS_e - b) \quad (X_{v,o} \approx 0) \quad (6.30)$$

If in this equation the units to be utilized are  $t$  (days),  $a$  (lb MLVSS/lb BOD<sub>r</sub>),  $k$  (hr<sup>-1</sup>),  $S_e$  (mg/liter), and  $b$  [lb MLVSS/(day)(lb MLVSS)], a conversion factor of 24 hr/day is used. Therefore, Eq. (6.29) is rewritten as Eq. (6.31).

$$t = 1/(24akS_e - b) \quad (X_{v,o} \approx 0) \quad (6.31)$$

### 5.9. TOTAL EFFLUENT BOD FOR AN AERATED LAGOON

So far, only soluble BOD of the effluent ( $S_e$ ) has been considered. To this, one must add the BOD contribution corresponding to volatile suspended solids present in the effluent.\* Assuming complete mixing, the concentration of VSS in the effluent is equal to that in the lagoon. BOD contribution due to VSS depends on sludge age, which for aerated lagoons is calculated from Eq. (5.150) [(or Eq. (5.151) if  $X_{v,o} = 0$ )]. As sludge stabilizes with age, its contribution to effluent BOD lowers. Correlation between sludge age and BOD contribution by VSS is shown in Fig. 6.12. The total BOD of the effluent is [Eq. (6.32)]

$$\text{Total BOD of effluent} = S_e + \psi X_{v,a} \quad (\text{mg/liter}) \quad (6.32)$$

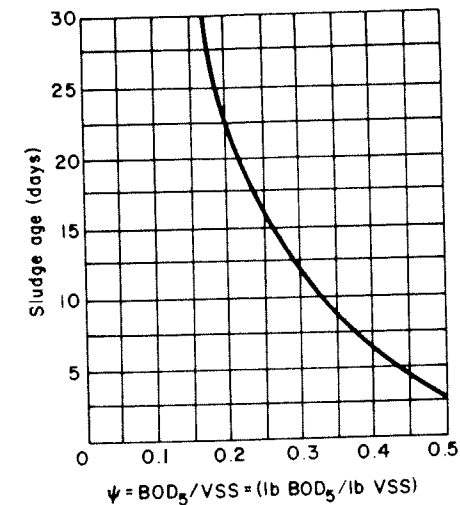


Fig. 6.12. Correlation for insoluble BOD [5].

\* For the activated sludge process, complete settling of MLVSS in the secondary clarifier is assumed. Therefore only soluble BOD is accounted for in the net effluent (refer to Fig. 5.1), where  $X_{v,e} \approx 0$ .

where  $S_e$  is calculated from Eq. (6.26) [or Eq. (6.27)],  $\psi$  is determined from Fig. 6.12, and  $X_{v,a}$  is calculated from Eq. (6.28).

### 5.10. DESIGN PROCEDURE FOR AERATED LAGOONS

The procedure proposed by Eckenfelder and associates is illustrated by Example 6.2.

#### Example 6.2

An aerated lagoon is contemplated for treating an industrial wastewater. An area of 5 acres is available. The following information is taken as a basis for design:

$$Q = 1.5 \text{ MGD (average flow rate)}$$

$$S_o = 600 \text{ mg/liter (BOD}_5 \text{ of influent)}$$

$$X_{i,o} = 20 \text{ mg/liter (VSS in influent)}$$

$$T_a = \text{average air temperature, } 80^\circ\text{F (summer); } 35^\circ\text{F (winter)}$$

$$T_i = 100^\circ\text{F (influent temperature)}$$

Data obtained from bench scale equipment:

$$K = 0.06 \text{ hr}^{-1} = 1.44 \text{ day}^{-1} \text{ (at } 20^\circ\text{C)}$$

$$a = 0.5 \text{ lb VSS/lb BOD}_r$$

$$b = 0.06 \text{ day}^{-1}$$

$$a' = 1.1 \text{ lb O}_2/\text{lb BOD}_r$$

$$h = 100 \text{ BTU}/(\text{day})(\text{ft}^2)(^\circ\text{F})$$

Calculate

1. Effluent soluble BOD<sub>5</sub> for summer and winter conditions. Base calculations on a lagoon depth of 8 ft.
2. MLVSS concentration at equilibrium for a completely mixed lagoon  $X_{v,a}$  for summer and winter conditions.
3. Total BOD<sub>5</sub> in the effluent for summer and winter conditions.
4. Surface aeration requirements: lb O<sub>2</sub>/day, required HP, and power level in HP/Mgal of basin volume.

SOLUTION: Part 1 BOD<sub>5</sub> of effluent

Step 1. Estimate  $T_w$  for summer and winter conditions [Eq. (6.21)]. Here

$$A = 5 \text{ acre} \times 43,560 \text{ ft}^2/\text{acre} = 217,800 \text{ ft}^2$$

and

$$f = 12 \times 10^{-6}$$

corresponding to

$$h = 100 \text{ BTU}/(\text{day})(\text{ft}^2)(^\circ\text{F})$$

$$\begin{aligned} \text{Summer: } T_w &= (217,800 \times 12 \times 10^{-6} \times 80 + 1.5 \times 100) / (217,800 \times 12 \times 10^{-6} + 1.5) \\ &= 87.1^\circ\text{F} \quad (30.6^\circ\text{C}) \end{aligned}$$

$$\begin{aligned} \text{Winter: } T_w &= (217,800 \times 12 \times 10^{-6} \times 35 + 1.5 \times 100) / (217,800 \times 12 \times 10^{-6} + 1.5) \\ &= 58.7^\circ\text{F} \quad (14.8^\circ\text{C}) \end{aligned}$$

Step 2. Estimate BOD removal rate  $K$  for summer and winter conditions [Eq. (6.22)].

$$\text{Summer: } K_{30.6^\circ\text{C}} = 1.44 \times 1.056^{(30.6-20)} = 2.57 \text{ day}^{-1}$$

$$\text{Winter: } K_{14.8^\circ\text{C}} = 1.44 \times 1.135^{(14.8-20)} = 0.745 \text{ day}^{-1}$$

Step 3. Calculate detention time [Eq. (6.16)].

$$t = \frac{217,800 \text{ ft}^2 \times 8 \text{ ft} \times 7.48 \text{ gal/ft}^3}{1,500,000 \text{ gal/day}} = 8.7 \text{ days}$$

Step 4. Calculate  $S_e$  (soluble BOD<sub>5</sub> of effluent) for summer and winter conditions.

$$\text{Summer: From Eq. (6.26) } S_e = 1.2 \times 600 / (1 + 2.57 \times 8.7) = 30.8 \text{ mg/liter}$$

$$\% \text{ soluble BOD removal: } [(600 - 30.8) / 600] \times 100 = 95\%$$

$$\text{Winter: From Eq. (6.27) } S_e = 1.05 \times 600 / (1 + 0.745 \times 8.7) = 84.2 \text{ mg/liter}$$

$$\% \text{ soluble BOD removal: } [(600 - 84.2) / 600] \times 100 = 86\%$$

SOLUTION: Part 2 MLVSS concentration [Eq. (6.28)]

$$\text{Summer: } X_{v,a} = [20 + 0.5(600 - 30.8)] / [1 + (0.06)(8.7)] = 200 \text{ mg/liter}$$

$$\text{Winter: } X_{v,a} = [20 + 0.5(600 - 84.2)] / [1 + (0.06)(8.7)] = 182.6 \text{ mg/liter}$$

SOLUTION: Part 3 Total BOD<sub>5</sub> for effluent

Soluble BOD<sub>5</sub> of the effluent has been calculated in Solution, Part 1, Step 4.

$$\text{Summer: } S_e = 30.8 \text{ mg/liter}$$

$$\text{Winter: } S_e = 84.2 \text{ mg/liter}$$

Step 1. Estimate  $\psi$  from Fig. 6.12. First calculate sludge age [Eq. (5.150)].

$$\text{Summer: } t_s = [200 / (200 - 20)] 8.7 = 9.67 \text{ days}$$

$$\text{Winter: } t_s = [182.6 / (182.6 - 20)] 8.7 = 9.77 \text{ days}$$

Then from Fig. 6.12,

Summer: For  $t_s = 9.67$  days, read  $\psi = 0.332$

Winter: For  $t_s = 9.77$  days, read  $\psi = 0.330$

Step 2. Estimate VSS contribution to effluent  $BOD_5$ .

Summer:  $\psi X_{v,a} = 0.332 \times 200 = 66.4$  mg/liter

Winter:  $\psi X_{v,a} = 0.330 \times 182.6 = 60.3$  mg/liter

Step 3. Calculate total  $BOD_5$  in effluent.

Summer: Soluble  $BOD_5$      30.8 mg/liter

$BOD_5$ (VSS)	$\frac{66.4 \text{ mg/liter}}{97.2 \text{ mg/liter}}$	}
---------------	---	---

Winter: Soluble  $BOD_5$      84.2 mg/liter

$BOD_5$ (VSS)	$\frac{60.3 \text{ mg/liter}}{144.5 \text{ mg/liter}}$
---------------	--

SOLUTION: Part 4 Surface aeration requirements

Step 1. Estimate oxygen requirements [Eq. (6.24)].

Summer:  $BOD_r = 600 - 30.8 = 569.2$  mg/liter =  $569.2 \times 10^{-6}$  lb  $BOD_r$ /lb liquor

$$BOD_r/\text{day} = 569.2 \times 10^{-6} \text{ lb } BOD_r/\text{lb liquor} \times 1.5 \times 10^6 \text{ gal liquor/day} \\ \times 8.34 \text{ lb liquor/gal liquor} = 7121 \text{ lb } BOD_r/\text{day}$$

$$\text{lb } O_2/\text{day} = 1.1 \text{ lb } O_2/\text{lb } BOD_r \times 7121 \text{ lb } BOD_r/\text{day} = 7833 \text{ lb } O_2/\text{day} \\ = 326 \text{ lb } O_2/\text{hr}$$

Winter:  $BOD_r = 600 - 84.2 = 515.8$  mg/liter =  $515.8 \times 10^{-6}$  lb  $BOD_r$ /lb liquor

$$BOD_r/\text{day} = 515.8 \times 10^{-6} \text{ lb } BOD_r/\text{lb liquor} \times 1.5 \times 10^6 \text{ gal liquor/day} \\ \times 8.34 \text{ lb liquor/gal liquor} = 6453 \text{ lb } BOD_r/\text{day} \\ = 269 \text{ lb } BOD_r/\text{hr}$$

$$\text{lb } O_2/\text{day} = 1.1 \text{ lb } O_2/\text{lb } BOD_r \times 6453 \text{ lb } BOD_r/\text{day} = 7098 \text{ lb } O_2/\text{day} \\ = 296 \text{ lb } O_2/\text{hr}$$

Step 2. Estimate lb  $O_2$ /(HP  $\times$  hr) [Eq. (4.34)]. Base estimate on the following values:

$$N_o = 2.5 \text{ lb } O_2/\text{HP} \times \text{hr}$$

$$\alpha = 0.8$$

$$C_{s,w} = 7.0 \text{ mg/liter (summer); } 9.5 \text{ mg/liter (winter)}$$

$$C_L = 1.0 \text{ mg/liter}$$

Then

Summer: For  $T_w = 30.6^\circ\text{C}$  (Part 1, Step 1)

$$N = 2.5[(7.0 - 1.0)/9.2]0.8 \times 1.024^{(30.6 - 20)} = 1.68 \text{ lb } O_2/(\text{HP} \times \text{hr})$$

Winter: For  $T_w = 14.8^\circ\text{C}$  (Part 1, Step 1)

$$N = 2.5[(9.5 - 1.0)/9.2]0.8 \times 1.024^{(14.8 - 20)} = 1.63 \text{ lb } O_2/(\text{HP} \times \text{hr})$$

Step 3. Calculate the required HP.

$$HP = \frac{\text{lb } O_2/\text{hr}}{\text{lb } O_2/\text{HP} \times \text{hr}}$$

where the numerator and denominator have been calculated in Part 4 (Steps 1 and 2, respectively).

Summer:  $HP = 326/1.68 = 194$  HP

Winter:  $HP = 296/1.63 = 182$  HP

Summer operation controls design.

Step 4. Estimate the power level based on 194 HP. The lagoon has a volume of  $217,800 \text{ ft}^2 \times 8 \text{ ft} = 1,742,400 \text{ ft}^3$ , or

$$1,742,400 \text{ ft}^3 \times 7.48 \text{ gal/ft}^3 = 13,033,152 \text{ gal}$$

$$\therefore \text{HP/Mgal} = 194/13.03 = 14.9 \text{ HP/Mgal} \quad (\text{faculative lagoon level})$$

## 6. Wastewater Stabilization Ponds

### 6.1. INTRODUCTION

The basic difference between the wastewater treatment process described in this section and those previously studied is that no aeration equipment is employed in stabilization ponds. Oxygen needs for ponds are provided by natural surface aeration and by algae, which produce oxygen by photosynthesis. Oxygen released by algae as a result of photosynthesis is utilized by bacteria for aerobic degradation of organic matter. Products of this degradation (carbon dioxide, ammonia, phosphates) are in turn utilized by algae. This cycle symbiotic relationship between algae and bacteria is shown diagrammatically in Fig. 6.13.

Wastewater stabilization ponds are feasible when large land areas are available at low cost and high quality effluent is not required. If BOD of the influent is high, oxygen demand is above that provided by photosynthesis and natural surface aeration. Under these circumstances DO concentration in the

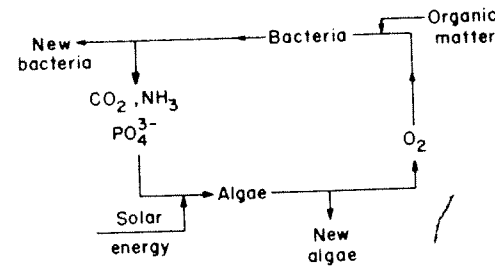


Fig. 6.13. Cyclic symbiotic relationship between algae and bacteria.

wastewater drops to a very low level and anaerobic decomposition prevails. Terminal products for anaerobic decomposition are  $\text{CH}_4 + \text{H}_2\text{O}$ , in contrast to  $\text{CO}_2 + \text{H}_2\text{O}$  for aerobic decomposition. Chemical equations pertinent to anaerobic decomposition are studied in Section 8.1.

Ponds in which the upper layers are aerobic and the lower are anaerobic are referred to as facultative ponds. Most stabilization ponds fall in this category.

Whenever organic loading is very high, oxygen demand may be such that pond operation is anaerobic. When several ponds are operating in series, the first one receiving raw wastewater discharge is anaerobic and the second, which receives partially stabilized wastewater from the first, may be a facultative pond. The last one receiving relatively low BOD wastewater discharge from the preceding one might function as an aerobic pond.

Because of high detention time, frequently about 2 months, removal of refractory organic materials which cannot be accomplished by activated sludge or aerated lagoon processes may become possible in stabilization ponds. Thus, a convenient arrangement may be to provide stabilization ponds following an activated sludge (or aerated lagoon) unit to complete stabilization.

## 6.2. KINETICS OF BOD REMOVAL FOR STABILIZATION PONDS

An idealized approach similar to that utilized for aerated lagoons is often employed for stabilization ponds. Concentration of MLVSS is not a relevant parameter in stabilization ponds, and thus the term  $X_{r,a}$  does not appear in the mathematical model. Equation 5.18 is rewritten as

$$(S_0 - S_e)/t = KS_e \quad (6.33)$$

Equation (6.33) indicates that a plot of  $(S_0 - S_e)/t$  vs.  $S_e$  yields a straight line, and the value of  $K$  is determined from the slope. Typical graphs of this type are shown in Figs. 6.18 and 6.19.

## 6.3. LABORATORY SIMULATION OF STABILIZATION PONDS

Batch or continuous models of stabilization ponds have been used on bench or pilot scale. Two of these models, recommended by Eckenfelder and Ford [5], are shown in Fig. 6.14.

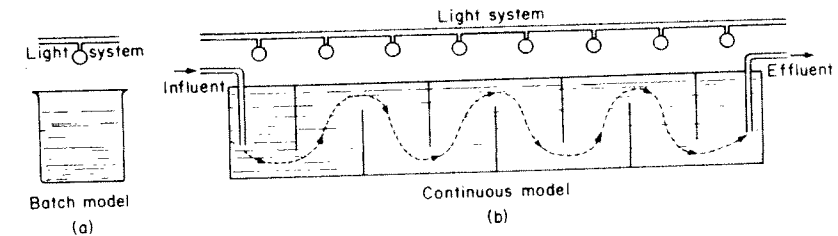


Fig. 6.14. Laboratory models for wastewater stabilization ponds.

From data obtained from these models, graphs such as Figs. 6.18 and 6.19 are constructed, and an estimate of  $K$  values is made.

## 6.4. MATHEMATICAL FORMULATION FOR SEVERAL STABILIZATION PONDS IN SERIES

Start from the modified form of the first-order kinetics removal equation for one pond. Solving Eq. (6.15) for  $t$  yields:

$$t = (1 - S_e/S_0)/K(S_e/S_0) \quad (6.34)$$

Ratio  $S_e/S_0$  represents the % BOD remaining in the effluent.

For two stabilization ponds in series (Fig. 6.15), one can write [Eq. (6.15)]

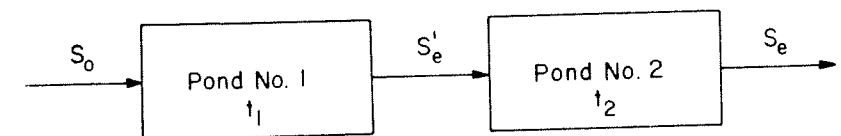


Fig. 6.15. Diagram for two stabilization ponds in series.

$$\text{For pond No. 1: } S_e'/S_0 = 1/(1 + K_1 t_1) \quad (6.35)$$

$$\text{For pond No. 2: } S_e/S_e' = 1/(1 + K_2 t_2) \quad (6.36)$$

Combining Eqs. (6.35) and (6.36) by multiplication,

$$S_e/S_0 = 1/(1 + K_1 t_1)(1 + K_2 t_2) \quad (6.37)$$

When retention period  $t$  and removal rate constant  $K$  are the same for both ponds, Eq. (6.37) yields

$$S_e/S_0 = 1/(1 + Kt)^2 \quad (6.38)$$

where  $t_1 = t_2 = t$  and  $K_1 = K_2 = K$ . Solving Eq. (6.38) for  $t$ ,

$$t = [1 - (S_e/S_o)^{1/2}] / K(S_e/S_o)^{1/2} \quad (6.39)$$

For  $n$  ponds in series, Eqs. (6.38) and (6.39) yield

$$S_e/S_o = 1/(1 + Kt)^n \quad (6.40)$$

$$t = [1 - (S_e/S_o)^{1/n}] / K(S_e/S_o)^{1/n} \quad (6.41)$$

Further simplification is possible whenever the product  $Kt$  is much less than unity. Exponential  $e^{Kt}$  is given by the power series in Eq. (6.42).

$$e^{Kt} = 1 + Kt + (Kt)^2/2! + (Kt)^3/3! + \dots \quad (6.42)$$

If  $Kt \ll 1$ , one can write as an approximation

$$e^{Kt} = 1 + Kt \quad (6.43)$$

Substituting in the denominator of Eq. (6.40)  $(1 + Kt)$  by the exponential  $e^{Kt}$ , gives Eq. (6.44),

$$S_e/S_o = 1/e^{nKt} = e^{-nKt} \quad (6.44)$$

from which

$$S_o/S_e = e^{nKt} \quad (6.45)$$

or

$$(S_o/S_e)^{1/n} = e^{Kt} \quad (6.46)$$

### 6.5. EFFECT OF TEMPERATURE ON REACTION RATE CONSTANT $K$

This effect is calculated from an empirical relationship recommended by Eckenfelder and Ford [5].

$$K_t = K_{25} \theta^{t-25} \quad (6.47)$$

where  $K_t$  is the reaction rate at  $t^\circ\text{C}$ ;  $K_{25}$  the reaction rate at  $25^\circ\text{C}$ ;  $t$  the temperature ( $^\circ\text{C}$ ); and  $\theta$  the temperature constant (1.06–1.09; take  $\theta = 1.07$ ).

### 6.6. OXYGEN PRODUCTION IN AEROBIC PONDS

Aerobic stabilization ponds depend on algae to provide the oxygen necessary to satisfy BOD requirements. Since this oxygen is produced by photosynthesis, sunlight is required. This restricts the depth of aerobic ponds to a range of 6–18 in.

The amount of oxygen produced by algae is estimated from Oswald's equation [Eq. (6.48)] [9].

$$OP = 0.25FI_L \quad (6.48)$$

where  $OP$  is the oxygen production [ $\text{lb O}_2/(\text{acre})(\text{day})$ ];  $F$  the light conversion efficiency (%); and  $I_L$  the light intensity [ $\text{cal}/(\text{cm}^2)(\text{day})$ ].

$F$  is usually assumed to be 4%. Thus  $F = 4$  and

$$OP \approx I_L \quad (6.49)$$

$I_L$  varies from about 100 to 300  $\text{cal}/(\text{cm}^2)(\text{day})$  during winter and summer, respectively, for a latitude of  $30^\circ$ . This means that the maximum BOD loading for aerobic operation of stabilization ponds to take place varies from 100 to 300  $\text{lb BOD}_5/(\text{acre})(\text{day})$ .

### 6.7. DEPTH OF OXYGEN PENETRATION IN STABILIZATION PONDS

Depth of oxygen penetration has been correlated by Oswald [10] to surface loading, expressed as  $\text{lb BOD}_5/(\text{acre})(\text{day})$  (Fig. 6.16). The greater the loading, the shallower the depth of oxygen penetration since oxygen demand is higher.

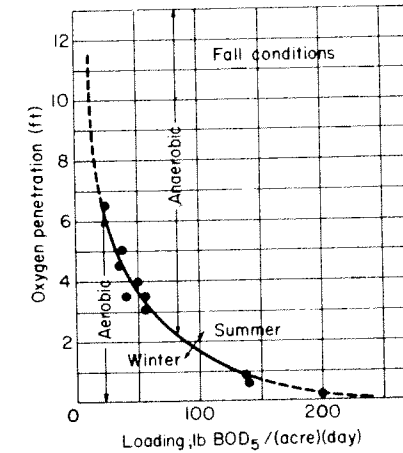


Fig. 6.16. Correlation for depth of oxygen penetration in stabilization ponds (adapted from Oswald [10]). (Reprinted with permission, copyright by the University of Texas Press).

### 6.8. FACULTATIVE PONDS: HERMANN AND GLOYNA'S EQUATION

The design formulation for stabilization ponds described in Sections 6.2 and 6.4 applies to facultative ponds. Facultative pond depths vary from 3 to 8 ft.

An empirical equation for facultative ponds has been developed by Hermann and Gloyna [6]. This formula is based on several assumptions and developed by analysis of results from bench scale, pilot-plant, and field ponds. It is applicable to domestic sewage for a 85–95% BOD reduction [Eq. (6.50)].

$$V = 10.7 \times 10^{-8} Q S_o (1.085^{35-t}) \quad (6.50)$$

where  $V$  is the pond volume ( $\text{acre} \times \text{ft}$ );  $Q$  the wastewater flow ( $\text{gal}/\text{day}$ );  $S_o$  the  $\text{BOD}_u$  of influent ( $\text{mg}/\text{liter}$ ); and  $t$  the pond temperature ( $^\circ\text{C}$ ).

### 6.9. ANAEROBIC PONDS

Loading of anaerobic ponds is such that anaerobic conditions prevail throughout the liquid. Organic loadings range between 250 and 4000 lb BOD<sub>5</sub>/(acre)(day). Removal efficiencies vary between 50 and 80%. Since this degree of BOD<sub>5</sub> removal is usually not adequate for discharging the effluent, anaerobic ponds are usually followed by facultative and aerobic ones. Depths from 8 to 15 ft are common, but greater depths are recommended to provide maximum heat retention, besides the resulting economy in terms of land cost.

### 6.10. SUMMARY OF DESIGN CRITERIA FOR WASTEWATER STABILIZATION PONDS

Design criteria are summarized in Table 6.2.

**TABLE 6.2**  
Summary of Design Criteria for Wastewater Stabilization Ponds<sup>a</sup>

Criteria	Ponds		
	Aerobic	Facultative	Anaerobic
Depth (ft)	0.5-1.5	3-8	8-15
Detention time (days)	2-6	7-50	5-50
Loading			
lb BOD <sub>5</sub> /(acre)(day)	100-200	200-500	250-4000
% BOD removal	80-95	70-95	50-80
Algae concentration (mg/liter)	100	10-50	—

<sup>a</sup> Adapted from Eckenfelder [4].

### 6.11. DESIGN CALCULATIONS FOR STABILIZATION PONDS

Design for stabilization ponds is illustrated by Example 6.3. The design procedure is that recommended by Eckenfelder and associates.

#### Example 6.3

Wastewater stabilization ponds are considered for treatment of organic chemicals waste. Total design flow is 1.0 MGD, and estimated pond temperatures are 15° and 30°C for winter and summer operations, respectively.

It is desired to reduce wastewater COD from 2000 to 400 mg/liter using two anaerobic ponds in series of equal detention time, and then to lower the COD of the effluent from the second anaerobic pond to 50 mg/liter by means of an

aerobic pond. Bench scale tests are performed with wastewater for both anaerobic and aerobic pond conditions, and the laboratory data obtained at 25°C are tabulated below.

	S <sub>0</sub> (mg/liter COD)	S <sub>e</sub> (mg/liter COD)	Detention time (days)
Anaerobic pond			
Run 1	3000	1000	40
Run 2	2000	667	40
Run 3	1200	400	40
Aerobic pond			
Run 4	700	49	40
Run 5	400	28	40
Run 6	300	21	40

Design the treatment system.

**SOLUTION** See Figs. 6.17-19.

*Step 1.* Obtain constant  $K$  at 25°C for the anaerobic and aerobic ponds, from plots of laboratory data  $[(S_0 - S_e)/t$  vs.  $S_e$ ] shown in tabulation below.

	S <sub>0</sub> (mg/liter COD)	S <sub>e</sub> (mg/liter COD)	Detention time (days)	(S <sub>0</sub> - S <sub>e</sub> )/t
Anaerobic				
Run 1	3000	1000	40	50
Run 2	2000	667	40	33.3
Run 3	1200	400	40	20
Aerobic				
Run 4	700	49	40	16.3
Run 5	400	28	40	9.3
Run 6	300	21	40	7.0

In summary

$$K \text{ (anaerobic ponds)} = 0.05 \text{ day}^{-1}$$

$$K \text{ (aerobic pond)} = 0.335 \text{ day}^{-1}$$

*Step 2.* Obtain values of  $K$  at 15°C since winter conditions control the design [Eq. (6.47)].

For the anaerobic ponds

$$K_{15} = 0.05 \times 1.07^{(15-25)} = 0.0254 \text{ day}^{-1}$$

(was 0.05 day<sup>-1</sup> at 25°C, laboratory conditions).

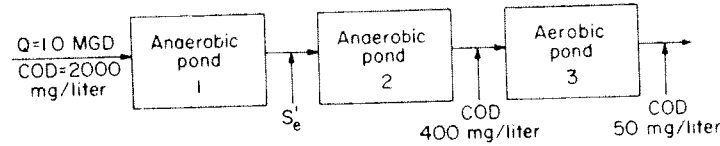


Fig. 6.17. Diagram for Example 6.3.

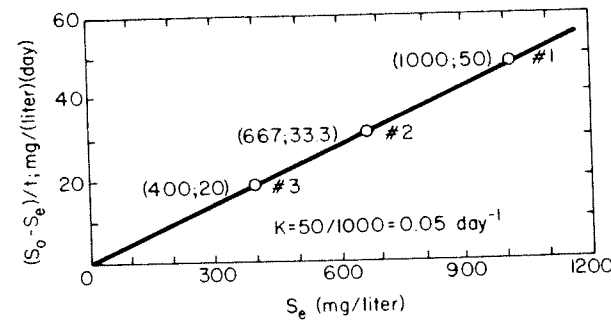


Fig. 6.18. Determination of  $K$  (anaerobic ponds) for Example 6.3.

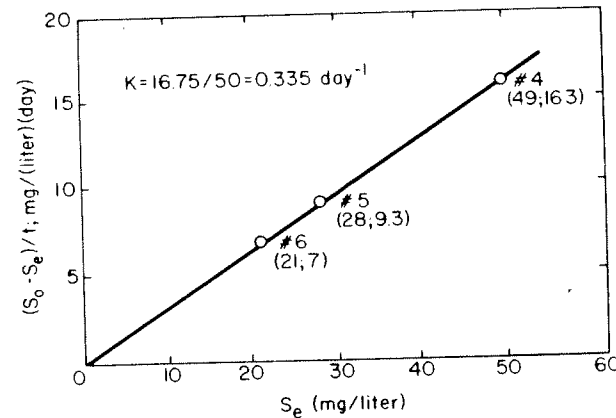


Fig. 6.19. Determination of  $K$  (aerobic pond) for Example 6.3.

For the aerobic pond

$$K_{1.5} = 0.335 \times 1.07^{(1.5-2.5)} = 0.17 \text{ day}^{-1}$$

(was  $0.335 \text{ day}^{-1}$  at  $25^\circ\text{C}$ , laboratory conditions).

Step 3. Calculate detention times.

For the two anaerobic ponds in series [Eq. (6.39)]

$$t = [1 - (400/2000)^{1/2}] / 0.0254 (400/2000)^{1/2} = 48.7 \text{ days} \quad (\text{each pond})$$

Since for the two anaerobic ponds in Example 6.3  $K_1 = K_2 = K_{1.5} = 0.0254 \text{ day}^{-1}$  and  $t_1 = t_2 = t = 48.7 \text{ days}$ , it follows that from Eqs. (6.35) and (6.36),

$$S_e'/S_0 = S_e/S_e'$$

$$\therefore S_e' = (S_0 S_e)^{1/2} = (2000 \times 400)^{1/2} = 894 \text{ mg/liter}$$

(COD of effluent from anaerobic pond 1)

For the aerobic pond [Eq. (6.34)]

$$t = (1 - 50/400) / 0.17 (50/400) = 41.2 \text{ days}$$

Step 4. Calculate pond area (acres). For the two anaerobic ponds assume a depth of 12 ft. Since  $t = V/Q = Ah/Q$ , where  $t$  is the residence time (day);  $V$  the volume of pond ( $\text{ft}^3$ );  $Q$  the flow rate ( $\text{ft}^3/\text{day}$ );  $A$  the area of pond ( $\text{ft}^2$ ); and  $h$  the depth of pond (ft), then  $A = tQ/h$  or

$$A = 48.7 \text{ days} \times (1 \times 10^6 \text{ gal/day} \times \text{ft}^3/7.48 \text{ gal} \times 1/12 \text{ ft} \times \text{acre}/43,560 \text{ ft}^2) = 12.5 \text{ acres per pond}$$

Therefore, design two 12.5 acre ponds, each 12 ft deep. Surface loading [ $\text{lb BOD}_5/(\text{acre})(\text{day})$ ] for each of the anaerobic ponds is shown below.

Anaerobic pond 1

$$\text{Since } 2000 \text{ mg/liter} = 2000 \times 10^{-6} \text{ lb COD/lb liquor}$$

$$\begin{aligned} \text{lb COD/day} &= 1 \times 10^6 \text{ gal liquor/day} \times 8.34 \text{ lb liquor/gal liquor} \\ &\quad \times 2000 \times 10^{-6} \text{ lb COD/lb liquor} \\ &= 1 \times 8.34 \times 2000 = 16,680 \text{ lb COD/day} \end{aligned}$$

Assume  $\text{BOD}_5/\text{COD} \approx 0.7$ . Then  $\text{lb BOD}_5/\text{day} = (0.7)(16,680) = 11,676$  and  $\text{lb BOD}_5/(\text{acre})(\text{day}) = 11,676/12.5 = 934$  (surface loading for anaerobic pond 1).

Anaerobic pond 2

$$\text{lb COD/day} = 1 \times 8.34 \times 894 = 7456$$

$$\text{lb BOD}_5/\text{day} = (0.7)(7456) = 5219$$

$$\text{lb BOD}_5/(\text{acre})(\text{day}) = 5219/12.5 = 418 \quad (\text{surface loading for anaerobic pond 2})$$

For the aerobic pond, the design procedure is as follows:

1. Assume a depth of oxygen penetration, e.g.,  $h = 3 \text{ ft}$ .
2. From knowledge of residence time and flow rate, calculate a first approximation of the pond area (acres):  $A = tQ/h$ .
3. Calculate surface loading in  $\text{lb BOD}_5/(\text{acre})(\text{day})$  based on the assumed value of  $h$ .



4. From Fig. 6.16 determine the depth of oxygen penetration and compare it with the assumed value under (1). For aerobic operation, the depth of oxygen penetration should be *at least* equal to the assumed depth. If necessary, assume another value of  $h$  and iterate steps (1)–(4).

5. Oxygen production by algae is checked to assure that it is sufficient to satisfy the surface loading [Eq. (6.49)].

Calculations are as follows:

1. Assume  $h = 3$  ft.
2. Calculate  $A$  in acres.

$$\begin{aligned} A &= 41.2 \text{ days} \times (1 \times 10^6 \text{ gal/day} \times \text{ft}^3/7.48 \text{ gal} \times 1/3 \text{ ft} \times \text{acre}/43,560 \text{ ft}^2) \\ &= 42.1 \text{ acres} \end{aligned}$$

3. Surface loading. First calculate loading in terms of lb COD/(acre)(day). Since  $400 \text{ mg/liter} = 400 \times 10^{-6} \text{ lb COD/lb liquor}$ ,

$$\begin{aligned} \text{lb COD/day} &= 1 \times 10^6 \text{ gal liquor/day} \times 8.34 \text{ lb liquor/gal liquor} \\ &\quad \times 400 \times 10^{-6} \text{ lb COD/lb liquor} \\ &= 3336 \text{ lb COD/day} \end{aligned}$$

Therefore,

$$\text{lb COD}/(\text{acre})(\text{day}) = 3336/42.1 = 79.2 \text{ lb COD}/(\text{acre})(\text{day})$$

Assume  $\text{BOD}_5/\text{COD} \approx 0.7$ . Then the surface loading in terms of  $\text{BOD}_5$  is

$$\text{lb BOD}_5/(\text{acre})(\text{day}) = 0.7 \times 79.2 \approx 55 \text{ lb BOD}_5/(\text{acre})(\text{day})$$

From Fig. 6.16 for this loading, read  $h \approx 3$  ft. Therefore, the assumed depth is appropriate and no further trial is necessary.

4. Check oxygen production by algae. From Eq. (6.49) it follows that oxygen production restricts maximum loadings from 100 to 300 lb  $\text{BOD}_5/(\text{acre})(\text{day})$ . Since actual loading is only 55 lb  $\text{BOD}_5/(\text{acre})(\text{day})$ , an excess of oxygen over  $\text{BOD}_5$  requirements is available.

## 7. Trickling Filters

### 7.1. INTRODUCTION

The trickling filter is a packed media covered with biological slime through which wastewater is percolated. The slime layer, which usually has a total thickness between 0.1 and 2.0 mm, consists of one aerobic and one anaerobic sublayer, as shown diagrammatically in Fig. 6.20.

The biological aerobic process which takes place in the aerobic sublayer is typical (Chapter 5, Section 4.1.1, Fig. 5.6). The substrate is partially oxidized

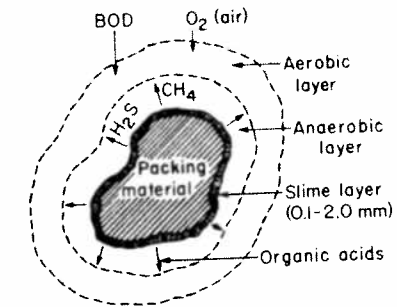


Fig. 6.20. Diagram of aerobic and anaerobic sublayers for a trickling filter.

to provide energy for the biological process. Another part of the substrate is utilized to synthesize new slime material.

In the anaerobic sublayer decomposition takes place with formation of organic acids,  $\text{CH}_4$ , and  $\text{H}_2\text{S}$  (Section 8.1). In the trickling filter, organic and colloidal matter are removed by aerobic oxidation, biosorption, coagulation, and anaerobic decomposition. Essentially there is no removal by mechanical filtration. The term "trickling filter" is misleading in this respect.

### 7.2. THICKNESS OF SLIME LAYER

Usually, the thickness of the slime layer is between 0.1 and 2.0 mm. It has an adverse effect on the operation of the trickling filter if it is thicker than 2.0 mm. Clogging of media may occur, thus impairing the wastewater flow and the transfer of oxygen to aerobic microorganisms.

Operational hydraulic loadings are low [ $0.4\text{--}4.0 \text{ gal}/(\text{min})(\text{ft}^2)$ ] and are not sufficient to keep the slime layer scoured off. Thus, hydraulic loading cannot be used in controlling the thickness of the slime layer. This control is exerted mostly by larvae and worms, which thrive on the accumulated slime.

As the slime layer increases in thickness, organic matter in the wastewater is metabolized before it can reach the layer of microorganisms clinging to the surface of the media. These microorganisms are left without sufficient food and tend to enter the endogenous respiration phase. Thus, the slime layer loses its ability to cling to the media surface and is washed away. This phenomena, called sloughing, is a function of organic and hydraulic loading of the filter.

### 7.3. COMPARISON BETWEEN TRICKLING FILTERS AND ACTIVATED SLUDGE PROCESS

For BOD removal efficiencies of about 60%, it is usually found that trickling filters are more economical than the activated sludge process, in particular for small flow rates of wastewater. For higher BOD removal efficiencies (90% or

above), the activated sludge process is more economical because packing material costs would be too high. These considerations suggest a possible two step operation: trickling filters followed by an activated sludge plant, a combination which in some cases may prove advantageous.

Some advantages of trickling filters over the activated sludge process are (1) no power requirements for aeration, (2) simple operation, and (3) slower response and quicker recovery to sudden changes of influent BOD.

#### 7.4. PHYSICAL ARRANGEMENT OF TRICKLING FILTERS

Trickling filters are beds from 3 to 40 ft deep filled with packing such as broken rock, clinkers, or synthetic media (trade names Surfpac, Flocor, Actifil). These plastic material packings are available commercially (Dow Chemical Co., Ethyl Corporation, B.F. Goodrich, Norton Co.) in honeycomb and other shapes. Influent wastewater is usually distributed over the filter by a mechanical rotating arm mechanism and percolates through the packing, coming in contact with the biological slime layer.

Whereas beds filled with rocks, clinkers, or other materials are limited in depth from 3 to 8 ft, beds of synthetic materials are commonly 20–40 ft deep. The higher percentage of void space for synthetic packing allows an easier flow and reduces the risk of flooding.

For ordinary packing (rocks, clinkers, etc.) the following characteristics are typical: diameters: 1½–2 in.; surface area: 24–34 ft<sup>2</sup>/ft<sup>3</sup> of bulk volume; void %: 45–55%; and maximum hydraulic loadings: 0.5 gal/(min)(ft<sup>2</sup>).

Advantages of synthetic packings are that they (1) allow packing depth up to 40 ft; (2) allow higher hydraulic loadings, up to 4 gal/(min)(ft<sup>2</sup>); (3) have surface areas up to 70 ft<sup>2</sup>/ft<sup>3</sup> of bulk volume; and (4) are less likely to be clogged by wastewaters carrying large amounts of suspended solids.

Disadvantages of synthetic packings are that they are (1) relatively expensive; and (2) inappropriate for wastewater treatment to arrive at a relatively high effluent quality as compared to ordinary packing.

#### 7.5. TRICKLING FILTER SYSTEMS

Most common arrangements for trickling filters are shown in Fig. 6.21. (a) *Single filter system*—May be operated with or without recycle. Recycling is indicated for higher effluent quality. If influent BOD is greater than 500 mg/liter, recycling is usually recommended. (b) *Alternating double filtration*—The first filter is responsible for most BOD removal; the second one is an effluent polisher. Consequently, most slime growth occurs in the first filter. The cycle is reversed periodically (daily or weekly) as indicated by dotted lines in Fig. 6.21 (b). In this manner control of the slime layer thickness is achieved, maintaining a uniform slime thickness in both units. Higher BOD effluent

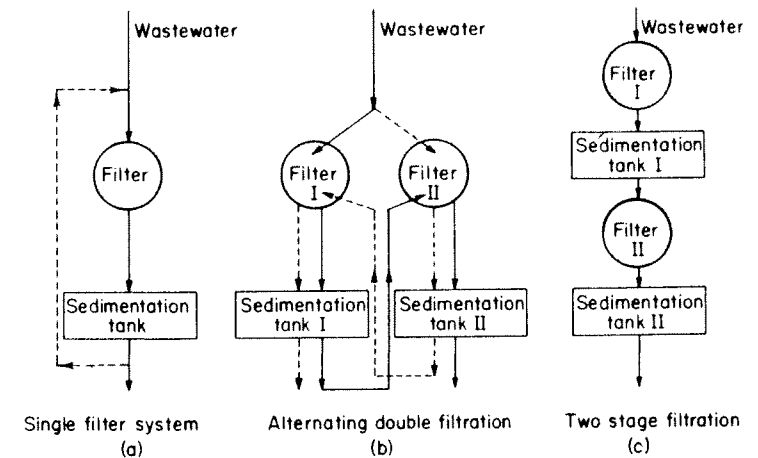


Fig. 6.21. Trickling filter systems.

quality is obtained by this system as compared to the single filters. (c) *Two stage filtration*—The first filter is a coarse one, usually filled with synthetic packing which removes 60–70% of the BOD. The second filter, where slime growth is considerably less, acts as the effluent polisher.

#### 7.6. PRETREATMENT FOR TRICKLING FILTRATION

A pretreatment similar to that for the activated sludge process may be required for trickling filtration. It may be necessary to adjust the pH by neutralization to an optimum range from 7 to 9, because excess alkalinity or acidity disturbs the biological process.

#### 7.7. DESIGN FORMULATION FOR TRICKLING FILTERS

The purpose of design formulation is to obtain a relationship among BOD removal, depth of the filter, and hydraulic loading. The following formulation is the one developed by Eckenfelder and associates. BOD removal is usually expressed as % BOD remaining in the effluent.

$$S_e/S_o = \text{effluent BOD (mg/liter)/influent BOD (mg/liter)}$$

The trickling filter depth is denoted by  $D$  (ft) and hydraulic loading by  $L$  [gal/(min)(ft<sup>2</sup>)]. Assuming BOD removal to follow first-order kinetics [Eq. (6.51)],

$$dS/dt = -k'X_v S = -K'S \quad (6.51)$$

where  $K' = k'X_v$ .

Rearranging and integrating,

$$\ln(S_e/S_o) = -k'X_v t = -K't$$

or

$$S_e/S_o = e^{-k'X_v t} = e^{-K't} \quad (6.52)$$

where  $S_e$  is the effluent BOD (mg/liter);  $S_o$  the influent BOD (mg/liter);  $k'$  the removal rate constant (no volatile solids included);  $X_v$  the volatile solids;  $K'$  the removal rate constant (volatile solids included,  $K' = k'X_v$ ); and  $t$  the residence time.

Residence time  $t$  is written as

$$t = CD/L^n \quad (6.53)$$

where  $D$  is the filter depth (ft);  $L$  the hydraulic loading [gal/(min)(ft<sup>2</sup>)]; and  $C, n$  the constants which are functions of the filter media and specific surface. Specific surface is defined as ft<sup>2</sup> of packing surface per ft<sup>3</sup> of bulk volume. Substitution of  $t$  [Eq. (6.53)] in Eq. (6.52) yields

$$S_e/S_o = e^{-K'CD/L^n} = e^{-KD/L^n} \quad (6.54)$$

where  $K = K'C$ .

Equation (6.54) is the basic mathematical model [4] for trickling filters. It relates % BOD remaining ( $S_e/S_o$ ) to depth of the filter ( $D$ ) and hydraulic loading ( $L$ ). Parameter  $K$  (rate constant) is a function of the ease of degradability of the substrate. Parameter  $n$  depends on the characteristics of the packing media.

### 7.8. APPLICATION OF BASIC MATHEMATICAL MODEL TO TRICKLING FILTERS WITHOUT AND WITH RECYCLE

Streams involved in operation of trickling filters without and with recycle are shown in the Fig. 6.22. Recycle improves efficiency of BOD removal and dilutes influent BOD to a level compatible with maintenance of aerobic conditions. When there is no recycle, Eq. (6.54) applies directly. For trickling filters with recycle, influent BOD ( $S_F$ ) is diluted to a value  $S_o$  prior to entering the filter. The relationship among  $S_o$ ,  $S_F$ ,  $S_e$ , and the recycle ratio is obtained by a material balance for the BOD [loop (---) of Fig. 6.22(b)].

$$S_e/S_o = e^{-KD/L^n} \quad (6.55)$$

$$Q_F S_F + Q_R S_e = (Q_F + Q_R) S_o$$

$$\therefore S_o = (Q_F S_F + Q_R S_e)/(Q_F + Q_R)$$

Dividing both numerator and denominator by  $Q_F$  and letting  $Q_R/Q_F = r =$  recycle ratio [Eq. (6.56)],

$$S_o = (S_F + rS_e)/(1 + r) \quad (6.56)$$

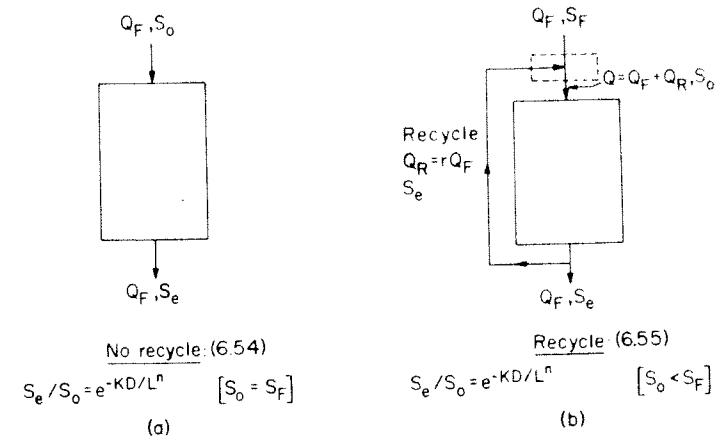


Fig. 6.22. Trickling filter without recycle (a) and with recycle (b).

Substituting this value in Eq. (6.55),

$$S_e/[(S_F + rS_e)/(1 + r)] = e^{-KD/L^n} \quad (6.57)$$

For simplicity, let  $KD/L^n = X$ . Dividing both numerator and denominator by  $S_e/S_F$  and solving for  $S_e/S_F$ ,

$$S_e/S_F = e^{-X}/(1 + r - re^{-X}) \quad (6.58)$$

For domestic sewage, an empirical relationship has been developed by Balakrishnan [2] based on Eq. (6.58):

$$S_e/S_o = \exp(-0.003A_p^{0.644} S_o^{0.54} D/L^n) \quad (6.59)$$

### 7.9. PROCEDURE FOR DESIGN OF TRICKLING FILTERS WHEN BENCH SCALE OR PILOT-PLANT DATA ARE AVAILABLE

Three items will be discussed: (1) A model of bench scale trickling filter and procedure for obtaining design data from this model, (2) treatment of data obtained in order to determine constants  $n$  and  $K$  in Eq. (6.54); and (3) application of these results to design of a plant scale trickling filter. Both cases, without and with recycle, are studied. The approach followed is that proposed by Eckenfelder and Ford [5]. Example 6.4 presents a numerical illustration.

#### 7.9.1. Model of Bench Scale Trickling Filter

A sketch of a bench scale model of a trickling filter developed by Eckenfelder and associates is shown in Fig. 6.23. Wastewater contained in the feed reservoir is pumped by a Sigmamotor pump to a perforated plate for flow distribution. There it is mixed with the recycle stream from the settling tank.

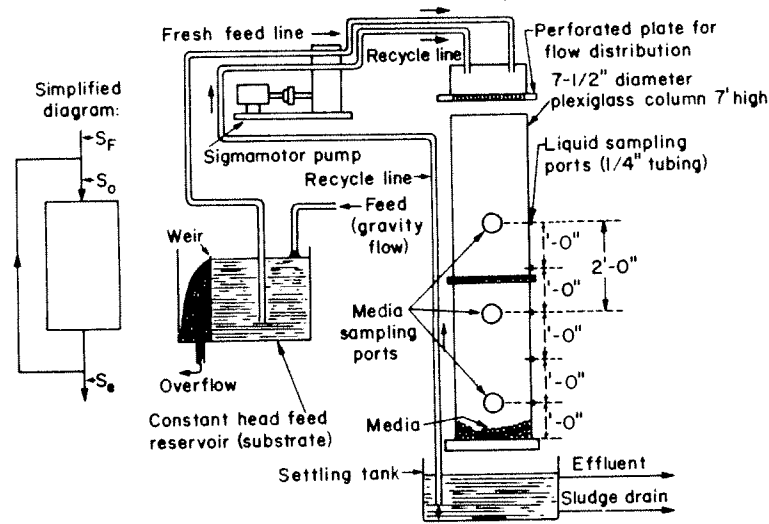


Fig. 6.23. Bench scale trickling filter.

As a preliminary step, it is necessary to generate an acclimated slime on the filter media. This may take from a few days to several weeks, depending on the nature of the wastewater. Samples are taken periodically at the media and liquid sampling ports, and BOD determinations are performed. Steady state BOD values are recorded. Pilot-plant units may also be utilized.

The procedure to obtain basic data is (1) select three or four hydraulic loadings [gal/(min)(ft<sup>2</sup>)]. For deep filters ( $D > 10$  ft) with plastic packing, flow rates of 0.5–4 gal/(min)(ft<sup>2</sup>) are selected. For the pilot-plant unit (Example 6.4), hydraulic loadings of 1, 2, 3, and 4 gal/(min)(ft<sup>2</sup>) are utilized (Table 6.3). (2) For each flow rate, sample at least three depths. For Example 6.4, four

TABLE 6.3  
Data for Example 6.4\*

D, depth (ft)	L, hydraulic loading [gal/(min)(ft <sup>2</sup> )]			
	L = 1	L = 2	L = 3	L = 4
5	57.5	67.5	73.0	76.0
10	33.5	46.0	53.0	57.0
15	19.5	31.0	38.7	43.0
20	11.3	21.0	28.2	32.8

\* Values in 4 × 4 matrix are those for  $(S_e/S_0) \times 100$ .

depths (5, 10, 15, and 20 ft) are sampled. Analyses of samples are usually expressed as % BOD remaining, i.e.,  $S_e/S_0 \times 100$ . Since in Example 6.4 four hydraulic loadings ( $L$ ) and four sampling depths are selected, the  $S_e/S_0$  values form a 4 × 4 matrix (Table 6.3). Analyses can also be made in terms of COD or TOC. Determinations of pH, Kjeldahl nitrogen, and water temperature are also performed.

7.9.2. Treatment of Data Obtained in Order to Determine Constants  $n$  and  $K$

Step 1. On semilog paper, plot the % BOD remaining ( $S_e/S_0 \times 100$ ) vs. depth ( $D$ ) for each value of hydraulic loadings ( $L$ ). From Eq. (6.54), it follows that [Eq. (6.60)]

$$\ln(S_e/S_0) = -KD/L^n = -(K/L^n)D \tag{6.60}$$

Therefore a family of straight lines is obtained. The absolute value of their slopes corresponds to  $K/L^n$ . For Example 6.4, this plot is shown in Fig. 6.24, where four lines correspond to four runs [ $L = 1, 2, 3, \text{ and } 4 \text{ gal}/(\text{min})(\text{ft}^2)$ ].

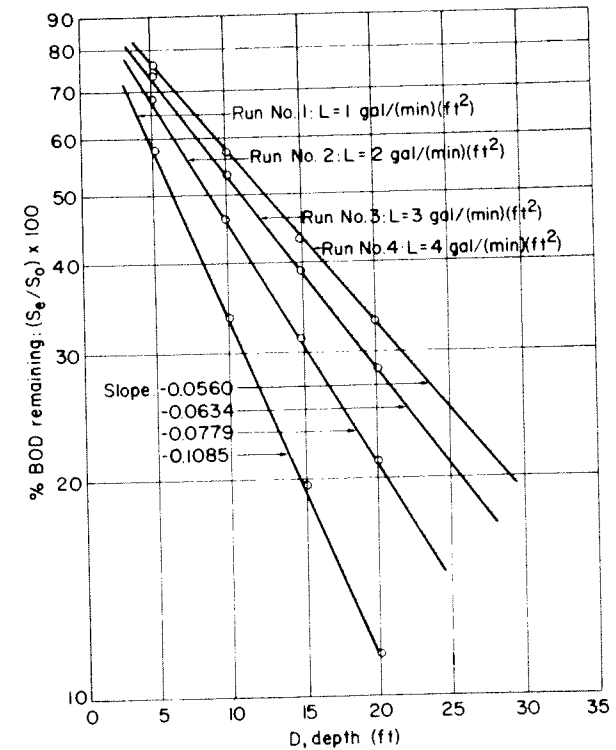


Fig. 6.24. Plot of % BOD remaining vs. depth.

Step 2. On log-log paper, plot the absolute values of the slopes for each of the lines from Step 1 vs.  $L$  (Table 6.4 and Fig. 6.25). Since

$$|\text{Slope}| = K/L^n \quad (6.61)$$

then

$$\log |\text{slope}| = \log K - n \log L \quad (6.62)$$

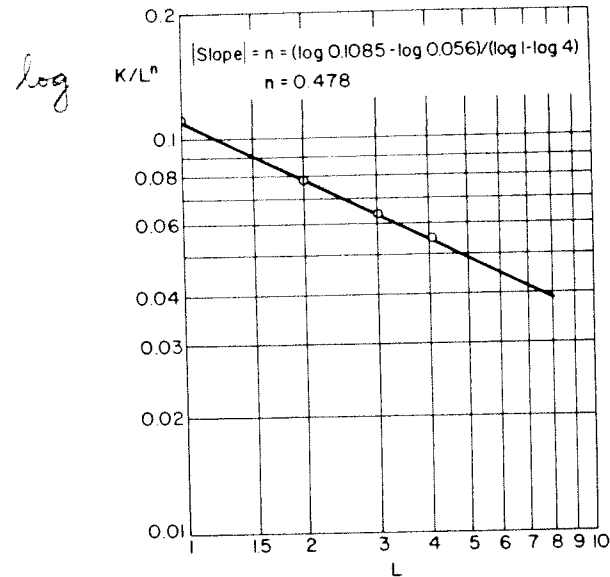


Fig. 6.25. Plot of  $K/L^n$  vs.  $L$ .

TABLE 6.4  
 $K/L^n$  vs.  $L$

$L$ [gal/(min)(ft <sup>2</sup> )]	*Absolute value of slopes, $K/L^n$
1	0.1085
2	0.0779
3	0.0634
4	0.0560

\* From Fig. 6.24.

One straight line is obtained, each point of which corresponds to one experiment. Its slope yields the value of  $n$  in accordance with Eq. (6.62). For Example 6.4 this plot is shown in Fig. 6.25, where four points are utilized for the construction.\*

\* The value of  $K$  may be estimated from the ordinate intercept (for abscissa  $L = 1$ ) of Fig. 6.25 as 0.11. It is preferable, however, to determine  $K$  from the slope of the straight line in Fig. 6.26, as described in Step 4.

Step 3. Perform the following calculations:

1. Determine the values of  $L^n$  for each experimental run utilizing the value of  $n$  determined in Step 2 (Table 6.5).

TABLE 6.5  
Values of  $L^n = L^{0.478}$  vs.  $L$

Run no.	$L$ (gal/(min)(ft <sup>2</sup> )]	$L^n = L^{0.478}$
1	1	1.0
2	2	1.393
3	3	1.690
4	4	1.941

2. Calculate the values of  $D/L^n$  for each experimental run at each depth. For Example 6.4 this results in the  $4 \times 4$  matrix (Table 6.6).

TABLE 6.6  
Matrix for Values of  $D/L^n = D/L^{0.478}$

$D$ , depth (ft)	Values of $L$			
	$L = 1$	$L = 2$	$L = 3$	$L = 4$
5	5.0	3.59	2.96	2.57
10	10.0	7.17	5.91	5.15
15	15.0	10.75	8.87	7.71
20	20.0	14.35	11.83	10.30

3. Construct a table of  $D/L^n$  vs.  $(S_e/S_o) \times 100$  from inspection of Tables 6.3 and 6.6. For Example 6.4, this is shown in Table 6.7. Values of  $D/L^n$  are read from Table 6.6 from left to right and from top to bottom, and corresponding values of  $(S_e/S_o) \times 100$  are read from Table 6.3 in the same way. For Example 6.4, a summary of calculated values is shown as a convenient array in Table 6.8.

Step 4. On semilog paper plot the values of  $(S_e/S_o) \times 100$  vs.  $D/L^n$  (Table 6.7). From Eq. (6.60) obtain

$$\ln(S_e/S_o) = -K(D/L^n) \quad (6.63)$$

The slope of the straight line thus obtained yields the value of  $K$ . For Example 6.4, this plot is shown in Fig. 6.26. Since in this example there is a  $4 \times 4$  matrix, 16 points are utilized in the construction of this line. Therefore, the values of constants  $n$  and  $K$  for the basic mathematical model have now been obtained.

**TABLE 6.7**  
 $D/L^n = D/L^{0.478}$  vs.  $(S_e/S_o) \times 100$

$D/L^n = D/L^{0.478}$	$(S_e/S_o) \times 100$
5	57.5
3.59	67.5
2.96	73.0
2.57	76.0
10.0	33.5
7.17	46.0
5.91	53.0
5.15	57.0
15.0	19.5
10.75	31.0
8.87	38.7
7.71	43.0
20.0	11.3
14.35	21.0
11.83	28.2
10.30	32.8

**TABLE 6.8**  
 Summary of Calculated Values for Example 6.4

$D$ , depth (ft)	$L$ [gal/(min)(ft <sup>2</sup> )]	$L^{0.478}$	$D/L^{0.478}$	$(S_e/S_o) \times 100$
5	1.0	1.0	5.0	57.5
10	1.0	1.0	10.0	33.5
15	1.0	1.0	15.0	19.5
20	1.0	1.0	20.0	11.3
5	2.0	1.393	3.59	67.5
10	2.0	1.393	7.17	46.0
15	2.0	1.393	10.75	31.0
20	2.0	1.393	14.35	21.0
5	3.0	1.690	2.96	73.0
10	3.0	1.690	5.91	53.0
15	3.0	1.690	8.87	38.7
20	3.0	1.690	11.83	28.2
5	4.0	1.941	2.57	76.0
10	4.0	1.941	5.15	57.0
15	4.0	1.941	7.71	43.0
20	4.0	1.941	10.30	32.8

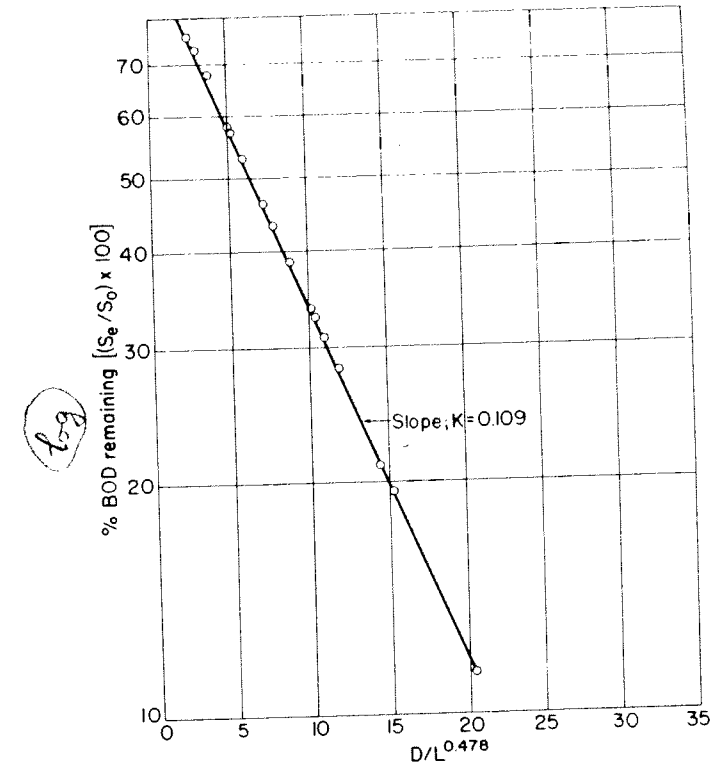


Fig. 6.26. Plot of  $(S_e/S_o) \times 100$  vs.  $D/L^{0.478}$ .

**7.9.3. Application of Results to the Design of a Plant Scale Trickling Filter**

*Case 1.* Trickling filters without recycle [Fig. 6.22(a)]  
 Design is based on Eq. (6.54), for which values of  $K$  and  $n$  are determined as described in Section 7.9.2.

*Step 1.* Calculate required hydraulic loading  $L$  for specified BOD removal corresponding to required  $S_e/S_o$ . Assume a value for depth  $D$  and solve for hydraulic loading  $L$  [gal/(min)(ft<sup>2</sup>)]. Solving Eq. (6.54) for  $L$ ,

$$L = [-KD/\ln(S_e/S_o)]^{1/n} \tag{6.64}$$

Equation (6.64) yields the required hydraulic loading  $L$  in gal/(min)(ft<sup>2</sup>).

*Step 2.* The required filter area  $A$  is

$$A = Q_r/L = \frac{\text{gal/day}}{\text{gal/(day)(ft}^2\text{)}} = \text{ft}^2 \tag{6.65}$$

Step 3. Filter diameter is given in Eq. (6.66).

$$d = (4A/\pi)^{1/2} = (A/0.785)^{1/2} \text{ ft} \quad (6.66)$$

Case 2. Trickling filters with recycle [Fig. 6.22(b)]

Step 1. Calculate  $S_e$  from Eq. (6.56) for a specified value of the recycle ratio  $r$ .

Step 2. Solving Eq. (6.55) for  $L$ , one obtains an equation similar to Eq. (6.64), from which the hydraulic loading is calculated. Steps 1 and 2 may be combined by solving Eq. (6.58) directly for  $L$ . Final result is

$$L = \left[ \frac{-KD}{\ln \frac{(S_e/S_F)(1+r)}{1+r(S_e/S_F)}} \right]^{1/n} \quad (6.67)$$

Step 3. The required filter area  $A$  is shown in Eq. (6.68).

$$A = Q/L = Q_F(r+1)/L \quad (\text{ft}^2) \quad (6.68)$$

Step 4. Filter diameter is calculated from Eq. (6.66).

#### Example 6.4

Data presented in Table 6.3 for % BOD remaining vs. depth are obtained from a pilot-plant trickling filter treating an industrial wastewater.

1. Determine values of parameters  $n$  and  $K$  and write the correlation of BOD remaining to depth and hydraulic loading.
2. Calculate trickling filter diameter necessary to obtain BOD reduction of 80% if wastewater flow is 2.0 MGD. A filter depth of 20 ft is provided with recycle ratio of 0.3. Base calculations on an influent BOD of 300 mg/liter.
3. If no recycle is used for the filter designed in Part 2, calculate the maximum wastewater flow permissible in Mgal/day to reach the desired BOD reduction of 80%.

SOLUTION: Part 1 Follow procedure described in Section 7.9.2.

Step 1. Plot of  $(S_e/S_o) \times 100$  vs.  $D$  for a set of four values of  $L$ . This graph is shown in Fig. 6.24.

Step 2. Table 6.4 presents the absolute values of the slopes read from Fig. 6.24 vs.  $L$ . Fig. 6.25 is a plot of these values. From Fig. 6.25 obtain  $n = 0.478$ .

Step 3.

1. Values of  $L^n = L^{0.478}$  for each experimental run are presented in Table 6.5.
2. Matrix for values of  $D/L^n = D/L^{0.478}$  for each experimental run at each depth is shown in Table 6.6.
3. Values of  $D/L^n = D/L^{0.478}$  vs.  $(S_e/S_o) \times 100$  are presented in Table 6.7.

#### 7. Trickling Filters

A summary of calculated values is given in Table 6.8.

Step 4. Plot  $\log[(S_e/S_o) \times 100]$  vs.  $D/L^{0.478}$  (Fig. 6.26). From the slope of the straight line in Fig. 6.26 one obtains  $K = 0.109$ . From Eq. (6.54) for  $n = 0.478$  and  $K = 0.109$ ,

$$S_e/S_o = e^{-0.109D/L^{0.478}}$$

which is the correlation of BOD remaining to depth and hydraulic loading.

SOLUTION: Part 2 Trickling filter diameter

Follow procedure described in Section 7.9.3 (Case 2).

Step 1. Calculate  $S_e$  [Eq. (6.56)].

Here,

$$S_e = (0.2)(300) = 60 \text{ mg/liter}$$

$$\therefore S_o = [300 + (0.3)(60)]/(1 + 0.3) = 244.6 \text{ mg/liter}$$

Step 2. Calculate  $L$  [Eq. (6.64)].

Here,

$$S_e/S_o = 60/244.6 = 0.245$$

$$\ln 0.245 = -1.405$$

and

$$n = 0.478$$

Therefore,

$$L = (-0.109 \times 20 / -1.405)^{1/0.478} = 2.506 \text{ gal}/(\text{min})(\text{ft}^2)$$

or

$$2.506 \text{ gal}/(\text{min})(\text{ft}^2) \times 60 \text{ min/hr} \times 24 \text{ hr/day} = 3609 \text{ gal}/(\text{day})(\text{ft}^2)$$

Alternative calculation procedure [Eq. (6.67)]

Here,  $S_e/S_F = 60/300 = 0.2$ ,  $K = 0.109$ ,  $n = 0.478$ ,  $D = 20$  ft, and  $r = 0.3$ .

Therefore,

$$\therefore L = \left[ \frac{0.109 \times 20}{-\ln [(0.2)(1 + 0.3)]/[1 + (0.3 \times 0.2)]} \right]^{1/0.478} = 2.506 \text{ gal}/(\text{min})(\text{ft}^2)$$

Step 3. Determine the filter area [Eq. (6.68)].

$$A = 2,000,000(0.3 + 1)/3609 = 720 \text{ ft}^2$$

Step 4. Determine the filter diameter [Eq. (6.66)].

$$d = (720/0.785)^{1/2} = 30.3 \text{ ft}$$

SOLUTION: Part 3 No recycle

$$A = 720 \text{ ft}^2$$

$$S_e/S_o = 0.2$$

Step 1. Calculate the allowable hydraulic loading [Eq. (6.64)].

Here,

$$S_o = S_F = 300 \text{ mg/liter}$$

$$\therefore L = (-0.109 \times 20 / \ln 0.2)^{1/0.478} = 1.89 \text{ gal}/(\text{min})(\text{ft}^2)$$

or

$$1.89 \text{ gal}/(\text{min})(\text{ft}^2) \times 60 \text{ min/hr} \times 24 \text{ hr/day} = 2722 \text{ gal}/(\text{day})(\text{ft}^2)$$

Step 2. Determine the maximum wastewater flow [Eq. (6.65)].

$$Q_F = AL = 720 \text{ ft}^2 \times 2722 \text{ gal}/(\text{min})(\text{ft}^2) = 1,960,000 \text{ gal/day}$$

or

$$Q_F = 1.960 \text{ MGD}$$

(was 2.0 MGD with  $r = 0.3$ ).

### 7.10. DESIGN PROCEDURE WHEN EXPERIMENTAL DATA ARE NOT AVAILABLE

Experimental data necessary to determine parameters  $n$  and  $K$  by the procedure described in Section 7.9.2, i.e., a network of data as shown in Table 6.3, are often unavailable. It may not be feasible to conduct the tedious experimental work required to obtain this network of data. If this is the case parameter  $n$  is estimated from available data for different types of filter media ( $n$  depends *only* on the characteristics of packing). Parameter  $K$ , which is only a function of the nature of the wastewater, is also estimated from available data for wastewaters from several sources. Eckenfelder [4] presents a tabulation of  $K$  values for Surfpac filter media ( $n = 0.5$ ) for several industrial wastewaters.

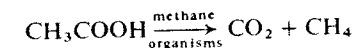
## 8. Anaerobic Treatment

### 8.1. INTRODUCTION

Anaerobic treatment is utilized for treatment of wastewaters as well as for digestion of sludges. Anaerobic treatment of wastewaters is described in this section. Anaerobic digestion of sludges is studied in Chapter 7 (Sections 2.6 and 2.7). The end products of anaerobic degradation are gases, mostly methane ( $\text{CH}_4$ ), carbon dioxide ( $\text{CO}_2$ ), and small quantities of hydrogen sulfide ( $\text{H}_2\text{S}$ ) and hydrogen ( $\text{H}_2$ ). The process comprises two stages: (1) acid fermentation and (2) methane fermentation.

In the acid fermentation stage, organic materials are broken down to organic acids, mainly acetic ( $\text{CH}_3\text{COOH}$ ), propionic ( $\text{CH}_3\text{CH}_2\text{COOH}$ ), and butyric ( $\text{CH}_3\text{CH}_2\text{CH}_2\text{COOH}$ ). In the methane fermentation stage, "methane

microorganisms" convert the longer chain acids to methane, carbon dioxide, and an acid having a shorter carbon chain. The acid molecule is repeatedly broken down in the same manner. The resulting acetic acid is directly converted to  $\text{CO}_2$  and  $\text{CH}_4$ .



The methane fermentation stage controls the rate of anaerobic degradation. In Section 8.3.1 it is shown that a plot of  $(S_o - S_e)/X_e t$  vs.  $S_e$  (Fig. 6.28) yields rate constant  $k$ . This is similar to the situation found in aerobic treatment [Eq. (5.18) and Fig. 5.5]. However, one straight line is obtained in aerobic treatment, whereas two result from anaerobic data. The rate constants can be evaluated from the slopes of these straight lines. From Fig. 6.28, the slope corresponding to the acid fermentation stage is much greater than that for the methane fermentation stage. If the two lines are plotted on the same scale as in Fig. 6.28, the one corresponding to the acid fermentation is nearly vertical by comparison with the other. This indicates that the methane fermentation stage controls the process rate. Therefore, for design purposes one should adopt the  $k$  value and other parameters evaluated from the methane fermentation stage. Since methane fermentation controls the process rate, it is important to maintain conditions of effective methane fermentation. Detention time for methane microorganisms must be adequate, or they are washed away from the system. Experimental data show that the required detention time varies from 2 to 20 days. Optimum pH range is 6.8–7.4.

Anaerobic treatment is relatively inexpensive because aeration equipment is not utilized. On the other hand, residence times required are much longer than for aerobic processes. Bad odors associated with anaerobic processes, due mainly to production of  $\text{H}_2\text{S}$  and mercaptans, may constitute a serious limitation, particularly in urban areas.

### 8.2. A QUANTITATIVE STUDY OF ANAEROBIC DEGRADATION OF AN ORGANIC WASTE

A quantitative study of anaerobic degradation of organic wastewaters has been made by Andrews [1], and results are summarized in Fig. 6.27.

The following observations may be made:

1. *pH*. At the beginning of anaerobic degradation (acid fermentation), the pH drops due to formation of organic acids. Since at a later stage (methane fermentation) these acids are broken down, pH increases. The rise starts after about 2 days, as depicted in Fig. 6.27.

2. *COD remaining*. This curve is relatively flat during the first 2 days of the acid fermentation stage, since at this time organic compounds are merely converted to soluble form; so there is no COD reduction. COD drops markedly after this initial stage.



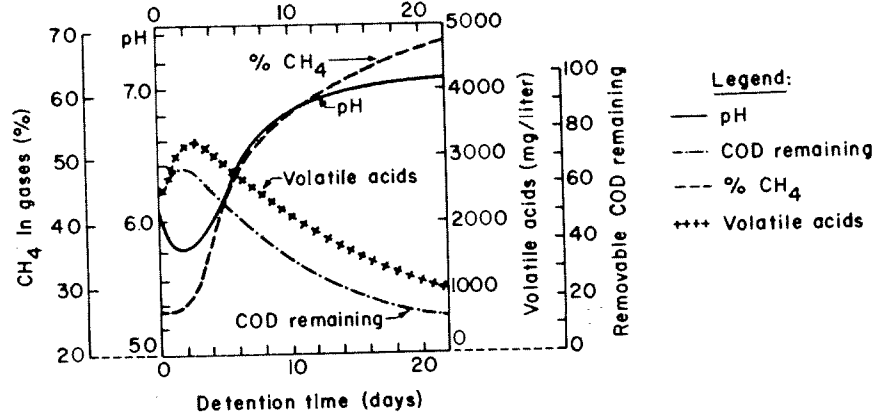


Fig. 6.27. Anaerobic degradation of an organic waste [1]. (Data obtained with a batch reactor.)

3. Methane percentage and volatile acids. There is a rapid increase in methane production with a corresponding decrease in volatile acids and an increase in pH after the first 2 days.

For long detention times, nearly all volatile acids are converted to CH<sub>4</sub> and CO<sub>2</sub>. Since not all volatile solids are biodegradable, the curve for COD remaining approaches a limiting ordinate for large values of residence time.

### 8.3. MATHEMATICAL FORMULATION FOR ANAEROBIC DIGESTION PROCESS

The mathematical formulation described is the one proposed by Eckenfelder and associates [5].

#### 8.3.1. BOD Removal Rate for Anaerobic Process

If BOD removal is assumed to follow first-order kinetics, one may utilize the formulation presented in Chapter 5, Section 3.2 for the aerobic process. Equation (6.69), which is similar to Eq. (5.18), is obtained.\*

$$(S_o - S_e)/X_v t = k S_e \tag{6.69}$$

Owing to the presence of nonremovable BOD and following considerations similar to those in Chapter 5, Section 3.2, one may write a modified form of Eq. (6.69).

$$(S_o - S_e)/X_v t = k(S_e - S_n) \tag{6.70}$$

\* Notation  $X_v$  is utilized in Eq. (6.69) instead of  $X_{v,a}$  used in Eq. (5.18). Since subscript  $a$  refers to the aerator, it is omitted in the formulation of the anaerobic process.

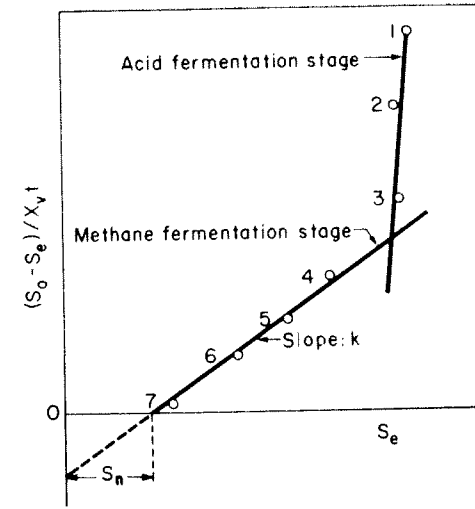


Fig. 6.28. Determination of BOD removal rate for anaerobic degradation.

Equation (6.70) is similar to Eq. (5.19). From Eq. (6.70) a plot of  $(S_o - S_e)/X_v t$  vs.  $S_e$  yields two straight lines, as shown in Fig. 6.28. For design purposes, the value of  $k$  obtained from the slope of the line corresponding to the methane fermentation stage is the one adopted. In Fig. 6.28 as in Fig. 5.14, the abscissa at the origin of the ordinate axis corresponds to the nonbiodegradable substrate ( $S_n$ ).

Figure 6.28 can be replotted by choosing as abscissa only the biodegradable portion of the effluent,  $S_e'$ .

$$S_e' = S_e - S_n \tag{6.71}$$

This corresponds to a translation of the axes indicated in Fig. 6.29, so that the straight line corresponding to methane fermentation passes through the origin.

The mathematical model corresponding to Fig. 6.29 is

$$(S_o - S_e)/X_v t = S_e'/X_v t = k S_e' \tag{6.72}$$

If there is no nonbiodegradable matter,  $S_n = 0$ , and  $S_e' = S_e$ , then Eq. (6.72) reduces to Eq. (6.69).

#### 8.3.2. Volatile Solids in the Anaerobic Reactor

Consider the anaerobic reactor depicted by Fig. 6.30. Let  $X_{v,o}$  and  $X_v$  be the VSS concentrations in the influent and in the reactor, respectively. Assuming steady state operation and perfect mixing conditions, VSS concentration in the effluent is also equal to  $X_v$ . A material balance for VSS leads

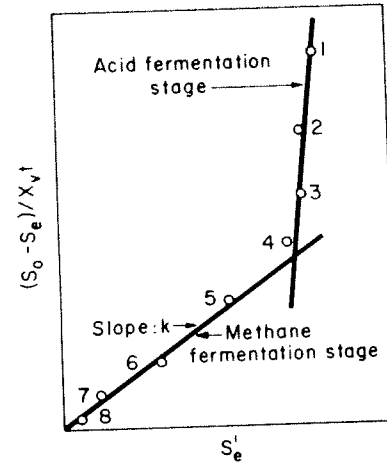


Fig. 6.29. Modified plot for determination of BOD removal rate for anaerobic degradation.

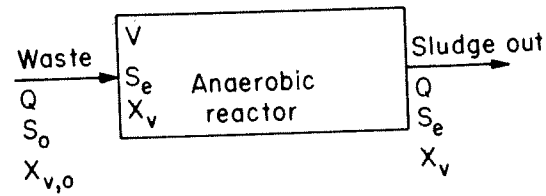


Fig. 6.30. Material balance for volatile solids in anaerobic reactor.

to an equation similar to Eq. (6.28) in Section 5.7, except for omission of subscript *a* in term  $X_v$ .

$$X_v = (X_{v,o} + aS_r)/(1 + bt) \tag{6.73}$$

Equation (6.73) is the expression for the concentration of VSS in the anaerobic reactor. If the concentration of VSS in the influent is negligible (i.e.,  $X_{v,o} \approx 0$ ), this equation is simplified to yield

$$X_v = aS_r/(1 + bt) \quad (X_{v,o} \approx 0) \tag{6.74}$$

Parameters *a* and *b* in Eq. (6.73) are determined by writing it in linear form. By cross multiplying, dividing through by  $aX_v$ , and rearranging, Eq. (6.74) yields

$$(1/a)(1 - X_{v,o}/X_v) + (b/a)t = S_r/X_v \tag{6.75}$$

Equation (6.75) is the basis for determination of parameters *a* and *b*. A plot of  $S_r/X_v = (S_o - S_e)/X_v$  vs. *t* yields a set of two straight lines: the first one, for lower *t*'s, corresponding to the acid fermentation stage, and the second, for

higher *t*'s, corresponding to the methane fermentation stage. A typical plot is shown in Fig. 6.31. The determination of parameters *a* and *b* is indicated in this graph.

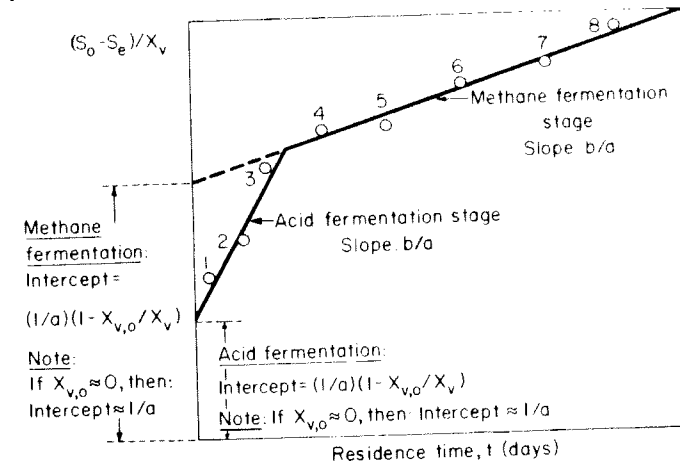


Fig. 6.31. Determination of parameters *a* and *b*.

When the concentration of VSS in the influent is negligible (i.e.,  $X_{v,o} \approx 0$ ) as shown in Fig. 6.35, Section 8.5, Eq. (6.75) yields Eq. (6.76).

$$1/a + (b/a)t = S_r/X_v \quad (X_{v,o} \approx 0) \tag{6.76}$$

For calculation of detention time, the values of parameters *a* and *b* corresponding to the methane fermentation are the ones to be adopted since this is the controlling stage.

### 8.3.3. Calculation of Detention Time for Anaerobic Treatment

Consider Eq. (6.72) for BOD removal and Eq. (6.73) for VSS concentration. From Eq. (6.72),

$$t = S_r/kX_vS_e' \tag{6.77}$$

From Eq. (6.73),

$$S_r = [X_v(1 + bt) - X_{v,o}]/a \tag{6.78}$$

Substituting *S<sub>r</sub>* in Eq. (6.77) by its value from Eq. (6.78) and solving for *t*,

$$t = (X_v - X_{v,o})/(akX_vS_e' - bX_v) \tag{6.79}$$

In Eq. (6.79),  $S_e'$  is given by Eq. (6.71). When the concentration of VSS in the influent is negligible (i.e.,  $X_{v,o} \approx 0$ ), Eq. (6.79) is simplified to yield

$$t = 1/(akS_e' - b) \quad (X_{v,o} \approx 0) \tag{6.80}$$

### 8.3.4. Gas Production in Anaerobic Treatment

Gas produced by anaerobic degradation is composed of  $\text{CH}_4$ ,  $\text{CO}_2$ , and small quantities of  $\text{H}_2\text{S}$  and  $\text{H}_2$ . Lawrence and McCarthy [7] have shown that methane gas production can be estimated as  $5.62 \text{ ft}^3$  (at STP)\* per lb of COD removed, *excluding* the COD removed which is converted to cells. The proposed relationship to estimate gas production is then

$$G = 5.62(QS_r - 1.42QX_v) = 5.62Q(S_r - 1.42X_v) \quad (6.81)$$

where  $G$  is the  $\text{ft}^3$  of gas (at STP) produced per day;  $QS_r$ , the lb COD removed/day; and  $1.42QX_v$ , the lb VSS in reactor effluent/day. [Ratio 1.42 corresponds to lb COD/lb VSS; see Chapter 5, Eq. (5.60).]

Attention should be given to the units in utilizing Eq. (6.81). If the following units are employed, where  $G$  is the  $\text{ft}^3$  gas/day (at STP);  $Q$  the MGD;  $S_r$ , the mg COD/liter ( $\approx$  lb COD/Mlb liquor); and  $X_v$ , the mg VSS/liter ( $\approx$  lb VSS/Mlb liquor), then

$$G = 5.62 \frac{\text{ft}^3}{\text{lb COD}} \left( Q \frac{\text{Mgal liquor}}{\text{day}} \times S_r \frac{\text{lb COD}}{\text{Mlb liquor}} \times 8.24 \frac{\text{lb liquor}}{\text{gal liquor}} - 1.42 \frac{\text{lb COD}}{\text{lb VSS}} \times Q \frac{\text{Mgal liquor}}{\text{day}} \times X_v \frac{\text{lb VSS}}{\text{Mlb liquor}} \times 8.24 \frac{\text{lb liquor}}{\text{gal liquor}} \right)$$

or finally [Eq. (6.82)],

$$G = 46.87Q(S_r - 1.42X_v) \text{ ft}^3/\text{day (STP)} \quad (6.82)$$

## 8.4. LABORATORY ANAEROBIC REACTORS FOR OBTAINING BASIC DESIGN INFORMATION

Two models of a laboratory anaerobic reactor are shown in Figs. 6.32 and 6.33. The reactor in Fig. 6.32 is a batch type and the one in Fig. 6.33 is continuous.

The operational procedure followed to obtain basic design data is

1. To provide a seed of microorganisms, obtain a sample of settled sludge from a municipal wastewater treatment plant and place it in the laboratory reactor. Add the wastewater to be treated and maintain a temperature of  $35^\circ\text{C}$  throughout the test period.
2. If significant amounts of air are trapped in the reactor, purge the system with an inert gas.
3. The mixture is mixed continuously, either mechanically (Fig. 6.32) or by recirculation of the gas produced, by means of a gas pump (Fig. 6.33). Do not start feeding the wastewater or withdrawing the mixed liquor until gas production is ascertained.

\* STP stands for standard temperature and pressure, taken as  $P = 1 \text{ atm}$  and  $t = 60^\circ\text{F}$ .

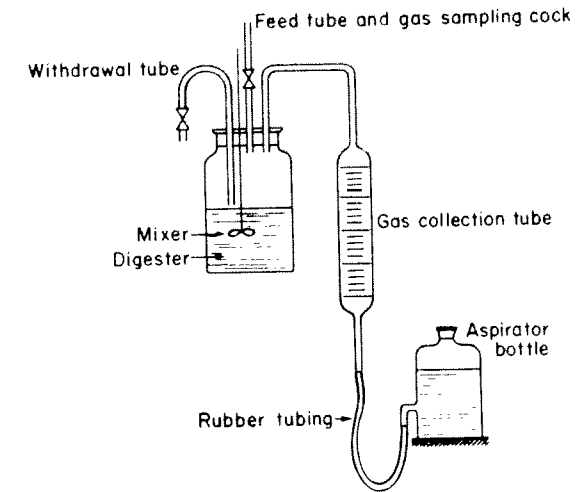


Fig. 6.32. Bench scale batch feed digester.

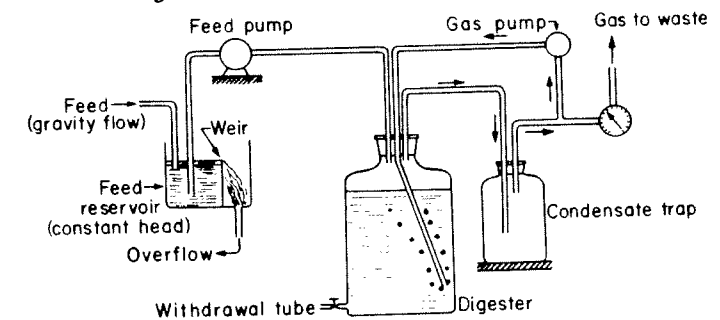


Fig. 6.33. Continuous feed digester.

4. Once gas production is noticed, feed portions of wastewater, withdrawing equal portions of mixed liquor.

5. During the entire start-up period pH is observed closely, and if it falls below 6.6, alkalinity is added to maintain the recommended range, i.e., pH 6.6–7.6.

6. Continue to feed and withdraw daily until effluent BOD (or COD) stabilizes. Analyze and record the data: raw wastewater: pH and COD (or  $\text{BOD}_5$ ); effluent: pH, COD (or  $\text{BOD}_5$ ), and VSS.

## 8.5. DESIGN PROCEDURE FOR ANAEROBIC DIGESTERS

Treatment of the data obtained in the laboratory to arrive at design parameters and utilization of this information in the design of an anaerobic digester are illustrated by Example 6.5.

**Example 6.5**

200,000 gal/day of a wastewater are to be treated by anaerobic degradation. The mean value for influent COD is 10,000 mg/liter. Effluent COD should be reduced to at the most 2000 mg/liter. Concentration of VSS in the influent is negligible.

A bench scale reactor is utilized to simulate the process, and MLVSS concentration is maintained at approximately 500 mg/liter in all experimental runs. Data obtained are presented in Table 6.9.

**TABLE 6.9**  
Laboratory Data for Example 6.5

Run no.	Residence time (days)	Influent COD (mg/liter)	Effluent COD (mg/liter)
1	5	8,550	3,800
2	10	8,400	2,400
3	15	9,190	1,940
4	20	10,200	1,700
5	30	12,470	1,470

1. Obtain design parameters  $k$ ,  $a$ , and  $b$ .
2. Calculate digester volume required (gal).
3. Calculate MLVSS concentration for the anaerobic digester (mg/liter).
4. Estimate gas production ( $\text{ft}^3/\text{day}$  at STP).

**SOLUTION: Part 1**

*Step 1.* Obtain values of  $(S_o - S_e)/X_v$  and  $(S_o - S_e)/X_v t$  necessary to plot Figs. 6.34 and 6.35. This is shown in Table 6.10.

*Step 2.* Prepare a plot of  $(S_o - S_e)/X_v t$  vs.  $S_e$  and determine  $k$  from the slope of the line for the methane fermentation stage. This is shown in Fig. 6.34. Since all residence times considered are longer than 5 days, only the straight line corresponding to the methane fermentation stage is shown in Fig. 6.34. From the slope of this line one obtains  $k = 0.0005 \text{ day}^{-1}$  and  $S_n \approx 0$  (straight line passes through origin).

*Step 3.* Determine parameters  $a$  and  $b$ . Plot  $(S_o - S_e)/X_v$  vs.  $t$ : This plot is shown in Fig. 6.35. As indicated in Step 2, only the straight line corresponding to the methane fermentation stage is obtained.

$$\text{Intercept} = 1/a = 7.0$$

$$\therefore a = 1/7 = 0.143$$

$$\text{Slope} = b/a = 0.5$$

$$\therefore b = a \cdot 0.5 = 0.143 \times 0.5 = 0.0715$$

**TABLE 6.10**  
Calculations for Example 6.5

Run no.	Residence time (days)	$S_o$ (mg/liter) (COD)	$S_e$ (mg/liter) (COD)	$S_o - S_e$ (mg/liter) (COD)	$(S_o - S_e)/X_v$ (lb COD/lb MLVSS)	$(S_o - S_e)/X_v t$ [(lb COD)/(day)(lb MLVSS)]
1	5	8,550	3,800	4,750	9.50	1.9
2	10	8,400	2,400	6,000	12.0	1.2
3	15	9,190	1,940	7,250	14.5	0.966
4	20	10,200	1,700	8,500	17.0	0.850
5	30	12,470	1,470	11,000	22.0	0.733

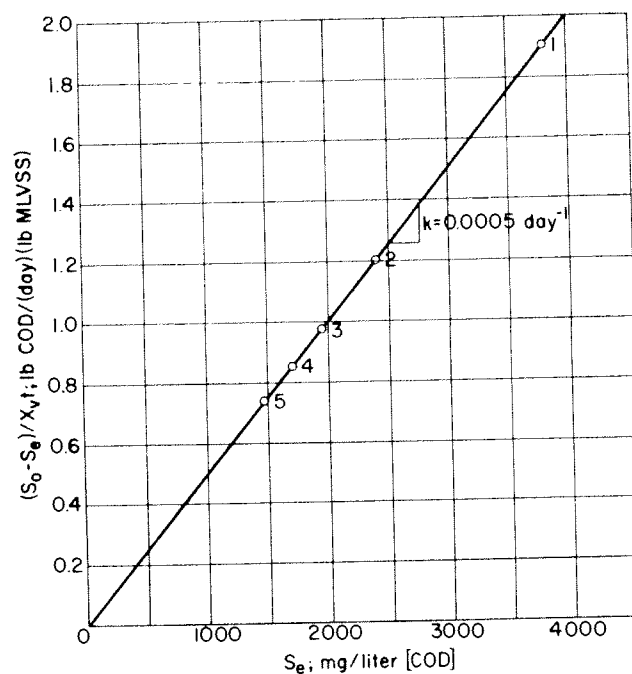


Fig. 6.34. Plot of  $(S_0 - S_e)/X_v t$  vs.  $S_e$  for Example 6.5.

SOLUTION: Part 2 Digester volume (gal)

Step 1. Calculate the residence time from Eq. (6.80). Since  $S_n \approx 0$ , then  $S_e' = S_e - S_n \approx S_e$ .

$$t = 1/(0.143 \times 0.0005 \times 2000 - 0.0715) = 14 \text{ days}$$

Step 2. Digester volume is then calculated.

$$V = Qt = 200,000 \text{ gal/day} \times 14 \text{ days} = 2,800,000 \text{ gal}$$

SOLUTION: Part 3 MLVSS concentration in anaerobic digester [Eq. (6.74)]

$$X_v = 0.143(10,000 - 2000)/(1 + 0.0715 \times 14) = 572 \text{ mg/liter}$$

SOLUTION: Part 4 Gas production from Eq. (6.82)

Since

$$Q = 200,000 \text{ gal/day} = 0.2 \text{ Mgal/day}$$

then

$$G = 46.87 \times 0.2[(10,000 - 2000) - 1.42 \times 572] = 67,380 \text{ ft}^3/\text{day at STP}$$

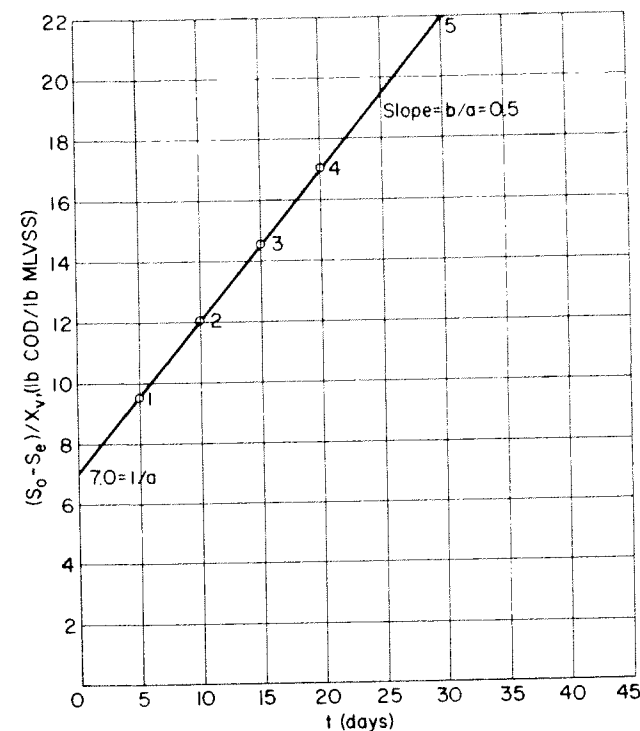


Fig. 6.35. Plot of  $(S_0 - S_e)/X_v$  vs.  $t$  for Example 6.5.

### Problems

I. An aerated lagoon is being considered for treatment of an industrial wastewater flow of 1.0 MGD.

The following data are available:

Influent soluble  $BOD_5 = 200$  mg/liter

Influent suspended solids concentration is negligible

Effluent soluble  $BOD_5 \leq 20$  mg/liter

Wastewater temperature =  $60^\circ\text{F}$

Since in most situations summer conditions control required aerator horsepower, base design upon an air temperature of  $85^\circ\text{F}$

Take  $h = 100 \text{ BTU}/(\text{day})(\text{ft}^2)(^\circ\text{F})$

Data obtained from laboratory simulation  $k = 0.025 \text{ day}^{-1}$  (at  $20^\circ\text{C}$ ),  $a = 0.65 \text{ lb VSS}/\text{lb BOD}_5 \text{ day}$ ,  $b = 0.07 \text{ day}^{-1}$ , and  $a' = 1.2 \text{ lb O}_2/\text{lb BOD}_5$

Lagoon depth = 10 ft

Oxygen concentration to be maintained in liquid is 1.5 mg/liter

Determine

1. Lagoon surface in acres

2. Total effluent BOD (soluble + insoluble)

3. Oxygen requirements [ $\text{lb O}_2/(\text{HP})(\text{hr})$ ] and total required HP and power level (HP/Mgal). Take  $C_{sw} = 8 \text{ mg/liter}$  and  $\alpha = 0.85$ . Assume aerators are conservatively rated at  $N_0 = 3.0 \text{ lb O}_2/(\text{HP})(\text{hr})$ .

II. It is proposed to treat an industrial wastewater (flow, 1.0 MGD) utilizing stabilization ponds. Plan the operation in three stages. Influent  $BOD_5$  (2000 mg/liter) should be reduced to a value not higher than 40 mg/liter (effluent from third stage).

*Stage 1—Anaerobic pond.* It should be designed for the purpose of reducing influent  $BOD_5$  (2000 mg/liter) to a value of 500 mg/liter in the effluent. Take  $K = 0.1 \text{ day}^{-1}$ . Plan on a 10-ft depth. Calculate the area of the pond in acres. Calculate surface loading in lb  $BOD_5$ /(acre)(day).

*Stages 2 and 3—Facultative ponds.* Design stages 2 and 3 for same residence time in each. Take  $K = 0.2 \text{ day}^{-1}$  for both stages. For stage 2, the design is to be based on a surface loading of 500 lb  $BOD_5$ /(acre)(day). Calculate the area in acres and the depth in ft. Estimate depth of oxygen penetration in ft. For stage 3, the design is to be based on a surface loading of 250 lb  $BOD_5$ /(acre)(day). Calculate the area in acres and the depth in ft. Estimate depth of oxygen penetration in ft.

III. The data tabulated below for %  $BOD_5$  remaining (i.e.,  $S_e/S_0 \times 100$ ) were obtained in a pilot-plant trickling filter treating an industrial wastewater.

D, depth (ft)	Hydraulic loading [gal/(min)(ft <sup>2</sup> )]			
	1	2	3	4
6	62	70	75	80
12	36	46	57	63
18	24	32	43	48.5
24	14	22	32	39

1. Correlate the data and develop a relationship between BOD removal, depth, and hydraulic loading.
2. Design a filter to obtain 80% BOD reduction from 5 MGD of settled wastewater with an initial  $BOD_5$  of 250 mg/liter, using a depth of 30 ft (a) without recycle; (b) with recycle ratio of 1.5.

IV. A wastewater for which the mean influent COD may be taken as 10,000 mg/liter is to be treated by anaerobic degradation. Laboratory tests indicated that the nondegradable COD is approximately 2000 mg/liter. Average MLVSS influent concentration is 15 mg/liter. Bench scale determinations yield the following values for design parameters:  $k = 0.0004 \text{ day}^{-1}$ ,  $a = 0.14 \text{ lb MLVSS yield/lb COD removed}$ , and  $b = 0.02 \text{ lb MLVSS oxidized/(day)(lb MLVSS in reactor)}$ . Calculate % COD removal for residence times of 80, 60, 40, and 20 days, and plot % removal vs. residence time. On the same graph plot the corresponding values of % COD removal, neglecting the MLVSS in the influent.

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