This chapter describes the basic responses and interactions among the waste constituents and process components of natural treatment systems. Many of these responses are common to more than one of the treatment concepts and are therefore discussed in this chapter. If a waste constituent is the limiting factor for design, it is also discussed in detail in the appropriate process design chapter. Water is the major constituent of all of the wastes of concern in this book, as even a “dried” sludge can contain more than 50% water. The presence of water is a volumetric concern for all treatment methods, but it has even greater significance for many of the natural treatment concepts because the flow path and the flow rate control the successful performance of the system. Other waste constituents of major concern include the simple carbonaceous organics (dissolved and suspended), toxic and hazardous organics, pathogens, trace metals, nutrients (nitrogen, phosphorus, potassium), and other micronutrients. The natural system components that provide the critical reactions and responses include bacteria, protozoa (e.g., algae), vegetation (aquatic and terrestrial), and the soil. The responses involved include a range of physical, chemical, and biological reactions.

3.1 WATER MANAGEMENT

Major concerns of water management include the potential for travel of contaminants with groundwater, the risk of leakage from ponds and other aquatic systems, the potential for groundwater mounding beneath a land treatment system, the need for drainage, and the maintenance of design flow conditions in ponds, wetlands, and other aquatic systems.

3.1.1 FUNDAMENTAL RELATIONSHIPS

Chapter 2 introduced some of the hydraulic parameters (e.g., permeability) that are important to natural systems and discussed methods for their determination in the field or laboratory. It is necessary to provide further details and definition before undertaking any flow analysis.
3.1.1.1 Permeability

The results from the field and laboratory test program described in the previous chapter may vary with respect to both depth and areal extent, even if the same basic soil type is known to exist over much of the site. The soil layer with the most restrictive permeability is taken as the design basis for those systems that depend on infiltration and percolation of water as a process requirement. In other cases, where there is considerable scatter to the data, it is necessary to determine a “mean” permeability for design.

If the soil is uniform, then the vertical permeability \( K_v \) should be constant with depth and area, and any differences in test results should be due to variations in the test procedure. In this case, \( K_v \) can be considered to be the arithmetic mean as defined by Equation 3.1:

\[
K_{am} = \frac{K_1 + K_2 + K_3 + K_n}{n}
\]  

(3.1)

where \( K_{am} \) is the arithmetic mean vertical permeability, and \( K_1 \) through \( K_n \) are individual test results.

Where the soil profile consists of a layered series of uniform soils, each with a distinct \( K_v \) generally decreasing with depth, the average value can be represented as the harmonic mean:

\[
K_{hm} = \frac{D}{\left( \frac{d_1}{K_1} \right) + \left( \frac{d_2}{K_2} \right) + \left( \frac{d_n}{K_n} \right)}
\]  

(3.2)

where

- \( K_{hm} \) = Harmonic mean permeability.
- \( D \) = Soil profile depth.
- \( d_n \) = Depth of \( n \)th layer.

If no pattern or preference is indicated by a statistical analysis, then a random distribution of the \( K \) values for a layer must be assumed, and the geometric mean provides the most conservative estimate of the true \( K_v \):

\[
K_{gm} = \left[ (K_1)(K_2)(K_3)(K_n) \right]^{1/n}
\]  

(3.3)

where \( K_{gm} \) is the geometric mean permeability (other terms are as defined previously).

Equation 3.1 or 3.3 can also be used with appropriate data to determine the lateral permeability, \( K_h \). Table 2.17 presents typical values for the ratio \( K_h/K_v \).
3.1.1.2 Groundwater Flow Velocity

The actual flow velocity in a groundwater system can be obtained by combining Darcy’s law, the basic velocity equation from hydraulics, and the soil porosity, because flow can occur only in the pore spaces in the soil.

\[
V = \frac{(K_v)(\Delta H)}{(n)(\Delta L)}
\]  

(3.4)

where

- \( V \) = Groundwater flow velocity (ft/d; m/d).
- \( K_v \) = Horizontal saturated permeability, mid (ft/d; m/d).
- \( \Delta H/\Delta L \) = Hydraulic gradient (ft/ft; m/m).
- \( n \) = Porosity (as a decimal fraction; see Figure 2.4 for typical values for in situ soils).

Equation 3.4 can also be used to determine vertical flow velocity. In this case, the hydraulic gradient is equal to 1 and \( K_v \) should be used in the equation.

3.1.1.3 Aquifer Transmissivity

The transmissivity of an aquifer is the product of the permeability of the material and the saturated thickness of the aquifer. In effect, it represents the ability of a unit width of the aquifer to transmit water. The volume of water moving through this unit width can be calculated using Equation 3.5:

\[
q = (K_v)(b)(w)\left(\frac{\Delta H}{\Delta L}\right)
\]  

(3.5)

where

- \( q \) = Volume of water moving through aquifer (ft³/d; m³/d).
- \( b \) = Depth of saturated thickness of aquifer (ft; m).
- \( w \) = Width of aquifer, for unit width \( w = 1 \) ft (1 m).
- \( \Delta H/\Delta L \) = Hydraulic gradient (ft/ft; m/m).

In many situations, well pumping tests are used to define aquifer properties. The transmissivity of the aquifer can be estimated using pumping rate and draw-down data from well tests (Bouwer, 1978; USDOI, 1978).

3.1.1.4 Dispersion

The dispersion of contaminants in the groundwater is due to a combination of molecular diffusion and hydrodynamic mixing. The net result is that the concentration of the material is less, but the zone of contact is greater at downgradient...
locations. Dispersion occurs in a longitudinal direction \( (D_x) \) and transverse to the flow path \( (D_y) \). Dye studies in homogeneous and isotropic granular media have indicated that dispersion occurs in the shape of a cone about 6° from the application point (Danel, 1953). Stratification and other areal differences in the field will typically result in much greater lateral and longitudinal dispersion. For example, the divergence of the cone could be 20° or more in fractured rock (Bouwer, 1978). The dispersion coefficient is related to the seepage velocity as described by Equation 3.6:

\[
D = (a)(v) \quad (3.6)
\]

where

- \( D = \) Dispersion coefficient: \( D_x \) longitudinal, \( D_y \) transverse \((\text{ft}^2/\text{d}; \text{m}^2/\text{d})\).
- \( a = \) Dispersivity: \( a_x \) longitudinal, \( a_y \) transverse \((\text{ft}; \text{m})\).
- \( v = \) Seepage velocity of groundwater system \((\text{ft/d}; \text{m/d}) = V/n\), where \( V \) is the Darcy’s velocity from Equation 3.5, and \( n \) is the porosity (see Figure 2.4 for typical values for \textit{in situ} soils).

The dispersivity is difficult to measure in the field or to determine in the laboratory. Dispersivity is usually measured in the field by adding a tracer at the source and then observing the concentration in surrounding monitoring wells. An average value of 10 \( \text{m}^2/\text{d} \) resulted from field experiments at the Fort Devens, Massachusetts, rapid infiltration system (Bedient et al., 1983), but predicted levels of contaminant transport changed very little after increasing the assumed dispersivity by 100% or more. Many of the values reported in the literature are site-specific, “fitted” values and cannot be used reliably for projects elsewhere.

### 3.1.1.5 Retardation

The hydrodynamic dispersion discussed in the previous section affects all the contaminant concentrations equally; however, adsorption, precipitation, and chemical reactions with other groundwater constituents retard the rate of advance of the affected contaminants. This effect is described by the retardation factor \( (R_d) \), which can range from a value of 1 to 50 for organics often encountered at field sites. The lowest values are for conservative substances, such as chlorides, which are not removed in the groundwater system. Chlorides move with the same velocity as the adjacent water in the system, and any change in observed chloride concentration is due to dispersion only, not retardation. Retardation is a function of soil and groundwater characteristics and is not necessarily constant for all locations. The \( R_d \) for some metals might be close to 1 if the aquifer is flowing through clean sandy soils with a low pH but close to 50 for clayey soils. The \( R_d \) for organic compounds depends on sorption of the compounds to soil organic matter plus volatilization and biodegradation. The sorptive reactions depend on the quantity of organic matter in the soil and on the solubility of the organic material in the groundwater. Insoluble compounds such as dichloro-diphenyl-trichloroethane (DDT), benzo[a]pyrenes, and some polychlorinated biphenyls
Basic Process Responses and Interactions

TABLE 3.1
Retardation Factors for Selected Organic Compounds

<table>
<thead>
<tr>
<th>Material</th>
<th>Retardation Factor ($R_d$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloride</td>
<td>1</td>
</tr>
<tr>
<td>Chloroform</td>
<td>3</td>
</tr>
<tr>
<td>Tetrachloroethylene</td>
<td>9</td>
</tr>
<tr>
<td>Toluene</td>
<td>3</td>
</tr>
<tr>
<td>Dichlorobenzene</td>
<td>14</td>
</tr>
<tr>
<td>Styrene</td>
<td>31</td>
</tr>
<tr>
<td>Chlorobenzene</td>
<td>35</td>
</tr>
</tbody>
</table>

(PCBs) are effectively removed by most soils. Highly soluble compounds such as chloroform, benzene, and toluene are removed less efficiently by even highly organic soils. Because volatilization and biodegradation are not necessarily dependent on soil type, the removal of organic compounds via these methods tends to be more uniform from site to site. Table 3.1 presents retardation factors for a number of organic compounds, as estimated from several literature sources (Bedient et al., 1983; Danel, 1953; Roberts et al., 1980).

3.1.2 **Movement of Pollutants**

The movement or migration of pollutants with the groundwater is controlled by the factors discussed in the previous section. This might be a concern for ponds and other aquatic systems as well as when utilizing the slow rate (SR) and rapid infiltration land treatment concepts. Figure 3.1 illustrates the subsurface zone of influence for the SAT basin.

FIGURE 3.1 Subsurface zone of influence for SAT basin.
influence for a rapid infiltration basin system or a treatment pond where significant seepage is allowed. It is frequently necessary to determine the concentration of a pollutant in the groundwater plume at a selected distance downgradient of the source. Alternatively, it may be desired to determine the distance at which a given concentration will occur at a given time or the time at which a given concentration will reach a particular point. Figure 3.2 is a nomograph that can be used to estimate these factors on the centerline of the downgradient plume (USEPA, 1985). The dispersion and retardation factors discussed above are included in the solution. Data required for use of the nomograph include:

- Aquifer thickness, $z$ (m)
- Porosity, $n$ (% as a decimal)
- Seepage velocity, $v$ (m/d)
- Dispersivity factors $a_x$ and $a_y$ (m)
- Retardation factor $R_d$ for the contaminant of concern
- Volumetric water flow rate, $Q$ (m$^3$/d)
- Pollutant concentration at the source, $C_0$ (mg/L)
- Background concentration in groundwater, $C_b$ (mg/L)
- Mass flow rate of contaminant $Q C_0$ (kg/d)
Basic Process Responses and Interactions

Use of the nomograph requires calculation of three scale factors:

\[ X_D = \frac{D_x}{v} \]  \hspace{1cm} (3.7)

\[ t_D = \frac{(R_d)(D_x)}{(v)^2} \]  \hspace{1cm} (3.8)

\[ Q_D = (16.02)(n)(z)[(D_x)(D_y)]^{1/2} \]  \hspace{1cm} (3.9)

The procedure is best illustrated with an example.

Example 3.1

Determine the nitrate concentration in the centerline of the plume, 600 m downgradient of a rapid infiltration system, 2 years after system startup. Data: aquifer thickness = 5 m; porosity = 0.35; seepage velocity = 0.45 m/d; dispersivity, \( a_x = 32 \) m, \( a_y = 6 \) m; volumetric flow rate = 90 \( m^3/d \); nitrate concentration in percolate = 20 mg/L; and nitrate concentration in background groundwater = 4 mg/L.

Solution

1. The downgradient volumetric flow rate combines the natural background flow plus the additional water introduced by the SAT system. To be conservative, assume for this calculation that the total nitrate at the origin of the plume is equal to the specified 20 mg/L. The residual concentration determined with the nomograph is then added to the 4-mg/L background concentration to determine the total downgradient concentration at the point of concern. Experience has shown that nitrate tends to be a conservative substance when the percolate has passed the active root zone in the soil, so for this case assume that the retardation factor \( R_d \) is equal to 1.

2. Determine the dispersion coefficients:

\[ D_x = (a_x)(v) = (32)(0.45) = 14.4 \, m^2/d \]

\[ D_y = (a_y)(v) = (6)(0.45) = 2.7 \, m^2/d \]

3. Calculate the scale factors:

\[ X_D = \frac{D_x}{v} = 14.4/0.45 = 32 \, m \]

\[ t_D = R_d(D_x)/(v)^2 = 1(14.4)/(0.45)^2 \]

\[ Q_D = (16.02)(n)(z)[(D_x)(D_y)]^{1/2} \]

\[ = (16.02)(0.35)(5)[(14.4)(2.7)]^{1/2} = 174.8 \, kg/d \]
4. Determine the mass flow rate of the contaminant:
\[(Q)(C_0) = (90 \text{ m}^3/\text{d})(20 \text{ mg/L})/(1000 \text{ g/kg}) = 1.8 \text{ kg/d}\]

5. Determine the entry parameters for the nomograph:
\[
\frac{x}{x_D} = \frac{600}{32} = 18.8
\]
\[
\frac{t}{t_D} = \frac{(2)(365)}{71} = 10.3 \quad \text{use} \quad \frac{t}{t_D} = 10 \text{ curve}
\]
\[
\frac{QC_0}{Q_D} = \frac{1.8}{174.8} = 0.01
\]

6. Enter the nomograph on the \(x/x_D\) axis with the value of 18.8, draw a vertical line to intersect with the \(t/t_D\) curve = 10. From that point, project a line horizontally to the \(A-A\) axis. Locate the calculated value 0.01 on the \(B-B\) axis and connect this with the previously determined point on the \(A-A\) axis. Extend this line to the \(C-C\) axis and read the concentration of concern, which is about 0.4 mg/L.

7. After 2 years, the nitrate concentration at a point 600 m downgradient is the sum of the nomograph value and the background concentration, or 4.4 mg/L.

Calculations must be repeated for each contaminant using the appropriate retardation factor. The nomograph can also be used to estimate the distance at which a given concentration will occur in a given time. The upper line on the figure is the “steady-state” curve for very long time periods and, as shown in Example 3.2, can be used to evaluate conditions when equilibrium is reached.

**Example 3.2**

Using the data in Example 3.1, determine the distance downgradient where the groundwater in the plume will satisfy the U.S. Environmental Protection Agency (EPA) limits for nitrate in drinking-water supplies (10 mg/L).

**Solution**

1. Assuming a 4-mg/L background value, the plume concentration at the point of concern could be as much as 6 mg/L. Locate 6 mg/L on the \(C-C\) axis.
2. Connect the point on the \(C-C\) axis with the value 0.01 on the \(B-B\) axis (as determined in Example 3.1). Extend this line to the \(A-A\) axis. Project a horizontal line from this point to intersect the steady-state line. Project a vertical line downward to the \(x/x_D\) axis and read the value \(x/x_D = 60\).
3. Calculate distance \(x\) using the previously determined value for \(x_D\):
\[
x = (x_D)(60) = (32)(60) = 1920 \text{ m}
\]
3.1.3 **GROUNDWATER MOUNDING**

Groundwater mounding is illustrated schematically in Figure 3.1. The percolate flow in the unsaturated zone is essentially vertical and controlled by \( K_v \). If a groundwater table, impeding layer, or barrier exists at depth, a horizontal component is introduced and flow is controlled by a combination of \( K_v \) and \( K_h \) within the groundwater mound. At the margins of the mound and beyond, the flow is typically lateral, and \( K_h \) controls.

The capability for lateral flow away from the source will determine the extent of mounding that will occur. The zone available for lateral flow includes the underground aquifer plus whatever additional elevation is considered acceptable for the particular project design. Excessive mounding will inhibit infiltration in a SAT system. As a result, the capillary fringe above the groundwater mound should never be closer than about 0.6 m (2 ft) to the infiltration surfaces in soil aquifer treatment (SAT) basins. This will correspond to a water table depth of about 1 to 2 m (3 to 7 ft), depending on the soil texture.

In many cases, the percolate or plume from a SAT system will emerge as base flow in adjacent surface waters, so it may be necessary to estimate the position of the groundwater table between the source and the point of emergence. Such an analysis will reveal if seeps or springs are likely to develop in the intervening terrain. In addition, some regulatory agencies require a specific residence time in the soil to protect adjacent surface waters, so it may be necessary to calculate the travel time from the source to the expected point of emergence. Equation 3.10 can be used to estimate the saturated thickness of the water table at any point downgradient of the source (USEPA, 1984). Typically, the calculation is repeated for a number of locations, and the results are converted to an elevation and plotted on maps and profiles to identify potential problem areas:

\[
\begin{align*}
    h &= \left[ (h_0)^2 - \left( \frac{2Q_i(d)}{K_h} \right) \right]^{1/2} \\
    &= \left[ (h_0)^2 - \left( \frac{2Q_i(d)}{K_h} \right) \right]^{1/2} \\
\end{align*}
\]  

where:

- \( h \) = Saturated thickness of the unconfined aquifer at the point of concern (ft; m).
- \( h_0 \) = Saturated thickness of the unconfined aquifer at the source (ft; m).
- \( d \) = Lateral distance from the source to the point of concern (ft; m).
- \( K_h \) = Effective horizontal permeability of the soil system, mid (ft/d).
- \( Q_i \) = Lateral discharge from the unconfined aquifer system per unit width of the flow system (ft³/d·ft; m³/d·m):

\[
Q_i = \frac{K_h}{2d} \left( h_0^2 - h_i^2 \right) 
\]  

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where

\[ d_i = \text{Distance to the seepage face or outlet point (ft; m).} \]

\[ h_i = \text{Saturated thickness of the unconfined aquifer at the outlet point (ft; m).} \]

The travel time for lateral flow is a function of the hydraulic gradient, the distance traveled, the \( K_h \), and the porosity of the soil as defined by Equation 3.12:

\[
t_D = \frac{(n)(d_i)^2}{(K_h)(h_0 - h_i)}
\]  \hspace{1cm} (3.12)

where

\[ t_D = \text{Travel time for lateral flow from source to the point of emergence in surface waters (ft; m).} \]

\[ K_h = \text{Effective horizontal permeability of the soil system (ft/d; m/d).} \]

\[ h_0, h_i = \text{Saturated thickness of the unconfined aquifer at the source and the outlet point, respectively (ft; m).} \]

\[ d_i = \text{Distance to the seepage face or outlet point (ft; m).} \]

\[ n = \text{Porosity, as a decimal fraction.} \]

A simplified graphical method for determining groundwater mounding uses the procedure developed by Glover (1961) and summarized by Bianchi and Muckel (1970). The method is valid for square or rectangular basins that lie above level, fairly thick, homogeneous aquifers of assumed infinite extent; however, the behavior of circular basins can be adequately approximated by assuming a square of equal area. When groundwater mounding becomes a critical project issue, further analysis using the Hantush method (Bauman, 1965) is recommended. Further complications arise with sloped water tables or impeding subsurface layers that induce “perched” mounds or due to the presence of a nearby outlet point. References by Brock (1976), Kahn and Kirkham (1976), and USEPA (1981) are suggested for these conditions. The simplified method involves the graphical determination of several factors from Figure 3.3, Figure 3.4, Figure 3.5, or Figure 3.6, depending on whether the basin is square or rectangular.

It is necessary to calculate the values of \( W/(4at)^{0.5} \) and \( R_i \), as defined in Equations 3.13 to 3.15:

\[
\frac{W}{[(4)(\alpha)(t)]^{0.5}} = \text{dimensionless scale factor}
\]  \hspace{1cm} (3.13)

where \( W \) is the width of the recharge basin (ft; m), and

\[
\alpha = \frac{(K_h)(h_i)}{Y_i}
\]  \hspace{1cm} (3.14)
where

\( K_h \) = Effective horizontal permeability of the aquifer (ft/d; m/d).

\( h_0 \) = Original saturated thickness of the aquifer beneath the center of the recharge area (ft; m).

\( Y_s \) = Specific yield of the soil (use Figure 2.5 or 2.6 to determine) (ft³/ft³; m³/m³).

**FIGURE 3.3** Groundwater mounding curve for center of a square recharge basin.

**FIGURE 3.4** Groundwater mounding curves for center of a rectangle recharge area with different ratios of length (\( L \)) to width (\( W \)).
Natural Wastewater Treatment Systems

\[(R)(t) = \text{scale factor (ft; m)} \quad (3.15)\]

where

\[R = \frac{I}{Y_s} \text{ (ft/d; m/d)}, \text{ where } I \text{ is the infiltration rate or volume of water infiltrated per unit area of soil surface (ft}^3/\text{ft}^2\cdot\text{d; m}^3/\text{m}^2\cdot\text{d).}\]

\[t = \text{Period of infiltration, d.}\]

Enter either Figure 3.3 or 3.4 with the calculated value of \(W/(4\alpha t)^{1/2}\) to determine the value for the ratio \(h_m/(R)(t)\), where \(h_m\) is the rise at the center of the mound. Use the previously calculated value for \((R)(t)\) to solve for \(h_m\). Figure 3.5 (for square areas) and Figure 3.6 (for rectangular areas, where \(L = 2W\)) can be used.
to estimate the depth of the mound at various distances from the center of the recharge area. The procedures involved are best illustrated with a design example.

**Example 3.3**

Determine the height and horizontal spread of a groundwater mound beneath a circular SAT basin 30 m in diameter. The original aquifer thickness is 4 m, and $K_h$ as determined in the field is 1.25 m/d. The top of the original groundwater table is 6 m below the design infiltration surface of the constructed basin. The design infiltration rate will be 0.3 m/d and the wastewater application period will be 3 days in every cycle (3 days of flooding, 10 days for percolation and drying; see Chapter 8 for details).
Natural Wastewater Treatment Systems

Solution

1. Determine the size of an equivalent area square basin:

\[ A = \frac{(3.14)(D)^2}{4} = 706.5 \text{ m} \]

Then the width \( W \) of an equivalent square basin is \((706.5)^{1/2} = 26.5 \text{ m}\).

2. Use Figure 2.5 to determine specific yield \( Y_s \):

\[ K_h = 1.25 \text{ m/d} = 5.21 \text{ cm/hr} \]
\[ Y_s = 0.14 \]

3. Determine the scale factors:

\[ \alpha = \frac{(K_h)(h_b)}{Y_s} = \frac{(1.25)(4)}{0.14} = 35.7 \text{ m}^2/\text{d} \]

\[ W = \frac{26.5}{(4\alpha)^{1/2}} = \frac{26.5}{[(4)(35.7)(3)]^{1/2}} = 1.28 \]

\[ R = \frac{0.3}{0.14} = 2 \text{ m/d} \]

\[ (R)(t) = (2)(3) = 6 \text{ m} \]

4. Use Figure 3.3 to determine the factor \( h_m/(R)(t) \):

\[ \frac{h_m}{(R)(t)} = 0.68 \]

\[ h_m = (0.68)(R)(t) = (0.68)(2)(3) = 4.08 \text{ m} \]

5. The original groundwater table is 6 m below the infiltration surface. The calculated rise of 4.08 m would bring the top of the mound within 2 m of the basin infiltration surface. As discussed previously, this is just adequate to maintain design infiltration rates. The design might consider a shorter (say, 2-day) flooding period, as discussed in Chapter 8, to reduce the potential for mounding somewhat.

6. Use Figure 3.5 to determine the lateral spread of the mound. Use the curve for \( W/(4(\alpha t)^{1/2}) \) with the previously calculated value of 1.28, enter the graph with selected values of \( x/W \) (where \( x \) is the lateral distance of concern), and read values of \( h_m/(R)(t) \). Find the depth to the top of the mound 10 m from the centerline of basin:

\[ x/W = 10/26.5 = 0.377 \]
Enter the \( x/W \) axis with this value, project up to \( W/(4\alpha t)^{1/2} = 1.28 \), then read 0.58 on the \( h_m/(R(t)) \) axis:

\[
h_m = (0.58)(2)(3) = 3.48 \text{ m}
\]

The depth to the mound at the 10-m point is \( 6 \text{ m} - 3.48 \text{ m} = 2.52 \text{ m} \). Similarly, at \( x = 13 \text{ m} \), the depth to the mound is \( 3.72 \text{ m} \), and at \( x = 26 \text{ m} \) the depth to the mound is \( 5.6 \text{ m} \). This indicates that the water level is almost back to the normal groundwater level at a lateral distance about equal to two times the basin width. Changing the application schedule to 2 days instead of 3 would reduce the peak water level to about 3 m below the infiltration surface of the basin.

The procedure demonstrated in Example 3.3 is valid for a single basin; however, as described in Chapter 8, SAT systems typically include multiple basins that are loaded sequentially, and it is not appropriate to do the mounding calculation by assuming that the entire treatment area is uniformly loaded at the design hydraulic loading rate. In many situations, this will result in the erroneous conclusion that mounding will interfere with system operation.

It is necessary first to calculate the rise in the mound beneath a single basin during the flooding period. When hydraulic loading stops at time \( t \), a uniform hypothetical discharge is assumed starting at \( t \) and continuing for the balance of the rest period. The algebraic sum of these two mound heights then approximates the mound shape just prior to the start of the next flooding period. Because adjacent basins may be flooded during this same period, it is also necessary to determine the lateral extent of their mounds and then add any increment from these sources to determine the total mound height beneath the basin of concern. The procedure is illustrated by Example 3.4.

**Example 3.4**

Determine the groundwater mound height beneath a SAT basin at the end of the operational cycle. Assume that the basin is square, 26.5 m on a side, and is one in a set of four arranged in a row (26.5 m wide by 106 m long). Assume the same site conditions as in Example 3.3. Also assume that flooding commences in one of the adjacent basins as soon as the rest period for the basin of concern begins. The operational cycle is 2 days flood, 12 days rest.

**Solution**

1. The maximum rise beneath the basin of concern would be the same as calculated in Example 3.3 with 2-day flooding: \( h_m = 3.00 \text{ m} \).
2. The influence from the next 2 days of flooding in the adjacent basin would be about equal to the mound rise at the 26-m point calculated in Example 3.3, or 0.4 \text{ m}. All the other basins are beyond the zone of influence, so the maximum potential rise beneath the basin of concern is \( (3.00) + (0.4) = 3.4 \text{ m} \). The mound will actually not rise that high, because during the 2 days the adjacent basin is being flooded the first
basin is draining. However, for the purposes of this calculation, assume that the mound will rise the entire 3.4 m above the static groundwater table.

3. The $R$ value for this “uniform” discharge will be the same as that calculated in Example 3.2, but $t$ will now be 12 days: $(R)(t) = (2)(12) = 24$ m/d.

4. Calculate a new $W/(4\alpha t)^{1/2}$, as the “new” time is 12 days:

$$W/(4\alpha t)^{1/2} = 26.5/[(4)(35.7)(12)]^{1/2} = 0.62$$

5. Use Figure 3.3 to determine $hm/(R)(t) = 0.30$: $hm = (24)(0.3) = 7.2$ m. This is the hypothetical drop in the mound that could occur during the 10-day rest period; however, the water level cannot actually drop below the static groundwater table, so the maximum possible drop would be 3.4 m. This indicates that the mound would dissipate well before the start of the next flooding cycle. Assuming that the drop occurs at a uniform rate of 0.72 m/d, the 3.4-m mound will be gone in 4.7 days.

In cases where the groundwater mounding analysis indicates potential interference with system operation, several corrective options are available. As described in Chapter 8, the flooding and drying cycles can be adjusted or the layout of the basin sets rearranged into a configuration with less inter-basin interference. The final option is to underdrain the site to control mound development physically.

Underdrainage may also be required to control shallow or seasonal natural groundwater levels when they might interfere with the operation of either a land or aquatic treatment system. Underdrains are also sometimes used to recover the treated water beneath land treatment systems for beneficial use or discharge elsewhere.

### 3.1.4 UNDERDRAINAGE

In order to be effective, drainage or water recovery elements must either be at or within the natural groundwater table or just above some other flow barrier. When drains can be installed at depths of 5 m (16 ft) or less, underdrains are more effective and less costly than a series of wells. It is possible using modern techniques to install semiflexible plastic drain pipe enclosed in a geotextile membrane by means of a single machine that cuts and then closes the trench.

In some cases, underdrains are a project necessity to control a shallow groundwater table so the site can be developed for wastewater treatment. Such drains, if effective for groundwater control, will also collect the treated percolate from a land treatment operation. The collected water must be discharged, so the use of underdrains in this case converts the project to a surface-water discharge system unless the water is otherwise used or disposed of. In a few situations, drains have been installed to control a seasonally high water table. This type of system may
require a surface-water discharge permit during the period of high groundwater but will function as a nondischarging system for the balance of the year.

The drainage design consists of selecting the depth and spacing for placement of the drain pipes or tiles. In the typical case, drains may be at a depth of 1 to 3 m (3 to 10 ft) and spaced 60 m (200 ft) or more apart. In sandy soils, the spacing may approach 150 m (500 ft). The closer spacings provide better water control, but the costs increase significantly.

The Hooghoudt method (Luthin, 1973) is the most commonly used method for calculating drain spacing. The procedure assumes that the soil is homogeneous, that the drains are spaced evenly apart, that Darcy’s law is applicable, that the hydraulic gradient at any point is equal to the slope of the water table above that point, and that a barrier of some type underlies the drain. Figure 3.7 defines the necessary parameters for drain design, and Equation 3.16 can be used for design:

$$S = \left[ \left( \frac{(4)(K_h)(h_m)}{L_w + P} \right) \left(2d + h_m\right) \right]^{1/2}$$

where

- $S$ = Drain spacing (ft; m).
- $K_h$ = Horizontal permeability of the soil (ft/d; m/d).
- $h_m$ = Height of groundwater mound above the drains (ft; m).
- $L_w$ = Annual wastewater loading rate expressed as a daily rate (ft/d; m/d).
- $P$ = Average annual precipitation expressed as a daily rate (ft/d; m/d).
- $d$ = Distance from drain to barrier (ft; m).
Natural Wastewater Treatment Systems

The position of the top of the mound between the drains is established by design or regulatory requirements for a particular project. SAT systems, for example, require a few meters of unsaturated soil above the mound in order to maintain the design infiltration rate; SR systems also require an unsaturated zone to provide desirable conditions for the surface vegetation. See Chapter 8 for further detail. Procedures and criteria for more complex drainage situations can be found in USDI (1978) and Van Schifgaarde (1974).

3.2 BIODEGRADABLE ORGANICS

Biodegradable organic contaminants, in either dissolved or suspended form, are characterized by the biochemical oxygen demand (BOD) of the waste. Table 1.1, Table 1.2, and Table 1.3 present typical BOD removal expectations for the natural treatment systems described in this book.

3.2.1 REMOVAL OF BOD

As explained in Chapters 4 through 7, the biological oxygen demand (BOD) loading can be the limiting design factor for pond, aquatic, and wetland systems. The basis for these limits is the maintenance of aerobic conditions within the upper water column in the unit and the resulting control of odors. The natural sources of dissolved oxygen (DO) in these systems are surface reaeration and photosynthetic oxygenation. Surface reaeration can be significant under windy conditions or if surface turbulence is created by mechanical means. Observations have shown that the DO in unaerated wastewater ponds varies almost directly with the level of photosynthetic activity, being low at night and early morning, and rising to a peak in the early afternoon. The phytosynthetic responses of algae are controlled by the presence of light, the temperature of the liquid, and the availability of nutrients and other growth factors.

Because algae are difficult to remove and can represent an unacceptable level of suspended solids in the effluent, some pond and aquaculture processes utilize mechanical aeration as the oxygen source. In partially mixed aerated ponds, the increased depth of the pond and the partial mixing of the somewhat turbid contents limit the development of algae as compared to a facultative pond. Most wetland systems (Chapters 6 and 7) restrict algae growth, as the vegetation limits the penetration of light to the water column.

Emergent plant species used in wetlands treatment have the unique capability to transmit oxygen from the leaf to the plant root. These plants do not themselves remove the BOD directly; rather, they serve as hosts for a variety of attached growth organisms, and it is this microbial activity that is primarily responsible for the organic decomposition. The stems, stalks, roots, and rhizomes of the emergent varieties provide the necessary surfaces. This dependence requires a relatively shallow reactor and a relatively low flow velocity to ensure optimum contact opportunities between the wastewater and the attached microbial growth.
Wu et al. (2001) reported that little oxygen escaped from the roots of *Typha latifolia* in a constructed wetland, and in this system the major pathway of oxygen was atmospheric diffusion. These results were reported to be species specific, and other results for *Spartina pectinata* by Wu et al. (2000) indicate that the potential oxygen release could be 15 times that for *T. latifolia*. They also concluded that the amount of oxygen transferred to the wetlands through macrophyte roots and atmospheric diffusion were relatively small compared to the amount of oxygen required to oxidize ammonia.

The BOD of the wastewater or sludge is seldom the limiting design factor for the land treatment processes described in Chapter 8. Other factors, such as nitrogen, metals, toxics, or the hydraulic capacity of the soils, control the design so the system almost never approaches the upper limits for successful biodegradation of organics. Table 3.2 presents typical organic loadings for natural treatment systems.

### 3.2.2 Removal of Suspended Solids

The suspended solids content of wastewater is not usually a limiting factor for design, but the improper management of solids within the system can result in process failure. One critical concern for both aquatic and terrestrial systems is the attainment of proper distribution of solids within the treatment reactor. The use of inlet diffusers in ponds, step feed (multiple inlets) in wetland channels, and higher pressure sprinklers in industrial overland-flow systems is intended to

---

**Table 3.2**

Typical Organic Loading Rates for Natural Treatment Systems

<table>
<thead>
<tr>
<th>Process</th>
<th>Organic Loading (kg/ha/d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxidation pond</td>
<td>40–120</td>
</tr>
<tr>
<td>Facultative pond</td>
<td>22–67</td>
</tr>
<tr>
<td>Aerated partial-mix pond</td>
<td>50–200</td>
</tr>
<tr>
<td>Hyacinth pond</td>
<td>20–50</td>
</tr>
<tr>
<td>Constructed wetland</td>
<td>100</td>
</tr>
<tr>
<td>Slow rate land treatment</td>
<td>45–450</td>
</tr>
<tr>
<td>Rapid infiltration land treatment</td>
<td>130–890</td>
</tr>
<tr>
<td>Overland flow land treatment</td>
<td>35–100</td>
</tr>
<tr>
<td>Land application of municipal sludge</td>
<td>27–930a</td>
</tr>
</tbody>
</table>

* These values were determined by dividing the annual rate by 365 days.
achieve a more uniform distribution of solids and avoid anaerobic conditions at
the head of the process. The removal of suspended solids in pond systems depends
primarily on gravity sedimentation, and, as mentioned previously, algae can be
a concern in some situations. Sedimentation and entrapment in the microbial
growths are both contributing factors in wetland and overland-flow processes.
Filtration in the soil matrix is the principal mechanism for SR and SAT systems.
Removal expectations for the various processes are listed in Table 1.1, Table 1.2,
and Table 1.3. Removal will typically exceed secondary treatment levels, except
for some of the pond systems that contain algal solids in their effluents.

3.3 ORGANIC PRIORITY POLLUTANTS

Many organic priority pollutants are resistant to biological decomposition. Some
are almost totally resistant and may persist in the environment for considerable
periods of time; others are toxic or hazardous and require special management.

3.3.1 REMOVAL METHODS

Volatilization, adsorption, and then biodegradation are the principal methods for
removing trace organics in natural treatment systems. Volatilization can occur at
the water surface of ponds, wetlands, and SAT basins; in the water droplets from
sprinklers used in land treatment; from the liquid films in overland-flow systems;
and from the exposed surfaces of sludge. Adsorption occurs primarily on the
organic matter in the treatment system that is in contact with the waste. In many
cases, microbial activity then degrades the adsorbed materials.

3.3.1.1 Volatilization

The loss of volatile organics from a water surface can be described using first-
order kinetics, because it is assumed that the concentration in the atmosphere
above the water surface is essentially zero. Equation 3.17 is the basic kinetic
equation, and Equation 3.18 can be used to determine the “half-life” of the
contaminant of concern (see Chapter 9 for further discussion of the half-life
concept and its application to sludge organics):

\[
\frac{C_t}{C_0} = e^{-(k_{vol})(y)}
\]

(3.17)

where
\[
\begin{align*}
C_t &= \text{Concentration at time } t \text{ (mg/L or g/L).} \\
C_0 &= \text{Initial concentration at } t = 0 \text{ (mg/L or g/L).} \\
k_{vol} &= \text{Volatilization mass transfer coefficient (cm/hr) = } (k)(y). \\
k &= \text{Overall rate coefficient (hr} \cdot \text{hr).} \\
y &= \text{Depth of liquid (cm).}
\end{align*}
\]
Basic Process Responses and Interactions

\[ t_{t/2} = \frac{(0.693)(y)}{k_{vol}} \]  
\[ \text{where } t_{t/2} \text{ is the time when concentration } C_t = (1/2)(C_0) \text{ (hr), and the other terms are as defined previously.} \]

The volatilization mass transfer coefficient is a function of the molecular weight of the contaminant and the air/water partition coefficient as defined by the Henry’s law constant, as shown by Equation 3.19:

\[ k_{vol} = \left( \frac{B_1}{y} \right) \left( \frac{H}{(B_2 + H)(M)^{1/2}} \right) \]  
\[ \text{where} \]
\[ k_{vol} = \text{Volatilization coefficient (hr}^{-1}). \]
\[ H = \text{Henry’s law constant (10^3 atm} \cdot \text{m}^3 \cdot \text{mol}^{-1}). \]
\[ M = \text{Molecular weight of contaminant of concern (g/mol).} \]

The coefficients \( B_1 \) and \( B_2 \) are specific to the physical system of concern. Dilling (1977) determined values for a variety of volatile chlorinated hydrocarbons at a well-mixed water surface:

\[ B_1 = 2.211, B_2 = 0.01042 \]

Jenkins et al. (1985) experimentally determined values for a number of volatile organics on an overland flow slope:

\[ B_1 = 0.2563, B_2 = (5.86)(10^{-4}) \]

The coefficients for the overland-flow case are much lower because the flow of liquid down the slope is nonturbulent and may be considered almost laminar flow (Reynolds number = 100 – 400). The average depth of flowing liquid on this slope was about 1.2 cm (Jenkins et al., 1985).

Using a variation of Equation 3.19, Parker and Jenkins (1986) determined volatilization losses from the droplets at a low-pressure, large-droplet wastewater sprinkler. In this case, the \( y \) term in the equation is equal to the average droplet radius; as a result, their coefficients are valid only for the particular sprinkler system used. The approach is valid, however, and can be used for other sprinklers and operating pressures. Equation 3.20 was developed by Parker and Jenkins for the organic compounds listed in Table 3.3:

\[ \ln \left( \frac{C_t}{C_0} \right) = (4.535) \left[ k_{vol}' + 11.02(10^{-4}) \right] \]  
\[ \text{where} \]
\[ k_{vol}' = \text{Volatilization coefficient (hr}^{-1}). \]
Volatile organics can also be removed by aeration in pond systems. Clark et al. (1984a) developed Equation 3.21 to determine the amount of air required to strip a given quantity of volatile organics from water via aeration:

\[
\left( \frac{A}{W} \right) = (76.4) \left( 1 - \frac{C_i}{C_0} \right)^{12.44} (S)^{0.37} (V)^{-0.45} (M)^{-0.18} (0.33)^{3/4}
\]

where
- \( (A/W) \) = Air-to-water ratio.
- \( S \) = Saturated condition of the compound of concern equal to 0, for unsaturated organics; 1, for saturated compounds).
- \( V \) = Vapor pressure (mmHg).
- \( M \) = Molecular weight (g/mol).
- \( s \) = Solubility of organic compound (mg/L).

The values in Table 3.4 can be used in Equation 3.21 to calculate the air-to-water ratio required for some typical volatile organics.
3.3.1.2 Adsorption

Sorption of trace organics to the organic matter present in the treatment system is thought to be the primary physicochemical mechanism of removal (USEPA, 1982a). The concentration of the trace organic that is sorbed relative to that in solution is defined by a partition coefficient $K_{p}$, which is related to the solubility of the chemical. This value can be estimated if the octanol–water partition coefficient ($K_{ow}$) and the percentage of organic carbon in the system are defined, as shown by Equation 3.22:

$$\log K_{oc} = (1.00)(\log K_{ow}) – 0.21$$  \hspace{1cm} (3.22)

where

- $K_{oc} = $ Sorption coefficient expressed on an organic carbon basis equal to $K_{sorb}/O_c$.
- $K_{sorb} = $ Sorption mass transfer coefficient (cm/hr).
- $O_c = $ Percentage of organic carbon present in the system.
- $K_{ow} = $ Octanol–water partition coefficient.

Hutchins et al. (1985) presented other correlations and a detailed discussion of sorption in soil systems.

Jenkins et al. (1985) determined that sorption of trace organics on an overland-flow slope could be described with first-order kinetics with the rate constant defined by Equation 3.23:

\begin{table}
\centering
\caption{Properties of Selected Volatile Organics for Equation 3.21}
\begin{tabular}{lccc}
\hline
Chemical & M & S & s \\
\hline
Trichloroethylene & 132 & 1000 & 0 \\
1,1,1-Trichloroethane & 133 & 5000 & 1 \\
Tetrachloroethylene & 166 & 145 & 0 \\
Carbon tetrachloride & 154 & 800 & 1 \\
cis-1,2-Dichloroethylene & 97 & 3500 & 0 \\
1,2-Dichloroethane & 99 & 8700 & 1 \\
1,1-Dichloroethylene & 97 & 40 & 0 \\
\hline
\end{tabular}
\end{table}

Natural Wastewater Treatment Systems

\[
k_{\text{surf}} = \left[ \frac{B_3}{y} \left( \frac{K_{\text{ow}}}{(B_4 + K_{\text{ov}})(M)^{1/2}} \right) \right]
\]  

(3.23)

where

\[k_{\text{surf}} = \text{Sorption coefficient (hr}^{-1}\text{).}\]

\[B_3 = \text{Coefficient specific to the treatment system, equal to 0.7309 for the overland-flow system studied.}\]

\[y = \text{Depth of water on the overland-flow slope (1.2 cm).}\]

\[K_{\text{ow}} = \text{Octanol–water partition coefficient.}\]

\[B_4 = \text{Coefficient specific to the treatment system = 170.8 for the overland-flow system studied.}\]

\[M = \text{Molecular weight of the organic chemical (g/mol).}\]

In many cases, the removal of trace organics is due to a combination of sorption and volatilization. The overall process rate constant \(k_{sv}\) is then the sum of the coefficients defined with Equations 3.19 and 3.23, and the combined removal is described by Equation 3.24:

\[
\frac{C_t}{C_0} = e^{(k_{sv})(t)}
\]  

(3.24)

where

\[C_t = \text{Concentration at time } t \text{ (mg/L or } \mu\text{g/L).}\]

\[C_0 = \text{Initial concentration at } t \text{ equal to 0 (mg/L or } \mu\text{g/L).}\]

\[k_{sv} = \text{Overall rate constant for combined volatilization and sorption equal to } k_{\text{vol}} + k_{\text{surf}}.\]

Table 3.5 presents the physical characteristics of a number of volatile organics for use in the equations presented above for volatilization and sorption.

**Example 3.5**

Determine the removal of toluene in an overland-flow system. Assume a 30-m-long terrace; hydraulic loading of 0.4 m³·hr⁻¹·m⁻¹ (see Chapter 8 for discussion); mean residence time on slope of 90 min; wastewater application with a low-pressure, large-droplet sprinkler; physical characteristics for toluene (Table 3.5) of \(K_{ow} = 490, H = 515, M = 92; \) depth of flowing water on the terrace = 1.5 cm; concentration of toluene in applied wastewater = 70 \(\mu\text{g/L}.\)

**Solution**

1. Use Equation 3.20 to estimate volatilization losses during sprinkling:

\[
\ln \frac{C_t}{C_0} = 4.535 \left[ k_{\text{vol}}' + 11.02(10)^{-4} \right]
\]

\[C_t = 70 \left[ e^{-4.535(0.220)+0.00112} \right] = 25.6 \mu\text{g} / \text{L}
\]
2. Use Equation 3.19 to determine the volatilization coefficient during flow on the overland-flow terrace:

\[
k_{vol} = \left( \frac{B_1}{y} \right) \left( \frac{H}{(B_2 + H)(M)^{1/2}} \right)
\]

\[
k_{vol} = \left( \frac{0.2563}{1.5} \right) \left( \frac{515}{(5.86)(10)^{-4} + (515)(92)^{1/2}} \right)
\]

\[= (0.17087)(0.1042)\]

\[= 0.0178\]

3. Use Equation 3.23 to determine the sorption coefficient during flow on the overland-flow terrace:

TABLE 3.5
Physical Characteristics for Selected Organic Chemicals

<table>
<thead>
<tr>
<th>Substance</th>
<th>(K_{ow}^a)</th>
<th>(H^b)</th>
<th>Vapor Pressure(^c)</th>
<th>(M^d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloroform</td>
<td>93.3</td>
<td>314</td>
<td>194</td>
<td>119</td>
</tr>
<tr>
<td>Benzene</td>
<td>135</td>
<td>435</td>
<td>95.2</td>
<td>78</td>
</tr>
<tr>
<td>Toluene</td>
<td>490</td>
<td>515</td>
<td>28.4</td>
<td>92</td>
</tr>
<tr>
<td>Chlorobenzene</td>
<td>692</td>
<td>267</td>
<td>12</td>
<td>113</td>
</tr>
<tr>
<td>Bromoform</td>
<td>189</td>
<td>63</td>
<td>5.68</td>
<td>253</td>
</tr>
<tr>
<td>m-Dichlorobenzene</td>
<td>2.4 \times 10^3</td>
<td>360</td>
<td>2.33</td>
<td>147</td>
</tr>
<tr>
<td>Pentane</td>
<td>1.7 \times 10^3</td>
<td>125,000</td>
<td>520</td>
<td>72</td>
</tr>
<tr>
<td>Hexane</td>
<td>7.1 \times 10^3</td>
<td>170,000</td>
<td>154</td>
<td>86</td>
</tr>
<tr>
<td>Nitrobenzene</td>
<td>70.8</td>
<td>1.9</td>
<td>0.23</td>
<td>123</td>
</tr>
<tr>
<td>m-Nitrotoluene</td>
<td>282</td>
<td>5.3</td>
<td>0.23</td>
<td>137</td>
</tr>
<tr>
<td>Diethylphthalate</td>
<td>162</td>
<td>0.056</td>
<td>7 \times 10^{-4}</td>
<td>222</td>
</tr>
<tr>
<td>PCB 1242</td>
<td>3.8 \times 10^3</td>
<td>30</td>
<td>4 \times 10^{-4}</td>
<td>26</td>
</tr>
<tr>
<td>Naphthalene</td>
<td>2.3 \times 10^3</td>
<td>36</td>
<td>8.28 \times 10^{-2}</td>
<td>128</td>
</tr>
<tr>
<td>Phenanthrene</td>
<td>2.2 \times 10^4</td>
<td>3.9</td>
<td>2.03 \times 10^{-4}</td>
<td>178</td>
</tr>
<tr>
<td>2,4-Dinitrophenol</td>
<td>34.7</td>
<td>0.001</td>
<td>—</td>
<td>184</td>
</tr>
</tbody>
</table>

\(^a\) Octanol-water partition coefficient.
\(^b\) Henry's law constant, 10^5 atm-m^3/mol at 20°C and 1 atm.
\(^c\) At 25°C.
\(^d\) Molecular weight (g/mol).
### TABLE 3.6
Removal of Organic Chemicals in Land Treatment Systems

<table>
<thead>
<tr>
<th>Substance</th>
<th>Sandy Soil (%)</th>
<th>Silty Soil (%)</th>
<th>Overland Flow&lt;sup&gt;bc&lt;/sup&gt; (%)</th>
<th>Rapid Infiltration&lt;sup&gt;c&lt;/sup&gt; (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chlorobenzene</td>
<td>99.97</td>
<td>99.98</td>
<td>98.99</td>
<td>&gt;99.99</td>
</tr>
<tr>
<td>Bromoform</td>
<td>99.93</td>
<td>99.96</td>
<td>97.43</td>
<td>&gt;99.99</td>
</tr>
<tr>
<td>Dibromochloromethane</td>
<td>99.72</td>
<td>99.72</td>
<td>98.78</td>
<td>&gt;99.99</td>
</tr>
<tr>
<td>m-Nitrotoluene</td>
<td>&gt;99.99</td>
<td>&gt;99.99</td>
<td>94.03</td>
<td>—</td>
</tr>
<tr>
<td>Naphthalene</td>
<td>99.98</td>
<td>99.98</td>
<td>98.49</td>
<td>96.15</td>
</tr>
<tr>
<td>Pentachlorophenol</td>
<td>&gt;99.99</td>
<td>&gt;99.99</td>
<td>98.06</td>
<td>—</td>
</tr>
<tr>
<td>2,4-Dinitrophenol</td>
<td>—</td>
<td>—</td>
<td>93.44</td>
<td>—</td>
</tr>
<tr>
<td>Nitrobenzene</td>
<td>&gt;99.99</td>
<td>&gt;99.99</td>
<td>88.73</td>
<td>—</td>
</tr>
<tr>
<td>m-Dichlorobenzene</td>
<td>&gt;99.99</td>
<td>&gt;99.99</td>
<td>82.27</td>
<td>—</td>
</tr>
<tr>
<td>Hexane</td>
<td>99.96</td>
<td>99.96</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Diethylphthalate</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>90.75</td>
</tr>
</tbody>
</table>

<sup>a</sup> Parker and Jenkins (1986).
<sup>b</sup> Jenkins et al. (1985).
<sup>c</sup> Love et al. (1983).
<sup>d</sup> Not reported.

\[
k_{w} = \frac{B_y}{y} \left( \frac{K_{w}}{(B_y + K_{w})(M)} \right)^{1/2}
\]

\[
k_{w} = \frac{0.7309}{1.5} \left( \frac{490}{(170.8 + 490)(92)} \right)^{1/2}
\]

\[
k_{w} = (0.4873)(0.0774)
\]

\[
k_{w} = 0.0377
\]
4. The overall rate constant is the sum of $k_{vol}$ and $k_{sorb}$:

$$k_t = 0.0178 + 0.0377 = 0.055$$

5. Use Equation 3.24 to determine the toluene concentration in the overland-flow runoff:

$$\frac{C_i}{C_0} = e^{-(k_t)\tau}$$

$$C_i = (25.68)e^{-(0.0558)(90)} = 0.17 \mu g / L$$

This represents about 99.8% removal.

3.3.2 REMOVAL PERFORMANCE

The land treatment systems are the only natural treatment systems that have been studied extensively to determine the removal of priority-pollutant organic chemicals. This is probably due to the greater concern about groundwater contamination with these systems. Results from these studies have been generally positive. As indicated previously, the more soluble compounds such as chloroform tend to move through the soil system more rapidly than the less soluble materials such as some PCBs. In all cases, the amount escaping the treatment system with percolate or effluent is very small. Table 3.6 presents removal performance for the three major land treatment concepts. The removals observed in the SR system were after 1.5 m of vertical travel in the soils indicated, and a low-pressure, large-droplet sprinkler was used for the application. The removals in the OF system were measured after flow on a terrace about 30 m long, with application via gated pipe at the top of the slope at a hydraulic loading of 0.12 m$^3$·m$^{-1}$·hr. The SAT data were obtained from wells about 200 m downgradient of the application basins.

The removals reported in Table 3.6 for SR systems represent concentrations in the applied wastewater ranging from 2 to 111 $\mu g/L$ and percolate concentrations ranging from 0 to 0.4 $\mu g/L$. The applied concentrations in the overland-flow system ranged from 25 to 315 $\mu g/L$ and the effluent from 0.3 to 16 $\mu g/L$. Concentrations of the reported substances applied to the SAT system ranged from 3 to 89 $\mu g/L$, and the percolate ranged from 0.1 to 0.9.

The results in Table 3.6 indicate that the SR system was more consistent and gave higher removals than the other two concepts. This is probably due to the use of the sprinkler and the enhanced opportunity for sorption on the organic matter in these finer-textured soils. Chloroform was the only compound to appear consistently in the percolate, and that was at very low concentrations. Although they were slightly less effective than SR, the other two concepts still produced very high removals. If sprinklers had been used in the OF system, it is likely that the removals would have been even higher. Based on these data, it appears that all three concepts are more effective for trace organic removal than activated sludge and other conventional mechanical treatment systems.
Quantitative relationships have not yet been developed for trace organic removal from natural aquatic systems. The removal due to volatilization in pond and free water surface wetland systems can at least be estimated with Equations 3.19 and 3.24. The liquid depth in these systems is much greater than on an OF slope, but the detention time is measured in terms of many days instead of minutes, so the removal can still be very significant. Organic removal in subsurface flow wetlands may be comparable to the SAT values in Table 3.6, depending on the media used in the wetland. See Chapters 6 and 7 for data on removal of priority pollutants in constructed wetlands.

In a modification of land treatment, Wang et al. (1999) have demonstrated the successful removal by hybrid poplar trees (H11-11) of carbon tetrachloride (15 mg/L in solution). The plant degrades and dechlorinates the carbon tetrachloride and releases the chloride ions to the soil and carbon dioxide to the atmosphere. Indian mustard and maize have been studied for the removal of metals from contaminated soils (Lombi et al., 2001). Alfalfa has been used to remediate a fertilizer spill (Russelle et al., 2001).

In microcosm studies, Bankston et al. (2002) concluded that trichloroethylene (TCE) could be attenuated in natural wetlands which would imply that similar results would be expected in constructed wetlands. The presence of broad-leaved cattails increased the rate of mineralization of TCE above that observed by the indigenous soil microorganisms.

### 3.3.3 Travel Time in Soils

The rate of movement of organic compounds in soils is a function of the velocity of the carrier water, the organic content of the soil, the octanol–water partition coefficient for the organic compound, and other physical properties of the soil system. Equation 3.25 can be used to estimate the movement velocity of an organic compound during saturated flow in the soil system:

\[
V_c = \frac{(K)(G)}{n - (0.63)(p)(O_c)(K_{ow})}
\]

(3.25)

where

- \(V_c\) = Velocity of organic compound (ft/d; m/d).
- \(K\) = Saturated permeability of soil (ft/d; m/d), in vertical or horizontal direction.
- \(K_v\) = Saturated vertical permeability (ft/d; m/d).
- \(K_h\) = Saturated horizontal permeability (ft/d; m/d).
- \(G\) = Hydraulic gradient of flow system (ft/ft; m/m), equal to 1 for vertical flow.
- \(\Delta H / \Delta L\) for horizontal flow (ft/ft; m/m); see Equation 3.4 for definition.
- \(n\) = Porosity of the soil (% , as a decimal); see Figure 2.4.
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\[
\begin{align*}
    p &= \text{Bulk density of soil (lb/in.}^3; \text{g/cm}^3). \\
    O_c &= \text{Organic content of soil (\%, as a decimal).} \\
    K_{ow} &= \text{Octanol–water partition coefficient.}
\end{align*}
\]

3.4 PATHOGENS

Pathogenic organisms may be present in both wastewaters and sludges, and their control is one of the fundamental reasons for waste management. Many regulatory agencies specify bacterial limits on discharges to surface waters. Other potential risks are impacts on groundwaters from both aquatic and land treatment systems, the contamination of crops or infection of grazing animals on land treatment sites, and the off-site loss of aerosolized organisms from pond aerators or land treatment sprinklers. Investigations have shown that the natural aquatic, wetland, and land treatment concepts provide very effective control of pathogens (Reed et al., 1979).

3.4.1 AQUATIC SYSTEMS

The removal of pathogens in pond-type systems is due to natural die-off, predation, sedimentation, and adsorption. Helminths, Ascaris, and other parasitic cysts and eggs settle to the bottom in the quiescent zone of ponds. Facultative ponds with three cells and about 20 days’ detention time and aerated ponds with a separate settling cell prior to discharge provide more than adequate helminth and protozoa removal. As a result, there is little risk of parasitic infection from pond effluents or from use of such effluents in agriculture. Some risk may arise when sludges are removed for disposal. These sludges can be treated, or temporary restrictions on public access and agricultural use can be placed on the disposal site.

3.4.1.2 Bacteria and Virus Removal

The removal of both bacteria and viruses in multiple-cell pond systems is very effective for both the aerated and unaerated types, as shown in Table 3.7 and Table 3.8. The effluent in all three of the cases in Table 3.8 was undisinfected. The viruses measured were the naturally occurring enteric types and not seeded viruses or bacteriophage. Table 3.8 presents seasonal averages; see Bausum (1983) for full details. The viral concentrations in the effluent were consistently low at all times, although, as shown in the table, the removal efficiency did drop slightly in the winter at all three locations.

Numerous studies have shown that the removal of fecal coliforms in ponds depends on detention time and temperature. Equation 3.25 can be used to estimate the removal of fecal coliforms in pond systems. The detention time used in the equation is the actual detention time in the system as measured by dye studies. The actual detention time in a pond can be as little as 45% of the theoretical design detention time due to short-circuiting of flow. If dye studies are not practical or possible, it would be conservative to assume for Equation 3.26 that the “actual” detention time is 50% of the design residence time:

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TABLE 3.7  
Fecal Coliform Removal in Pond Systems

<table>
<thead>
<tr>
<th>Location</th>
<th>Number of Cells</th>
<th>Detention Time (d)</th>
<th>Fecal Coliforms (No./100 mL)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Influent</td>
</tr>
<tr>
<td><strong>Facultative ponds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peterborough, New Hampshire</td>
<td>3</td>
<td>57</td>
<td>$4.3 \times 10^6$</td>
</tr>
<tr>
<td>Eudora, Kansas</td>
<td>3</td>
<td>47</td>
<td>$2.4 \times 10^6$</td>
</tr>
<tr>
<td>Kilmichael, Mississippi</td>
<td>3</td>
<td>79</td>
<td>$1.2.8 \times 10^6$</td>
</tr>
<tr>
<td>Corinne, Utah</td>
<td>7</td>
<td>180</td>
<td>$1.0 \times 10^6$</td>
</tr>
<tr>
<td><strong>Partial-mix aerated ponds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Windber, Pennsylvania</td>
<td>3</td>
<td>30</td>
<td>$1 \times 10^6$</td>
</tr>
<tr>
<td>Edgerton, Wisconsin</td>
<td>3</td>
<td>30</td>
<td>$1 \times 10^6$</td>
</tr>
<tr>
<td>Pawnee, Illinois</td>
<td>3</td>
<td>60</td>
<td>$1 \times 10^6$</td>
</tr>
<tr>
<td>Gulfport, Mississippi</td>
<td>2</td>
<td>26</td>
<td>$1 \times 10^6$</td>
</tr>
</tbody>
</table>


\[
\frac{C_f}{C_i} = \frac{1}{\left(1 + t(k_f)n\right)^{\frac{1}{n}}} \quad (3.26)
\]

where

- \( C_f \) = Effluent fecal coli concentration (number/100 mL).
- \( C_i \) = Influent fecal coli concentration (number/100 mL).
- \( t \) = Actual detention time in the cell (d).
- \( k_f \) = Temperature-dependent rate constant (d\(^{-1}\)), equal to \((2.6)(1.19)^{(T_w-20)}\).
- \( T_w \) = Mean water temperature in pond (°C).
- \( n \) = Number of cells in series.

See Chapter 4 for a method of determining the temperature in the pond; for the general case, it is safe to assume that the water temperature will be about equal to the mean monthly air temperature, down to a minimum of 2°C.

Equation 3.26 in the form presented assumes that all cells in the system are the same size. See Chapter 4 for the general form of the equation when the cells are different sizes. The equation can be rearranged and solved to determine the
optimum number of cells needed for a particular level of pathogen removal. In general, a three- or four-cell (in series) system with an actual detention time of about 20 days will remove fecal coliforms to desired levels. Model studies with polio and coxsackie viruses indicated that the removal of viruses proceeds similarly to the first-order reaction described by Equation 3.26. Hyacinth ponds and similar aquatic units should also perform in accordance with Equation 3.26.

### 3.4.2 Wetland Systems

Pathogen removal in many wetland systems is due to essentially the same factors described above for pond systems. Equation 3.26 can also be used to estimate the removal of bacteria or virus in wetland systems where the water flow path is above the surface. The detention time will be less in most constructed wetlands as compared to ponds, but the opportunities for adsorption and filtration will be greater. The subsurface-flow wetland systems described in Chapter 7 remove pathogens in essentially the same ways as land treatment systems. Table 3.9 summarizes pathogen removal information for selected wetlands. A study of over

---

**TABLE 3.8**

**Enteric Virus Removal in Facultative Ponds**

<table>
<thead>
<tr>
<th>Location</th>
<th>Enteric Virus (PFU/L)</th>
<th>Influent</th>
<th>Effluent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shelby, Mississippi (3 cells, 72 d)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Summer</td>
<td>791</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>Winter</td>
<td>52</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Spring</td>
<td>53</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>El Paso, Texas (3 cells, 35 d)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Summer</td>
<td>348</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>Winter</td>
<td>87</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Spring</td>
<td>74</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Beresford, South Dakota (2 cells, 62 d)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Summer</td>
<td>94</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Winter</td>
<td>44</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>Spring</td>
<td>50</td>
<td>0.4</td>
<td></td>
</tr>
</tbody>
</table>

* PFU/L, plaque-forming units per liter.

TABLE 3.9
Pathogen Removal in Constructed Wetland Systems

<table>
<thead>
<tr>
<th>Location</th>
<th>System Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Influent</td>
</tr>
<tr>
<td>San Diego, California (bullrush wetland)</td>
<td></td>
</tr>
<tr>
<td>Winter season (October–March)</td>
<td></td>
</tr>
<tr>
<td>Total coliforms (number/100 mL)</td>
<td>$5 \times 10^7$</td>
</tr>
<tr>
<td>Bacteriophage (PFU/mL)</td>
<td>1900</td>
</tr>
<tr>
<td>Summer season (April–September)</td>
<td></td>
</tr>
<tr>
<td>Total coliforms (number/100 mL)</td>
<td>$6.5 \times 10^7$</td>
</tr>
<tr>
<td>Bacteriophage (PFU/mL)</td>
<td>2300</td>
</tr>
<tr>
<td>Iselin, Pennsylvania (cattails and grasses)</td>
<td></td>
</tr>
<tr>
<td>Winter season (November–April)</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>$1.7 \times 10^6$</td>
</tr>
<tr>
<td>Summer season (May–October)</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>$1.0 \times 10^6$</td>
</tr>
<tr>
<td>Arcata, California (bullrush wetland)</td>
<td></td>
</tr>
<tr>
<td>Winter season</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>4300</td>
</tr>
<tr>
<td>Summer season</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>1800</td>
</tr>
<tr>
<td>Listowel, Ontario (cattails)</td>
<td></td>
</tr>
<tr>
<td>Winter season</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>556,000</td>
</tr>
<tr>
<td>Summer season</td>
<td></td>
</tr>
<tr>
<td>Fecal coliforms (number/100 mL)</td>
<td>198,000</td>
</tr>
</tbody>
</table>

a Undisinfected.
b Gravel bed, subsurface flow.
c Sand bed, subsurface flow.
d Free water surface.


40 constructed wetlands in Colorado was funded by the Colorado Governor’s Office of Energy Management and Conservation (2000) and performed by HDR Engineering, Inc., and ERO Resources. The performance of and deficiencies in
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the various systems were evaluated, and a comprehensive report is available by contacting the office listed in the references.

3.4.3 Land Treatment Systems

Because land treatment systems in the United States are typically preceded by some form of preliminary treatment or a storage pond, parasites should be of little concern. The evidence in the literature of infection of grazing animals (Reed, 1979) is due to the direct ingestion of, or irrigation with, essentially raw wastewater. The removal of bacteria and viruses in land treatment systems is due to a combination of filtration, dessication, adsorption, radiation, and predation.

3.4.3.1 Ground Surface Aspects

The major concerns relate to the potential for the contamination of surface vegetation or off-site runoff, as the persistence of bacteria or viruses on plant surfaces could then infect people or animals if the plants were consumed raw. To eliminate these risks, it is generally recommended in the United States that agricultural land treatment sites not be used to grow vegetables that may be eaten raw. The major risk is then to grazing animals on a pasture irrigated with wastewater. Typical criteria specify a period ranging from 1 to 3 weeks after sprinkling undisinfected effluent before allowing animals to graze. Systems of this type are divided into relatively small paddocks, and the animals are moved in rotation around the site. Control of runoff is a design requirement of SR and SAT land treatment systems (as described in Chapter 8), so these sources should present no pathogenic hazard. Runoff of the treated effluent is the design intention of overland flow systems, which typically can achieve about 90% removal of applied fecal coliforms. It is a site-specific decision by the regulatory agency regarding the need for final disinfection of treated OF runoff. Overland flow slopes also collect precipitation of any intensity that may happen to occur. The runoff from these rainfall events can be more intense than the design treatment rate, but the additional dilution provided results in equal or better water quality than the normal runoff.

3.4.3.2 Groundwater Contamination

Because percolate from SR and SAT land treatment can reach groundwater aquifers, the risk of pathogenic contamination must be considered. The removal of bacteria and viruses from the finer-textured agricultural soils used in SR systems is quite effective. A 5-year study in Hanover, New Hampshire, demonstrated almost complete removal of fecal coliforms within the top 5 ft of the soil profile (Reed, 1979). Similar studies in Canada (Bell and Bole, 1978) indicated that fecal coliforms were retained in the top 8 cm (3 in.) of the soil. About 90% of the bacteria died within the first 48 hr, and the remainder was eliminated over the next 2 weeks. Virus removal, which depends initially on adsorption, is also very effective in these soils.
The coarse-textured soils and high hydraulic loading rates used in SAT systems increase the risk of bacteria and virus transmission to groundwater aquifers. A considerable research effort, both in the laboratory and at operational systems, has focused on viral movement in SAT systems (Reed, 1979). The results of this work indicate minimal risk for the general case; movement can occur with very high viral concentrations if the wastewater is applied at very high loading rates on very coarse-textured soils. It is unlikely that all three factors will be present in the majority of cases. Chlorine disinfection prior to wastewater application in a SAT system is not recommended, as the chlorinated organic compounds formed represent a greater threat to the groundwater than does the potential transmission of a few bacteria or viruses.

### 3.4.4 Sludge Systems

As shown by the values in Table 3.10, the pathogen levels in raw and digested sludge can be quite high. The pathogen content of sludge is especially critical when the sludge is to be used in agricultural operations or when public exposure is a concern. The sludge utilization guidelines developed by the U.S. EPA are discussed in detail in Chapter 9. Sludge stabilization with earthworms (vermicomposting) is also described in Chapter 9, and some evidence suggests that a reduction in pathogenic bacteria occurs during the process. The freeze–dewatering process will not kill pathogens but can reduce the concentration in the remaining sludge due to enhanced drainage upon thawing. The reed-bed drying concept can achieve significant pathogen reduction due to desiccation and the long detention time in the system. Pathogens are further reduced after sludge is land applied, by the same mechanisms discussed previously for land application of wastewater. There is little risk of transmission of sludge pathogens to groundwater or in runoff to surface waters if the criteria in Chapter 9 are used in system design.

#### Table 3.10

<table>
<thead>
<tr>
<th>Pathogen</th>
<th>Untreated (No./100 mL)</th>
<th>Anaerobically Digested (No./100 mL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viruses</td>
<td>2500–70,000</td>
<td>100–1000</td>
</tr>
<tr>
<td>Fecal coliforms</td>
<td>$1.0 \times 10^6$</td>
<td>$30,000–6 \times 10^6$</td>
</tr>
<tr>
<td><em>Salmonella</em></td>
<td>8000</td>
<td>3–62</td>
</tr>
<tr>
<td><em>Ascaris lumbricoides</em></td>
<td>200–1000</td>
<td>0–1000</td>
</tr>
</tbody>
</table>
Aerosol particles may be up to 20 µm in diameter, which is large enough to transport bacteria or virus. Aerosols will be produced any time that liquid droplets are sprayed into the air, or at the boundary layer above agitated water surfaces, or when sludges are moved about or aerated. Aerosol particles can travel significant distances, and the contained pathogens remain viable until inactivated by desiccation or ultraviolet light. The downwind travel distance for aerosol particles depends on the wind speed, turbulence, temperature, humidity, and presence of any barrier that might entrap the particle. With the impact sprinklers commonly used in land application of wastewater, the volume of aerosols produced amounts to about 0.3% of the water leaving the nozzle (Sorber et al., 1976). If no barrier is present, the greatest travel distance will occur with steady, nonturbulent winds under cool, humid conditions, which are generally most likely to happen at night. The concentration of organisms entering a sprinkler nozzle should be no different than the concentration in the bulk liquid or sludge. Immediately after aerosolization, temperature, sunlight, and humidity have an immediate and significant effect on organism concentration. This aerosol shock is demonstrated in Table 3.11.

### TABLE 3.11
Organism Concentration in Wastewater and Downwind Aerosol

<table>
<thead>
<tr>
<th>Organism</th>
<th>Wastewater Concentration [(No./100 mL) × 10⁶]</th>
<th>Aerosol Concentration at Edge of Sprinkler Impact Circle (No./m³ of air sampled)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard plate count</td>
<td>69.9</td>
<td>2578</td>
</tr>
<tr>
<td>Total coliforms</td>
<td>7.5</td>
<td>5.6</td>
</tr>
<tr>
<td>Fecal coliforms</td>
<td>0.8</td>
<td>1.1</td>
</tr>
<tr>
<td>Coliphage</td>
<td>0.22</td>
<td>0.4</td>
</tr>
<tr>
<td>Fecal streptococci</td>
<td>0.007</td>
<td>11.3</td>
</tr>
<tr>
<td><em>Pseudomonas</em></td>
<td>1.1</td>
<td>71.7</td>
</tr>
<tr>
<td><em>Klebsiella</em></td>
<td>0.39</td>
<td>&lt;1.0</td>
</tr>
<tr>
<td><em>Clostridium perfringes</em></td>
<td>0.005</td>
<td>1.4</td>
</tr>
</tbody>
</table>

As the aerosol particle travels downwind, the microorganisms continue to die off at a slower, first-order rate due to desiccation, ultraviolet radiation, and possibly trace compounds in the air or in the aerosol. This die-off can be very significant for bacteria, but the rates for viruses are very slow so it is prudent to assume no further downwind inactivation of viruses by these factors. Equations 3.27 and 3.28 form a predictive model that can be used to estimate the downwind concentration of aerosol organisms:

\[
C_d = \left( C_n \right) \left( D_d \right) (e)^{(-ax)} + B
\]  

where
- \( C_d \) = Concentration at distance \( d \) (number/ft\(^3\); number/m\(^3\)).
- \( C_n \) = Concentration released at source (number/s).
- \( D_d \) = Atmospheric diffusion factor (s/ft\(^3\); s/m\(^3\)).
- \( x \) = Decay or die-off rate (s\(^{-1}\)).
  - = –0.023 for bacteria (derived for fecal coliforms).
  - = 0.00 for viruses (assumed).
- \( a \) = Downwind distance \( d \)/wind velocity (ft·ft/s; m·m/s),
- \( B \) = Background concentration in upwind air (number/ft\(^3\); number/m\(^3\)).

The initial concentration \( C_n \) leaving the nozzle area is a function of the original concentration in the bulk wastewater (\( W \)), the wastewater flow rate (\( F \)), the aerosolization efficiency (\( E \)), and a survival factor (\( I \)), all as described by Equation 3.28:

\[
C_n = \left( W \right) \left( F \right) \left( E \right) \left( I \right)
\]  

where
- \( C_n \) = Organisms released at source (number/ft\(^3\); number/m\(^3\)).
- \( W \) = Concentration in bulk wastewater (number/100 mL).
- \( F \) = Flow rate (0.631 gal/min; L/s).
- \( E \) = Aerosolization efficiency.
  - = 0.003 for wastewater.
  - = 0.0004 for sludge spray guns.
  - = 0.000007 for sludge applied with tank truck sprinklers.
- \( I \) = Survival factor.
  - = 0.34 for total coliforms.
  - = 0.27 for fecal coliforms.
  - = 0.71 for coliphage.
  - = 3.6 for fecal streptococci.
  - = 80.0 for enteroviruses.

The atmospheric dispersion factor (\( D_d \)) in Equation 3.27 depends on a number of related meteorological conditions. Typical values for a range of expected conditions are given below; USEPA (1982b) should be consulted for a more exact determination:
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Example 3.6

Find the fecal coliform concentration in aerosols 8 m downwind of a sprinkler impact zone. The sprinkler has a 23-m impact circle and is discharging at 30 L/s, fecal coliforms in the bulk wastewater are $1 \times 10^5$, the sprinkler is operating on a cloudy day with a wind speed of about 8 km/hr, and the background concentration of fecal coliforms in the upwind air is zero.

Solution

1. The distance of concern is 31 m downwind of the nozzle source, and
   the wind velocity is 2.22 m/s, so we can calculate the $a$ factor:
   
   $$ a = \frac{\text{Downwind distance}}{\text{Wind velocity}} = \frac{31}{2.22} = 13.96 \text{ s}^{-1} $$

2. Calculate the concentration leaving the nozzle area using Equation 3.28:

   $$ C_n = (W)(F)(E)(I) $$

   $$ = (1 \times 10^5)(30 \text{ L/s})(0.003)(0.27) $$

   $$ = 2430 \text{ fecal coliforms released per second at the nozzle} $$

3. Calculate the concentration at the downwind point of concern using Equation 3.27:

   $$ D_d = 318 \times 10^{-6} $$

   $$ C_d = (C_n)(D_d)(e)^{-0.023(13.96)} + B $$

   $$ = (2340)(318 \times 10^{-6})(e)^{-0.023(13.96)} + 0.0 $$

   $$ = 0.54 \text{ fecal coliforms per m}^3 \text{ of air,} $$

   8 m downwind of the wetted zone of the sprinkler

This is an insignificant level of risk.

The very low concentration predicted in Example 3.6 is typical of the very low concentrations actually measured at a number of operational land treatment sites. Table 3.12 provides a summary of data collected at an intensively studied
### TABLE 3.12
Aerosol Bacteria and Viruses at Pleasanton, California, Land Treatment System Using Undisinfected Effluent

<table>
<thead>
<tr>
<th>Location</th>
<th>Fecal Coliform</th>
<th>Fecal <em>Streptococcus</em></th>
<th>Coliphage</th>
<th><em>Pseudomonas</em></th>
<th>Enteroviruses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wastewater (number/100 mL)</td>
<td>(1 \times 10^5)</td>
<td>(8.8 \times 10^3)</td>
<td>(2.6 \times 10^5)</td>
<td>(2.6 \times 10^5)</td>
<td>2.8</td>
</tr>
<tr>
<td>Upwind (number/m³)</td>
<td>0.02</td>
<td>0.23</td>
<td>0.01</td>
<td>0.03</td>
<td>ND</td>
</tr>
<tr>
<td>Downwind (number/m³):</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10–30 m</td>
<td>0.99</td>
<td>1.45</td>
<td>0.34</td>
<td>81</td>
<td>0.01</td>
</tr>
<tr>
<td>31–80 m</td>
<td>0.46</td>
<td>0.6</td>
<td>0.39</td>
<td>46</td>
<td>ND</td>
</tr>
<tr>
<td>81–200 m</td>
<td>0.23</td>
<td>0.42</td>
<td>0.21</td>
<td>25</td>
<td>ND</td>
</tr>
</tbody>
</table>

* ND, none detected
system where undisinfected effluent was applied to the land. It seems clear that
the very low aerosolization efficiencies ($E$) as defined in Equation 3.27 for sludge
spray guns and truck-mounted sprinklers indicate very little risk of aerosol trans-
port of pathogens from these sources, and this has been confirmed by field
investigations (Sorber et al., 1984)

Composting is a very effective process for inactivating most microorganisms,
including viruses, due to the high temperatures generated during the treatment
(see Chapter 9 for details); however, the heat produced in the process also
stimulates the growth of thermophilic fungi and actinomycetes, and concerns
have been expressed regarding their aerosol transport. The aerosols in this case
are dust particles released when the compost materials are aerated, mixed,
screened, or otherwise moved about the site.

A study was conducted at four composting operations involving 400 on-site
and off-site workers (Clark et al., 1984b). The most significant finding was a
higher concentration of the fungus *Aspergillus fumigatus* in the throat and nasal
cultures of the actively involved on-site workers, but this finding was not cor-
related with an increased incidence of infection or disease. The fungus was rarely
detected in on-site workers involved only occasionally or in the off-site control
group.

The presence of *Aspergillus fumigatus* is due to the composting process itself
and not because wastewater sludges are involved. The study results suggest that
workers who are directly and frequently involved with composting operations
have a greater risk of exposure, but the impact on those who are exposed only
occasionally or on the downwind off-site population is negligible. It should be
possible to protect all concerned with respirators for the exposed workers and a
boundary screen of vegetation around the site.

### 3.5 METALS

Metals at trace-level concentrations are found in all wastewaters and sludges.
Industrial and commercial activities are the major sources, but wastewater from
private residences can also have significant metal concentrations. The metals of
greatest concern are copper, nickel, lead, zinc, and cadmium, and the reason for
the concern is the risk of their entry into the food chain or water supply. A large
percentage of the metals present in wastewater will accumulate in the sludges
produced during the wastewater treatment process. As a result, metals are often
the controlling design parameter for land application of sludge, as described in
detail in Chapter 9. Metals are not usually the critical design parameter for
wastewater treatment or reuse, with the possible exception of certain industries.
Table 3.13 compares the metal concentrations in untreated municipal wastewaters
and the requirements for irrigation and drinking-water supplies.

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3.5.1 AQUATIC SYSTEMS

Trace metals are not usually a concern for the design or performance of pond systems that treat typical municipal wastewaters. The major pathways for removal are adsorption on organic matter and precipitation. Because the opportunity for both is somewhat limited, the removal of metals in most pond systems will be less effective than with activated sludge — for example, where more than 50% of the metals present in the untreated wastewater can be transferred to the sludge in a relatively short time period. Sludges from pond systems can, however, contain relatively high concentrations of metals due to the long retention times and infrequent sludge removal. The metal concentrations found in lagoon sludges at several locations are summarized in Table 3.14. The concentrations shown in Table 3.14 are within the range normally found in unstabilized primary sludges and therefore would not inhibit further digestion or land application as described in Chapter 9. Table 9.4 and Table 9.5 in Chapter 9 list other characteristics of pond sludges. The data in Table 3.14 are from lagoons in cold climates. It is likely that sludge metal concentrations may be higher than these values in lagoons in warm climates that receive a significant industrial wastewater input. In these cases, the benthic sludge will undergo further digestion, which reduces the organic content and sludge mass but not the metals content so their concentrations should increase with time.

### Table 3.13
Metal Concentrations in Wastewater and Requirements for Irrigation and Drinking Water Supplies

<table>
<thead>
<tr>
<th>Metal</th>
<th>Untreated Wastewater(a) (mg/L)</th>
<th>Drinking Water (mg/L)</th>
<th>Irrigation (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Continuous(b)</td>
<td>Short-Term(c)</td>
</tr>
<tr>
<td>Cadmium</td>
<td>&lt;0.005</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Lead</td>
<td>0.008</td>
<td>0.05</td>
<td>5.0</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.04</td>
<td>0.05</td>
<td>2.0</td>
</tr>
<tr>
<td>Copper</td>
<td>0.18</td>
<td>1.0</td>
<td>0.2</td>
</tr>
<tr>
<td>Nickel</td>
<td>0.04</td>
<td>—</td>
<td>0.2</td>
</tr>
</tbody>
</table>

\(a\) Median values for typical municipal wastewater.

\(b\) For waters used for an infinite time period on any kind of soil.

\(c\) For waters used for up to 20 years on fine-textured soils when sensitive crops are to be grown.

If metal removal is a process requirement and the local climate is close to subtropical, the use of water hyacinths in shallow ponds may be considered. Tests with full-scale systems in both Louisiana and Florida (Kamber, 1982) have documented excellent removal, with uptake by the plant itself being a major factor. The plant tissue concentrations may range from hundreds to thousands of times that of the water or sediment concentrations, indicating that bioaccumulation of trace elements by the plant occurs. Metal removals in a pilot hyacinth system in central Florida are presented in Table 3.15. Hyacinths have also been shown to be particularly effective in extracting metals from photoprocessing wastewater at a system in Louisiana (Kamber, 1982)

### TABLE 3.14
**Metal Concentrations in Sludges from Treatment Lagoons**

<table>
<thead>
<tr>
<th>Metal</th>
<th>Facultative Lagoons</th>
<th>Partial-Mix Aerated Lagoons</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wet sludge (mg/L)</td>
<td>Dry solids (mg/kg)</td>
</tr>
<tr>
<td>Copper</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>53.8</td>
</tr>
<tr>
<td>Iron</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>9.0</td>
</tr>
<tr>
<td>Lead</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.9</td>
<td>144</td>
</tr>
<tr>
<td>Mercury</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>2.4</td>
</tr>
<tr>
<td>Zinc</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>54.6</td>
<td>840</td>
</tr>
</tbody>
</table>

*a* Average of values from two facultative lagoons in Utah.

*b* Average of values from two partial-mix aerated lagoons in Alaska.

Excellent metal removals have been demonstrated in the type of constructed wetlands described in Chapter 6 and Chapter 7. Tests at pilot wetlands in southern California, with about 5.5 days' hydraulic residence time, indicated 99, 97, and 99% removal for copper, zinc, and cadmium, respectively (Gersberg et al., 1983); however, plant uptake by the vegetation accounted for less than 1% of the metals involved. The major mechanisms responsible for metal removal were precipitation and adsorption interactions with the organic benthic layer.

Removal of metals in land treatment systems can involve both uptake by any vegetation and adsorption, ion exchange, precipitation, and complexation in or on the soil. As explained in Chapter 9, zinc, copper, and nickel are toxic to vegetation long before they reach a concentration in the plant tissue that would represent a risk to human or animal food chains. Cadmium, however, can accumulate in many plants without toxic effects and may represent some health risk. As a result, cadmium is the major limiting factor for application of sludge on agricultural land.

The near-surface soil layer in land treatment systems is very effective for removal, and most retained metals are found in this zone. Investigations at a rapid-infiltration system that had operated for 33 years on Cape Cod, Massachusetts,

<table>
<thead>
<tr>
<th>Metal</th>
<th>Influent Concentration</th>
<th>Percent Removal^a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boron</td>
<td>0.14 mg/L</td>
<td>37</td>
</tr>
<tr>
<td>Copper</td>
<td>27.6 g/L</td>
<td>20</td>
</tr>
<tr>
<td>Iron</td>
<td>457.8 g/L</td>
<td>34</td>
</tr>
<tr>
<td>Manganese</td>
<td>18.2 g/L</td>
<td>37</td>
</tr>
<tr>
<td>Lead</td>
<td>12.8 g/L</td>
<td>68</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.4 g/L</td>
<td>46</td>
</tr>
<tr>
<td>Chromium</td>
<td>0.8 g/L</td>
<td>22</td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.9 g/L</td>
<td>18</td>
</tr>
</tbody>
</table>

^a Average of three parallel channels, with a detention time about 5 days.

### TABLE 3.16
Metal Content of Grasses at Land Treatment Sites

<table>
<thead>
<tr>
<th>Locations (Concentrations (mg/kg))</th>
<th>Melbourne, Australia</th>
<th>Fresno, California</th>
<th>Manteca, California</th>
<th>Livermore, California</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Started 1896</td>
<td>Started 1907</td>
<td>Started 1961</td>
<td>Started 1964</td>
</tr>
<tr>
<td>Metal</td>
<td>Control Site</td>
<td>Measurement</td>
<td>Control Site</td>
<td>Measurement</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.77</td>
<td>0.89</td>
<td>0.9</td>
<td>1.6</td>
</tr>
<tr>
<td>Copper</td>
<td>6.5</td>
<td>12.0</td>
<td>16.0</td>
<td>13.0</td>
</tr>
<tr>
<td>Nickel</td>
<td>2.7</td>
<td>4.9</td>
<td>5.0</td>
<td>45.0</td>
</tr>
<tr>
<td>Lead</td>
<td>2.5</td>
<td>2.5</td>
<td>13.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Zinc</td>
<td>50.0</td>
<td>63.0</td>
<td>93.0</td>
<td>161.0</td>
</tr>
</tbody>
</table>
Natural Wastewater Treatment Systems

indicated that essentially all of the metals applied could be accounted for in the top 50 cm (20 in.) of the sandy soil, and over 95% were contained within the top 15 cm (6 in.) (Reed, 1979).

Although the metal concentrations in typical wastewaters is low, concerns have been expressed regarding long-term accumulation in the soil that might then affect the future agricultural potential of the site. Work by Hinesly and others as reported by Reed (1979) seems to indicate that most of the metals retained over a long period in the soil are in forms that are not readily available to most vegetation. The plants will respond to the metals applied during the current growing season but are not significantly affected by previous accumulations in the soil. The data in Table 3.16 demonstrate the same relationship. At Melbourne, Australia, after 76 years of application of raw sewage, the cadmium concentration in the grass was just slightly higher than in the grass on the control site, which received no wastewaster. The other locations are newer systems in California, where the cadmium content is the same order of magnitude as measured at Melbourne, suggesting that the vegetation in all these locations is responding to the metals applied during the current growing season and not to prior soil accumulation. The significantly higher lead in the three California sites as compared to Melbourne is believed to be due to motor vehicle exhaust from adjacent highways.

Metals do not pose a threat to groundwater aquifers, even at the very high hydraulic loadings used in rapid-infiltration systems. Experience at Hollister, California, demonstrates that the concentration of cadmium in the shallow groundwater beneath the site is not significantly different than normal offsite groundwater quality (Pound and Crites, 1979). After 33 years of operation at this site, the accumulation of metals in the soil was still below or near the low end of the range normally expected for agricultural soils. Had the site been operated in the slow rate mode, it would have taken over 150 years to apply the same volume of wastewater and contained metals.

3.6 NUTRIENTS

A dual concern with respect to nutrients is that their control is necessary to avoid adverse health or environmental effects but the same nutrients are essential for the performance of the natural biological treatment systems discussed in this book. The nutrients of major importance for both purposes are nitrogen, phosphorus, and potassium. Nitrogen is the controlling parameter for the design of many land treatment and sludge application systems, and those aspects are discussed in detail in Chapter 6, Chapter 7, and Chapter 9. This section covers the potential for nutrient removal using the other treatment concepts and the nutrient requirements of the various system components.

3.6.1 NITROGEN

Nitrogen is limited in drinking water to protect the health of infants and may be limited in surface waters to protect fish life or to avoid eutrophication. As described in Chapter 8, land treatment systems are typically designed to meet the
10-mg/L nitrate drinking-water standard for any percolate or groundwater leaving the project boundary. In some cases, nitrogen removal may also be necessary prior to discharge to surface waters. More often, it is necessary to oxidize or otherwise remove the ammonia form of nitrogen, as this is toxic to many fish and can also represent a significant oxygen demand on the stream.

Nitrogen is present in wastewaters in a variety of forms because of the various oxidation states represented, and it can readily change from one state to another depending on the physical and biochemical conditions present. The total nitrogen concentration in typical municipal wastewaters ranges from about 15 to over 50 mg/L. About 60% of this is in ammonia form, and the remainder is in organic form.

Ammonia can be present as molecular ammonia (NH₃) or as ammonium ions (NH₄⁺). The equilibrium between these two forms in water is strongly dependent on pH and temperature. At pH 7 essentially only ammonium ions are present, while at pH 12 only dissolved ammonia gas is present. This relationship is the basis for air-stripping operations in advanced wastewater treatment plants and for a significant portion of the nitrogen removal that occurs in wastewater treatment ponds.

### 3.6.1.1 Pond Systems

Nitrogen can be removed in pond systems by plant or algal uptake, nitrification and denitrification, adsorption, sludge deposition, and loss of ammonia gas to the atmosphere (volatilization). In facultative wastewater treatment ponds, the dominant mechanism is believed to be volatilization, and under favorable conditions up to 80% of the total nitrogen present can be lost. The rate of removal depends on pH, temperature, and detention time. The amount of gaseous ammonia present at near-neutral pH levels is relatively low, but when some of this gas is lost to the atmosphere additional ammonium ions shift to the ammonia form to maintain equilibrium. Although the unit rate of conversion and loss may be very low, the long detention time in these ponds compensates, resulting in very effective removal over the long term. Chapter 4 presents equations describing this nitrogen removal in ponds that can be used for design. Because nitrogen is often the controlling design parameter for land treatment, a reduction in pond effluent nitrogen can often permit a very significant reduction in the land area needed for wastewater application, with a comparable savings in project costs.

### 3.6.1.2 Aquatic Systems

Nitrogen removal in hyacinth ponds, due primarily to nitrification/denitrification and plant uptake, can be very effective. The plant uptake will not represent permanent removal, however, unless the plants are routinely harvested. A complete harvest is not typically possible, as another function of the hyacinth plant is to shade the water surface so restricted light penetration will limit algal growth. Because harvest might remove only 20 to 30% of the plants in the basin at any one time, the full nitrogen-removal potential of the plants is never realized.

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Nitrification and denitrification are possible in shallow hyacinth ponds even if mechanical aeration is used, due to the presence of aerobic and anaerobic microsites within the dense root zone of the floating plant and the presence of the carbon sources needed for denitrification. Nitrogen removals observed in hyacinth ponds range from less than 10 to over 50 kg/ha/d (9 to 45 lb/ac/d), depending on the season and frequency of harvest. Some of these were carefully managed pilot-scale or research facilities.

3.6.1.3 Wetland Systems

Volatilization of ammonia, denitrification, and plant uptake (if the vegetation is harvested) are the potential methods of nitrogen removal in wetland systems (Gersberg et al., 1983). Studies in Canada (Wile et al., 1985) demonstrated that a regular harvest of cattails still accounted for only about 10% of the nitrogen removed by the system. These findings have been confirmed elsewhere, which indicates that the major pathway for nitrogen removal is nitrification followed by denitrification.

3.6.1.4 Land Treatment Systems

Nitrogen is usually the limiting design parameter for slow-rate land treatment of wastewater, and the criteria and procedures for nitrogen are presented in Chapter 8. Nitrogen can also limit the annual application rate for many sludge systems, as described in Chapter 9. The removal pathways for both types of systems are similar, and include plant uptake, ammonia volatilization, and nitrification/denitrification. Ammonium ions can be adsorbed onto soil particles, thus providing a temporary control; soil microorganisms then nitrify this ammonium, restoring the original adsorptive capacity. Nitrate, on the other hand, will not be chemically retained by the soil system. Nitrate removal by plant uptake or denitrification can occur only during the hydraulic residence time of the carrier water in the soil profile. The overall capability for nitrogen removal will be improved if the applied nitrogen is ammonia or other less well-oxidized forms. Nitrification and denitrification are the major factors for nitrogen removal in rapid-infiltration systems, and crop uptake is a major method for both slow rate and overland flow systems. Volatilization and denitrification also occur with the latter two types of system and may account for from 10 to over 50% of the applied nitrogen, depending on waste characteristics and application methods, as described in Chapter 8. Design procedures based on nitrogen uptake of agricultural and forest vegetation can be found in Chapter 8.

3.6.2 Phosphorus

Phosphorus has no known health significance but is the wastewater constituent that is most often associated with eutrophication of surface waters. Phosphorus in wastewater can occur as polyphosphates, orthophosphates (which can originate from a number of sources), and organic phosphorus, which is more commonly
found in industrial discharges. The potential removal pathways in natural treatment systems include vegetation uptake, other biological processes, adsorption, and precipitation. The vegetative uptake can be significant in the slow-rate and overland flow land treatment processes when harvest and removal are routinely practiced. In these cases, the harvested vegetation might account for 20 to 30% of the applied phosphorus. The vegetation typically used in wetland systems is not considered a significant factor for phosphorus removal, even if harvesting is practiced. If the plants are not harvested, their decomposition releases phosphorus back to the water in the system. Phosphorus removal by water hyacinths and other aquatic plants is limited to plant needs and will not exceed 50 to 70% of the phosphorus present in the wastewater, even with careful management and regular harvests.

Adsorption and precipitation reactions are the major pathways for phosphorus removal when wastewater has the opportunity for contact with a significant volume of soil. This is always the case with slow rate and rapid infiltration systems, as well as some wetland systems where infiltration and lateral flow through the subsoil are possible. The possibilities for contact between the wastewater and the soil are more limited with the overland flow process, as relatively impermeable soils are used.

The soil reactions involve clay, oxides of iron and aluminum, calcium compounds present, and the soil pH. Finer-textured soils tend to have the greatest potential for phosphorus sorption due to the higher clay content but also to the increased hydraulic residence time. Coarse-textured, acidic, or organic soils have the lowest capacity for phosphorus. Peat soils are both acidic and organic, but some have a significant sorption potential due to the presence of iron and aluminum.

A laboratory-scale adsorption test can estimate the amount of phosphorus that a soil can remove during short application periods. Actual phosphorus retention in the field will be at least two to five times the value obtained during a typical 5-day adsorption test. The sorption potential of a given soil layer will eventually be exhausted, but until that occurs the removal of phosphorus will be almost complete. It has been estimated that a 30-cm depth of soil in a typical slow-rate system might become saturated with phosphorus every 10 years. The phosphorus concentrations in the percolate from slow-rate systems usually approach background levels for the native groundwater within 2 m of travel in the soil. The coarser textured soils utilized for rapid infiltration might require an order-of-magnitude greater travel distance.

Phosphorus is not usually a critical issue for groundwater quality; however, when the groundwater emerges in a nearby surface stream or pond, eutrophication concerns may arise. Equation 3.29 can be used to estimate the phosphorus concentration at any point on the infiltration/percolation, groundwater flow path. The equation was originally developed from rapid infiltration system responses, so it provides a very conservative basis for all soil systems (USEPA, 1981):

\[ P_t = P_0e^{kt} \]  

(3.29)
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where

\[ P_x = \text{Total } P \text{ at a distance } x \text{ on the flow path (mg/L).} \]
\[ P_0 = \text{Total } P \text{ in applied wastewater (mg/L).} \]
\[ k_p = 0.048 \text{ at pH 7 (d}^{-1}) \text{ (pH 7 gives the lowest value).} \]
\[ t = \text{Detention time (d)} = (x)(W)/(K_x)(G), \text{ where:} \]
\[ x = \text{Distance along flow path (ft; m).} \]
\[ W = \text{Saturated soil water content; assume 0.4.} \]
\[ K_x = \text{Hydraulic conductivity of soil in direction } x \text{ (ft/d; m/d); thus, } K_v = \text{vertical and } K_h = \text{horizontal.} \]
\[ G = \text{Hydraulic gradient for flow system:} \]
\[ = 1 \text{ for vertical flow.} \]
\[ = \Delta H/\Delta L \text{ for lateral flow.} \]

The equation is solved in two steps: first for the vertical flow component, from the soil surface to the subsurface flow barrier (if one exists), and then for the lateral flow to the adjacent surface water. The calculations are based on assumed saturated conditions, so the lowest possible detention time will result. The actual vertical flow in most cases will be unsaturated, so the actual detention time will be much longer than is calculated with this procedure. If the equation predicts acceptable removal, we have some assurance that the site should perform reliably and detailed tests should not be necessary for preliminary work. Detailed tests should be conducted for final design of large-scale projects.

### 3.6.3 Potassium and Other Micronutrients

As a wastewater constituent, potassium usually has no health or environmental effects. It is, however, an essential nutrient for vegetative growth, and it is not typically present in wastewaters in the optimum combination with nitrogen and phosphorus. If a land or aquatic treatment system depends on vegetation for nitrogen removal, it may be necessary to add supplemental potassium to maintain plant uptake of nitrogen at the optimum level. Equation 3.30 can be used to estimate the supplemental potassium that may be required for aquatic systems and for land systems where the soils have a low level of natural potassium:

\[ K = (0.9)(U) - K_{ww} \quad (3.30) \]

where

\[ K = \text{Annual supplemental potassium needed (kg/ha).} \]
\[ U = \text{Estimated annual nitrogen uptake of vegetation (kg/ha).} \]
\[ K_{ww} = \text{Amount of potassium in the applied wastewater (kg/ha).} \]

Most plants also require magnesium, calcium, and sulfur and, depending on soil characteristics, there may be deficiencies in some locations. Iron, manganese, zinc, boron, copper, molybdenum, and sodium are other micronutrients that are important for vegetative growth. Generally, wastewater contains a sufficient
amount of these elements, and in some cases the excess can lead to phytotoxicity problems. Some high-rate hyacinth systems may require supplemental iron to maintain vigorous plant growth.

3.6.3.1 Boron

Boron is at the same time essential for plant growth and toxic to sensitive plants at low concentrations. Experience has shown that soil systems have very limited capacity for boron adsorption, so it is conservative to assume a zero removal potential for land treatment systems. Industrial wastewaters may have a higher boron content than typical municipal effluents; the boron content may influence the type of crop selected but will not control the feasibility of land treatment. Tolerant crops such as alfalfa, cotton, sugar beets, and sweet clover might accept up to 2 to 4 mg/L boron in the wastewater; semi-tolerant crops such as corn, barley, milo, oats, and wheat might accept 1 to 2 mg/L; and sensitive crops such as fruits and nuts should receive less than 1 mg/L.

3.6.3.2 Sulfur

Wastewaters contain sulfur in either the sulfite or the sulfate form. Municipal wastewaters do not usually contain enough sulfur to be a design problem, but industrial wastewaters from petroleum refining and Kraft paper mills can be a concern. Sulfate is limited to 250 mg/L in drinking waters and 200 to 600 mg/L for irrigation, depending on the type of vegetation. Sulfur is weakly adsorbed on soils, so the major pathway for removal is by plant uptake. The grasses typically used in land treatment can remove 2 to 3 kg of sulfur per 1000 kg (4 to 7 lb per 2200 lb) of material harvested (Overcash and Pal, 1979). The presence of sulfites or sulfates in wastewater can lead to serious odor problems if anaerobic conditions develop. This has occurred with some hyacinth systems, and supplemental aeration is then needed to maintain aerobic conditions in the basin.

3.6.3.3 Sodium

Sodium is not limited by primary drinking-water standards, and the sodium content of typical municipal wastewaters is not a significant water-quality concern. A sudden change to high sodium content will adversely affect the biota in an aquatic system, but most systems can acclimate to gradual changes. Sodium and also calcium influence soil alkalinity and salinity, which in turn can affect the vegetation in land treatment systems. The growth of the plant and its ability to absorb moisture from the soil are influenced by salinity. The structure of clay soils can be damaged when there is an excess of sodium with respect to calcium and magnesium in the wastewater. The resulting swelling of some clay particles changes the hydraulic capacity of the soil profile. The sodium adsorption ratio (SAR) as shown by Equation 3.31 defines the relationship among these three elements:
Natural Wastewater Treatment Systems

\[
SAR = \frac{[Na]}{\left( \frac{([Ca] + [Mg])}{2} \right)^{1/2}}
\]

(3.31)

where

\[\begin{align*}
SAR &= \text{Sodium adsorption ratio.} \\
[Na] &= \text{Sodium concentration (mEq/L) = (mg/L in wastewater)/22.99.} \\
[Ca] &= \text{Calcium concentration (mEq/L) = (mg/L in wastewater)(2)/40.08.} \\
[Mg] &= \text{Magnesium concentration (mEq/L) = (mg/L in wastewater)(2)/24.32.}
\end{align*}\]

The SAR for typical municipal effluents seldom exceeds a value of 5 to 8, so it should not be a problem with most soils in any climate. Soils with up to 15% clay can tolerate a SAR of 10 or less, while soils with little clay or with nonswelling clays can accept SARs up to about 20. Industrial wastewaters can have a high SAR, and periodic soil treatment with gypsum or some other inexpensive source of calcium may be necessary to reduce clay swelling. Soil salinity is managed by adding an excess of water above that required for crop growth to leach the salts from the soil profile. A “rule of thumb” for total water required to prevent salt buildup in arid climates is to apply the crop needs plus about 10% (Pettygrove and Asano, 1985). A report by the USEPA (1984) provides further details.

REFERENCES


Basic Process Responses and Interactions


