Example 7.17
A sample of water has been found to contain the following:

alkalinity 220 mg/l as CaCO₃
hardness 180 mg/l as CaCO₃
calcium (Ca²⁺) 140 mg/l as CaCO₃

(a) What is the non-carbonate hardness?
(b) What is the Mg²⁺ content in mg/l as substance?

To use figure 7.1, the absence of any significant hydroxides must be assumed. Since the alkalinity is greater than the hardness, the figure indicates the following compounds in the water:

$$NaHCO₃ = 220 - 180 = 40 \text{ mg/l as CaCO}_3$$
$$Mg(HCO₃)_2 = 180 - 140 - 40 \text{ mg/l as CaCO}_3$$
$$Ca(HCO₃)_2 = 140 \text{ mg/l}$$

There is no non-carbonate hardness in the water.

The Mg²⁺ ion content in CaCO₃ is equal to the Mg(HCO₃)₂ content as CaCO₃. Appendix A can be used to convert CaCO₃ equivalents to amounts as substance

$$Mg^{2+} = \frac{40}{41} = 9.6 \text{ mg/l as substance}$$

### 7 WATER QUALITY STANDARDS

Minimum drinking water quality standards have been set by the Safe Drinking Water Act. Typical minimum standards are given in Table 7.9 as Maximum Contaminant Levels (MCL's). Values in Table 7.9 are subject to change as new legislation is enacted.

### 8 WATER DEMAND

Water demand comes from a number of sources, including residential, commercial, industrial, and public consumers, as well unavoidable loss and waste. In project planning, a minimum of about 165 gallons per capita per day should be considered. This 165 gpcd is a total of all demands, as given in Table 7.10. If large industries are present (such as canning, steel making, automobile production, electronics, etc.), then those industries' special needs must also be considered.

For ordinary domestic use, the water pressure should be 25 to 40 psi. A minimum of 60 psi at the fire hydrant is usually adequate, since that allows for up to 20 psi pressure drop in fire hoses. 75 psi and higher is common in commercial and industrial districts.

<table>
<thead>
<tr>
<th>Contaminant/Quality</th>
<th>MCL</th>
</tr>
</thead>
<tbody>
<tr>
<td>inorganic compounds</td>
<td></td>
</tr>
<tr>
<td>arsenic</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>barium</td>
<td>1.0 mg/l</td>
</tr>
<tr>
<td>cadmium</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>chloride</td>
<td>250 mg/l</td>
</tr>
<tr>
<td>chromium</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>copper</td>
<td>1.0 mg/l</td>
</tr>
<tr>
<td>cyanide</td>
<td>0.005 mg/l</td>
</tr>
<tr>
<td>iron</td>
<td>0.3 mg/l</td>
</tr>
<tr>
<td>lead</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>manganese</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>mercury</td>
<td>0.002 mg/l</td>
</tr>
<tr>
<td>nitrate</td>
<td>10 mg/l</td>
</tr>
<tr>
<td>selenium</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>silver</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>sulfate</td>
<td>250 mg/l</td>
</tr>
<tr>
<td>zinc</td>
<td>5.0 mg/l</td>
</tr>
<tr>
<td>organic compounds</td>
<td></td>
</tr>
<tr>
<td>trihalomethanes (total)</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>organic pesticides</td>
<td></td>
</tr>
<tr>
<td>endrin</td>
<td>0.0002 mg/l</td>
</tr>
<tr>
<td>lindane</td>
<td>0.004 mg/l</td>
</tr>
<tr>
<td>methoxychlor</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>toxaphene</td>
<td>0.005 mg/l</td>
</tr>
<tr>
<td>2,4-D</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>2,4,5-TP(sivex)</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>miscellaneous regulations</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>6.5-8.5</td>
</tr>
<tr>
<td>turbidity</td>
<td>1 NTU</td>
</tr>
<tr>
<td>color</td>
<td>15 units</td>
</tr>
<tr>
<td>microbiological</td>
<td>1 coliform/100 ml</td>
</tr>
<tr>
<td>total dissolved solids</td>
<td>500 mg/l</td>
</tr>
<tr>
<td>odor (in threshold)</td>
<td>3 T O N</td>
</tr>
<tr>
<td>odor numbers</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 7.10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual Average Water Requirements (gpcd)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>(Excluding fire fighting)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>residential</td>
</tr>
<tr>
<td>commercial &amp; industrial</td>
</tr>
<tr>
<td>public</td>
</tr>
<tr>
<td>loss &amp; waste</td>
</tr>
<tr>
<td>165-270 total</td>
</tr>
</tbody>
</table>

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Example 1-1

Find the normality of the following solutions:

1. 60 mg CO$_3^{2-}$/liter, given that CO$_3^{2-}$ participates in the precipitation reaction:
   \[ \text{Ca}^{2+} + \text{CO}_3^{2-} \rightarrow \text{CaCO}_3(s) \]

2. 155 mg Ca$_3$(PO$_4$)$_2$/liter given that Ca$_3$(PO$_4$)$_2$ participates in the dissolution reaction:
   \[ \text{Ca}_3\text{(PO}_4)_2 \rightarrow 3\text{Ca}^{2+} + 2\text{PO}_4^{3-} \]

Solution

1. The molecular weight of CO$_3^{2-}$ is 60.
   
   \[
   \text{Gram equivalent weight} = \frac{\text{gram molecular weight}}{\text{ion charge}} = \frac{60 \text{ g/mole}}{2 \text{ eq/mole}} = 30 \text{ g/eq}
   \]
   
   \[= 30 \text{ mg/meq} \]
   
   \[
   \text{Normality} = \frac{60 \text{ mg/liter}}{30 \text{ mg/meq}} = 2 \text{ meq/liter}
   \]

2. The molecular weight of Ca$_3$(PO$_4$)$_2$ is 310. Because each Ca$_3$(PO$_4$)$_2$ forms six positive and six negative charges.
   
   \[
   \text{Gram equivalent weight} = \frac{310 \text{ g/mole}}{6 \text{ eq/mole}} = 51.67 \text{ g/eq} = 51.67 \text{ mg/meq}
   \]
   
   \[
   \text{Normality} = \frac{155 \text{ mg/liter}}{51.67 \text{ mg/meq}} = 3 \text{ meq/liter}
   \]
Example 1-2

Find the normality of the following solutions:

1. 36.5 mg HCl/liter, with respect to the reaction
   \[ \text{HCl} + \text{NaOH} \rightleftharpoons \text{NaCl} + \text{H}_2\text{O} \]
2. 49 mg H$_3$PO$_4$/liter, with respect to the reaction
   \[ \text{H}_3\text{PO}_4 \rightleftharpoons 2\text{H}^+ + \text{HPO}_4^{2-} \]
3. 45 mg CO$_3^{2-}$/liter, with respect to the reaction
   \[ \text{CO}_3^{2-} + \text{H}_2\text{O} \rightleftharpoons \text{HCO}_3^- + \text{OH}^- \]
4. 45 mg CO$_3^{2-}$/liter, with respect to the reaction
   \[ \text{CO}_3^{2-} + 2\text{H}^+ \rightleftharpoons \text{H}_2\text{CO}_3 \]

Solution

1. One H$^+$ reacts per HCl. Therefore, we find
   \[
   \text{Gram equivalent weight} = \frac{\text{gram molecular weight}}{1 \text{ eq/mole}} = \frac{36.5 \text{ g/mole}}{1 \text{ eq/mole}} = 36.5 \text{ g/eq} = 36.5 \text{ mg/Meq} \\
   \text{Normality} = \frac{36.5 \text{ mg/liter}}{36.5 \text{ mg/Meq}} = 1 \text{ Meq/liter}
   \]

2. 2H$^+$ react per H$_3$PO$_4$. Therefore,
   \[
   \text{Gram equivalent weight} = \frac{\text{gram molecular weight}}{2 \text{ eq/mole}} = \frac{98 \text{ g/mole}}{2 \text{ eq/mole}} = 49 \text{ g/eq} = 49 \text{ mg/Meq} \\
   \text{Normality} = \frac{49 \text{ mg/liter}}{49 \text{ mg/Meq}} = 1 \text{ Meq/liter}
   \]

3. One OH$^-$ results from this reaction. Thus
   \[
   \text{Gram equivalent weight} = \frac{\text{gram molecular weight}}{1 \text{ eq/mole}} = \frac{60 \text{ g/mole}}{1 \text{ eq/mole}} = 60 \text{ g/eq} = 60 \text{ mg/Meq} \\
   \text{Normality} = \frac{45 \text{ mg/liter}}{60 \text{ mg/Meq}} = 0.75 \text{ Meq/liter}
   \]

4. Two H$^+$ react with each CO$_3^{2-}$. Thus
   \[
   \text{Gram equivalent} = \frac{\text{gram molecular weight}}{2 \text{ eq/mole}} = \frac{60 \text{ g}}{2 \text{ eq/mole}} = 30 \text{ g/eq} = 30 \text{ mg/Meq} \\
   \text{Normality} = \frac{45 \text{ mg/liter}}{30 \text{ mg/Meq}} = 1.5 \text{ Meq/liter}
   \]
Example 2-2: Calculating equivalents How many grams of calcium will be required to combine with 90 g of carbonate to form calcium carbonate?

Solution
1. Carbonate (CO$_3^{2-}$) is a radical composed of carbon and oxygen. In this particular combination, carbon has an atomic mass of 12 and a valence of +4, while oxygen has an atomic mass of 16 and a valence of −2. Therefore, the radical has a total valence of −2 and an equivalence of 2. One equivalent of carbonate is
   \[
   \frac{12 + 3(16)}{2} = 30 \text{ g/equiv}
   \]
2. The calcium ion has an atomic mass of 40 and a valence of +2; therefore, one equivalent of calcium is
   \[
   \frac{40}{2} = 20 \text{ g/equiv}
   \]
3. The number of equivalents of calcium must equal the number of equivalents of carbonate, therefore
   \[
   \frac{90 \text{ g}}{30 \text{ g/equiv}} = 3 \text{ equiv of carbonate}
   \]
Therefore, 3 equiv × 20 g equiv = 60 g of calcium, and that amount will be required to react with 90 g of carbonate.

Example 2-3: Determining equivalent concentrations What is the equivalent calcium carbonate concentration of (a) 117 mg/L of NaCl and (b) 2 × 10$^{-3}$ mol of NaCl?

Solution
(a) 1. One equivalent of calcium carbonate is
   \[
   \frac{40 + 12 + 3(16)}{2} = 50 \text{ g/equiv} = 50,000 \text{ mg/equiv} = 50 \text{ mg/mequiv}
   \]
2. One equivalent of sodium chloride is
   \[
   \frac{23 + 35.5}{1} = 58.5 \text{ g/equiv} = 58.5 \text{ mg/mequiv}
   \]
3. By Eq (2-2)
   \[
   \frac{117 \text{ mg/L}}{58.5 \text{ mg/mequiv}} \times 50 \text{ mg/mequiv} = 100 \text{ mg/L of NaCl as CaCO}_3
   \]
(b) 1. One mole of a substance divided by its valence is one equivalent
   \[
   \frac{2 \times 10^{-3} \text{ mol/L}}{1 \text{ mol/equiv}} = 2 \times 10^{-3} \text{ equiv L}
   \]
2. Thus, 2 × 10$^{-3}$ equiv L × 50,000 mg/equiv = 100 mg L.
Example 2-5: Testing for ion balance Tests for common ions are run on a sample of water and the results are shown below. If a 10 percent error in the balance is acceptable, should the analysis be considered complete?

Constituents

\[
\begin{align*}
\text{Ca}^{2+} & = 55 \text{ mg/L} & \text{HCO}_3^- & = 750 \text{ mg/L} \\
\text{Mg}^{2+} & = 18 \text{ mg/L} & \text{SO}_4^{2-} & = 60 \text{ mg/L} \\
\text{Na}^+ & = 98 \text{ mg/L} & \text{Cl}^- & = 89 \text{ mg/L}
\end{align*}
\]

Solution

1. Convert the concentrations of cations and anions from milligrams per liter to milliequivalents per liter and sum them.

<table>
<thead>
<tr>
<th>Ion</th>
<th>Conc, mg/L</th>
<th>Equiv, mg/mequiv</th>
<th>Equiv conc, meq/L</th>
<th>Ion</th>
<th>Conc, mg/L</th>
<th>Equiv, mg/mequiv</th>
<th>Equiv conc, meq/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca(^{2+})</td>
<td>55</td>
<td>40/2</td>
<td>2.75</td>
<td>HCO(_3)^-</td>
<td>250</td>
<td>61/1</td>
<td>4.30</td>
</tr>
<tr>
<td>Mg(^{2+})</td>
<td>18</td>
<td>24/3/2</td>
<td>1.48</td>
<td>SO(_4)^{2-}</td>
<td>60</td>
<td>96/2</td>
<td>1.25</td>
</tr>
<tr>
<td>Na(^+)</td>
<td>98</td>
<td>23/1</td>
<td>4.26</td>
<td>Cl(^-)</td>
<td>89</td>
<td>35/5/1</td>
<td>2.51</td>
</tr>
<tr>
<td>Total ions</td>
<td></td>
<td></td>
<td>8.49</td>
<td></td>
<td></td>
<td></td>
<td>7.86</td>
</tr>
</tbody>
</table>

2. Calculate percent of error.

\[
\frac{8.49 - 7.86}{7.86} \times 100 = 8\%
\]

Therefore, accept analysis.

A common ion balance can be displayed conveniently in the form of a bar diagram. A bar diagram for the water in Example 2-5 can be drawn as shown below.
Example 2-6: Determining total alkalinity

A 200-ml sample of water has an initial pH of 10. Thirty milliliters of 0.02 N H₂SO₄ is required to titrate the sample to pH 4.5. What is the total alkalinity of the water in milligrams per liter as CaCO₃?

**Solution**

Because each mg of 0.02 N H₂SO₄ will neutralize 1 mg of alkalinity, there is 30 mg of alkalinity in the 200 mL sample. Therefore, the concentration of alkalinity expressed as milligrams per liter will be

\[
\frac{30 \text{ mg}}{200 \text{ mL}} \times \frac{1000 \text{ mL}}{1 \text{ L}} = 150 \text{ mg/L.}
\]

If the volume of acid needed to reach the 8.3 endpoint is known, the species of alkalinity can also be determined. Because all of the hydroxide and one-half of the carbonate have been neutralized at pH 8.3, the acid required to lower the pH from 8.3 to 4.5 must measure the other one-half of the carbonate, plus all of the original bicarbonate. If P is the amount of acid required to reach pH 8.3 and M is the total quantity of acid required to reach 4.5, the following generalizations concerning the forms of alkalinity can be made:

- If \( P = M \), all alkalinity is \( \text{OH}^- \)
- \( P = M/2 \), all alkalinity is \( \text{CO}_3^{2-} \)
- \( P = 0 \) (i.e., initial pH is below 8.3), all alkalinity is \( \text{HCO}_3^- \)
- \( P < M/2 \), predominant species are \( \text{CO}_3^{2-} \) and \( \text{HCO}_3^- \)
- \( P > M/2 \), predominant species are \( \text{OH}^- \) and \( \text{CO}_3^{2-} \)

In observing the pH dependency of the species in Fig. 2-3, it is noted that the quantity of \( \text{OH}^- \) becomes significant at pH less than about 9.0. Without introducing significant error, it can be assumed that the \( \text{OH}^- \) of samples with pH less than 9.0 is insignificant. The \( \text{CO}_3^{2-} \) would then be measured by \( 2P \) and the \( \text{HCO}_3^- \) would be measured by the remainder \((M - 2P)\). One method of calculating the quantities of each species is illustrated in the following example.
Example 2.7: Determining alkalinity species. Determine the species and the quantity of each species of alkalinity in Example 2.6 if the 8.3 equivalence point is reached at 11 mL of acid.

SOLUTION

1. Because the initial pH is 10, the initial pOH of the water is 4. A determination of the OH\(^-\) concentration can be made as follows.

\[
[\text{OH}^-] = 10^{-4} \text{ mol OH}^- \times \frac{1 \text{ equiv}}{\text{mol OH}^-} \times \frac{50,000 \text{ mg CaCO}_3}{1 \text{ equiv}}
\]

\[
= 5 \text{ mg/L as CaCO}_3
\]

2. Five milliliters of acid would be required to measure the OH\(^-\) in a 1-L sample. However, this sample is only 200 mL so the necessary volume of acid is:

\[
\frac{5}{1000} = 0.005 \text{ mL}
\]

3. If 1 mL of acid measures the OH\(^-\), then 10 mL of acid measures one-half of the carbonate and 10 more will be required to measure the remaining one-half of the CO\(_3^{2-}\), leaving 9 mL to measure the HCO\(_3^-\). (See Fig. 2-4.) Thus, the quantity of each species is as follows:

\[
\text{OH}^- \text{ (calculated from pH)} = 5 \text{ mg/L}
\]

\[
\text{CO}_3^{2-} = \frac{20 \text{ mg}}{200 \text{ mL}} \times \frac{1000 \text{ mL}}{\text{L}} = 100 \text{ mg/L}
\]

\[
\text{HCO}_3^- = \frac{9 \text{ mg}}{200 \text{ mL}} \times \frac{1000 \text{ mL}}{\text{L}} = 45 \text{ mg/L}
\]

Total alkalinity = 150 mg/L
PROBLEMS

11.1 (a) Using atomic weights from the table of elements given in Table A7, calculate the molecular and equivalent weights of alum (aluminum sulfate), ferric sulfate, and soda ash (sodium carbonate). The formulas of these compounds are given in Table 11.1. (b) Using atomic weights, compute the equivalent weights of the ammonium ion, bicarbonate ion, calcium carbonate, and carbon dioxide. Values are given in Table 11.2.

11.1 (a) \( \text{Al}_2(\text{SO}_4)_3 \cdot 14.3\text{H}_2\text{O} = 2\text{Al} + 3\text{S} + 26.3\text{(O)} + 28.6\text{H} \)

\[ MW = 2 \times 27.0 + 3 \times 32.1 + 26.3 \times 16.0 + 28.6 \times 1.0 = 600 \]

\[ EW = \frac{600}{6} = 100 \]

\( \text{Fe}_2(\text{SO}_4)_3 = 2\text{Fe} + 3\text{S} + 12\text{(O)} \)

\[ MW = 2 \times 55.8 + 3 \times 32.1 + 12 \times 16.0 = 400 \]

\[ EW = \frac{400}{6} = 66.7 \]

\( \text{Na}_2\text{CO}_3 = 2\text{Na} + 1\text{C} + 3\text{(O)} \)

\[ MW = 2 \times 23.0 + 12.0 + 3 \times 16.0 = 106 \]

\[ EW = \frac{106}{2} = 53 \]

(b) \( EW \text{ of } \text{NH}_4^+ = (14.0 + 4.0)/1.0 = 18.0 \)

\( EW \text{ of } \text{HCO}_3^- = (1.0 + 12.0 + 48.0)/1.0 = 61.0 \)

\( EW \text{ of } \text{CO}_3^{2-} = (12.0 + 48.0)/2.0 = 30.0 \)

Since \( \text{CO}_2 \) reacts with \( \text{H}_2\text{O} \) to form \( \text{H}_2\text{CO}_3 \), the electrical charge of \( \text{CO}_2 \) after dissolution is taken as 2.

\[ EW \text{ of } \text{CO}_2 = (12.0 + 32.0)/2 = 22.0 \]
11.2 (a) Water contains 50 mg/l of calcium ion and 15 mg/l of magnesium ion. Express the hardness as mg/l of CaCO₃. (b) Alkalinity in water consists of 150 mg/l of bicarbonate ion and 15 mg/l of carbonate ion. Express the alkalinity in units of mg/l of CaCO₃.

11.2 (a) Total hardness = $50 \frac{50}{20} + 15 \frac{50}{12.2} = 186$ mg/l

(b) Total alkalinity = $150 \frac{50}{61} + 15 \frac{50}{50} = 146$ mg/l

11.3 Draw a milliequivalent per liter bar graph and list the hypothetical combinations for the following analysis of a soft water:

<table>
<thead>
<tr>
<th>Component</th>
<th>mg/l</th>
<th>FW</th>
<th>meq/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca²⁺</td>
<td>36</td>
<td>20.0</td>
<td>1.80</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>14</td>
<td>12.2</td>
<td>1.15</td>
</tr>
<tr>
<td>Na⁺</td>
<td>43</td>
<td>23.0</td>
<td>1.87</td>
</tr>
<tr>
<td>K⁺</td>
<td>7</td>
<td>39.1</td>
<td>0.18</td>
</tr>
<tr>
<td>HCO₃⁻</td>
<td>208</td>
<td>61.0</td>
<td>3.41</td>
</tr>
<tr>
<td>SO₄²⁻</td>
<td>14</td>
<td>48.0</td>
<td>0.29</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>44</td>
<td>35.5</td>
<td>1.24</td>
</tr>
</tbody>
</table>

1.80 Ca(HCO₃)₂; 1.15 Mg(HCO₃)₂; 0.46 NaHCO₃
0.29 Na₂SO₄; 1.12 NaCl; 0.15 KCl

BC - 8
11.4 Draw a milliequivalents-per-liter bar graph and list the hypothetical combinations for the following analysis of a groundwater:

\[
\begin{align*}
\text{Ca}^{2+} &= 94 \text{ mg/l} & \text{HCO}_3^- &= 317 \text{ mg/l} \\
\text{Mg}^{2+} &= 24 \text{ mg/l} & \text{SO}_4^{2-} &= 67 \text{ mg/l} \\
\text{Na}^+ &= 14 \text{ mg/l} & \text{Cl}^- &= 24 \text{ mg/l}
\end{align*}
\]

11.4 Calcium = 94/20.0 = 4.7 meq/l  
Magnesium = 24/12.2 = 2.0  
Sodium = 14/23.0 = 0.6  
Bicarbonate = 317/61.0 = 5.2  
Sulfate = 67/48.0 = 1.4  
Chloride = 24/35.5 = 0.7

\[
\begin{array}{c|c|c}
\text{Ca} & \text{Mg} & \text{Na} \\
\hline
\text{HCO}_3^- & \text{SO}_4^- & \text{Cl}^- \\
\end{array}
\]

4.7 \text{Ca(HCO}_3^-}_2: 0.5 \text{Mg(HCO}_3^-)_2: 1.4 \text{MgSO}_4: 0.1 \text{MgCl}_2: 0.6 \text{NaCl}

\[BC - 9\]
Draw a milliequivalents-per-liter bar graph for the following water analysis:

- calcium hardness = 185 mg/l
- magnesium hardness = 50 mg/l
- sodium ion = 23 mg/l
- potassium ion = 20 mg/l
- alkalinity = 200 mg/l
- sulfate ion = 58 mg/l
- chloride ion = 36 mg/l
- pH = 7.7

11.5 Calcium hardness = \( \frac{185}{50.0} = 3.7 \) meq/l
Magnesium hardness = \( \frac{50}{50.0} = 1.0 \)
Sodium = \( \frac{23}{23.0} = 1.0 \)
Potassium = \( \frac{20}{39.1} = 0.51 \)
Alkalinity = \( \frac{200}{50.0} = 4.0 \)
Sulfate = \( \frac{58}{48.0} = 1.21 \)
Chloride = \( \frac{36}{35.5} = 1.01 \)

All alkalinity is in the form of bicarbonate ion since the pH is between 4.5 and 8.3.
Example 7-12

A chlorine dose of 1 mg/liter as Cl₂ satisfactorily disinfects a water at pH 7.0. What dose of chlorine would be required to achieve the same disinfection efficiency if the pH of the water was 8.5? Assume that HOCl is 100 times more effective as a disinfectant than OCl⁻ and that the temperature is 25°C.

Solution

From Fig. 7.21 at pH 7,

\[ \alpha_{\text{Cl}_2} = 0 \]
\[ \alpha_{\text{HOCl}} = 0.76 \]
\[ \alpha_{\text{OCl}^-} = 0.24 \]

Since \( C_{\text{T,Cl}_2} = 1 \) mg/liter as Cl₂, HOCl = 0.76 mg/liter as Cl₂, and OCl⁻ = 0.24 mg/liter.

The relative disinfection effectiveness, HOCl = 100, OCl⁻ = 1.0.

Disinfection effectiveness = concentration \times \text{relative disinfection effectiveness}

\[ = 0.76 \times 100 + 0.24 \times 1.0 \]
\[ = 76 \text{ units per mg/liter of chlorine} \]

From Fig. 7.21 at pH 8.5,

\[ \alpha_{\text{Cl}_2} = 0 \]
\[ \alpha_{\text{HOCl}} = 0.09 \]
\[ \alpha_{\text{OCl}^-} = 0.91 \]

The disinfection effectiveness is

\[ = 0.09 \times 100 + 0.91 \times 1.0 \]
\[ = 9.9 \text{ units per mg/liter of chlorine} \]

To achieve the same effectiveness of disinfection at pH 8.5, we would therefore need 76/9.9 = 7.7 times the dose applied at pH 7.

![Fig. 7.21. Distribution diagram for chlorine species, 25°C, [Cl⁻] \( = 10^{-3} \text{ M} \), \( C_{\text{T,Cl}_2} = [\text{Cl}_2] + [\text{HOCl}] + [\text{OCl}^-] \).](image-url)
Chlorination

Chlorine is the most widely used disinfectant because it is effective at low concentration, cheap, and forms a residual if applied in sufficient dosage. It may be applied as a gas or as a hypochlorite, the gas form being most common. The gas is liquified at five to ten atmospheres and shipped in steel cylinders. Pressurized liquid chlorine (99.8 percent Cl₂) is available in cylinders containing 100, 150, or 2000 lb of the liquifed gas. The disinfecting ability of chlorine is due to its powerful oxidizing properties, which oxidize those enzymes of microbial cells that are essential in the cells' metabolic processes (Butterfield, C. T., Wattie, E., Megregian, S., & Chambers, C. W. 1943).

Reaction
Chlorine gas reacts readily with water to form hypochlorous acid, HOCl, and hydrochloric acid:

\[ \text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{HOCl} + \text{HCl} \]  \hspace{1cm} (17.3)

In dilute solution and with pH greater than 3, the reaction is appreciably displaced to the right and very little molecular chlorine gas will remain dissolved and unreacted. The hypochlorous acid produced then dissociates to yield hypochlorite ion:

\[ \text{HOCl} \rightarrow \text{H}^+ + \text{OCl}^- \]  \hspace{1cm} (17.4)

The relative distribution of HOCl and OCl⁻ is a function of pH, as shown in Figure 17.3.

Hypochlorite salts are available in dry (calcium hypochlorite) or liquid (sodium hypochlorite) form. The dry form is cheaper but must be dissolved in water:

\[ \text{H}_2\text{O} \rightarrow \text{Ca(OCl)}_2 \rightarrow \text{Ca}^{+2} + 2\text{OCl}^- \]  \hspace{1cm} (17.5)

The OCl⁻ will then seek an equilibrium with the hydrogen ions as indicated in Eq. (17.4), and therefore when hypochlorites are used in such applications as swimming pools, it is often necessary to add acid.

Although both hypochlorous acid and hypochlorite ion are excellent disinfecting agents, the acid form is the more effective (Engelbrecht, R. S.). They also react with certain inorganic and organic materials in water. One of the important reactions is with ammonia:

\[ \text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{HOCl} + \text{HCl} \]

Figure 17.3. Relative Amounts of Chlorine as HOCl and OCl⁻ at 20°C versus pH
\[ \text{NH}_3 + \text{HOCl} \rightarrow \text{NH}_2\text{Cl} + \text{H}_2\text{O} \]
monochloramine \hspace{1cm} (17.6)

\[ \text{NH}_2\text{Cl} + 2\text{HOCl} \rightarrow \text{NHCl}_2 + 2\text{H}_2\text{O} \]
dichloramine \hspace{1cm} (17.7)

\[ \text{NHCl}_2 + 3\text{HOCl} \rightarrow \text{NCl}_3 + 3\text{H}_2\text{O} \]
nitrogen trichloride \hspace{1cm} (17.8)

The relative amounts of the various chloramines that are formed are mainly a function of the hypochlorous acid present and the pH. The monochloramine form predominates at a pH greater than about 6.0, and the dichloramine at about pH 5.

Reactions also occur with reduced substances and with organic materials. Dissolved chlorine gas will react with hydrogen sulfide to produce sulfuric and hydrochloric acids. It will react with other inorganic reducing ions or substances such as Fe^{2+}, Mn^{2+}, and NO_3^-\hspace{1cm} 1. Dissolved chlorine will also react with organic compounds, particularly unsaturated compounds. Two of the organo reactions are particularly important—those that result in chlorophenols and those that produce trithalothanethanes. Chlorophenols, formed from the reaction of chlorine with phenols, impart undesirable tastes and odors to water that are detectable at phenol concentrations less than one microgram per liter. Reaction of chlorine with innocuous humic substances results in the formation of trithalothanethanes including:

- CHCl_3 \hspace{1cm} chloroform
- CHCl_2Br \hspace{1cm} bromodichloromethane
- CHBrCl_2 \hspace{1cm} dibromochloromethane

These compounds are limited by drinking water regulations to an in toto 0.1 milligram per liter because of tumorogenic properties.

Chloramines are effective compounds against bacteria but are not nearly so effective against viruses. The difference in effectiveness of chloramines was illustrated in a bench study (Figure 17.4) by Kruse, Hsu, Griffiths, and Stringer (1970), who utilized a synthetic waste, Escherichia coli, and F2 coliphage. A plot of the data of Durham and Wolf (1973) shows the same type of results on effluents from two trickling filter plants using total coliforms and any phages accepted by E. coli K12 (') cells. The extrapolation of these findings on coliphages to all animal viruses is not warranted, but the results are meant to convey some inherent differences that can exist between bacteria and viruses in their susceptibility to chloramines.
Figure 17.4. Chloramine Effects on Bacteria and Virus

(a) Chloramine Effects, Synthetic Medium

(b) Chlorination Effects on Trickling Filter Effluent

Figure 17.5. Chlorine Dosages, Demands, and Residuals

Adapted from Chemistry for Environmental Engineers by C. N. Sawyer and P. L. McCarty

Dosages, Demands, and Residuals

The dosage is the amount of chlorine added, the demand is the amount used for oxidation of materials present, and the residual is the amount remaining after oxidation. The relationships between these are shown in Figure 17.5. The residual equals the dosage minus the demand.

Contact time is very important in the disinfection process. In chlorination, increased time of contact results not only in greater destruction of microorganisms but in an increased demand and, if appropriate precursors and free chlorine are present, in an increased amount of various chlorinated by-products.

Chlorine gas, hypochlorous acid, and the hypochlorite ion remaining after the demand is satisfied are collectively termed free chlorine residuals. The chloramines and other reactive chlorine forms remaining after the demand is satisfied are referred to as combined chlorine residuals. Free chlorine residuals are faster acting than combined residuals, and, for the same concentration and time, the free chlorine residuals have much greater disinfecting capacity than combined residuals, especially for viruses.

As depicted in Figure 17.5, an increase in chlorine dosage results in an evident increase in the residual up to a molar ratio of chlorine to ammonia nitrogen of 1:1. The residual formed is predominantly mono- and dichloramine. If the chlorine dosage is increased above this ratio, some nitrogen trichloride will be formed; however, as the dosage is increased, most of the chloramines will be oxidized to nitrogen gases. The oxidation reaction is essentially complete, for any particular point in time, at the minimum dip in the residual curve, which is termed the breakpoint. The breakpoint occurs at a chlorine dosage of about 1 1/2 to 2 moles of chlorine per 1 mole of ammonia nitrogen and represents the dosage when the chloramines have been converted to the nitrogen gases. Some of the gases have been identified as free nitrogen, nitrous oxide, and nitrogen trichloride, with free nitrogen being the most predominant. Continued addition of chlorine beyond the breakpoint gives a residual that is predominantly free chlorine.

The breakpoint dosage is very much dependent upon water quality, but for many drinking water supplies it ranges over 4 to 10 mg/l. The desirable residual to be maintained at the farthest tap on the distribution system is at least 0.2 mg/l., all free chlorine. The National Primary Drinking Water Regulations promote such a residual by allowing a reduction in the number of bacteriological samplings if that residual is maintained.

Wastewater chlorination practice will vary with states' policies. Some states utilize a fecal coliform criterion such as 200/100 ml. Other states specify a type of residual after a specific contact period. Texas specifies 1.0 mg/l total residual after 20 minutes of contact. Breakpoint chlorination of wastewater is seldom practiced. Dosages of 50–70 mg/l may be necessary to reach breakpoint in many wastewaters, and this renders the effluents highly toxic to much of the aquatic life (Brungs, W. A., 1973).

Chlorine Application

Gaseous chlorine may be dissolved in water using any one of a variety of proprietary chlorinators, and the concentrated solution is then piped to the water stream to be disinfected. Hypochlorites can be added using solution-type feeders. Dry hypochlorites are first dissolved in water in a plastic or clay vessel; the liquid is then decanted off by a solution-type feeder. Some hypochlorites are packaged in polymers to render them amenable to dry feed equipment, but for the usual size of installation these cannot compete economically with gaseous chlorine.

In water and wastewater treatment, prechlorination is the application of chlorine prior to any treatments, whereas postchlorination is chlorination after all treatments. Prechlorination is practiced to control undesirable growth such as might occur in a pipeline aqueduct. Similarly, prechlorination in wastewater treatment might be applied to sewers to control odors that develop as a result of undesirable growth. Postchlorination is sometimes called terminal disinfection.
Aluminum Sulfate (Filter Alum)

Aluminum sulfate is the standard coagulant used in water treatment. The commercial product strength ranges from 15% to 22% as Al₂O₃, with a hydration of about 14 moles of water. A formula used for filter alum is Al₂(SO₄)₃·14·3H₂O with a molecular weight of 600. The material is commonly shipped and fed in a dry granular form, although it is available as a powder or liquid alum syrup.

Aluminum sulfate reacts with natural alkalinity in water to form aluminum hydroxide floc:

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 14 \cdot 3\text{H}_2\text{O} + 3\text{Ca(HCO}_3\text{)}_2
\rightarrow 2\text{Al(OH)}_3 \downarrow + 3\text{CaSO}_4 + 14 \cdot 3\text{H}_2\text{O} + 6\text{CO}_2
\]  

Each mg/l of alum decreases water alkalinity by 0.50 mg/l (as CaCO₃) and produces 0.44 mg/l of carbon dioxide. Production of carbon dioxide is undesirable since this increases the corrosiveness of water.

If water does not contain sufficient alkalinity to react with the alum, lime or soda ash is fed to provide the necessary alkalinity.

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 14 \cdot 3\text{H}_2\text{O} + 3\text{Ca(OH)}_2
\rightarrow 2\text{Al(OH)}_3 \downarrow + 3\text{CaSO}_4 + 14 \cdot 3\text{H}_2\text{O}
\]

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 14 \cdot 3\text{H}_2\text{O} + 3\text{Na}_2\text{CO}_3 + 3\text{H}_2\text{O}
\rightarrow 2\text{Al(OH)}_3 \downarrow + 3\text{Na}_2\text{SO}_4 + 3\text{CO}_2 + 14 \cdot 3\text{H}_2\text{O}
\]

An advantage of using sodium carbonate (soda ash) is that unlike lime it does not increase water hardness, only corrosiveness. Lime, which is more popular, is less expensive than soda ash.

The dosage of alum used in water treatment is in the range 5–50 mg/l. The effective pH range for alum coagulation is 5.5–8.0. Alum is preferred in treating relatively high-quality surface waters because it is the only chemical needed for coagulation.

### 11.12 The results from a jar test for coagulation of a turbid alkaline raw water are given in the table. Each jar contained 1000 ml of water. The aluminum sulfate solution used for chemical addition had such strength that each milliliter of the solution added to a jar of water produced a concentration of 8.0 mg/l of aluminum sulfate. Based on the jar test results, what is the most economical dosage of aluminum sulfate in mg/l?

<table>
<thead>
<tr>
<th>Jar</th>
<th>Aluminum sulfate solution (ml)</th>
<th>Floc formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>Smoky</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>Fair</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>Good</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>Good</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>Very heavy</td>
</tr>
</tbody>
</table>

If another jar had been filled with freshly distilled water and dosed with 5 ml of aluminum sulfate solution, what would have been the degree of floc formation?

### 11.12 Jar 4 with a dosage \( \frac{4}{7} \times 8.0 = \frac{32}{7} \text{ mg/l} \) with distilled water, no coagulation occurs because of the lack of alkalinity for the chemical reaction, Eq. 11.34.
11.20 The water defined by the analysis given below is to be softened by excess lime treatment. (a) Sketch a meq/l bar graph. (b) Calculate the softening chemicals required. (c) Draw a bar graph for the softened water after recarbonation and filtration assuming 80% of the alkalinity is in the bicarbonate form.

\[
\begin{align*}
\text{CO}_2 & = 8.8 \text{ mg/l} & \text{Alk} (\text{HCO}_3^-) & = 135 \text{ mg/l} \\
\text{Ca}^{2+} & = 40.0 \text{ mg/l} & \text{SO}_4^{2-} & = 29.0 \text{ mg/l} \\
\text{Mg}^{2+} & = 14.7 \text{ mg/l} & \text{Cl}^- & = 17.8 \text{ mg/l} \\
\text{Na}^+ & = 13.7 \text{ mg/l} & \\
\end{align*}
\]

11.20 \[ \text{CO}_2 = \frac{8.8}{22.0} = 0.40 \text{ meq/l} \]

\[
\begin{align*}
\text{Ca} & = \frac{40.0}{20.0} = 2.00 & \text{Alk} & = \frac{135}{50.0} = 2.70 \\
\text{Mg} & = \frac{14.7}{12.2} = 1.21 & \text{SO}_4 & = \frac{29.0}{48.0} = 0.60 \\
\text{Na} & = \frac{13.7}{23.0} = 0.60 & \text{Cl} & = \frac{17.8}{35.5} = 0.51 \\
\end{align*}
\]

![Bar Graph](image)

<table>
<thead>
<tr>
<th>Component</th>
<th>meq/l</th>
<th>Lime</th>
<th>Soda ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO(_2)</td>
<td>0.40</td>
<td>0.40</td>
<td>0</td>
</tr>
<tr>
<td>Ca(\text{HCO}_3)(_2)</td>
<td>2.00</td>
<td>2.00</td>
<td>0</td>
</tr>
<tr>
<td>Mg(\text{HCO}_3)(_2)</td>
<td>0.70</td>
<td>1.40</td>
<td>0</td>
</tr>
<tr>
<td>Mg\text{SO}_4</td>
<td>0.51</td>
<td>0.51</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.31</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Lime required = 4.31 x 28 + 35 = 156 mg/l CaO

Soda ash required = 0.51 x 53 = 27 mg/l Na\(_2\)CO\(_3\)
Iron and Manganese Removal

Iron and manganese in concentrations greater than 0.3 mg/l of iron and 0.05 mg/l of manganese stain plumbing fixtures and laundered clothes. Although discoloration from precipitates is the most serious problem associated with water supplies having excessive iron and manganese, foul tastes and odors can be produced by growth of iron bacteria in water distribution mains. These filamentous bacteria, using reduced iron as an energy source, precipitate iron causing pipe incrustations. Decay of the accumulated bacterial slimes creates offensive tastes and odors.

Dissolved iron and manganese are often found in groundwater from wells located in shale, sandstone, and alluvial deposits. Impounded surface water supplies may also have troubles with iron and manganese. An anaerobic hypolimnion (stagnant bottom-water layer) in a reservoir dissolves precipitated iron and manganese from the bottom muds, and during periods of overturn these minerals are dispersed throughout the entire depth.

CHEMISTRY OF IRON AND MANGANESE

Iron (II) (Fe²⁺) and manganese (II) (Mn²⁺) are chemically reduced, soluble forms that exist in a reducing environment (absence of dissolved oxygen and low pH). These conditions exist in groundwater and anaerobic reservoir water. When it is pumped from underground or an anaerobic hypolimnion, carbon dioxide and hydrogen sulfide are released, raising the pH. In addition, the water is exposed to air, creating an oxidizing environment. The reduced iron and manganese start transforming to their stable, oxidized, insoluble forms of iron (III) (Fe³⁺) and manganese (IV) (Mn⁴⁺).

The rate of oxidation of iron and manganese depends on the type and concentration of the oxidizing agent, pH, alkalinity, organic content, and presence of catalysts [10].

Oxygen, chlorine, and potassium permanganate are the most frequent oxidizing agents. The natural reaction by oxygen is enhanced in water treatment by using spray nozzles or waterfall-type aerators. Chlorine and potassium permanganate (KMnO₄) are the chemicals commonly used in iron and manganese-removal plants. Oxidation reactions using potassium permanganate are

\[ 3\text{Fe}^{2+} + \text{MnO}_4^- \rightarrow 3\text{Fe}^{3+} + \text{MnO}_2 \]  \hspace{1cm} (11.55)

\[ 3\text{Mn}^{2+} + 2\text{MnO}_4^- \rightarrow 5\text{MnO}_2 \]  \hspace{1cm} (11.56)

Rates of oxidation of the ions depend on the pH and bicarbonate ion concentration. The pH for oxidation of iron should be 7.5 or higher. Manganese oxidizes readily at pH 9.5 or higher. Organic substances (i.e., humic or tannic acids) can create complexes with iron (II) and manganese (II) ions holding them in the soluble state to higher pH levels. If a large concentration
of organic matter is present, iron can be held in solution at pH levels of up to 9.5.

Copper ions and silica have a catalytic effect on the oxidation of iron and manganese [11]. The presence of about 0.1 mg/l of copper increases the rate of iron oxidation 5–6 times. Silica increases oxidation rates of both metals. Manganese oxides are catalytic in the oxidation of manganese. Tray aerators frequently contain coke or stone contact beds through which the water percolates. These media develop and support a catalytic coating of manganese oxides.

11.33 Untreated well water contains 1.2 mg/l of iron and 0.8 mg/l of manganese at a pH of 7.5. Calculate the theoretical dosage of potassium permanganate required for iron and manganese oxidation.

\[
\begin{align*}
1.2 \frac{1.0}{1.06} + 0.8 \frac{1.0}{0.52} &= 2.7 \text{ mg/l of KMnO}_4 \\
\end{align*}
\]
11.18 Presedimentation reduces the turbidity of a raw river water from 1500 mg/l suspended solids to 200 mg/l. How many pounds of dry solids are removed per million gallons? If the settled sludge has a concentration of 8% solids and a specific gravity of 1.03, calculate the sludge volume produced per million gallons of river water processed.

\[
\text{Dry solids} = (1500 - 200) \times 8.34 = 10,800 \text{ lb/mil gal}
\]

\[
\text{Volume} = \frac{10,800}{(0.08)(1.03)} = 131,000 \text{ gal/mil gal} \quad \text{(Refer to Eq. 13.3)}
\]

11.31 A small community has used an unchlorinated well-water supply containing approximately 0.3 mg/l of iron and manganese for several years without any apparent iron and manganese problems. A health official suggested that the town install chlorination equipment to disinfect the water and provide a chlorine residual in the distribution system. After initiating chlorination, consumers complained about water staining washed clothes and bathroom fixtures. Explain what is occurring due to chlorination.

11.31 Chlorine is oxidizing the iron and manganese forming precipitates that settle out in the distribution system. Periodically these are flushed out of mains and storage tanks to cause "rusty" water that results in staining. Apparently prior to chlorination most of the iron and manganese remained in solution and colloidal suspension and passed through the system unnoticed.
11.35 The results of a chlorine demand test on a raw water at 20°C are given in the following table.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Chlorine dosage (mg/l)</th>
<th>Residual chlorine after 10 min of contact (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.20</td>
<td>0.19</td>
</tr>
<tr>
<td>2</td>
<td>0.40</td>
<td>0.37</td>
</tr>
<tr>
<td>3</td>
<td>0.60</td>
<td>0.51</td>
</tr>
<tr>
<td>4</td>
<td>0.80</td>
<td>0.50</td>
</tr>
<tr>
<td>5</td>
<td>1.00</td>
<td>0.20</td>
</tr>
<tr>
<td>6</td>
<td>1.20</td>
<td>0.40</td>
</tr>
<tr>
<td>7</td>
<td>1.40</td>
<td>0.60</td>
</tr>
<tr>
<td>8</td>
<td>1.60</td>
<td>0.80</td>
</tr>
</tbody>
</table>

(a) Sketch the chlorine demand curve.
(b) What is the breakpoint chlorine dosage?
(c) What is the chlorine demand at a chlorine dosage of 1.20 mg/l?
11.51 List three possible methods for controlling crown corrosion in a large concrete sanitary sewer.

11.51 Crown corrosion in large concrete sewer pipe can be retarded by ventilation and chlorination to control hydrogen sulfide generation and synthetic coatings and linings. (Section 11.35)

11.57 Outlined below is the sequence of unit operations and chemical additions used in the treatment of a well-water supply. Briefly state the function or purpose of each unit process and the reason for each chemical addition.

1. Mixing and flocculation with the addition of lime.
2. Sedimentation.
3. Recarbonation.
5. Postchlorination.

11.57
1. Lime softening for precipitating carbonate hardness.
2. Sedimentation removes settleable solids.
3. Recarbonation stabilizes the water by lowering the pH.
4. Filtration to remove nonsettleable solids.
5. Postchlorination establishes a disinfecting residual.

11.58 Outlined below is the sequence of unit operations and chemical additions used in the treatment of a well-water supply. Briefly state the function or purpose of each unit process and the reason for each chemical addition.

1. Prechlorination at the wells.
2. Aeration over a tray aerator.
3. Rechlorination.
4. Detention in a settling basin.
5. Granular-media filtration.
6. Addition of anhydrous ammonia.

11.58
1. Prechlorination starts oxidation of reduced iron and manganese, suppresses growth of iron bacteria in pipelines, and controls algal growth on open aerators.
2. Aeration adds dissolved oxygen, removes carbon dioxide, and oxidizes iron and a portion of the manganese.
3. Rechlorination oxidizes the remainder of iron and manganese.
4. Detention allows chemical reaction time.
5. Filtration removes metal oxides.
6. Ammonia converts a portion of the free chlorine to combined chlorine residual.
11.59 Outlined below is the sequence of unit operations and chemical additions used in the treatment of a well-water supply. Briefly state the function or purpose of each unit process and the reason for each chemical addition.

1. Prechlorination at the wells.
2. Mixing–flocculation–sedimentation in flocculator–clarifiers using split treatment with lime and alum added to one leg and potassium permanganate to the other leg.
4. Postchlorination.

11.59
1. Prechlorination suppresses iron bacteria and initiates oxidation of iron and manganese.
2. Flocculator–clarifiers provide chemical mixing, flocculation and sedimentation. Lime provides partial precipitation softening, and alum coagulates the calcium carbonate precipitate. Potassium permanganate oxides iron and manganese.
3. Filtration removes suspended solids.
4. Postchlorination is to establish a disinfecting chlorine residual.

11.60 Consider the following sequence of unit operations and chemical additions used in the treatment of a river-water supply. Briefly state the function or purpose of each unit process and the reason for each chemical addition.

1. Presedimentation with polymer addition.
2. Activated carbon available when needed.
3. Mixing and flocculation with the addition of alum and polymer.
4. Sedimentation.
5. Addition of activated carbon.
7. Postchlorination.

11.60
1. Presedimentation reduces turbidity, particularly silt, by sedimentation. Polymer is added to enhance settling.
2. Activated carbon adsorbs taste and odor producing compounds.
3. Alum is a coagulant and polymer is a flocculation aid.
4. Sedimentation removes settleable floc.
5. Same as 2.
6. Filtration removes nonsettleable floc.
7. Postchlorination is for disinfection and to establish a disinfecting chlorine residual.
11.61 Outlined below is the sequence of unit operations and chemical additions used in the treatment of a reservoir water supply. Briefly state the function or purpose of each unit process and the reason for each chemical addition.

1. Intermittent applications of copper sulfate to reservoir during summer and fall.
2. Chlorine dioxide available when needed.
3. Mixing and flocculation with the addition of alum and polymer.
4. Sedimentation.
5. Addition of activated carbon.
7. Postchlorination.

11.61
1. Copper sulfate controls algal growth in reservoir water.
2. Chlorine dioxide is for oxidation of taste and odor producing compounds and for disinfection.
3. Alum and polymer are to remove turbidity.
4. Sedimentation removes settleable floc.
5. Activated carbon adsorbs taste and odor producing compounds.
7. Postchlorination is for disinfection and to establish a disinfecting chlorine residual.
WATER FILTRATION

> Settling > Filter > Clearwell

RAPID SAND FILTER

- Depth of sand bed, 3 feet or more
- Depth of filter bcd, 10 feet
- Hydraulic head, 1 to 8 feet
- Flow rate, historic value 2 - 10 gpm/sq ft
  modern rates up to 8 gpm/sq ft
  reasonable rate 4 gpm/sq ft (maximum design rate)
- Total Capacity per Filter, 350 gpm to 3,500 gpm
- Try to have at least 3 filters, be able to meet maximum demand with 2 filters (have a filter rccrvc)

Water Filtration - 1
Service to Rapid Sand Filter - Backwashing

Normal backwashing occurs after 1 to 3 days.
Backwashing occurs when head loss reaches some design value, usually a maximum of 8 feet.
Backwashing expands bed by up to 50%.
Backwash for 3 to 5 minutes at 8-15 gpm/sq ft.
Backwashing normally takes 1% to 5% of total filtered water.
(if it takes more improve upstream processes)

Water Filtration - 2
DIRECT FILTRATION

Direct filtration is filtration without pretreatment. Direct filtration is feasible in waters with less than 40 units of color, turbidity consistently below 5 units, iron and manganese concentrations of less than 0.3 and 0.05 mg/l, respectively, and algal counts below 2000 /ml.

Direct Filtration is normally done at rates "somewhat" lower than normal rates (10% to 25% less) for rapid or slow sand filters.

Water Filtration - 3
CLEARWELL

Storage volume = 30% to 60% of daily output with a minimum of 12 hours of maximum daily consumption. Clearwell capacity should meet demand when filters are being washed.

Water Filtration - 4
SLOW SAND FILTER

Thickness of sand layer 24" to 48"
Gravel layer 6" to 12"
Flow rate is 0.05 gpm/sq ft to 0.10 gpm/sq ft
Slow sand filters limited to low-turbidity applications with no chemical treatment
Usually no backwashing, filter cleaned by removing a few inches of sand.
Normal head loss is 0.2 to 4.0 feet

PRESSURE FILTERS

Pressure up to 25 feet of head loss
Filter rates of 2 to 4 gpm/sq ft
Backwash to clean

Water Filtration - 5
POTABLE WATER

WATER DEMAND

Table 7.10, p 7-16, text
Table 7.11, p 7-17, text

DESIGN WATER DEMAND

Distribution System - The distribution system is usually designed to handle the maximum hourly flow plus some future growth or the fire demand plus the maximum daily flow plus some future growth.

Treatment Plant - The treatment plant is usually designed to operate at a fixed, uniform rate. The plant capacity should be at least the maximum daily demand plus some part of the fire fighting demand and some future growth (part of fire fighting demand can come from storage).

Growth is up to 50% for small systems and 25% for large systems.

<table>
<thead>
<tr>
<th>Table 7.10</th>
<th>Table 7.11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual Average Water Requirements (gpcd)</td>
<td>Demand Multipliers For Peak Periods</td>
</tr>
<tr>
<td>(Excluding fire fighting)</td>
<td>consumption time/period</td>
</tr>
<tr>
<td>165-270 total</td>
<td>multiplier</td>
</tr>
<tr>
<td>residential</td>
<td>winter</td>
</tr>
<tr>
<td>commercial &amp; industrial</td>
<td>summer</td>
</tr>
<tr>
<td>public</td>
<td>maximum daily</td>
</tr>
<tr>
<td>loss &amp; waste</td>
<td>maximum hourly</td>
</tr>
<tr>
<td></td>
<td>early morning</td>
</tr>
<tr>
<td></td>
<td>noon</td>
</tr>
</tbody>
</table>

Potable Water # 1
EXAMPLE

Population = 2,500  Calculate the Maximum Hourly Flow for the Distribution System

\[
(2500) \times \frac{60}{24} \times \frac{60}{12} = 412,500 \text{ gal/day}
\]

\[
\frac{412,500}{24} = 17,187.5 \text{ gal/hr average}
\]

\[
\text{Summer Multiplier Max Hour Multiplier}
\]

\[
(17,187.5) \times (1.30)(3.02) = 67,031.2 \text{ gal/hr max}
\]

\[
\frac{67,031.2}{12} = 5,587 \text{ gal/min}
\]

\[
Q = 1020 \sqrt{P} (1 - 0.01 \sqrt{P}) = 1020 \sqrt{12.5} (1 - 0.01 \sqrt{12.5})
\]

\[
= 15,87 \text{ gal/min}
\]

\[
(15,87) \times \frac{60}{12} = 95,235.7 \text{ gal/hr fire}
\]

\[
\text{Growth (take total + 50% of total)}
\]

\[
67,031.2 + 95,235.7 = 162,766.9
\]

\[
\frac{162,766.9 + 81,133.5}{2} = 244,400 \text{ gal/hr}
\]

Design Flow for Distribution System

Potable Water 2
Calculate the Design Flow for the Water Treatment Plant

**Max Daily Demand**

\[
(250 \text{ people})(165 \text{ gpd}) = 41,250 \quad \text{gpd}
\]

\[
\frac{41,250 \text{ gpd}}{250 \text{ people}} = \frac{165 \text{ gpd}}{} \quad \text{per person}
\]

\[
(41,250 \text{ gpd})(1.30)(1.3) = 99,525 \quad \text{gpd}
\]

\[
\text{Summer Multiplier} \times \text{Max Day Multiplier}
\]

**Fire Demand (depends on storage)**

1587 gpm requires 2 hrs. Duration TABLE 7.13

\[
\frac{1587 \text{ gpm}}{2 \text{ hrs}} = 793,990 \quad \text{gpm}
\]

Maybe \( \frac{1}{2} \) comes from water plant and \( \frac{1}{2} \) comes from storage.

**Growth** (take total + 50% total)

\[
99,525 \quad \text{gpm}
\]

\[
95,220
\]

\[
1,030,470 \quad \frac{2}{3} + 545,235 = 1,613,005 \quad \text{gpm}
\]

Potable Water 3
A primary clarifier is to be designed to treat an industrial wastewater having 320 mg/l suspended solids and a flow of 2.0 MGD. A batch settling test was performed using an 8-in.-diameter column that was 10 ft long and had withdrawal ports every 2.0 ft. The reduced data giving the percent removals are as shown in Table 3.1.

Table 3.1. Percent Suspended Solids Removal at Given Depths

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>2 ft</th>
<th>4 ft</th>
<th>6 ft</th>
<th>8 ft</th>
<th>10 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>28</td>
<td>18</td>
<td>18</td>
<td>12</td>
<td>*</td>
</tr>
<tr>
<td>20</td>
<td>48</td>
<td>39</td>
<td>25</td>
<td>27</td>
<td>*</td>
</tr>
<tr>
<td>30</td>
<td>68</td>
<td>50</td>
<td>34</td>
<td>31</td>
<td>*</td>
</tr>
<tr>
<td>45</td>
<td>70</td>
<td>56</td>
<td>53</td>
<td>41</td>
<td>*</td>
</tr>
<tr>
<td>60</td>
<td>85</td>
<td>66</td>
<td>59</td>
<td>53</td>
<td>*</td>
</tr>
<tr>
<td>90</td>
<td>88</td>
<td>82</td>
<td>73</td>
<td>62</td>
<td>*</td>
</tr>
</tbody>
</table>

* Data showed an increase in solids concentration

Determine:

1. The design detention time and design overflow rate if 65 percent of the suspended solids are to be removed.
2. The design diameter and depth.

Solution

A plot of the percent removals at the various depths and times is shown in Figure 3.11. Interpolations have been

Figure 3.11. Graph Showing Suspended Solids Removal (as a Percent) at Various Depths and Settling Times, for Example Problem 3.1.
made to locate the 20, 30, 40, 50, 60, and 70 percent removal curves, and the curves have been drawn on the plot. The 20 percent curve intersects the x-axis at 16 minutes; thus, the overflow rate at that time is

\[ V_0 = \frac{10 \text{ ft}}{16 \text{ min}} \times 1440 \text{ min/day} \times 7.48 \text{ gal/ft}^3 = 6730 \text{ gal/day-ft}^3 \]

The detention time in hours is 16/60 or 0.27 hours. The point midway between the 20 and 30 percent curves at 16 min is located as shown and is at a depth of 6.7 ft. In a like manner, the points midway between the 30 and 40, 40 and 50, 50 and 60, and 60 and 70 percent curves are located and the respective depths are 2.9, 2.0, 1.3, and 0.8 ft. Using these values, the total fraction removed \( (R_f) \) at 16 min (0.27 hr) is

\[ R_f = 20 + (6.7/10)(30 - 20) + (2.9/10)(40 - 30) + (2.0/10)(50 - 40) + (1.3/10)(60 - 50) + (0.8/10)(70 - 60) - 33.7 \text{ percent} \]

Similarly, the overflow rates, detention times, and total fractions removed are computed for the 30, 40, 50, and 60 percent curves and a summary of the reduced data is shown in Table 3.2.

Table 3.2. Reduced Data for 30, 40, 50, and 60 Percent Curves

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>Overflow Rate ( V_0 ) (gal/day ft(^3))</th>
<th>Fraction Removed ( R_f ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.27</td>
<td>6730</td>
<td>33.7</td>
</tr>
<tr>
<td>0.55</td>
<td>3260</td>
<td>48.7</td>
</tr>
<tr>
<td>0.77</td>
<td>2340</td>
<td>56.7</td>
</tr>
<tr>
<td>1.13</td>
<td>1590</td>
<td>63.8</td>
</tr>
<tr>
<td>1.60</td>
<td>1120</td>
<td>68.6</td>
</tr>
</tbody>
</table>

A plot of the fraction removed \( (R_f) \) versus detention time \( (t) \) is shown in Figure 3.12. Also, a plot of the fraction removed \( (R_f) \) versus overflow rate \( (V_0) \) is shown in Figure 3.13. For 65 percent removal the detention time is 1.22 hr; thus, the design detention time is \( (1.22)(1.75) = 2.14 \text{ hr} \). For 65 percent removal the overflow rate is 1420 gal/day ft\(^3\); thus, the design overflow rate is \((1420)(0.65) = 923 \text{ gal/day-ft}^3\). The required area is

\[ A = \frac{2,000,000 \text{ gal}}{\text{day}} \times 923 \text{ gal/day-ft}^3 = 2167 \text{ ft}^3 \]

Thus, the diameter, \( D \), is

\[ D = \left[ \frac{4}{\pi (2167)} \right]^{1/2} = 52.5 \text{ ft or 55 ft for standard size} \]

The required depth, \( H \), is

\[ H = \left( \frac{2,000,000 \text{ gal}}{24 \text{ hr}} \right) \times \left( \frac{7.48 \text{ gal}}{\text{ft}^3} \right) \times \frac{4}{(55 \text{ ft})^3} - 10.03 \text{ ft} \]

Use 10 ft; 3 in.

Figure 3.12. Suspended Solids Removal versus Detention Time for Example Problem 3.1

Figure 3.13. Suspended Solids Removal versus Overflow Rate for Example Problem 3.1
A final clarifier is to be designed for an activated sludge plant treating an industrial wastewater having a design flow of 1.2 MGD. Batch settling studies have been performed in the laboratory using an acclimated culture of activated sludge and a graduate cylinder with a very slow rotating stirrer. The MLSS in the test was 2500 mg/l. The interface height versus settling time is shown in Figure 3.17. The design MLSS is 2500 mg/l and the design underflow concentration is 10.000 mg/l. Determine:

1. The area required for clarification.
2. The area required for thickening.
3. The design diameter.

Solution

A material balance for the recyle is

\[
(1.2 \text{ MGD})(0) + (R)(10.000) = (1.2 \text{ MGD} + R)(2500)
\]

Thus, \( R = 0.40 \text{ MGD} \). The interface height of the underflow is

\[
H_o = C_o h_o = (2500)(2.13/10.000) = 0.53 \text{ ft}
\]

The bisecting angle and tangent construction is shown on the graph, and the intersection within the \( H_o \) line gives a \( t_o \) value of 48 min. The settling velocity, \( V_o \), is 3.48 ft/hr. The area required for thickening is

\[
A_t = \frac{1.5(Q + R) t_o}{H_o} = \frac{1.5(1.60 \times 10^6 \text{ gal})}{1440 \text{ min} / 2.13 \text{ ft} / 7.48 \text{ gal}} = 5021 \text{ ft}^2
\]

The area required for clarification is

\[
A_c = \frac{Q}{V_o} = \frac{2.0(1.2 \times 10^6 \text{ gal})}{24 \text{ hr} / 3.48 \text{ ft} / 7.48 \text{ gal}} = 3842 \text{ ft}^2
\]

The area for thickening controls, thus the diameter, is

\[
D = \left[ \frac{4}{\pi} (5021 \text{ ft}^2) \right]^{1/2} = 80.0 \text{ ft}
\]

Use 80.0 ft for standard size.
Batch settling tests have been performed using an acclimated activated sludge to give the data in Table 3.3.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>C (mg/l)</th>
<th>V (ft/hr)</th>
<th>G = CV (lb/hr-ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12460</td>
<td>0.400</td>
<td>0.318</td>
</tr>
<tr>
<td>2</td>
<td>9930</td>
<td>0.817</td>
<td>0.506</td>
</tr>
<tr>
<td>3</td>
<td>7450</td>
<td>1.525</td>
<td>0.709</td>
</tr>
<tr>
<td>4</td>
<td>5220</td>
<td>3.281</td>
<td>1.068</td>
</tr>
<tr>
<td>5</td>
<td>3140</td>
<td>9.646</td>
<td>1.900</td>
</tr>
<tr>
<td>6</td>
<td>1580</td>
<td>13.710</td>
<td>1.351</td>
</tr>
</tbody>
</table>

The design mixed liquor flow to the final clarifier is 2530 gpm (gallons per minute), the MLSS is 2500 mg/l, and the underflow concentration is 12,000 mg/l. Determine the diameter of the final clarifier.

Solution

The settling curve showing the settling velocity versus solids concentration is shown in Figure 3.21. The flux curve showing the solids flux versus solids concentration is shown in Figure 3.22. A tangent to the curve drawn from \( C_0 = 12,000 \) mg/l gives a \( G_L \) value of 1.80 lb/hr-ft². Using a scale-up factor of 1.5 gives \( G_L = 1.80 \times 1.5 = 2.70 \) lb/hr-ft². The rate at which the solids settle, \( M_L \), is equal to \( Q_0 C_0 \), or \( M_L = (2530 \text{ gal/min})(60 \text{ min/hr})(8.34 \text{ lb/gal})(2500 \times 10^6) \) or 3165 lb/hr. From Eq. (3.42) the area required is \( M_L / G_L \), or \( A = (3165 \text{ lb/hr})(1.20 \text{ lb/hr-ft}^2) \) or 2638 ft². The required diameter is given by

\[
D = \left[ \frac{4}{\pi} \left(\frac{2638 \text{ ft}^2}{2}\right) \right]^{1/2} \quad 58.0 \text{ ft}.
\]

Use 60 ft for standard size.

---

**Figure 3.21. Example Problem 3.3**

**Figure 3.22. Example Problem 3.3**
Example Problem 3.3 SI
Final Clarifier

Batch settling tests have been performed using an acclimated activated sludge to give the data in Table 3.4.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>C (mg/l)</th>
<th>V (m/h)</th>
<th>G = CV (kg/h·m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12460</td>
<td>0.125</td>
<td>1.558</td>
</tr>
<tr>
<td>2</td>
<td>9930</td>
<td>0.249</td>
<td>2.473</td>
</tr>
<tr>
<td>3</td>
<td>7450</td>
<td>0.465</td>
<td>3.464</td>
</tr>
<tr>
<td>4</td>
<td>5220</td>
<td>1.000</td>
<td>5.220</td>
</tr>
<tr>
<td>5</td>
<td>3140</td>
<td>2.941</td>
<td>9.235</td>
</tr>
<tr>
<td>6</td>
<td>1580</td>
<td>4.180</td>
<td>6.604</td>
</tr>
</tbody>
</table>

The design mixed liquor flow to the final clarifier is 160 t/s, the mass is 2500 mg/l, and the underflow concentration is 12,000 mg/l. Determine the diameter of the final clarifier.

Solution

The settling curve showing the settling velocity versus solids concentration is shown in Figure 3.23. From the previous data, the flux curve showing the solids flux versus solids concentration is shown in Figure 3.24. A tangent to the curve drawn from C₀ = 12,000 mg/l gives a G₁ value of 8.90 kg/h·m². Using a scale-up factor of 1.5 gives Gₗ = 8.90/1.5 or 5.93 kg/h·m². The rate at which the solids settle, Mᵣ, is equal to QᵣC₀ or Mᵣ = (160 t/s)(60 s/min)(60 min/h)(2.50 g/l)(1000 g)/1440 kg/h. From Eq. (3.42) the area required is Mᵣ/Gₗ.

Of A = (1440 kg/h)(h·m²/5.93 kg) = 242.8 m². The required diameter is given by

\[ D = \left( \frac{4}{\pi} \times 242.8 \text{ m}^2 \right)^{1/2} \]

= 17.58 m

\[ S E R - S \]
precipitate out until the product of the remaining ion concentrations attains the standard value. If the product is less than the standard value, the solution is not saturated.

### Table 7.2
Approximate Solubility Products

<table>
<thead>
<tr>
<th>Substance</th>
<th>12°C</th>
<th>15°C</th>
<th>18°C</th>
<th>25°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al(OH)₃</td>
<td>4 EE-13</td>
<td>1.1 EE-13</td>
<td>3.7 EE-13</td>
<td>0.87 EE-13</td>
</tr>
<tr>
<td>CaCO₃</td>
<td>0.99 EE-8</td>
<td>0.87 EE-8</td>
<td>0.87 EE-8</td>
<td>0.87 EE-8</td>
</tr>
<tr>
<td>CaF₂</td>
<td>3.4 EE-11</td>
<td>4.0 EE-11</td>
<td>4.0 EE-11</td>
<td>4.0 EE-11</td>
</tr>
<tr>
<td>Fe(OH)₃</td>
<td>1.1 EE-36</td>
<td>1.1 EE-36</td>
<td>1.1 EE-36</td>
<td>1.1 EE-36</td>
</tr>
<tr>
<td>Mg(OH)₂</td>
<td>1.2 EE-11</td>
<td>1 EE-11</td>
<td>1 EE-11</td>
<td>1 EE-11</td>
</tr>
<tr>
<td>MgCO₃</td>
<td>2.6 EE-5</td>
<td>1 EE-5</td>
<td>1 EE-5</td>
<td>1 EE-5</td>
</tr>
<tr>
<td>BaCO₃</td>
<td>7 EE-9</td>
<td>8.1 EE-9</td>
<td>8.1 EE-9</td>
<td>8.1 EE-9</td>
</tr>
<tr>
<td>BaSO₄</td>
<td>0.87 EE-10</td>
<td>1 EE-10</td>
<td>1 EE-10</td>
<td>1 EE-10</td>
</tr>
<tr>
<td>CaSO₄</td>
<td>2 EE-4</td>
<td>2 EE-4</td>
<td>2 EE-4</td>
<td>2 EE-4</td>
</tr>
<tr>
<td>Fe(OH)₃</td>
<td>1.6 EE-14</td>
<td>8 EE-6</td>
<td>8 EE-6</td>
<td>8 EE-6</td>
</tr>
<tr>
<td>MnCO₃</td>
<td>1.8 EE-11</td>
<td>1.8 EE-11</td>
<td>1.8 EE-11</td>
<td>1.8 EE-11</td>
</tr>
<tr>
<td>Mn(OH)₂</td>
<td>4 EE-14</td>
<td>6 EE-9</td>
<td>6 EE-9</td>
<td>6 EE-9</td>
</tr>
<tr>
<td>MgF₂</td>
<td>7.1 EE-9</td>
<td>6 EE-9</td>
<td>6 EE-9</td>
<td>6 EE-9</td>
</tr>
</tbody>
</table>

**Example 7.12**

What is the solubility product of lead sulfate (PbSO₄) if its solubility is 38 mg/l?

\[ \text{PbSO}_4 \rightleftharpoons \text{Pb}^{2+} + \text{SO}_4^{2-} \quad \text{(in water)} \]

The molecular weight of PbSO₄ is 207.19 + 32.06 + (4)(16) = 303.25

The number of moles of PbSO₄ in a liter of saturated solution is

\[ 0.038/303.25 = 1.25 \text{ EE-4} \]

This is also the number of moles of Pb²⁺ and SO₄²⁻ that will form in the solution. Therefore,

\[ K_{sp} = [\text{Pb}^{2+}][\text{SO}_4^{2-}] = (1.25 \text{ EE-4})^2 = 1.56 \text{ EE-8} \]

The method used in example 7.12 to find the solubility product works well with chromates (CrO₄²⁻), halides (F⁻, Cl⁻, Br⁻, I⁻), sulfates (SO₄²⁻), and iodates (IO₃⁻). However, sulfides (S⁻⁻), carbonates (CO₃⁻⁻), phosphates (PO₄³⁻⁻), and the salts of transition elements (such as iron) hydrolyze and must be treated differently.

### 4 QUALITIES OF SUPPLY WATER

#### A ACIDITY AND ALKALINITY

**Acidity** is a measure of acids in solution. Acidity in surface water is caused by formation of carbonic acid \((\text{H}_2\text{CO}_3)\) from carbon dioxide in the air.¹

\[
\text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{H}_2\text{CO}_3
\]

\[
\text{H}_2\text{CO}_3 + \text{H}_2\text{O} \rightarrow \text{HCO}_3^- + \text{H}_3\text{O}^+ \quad (\text{pH} > 4.5) \quad 7.16
\]

\[
\text{HCO}_3^- + \text{H}_2\text{O} \rightarrow \text{CO}_3^{2-} + \text{H}_3\text{O}^+ \quad (\text{pH} > 8.3) \quad 7.17
\]

Measurement of acidity is done by titration with a standard basic measuring solution. Acidity in water is typically given in terms of the CaCO₃ equivalent that would neutralize the acid.

\[
\text{acidity (mg/l of CaCO}_3) = \left( \frac{\text{vol. titrant})(\text{titrant normality})(50.000)}{\text{vol. sample}} \right) \quad 7.18
\]

**Alkalinity** is a measure of the amount of negative ions in the water. Specifically, OH⁻, CO₃⁻⁻, and HCO₃⁻ all contribute to alkalinity.² The measure of alkalinity is the sum of concentrations of each of the substances measured as CaCO₃.

**Example 7.15**

Water from a city well is analyzed and is found to carry 20 mg/l as substance of HCO₃⁻ and 40 mg/l as substance of CO₃⁻⁻. What is the alkalinity of this water?

From appendix A, the factors converting HCO₃⁻ and CO₃⁻⁻ ions to CaCO₃ equivalents are 0.82 and 1.67 respectively.

\[
\text{alkalinity} = (0.82)(20) + (1.67)(40) = 83.2 \text{ mg/l as CaCO}_3
\]

Alkalinity can also be found by using an acidic titrant.

\[
\text{alkalinity (mg/l of CaCO}_3) = \left( \frac{\text{vol. titrant})(\text{titrant normality})(50.000)}{\text{vol. sample}} \right) \quad 7.19
\]

Alkalinity and acidity of a titrated sample is determined from color changes in indicators added to the titrant. Table 7.3 lists indicators that are commonly used.

---

¹ Carbonic acid is very aggressive and must be neutralized to eliminate the cause of water pipe corrosion. If the pH of water is greater than 4.5, carbonic acid ionizes to form bicarbonate (equation 7.16). If the pH is greater than 8.3, carbonic ions form which cause water hardness by combining with calcium (see equation 7.11).

² Other radicals, such as NO₃⁻, also contribute to alkalinity, but their presence is rare. If detected, they should be included in the calculation of alkalinity.
Table 7.3
Commonly Used Indicator Solutions

<table>
<thead>
<tr>
<th>Indicator Name</th>
<th>pH Visual Transition</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>cresol red</td>
<td>0.2-1.8</td>
<td>red</td>
</tr>
<tr>
<td>thymol blue</td>
<td>1.2-2.8</td>
<td>red</td>
</tr>
<tr>
<td>methyl yellow</td>
<td>2.4-4.0</td>
<td>red</td>
</tr>
<tr>
<td>bromophenol blue</td>
<td>3.0-4.6</td>
<td>yellow</td>
</tr>
<tr>
<td>methyl orange</td>
<td>3.2-4.4</td>
<td>red</td>
</tr>
<tr>
<td>methyl orange + xylene cyanole FF 40-56</td>
<td>(3.8-4.1)</td>
<td>violet</td>
</tr>
<tr>
<td>bromocresol green</td>
<td>3.9-5.4</td>
<td>yellow</td>
</tr>
<tr>
<td>methyl red</td>
<td>4.2-6.2</td>
<td>pink</td>
</tr>
<tr>
<td>methyl red + methylene blue 1-2</td>
<td>(~3.3)</td>
<td>red-violet</td>
</tr>
<tr>
<td>bromocresol purple</td>
<td>5.2-6.8</td>
<td>yellow</td>
</tr>
<tr>
<td>bromothymol blue</td>
<td>6.0-7.6</td>
<td>yellow</td>
</tr>
<tr>
<td>cresol red</td>
<td>6.8-8.2</td>
<td>yellow</td>
</tr>
<tr>
<td>phenol red</td>
<td>8.0-9.2</td>
<td>yellow</td>
</tr>
<tr>
<td>phenol-phenothalein</td>
<td>(8.0-9.8)</td>
<td>colorless</td>
</tr>
<tr>
<td>phenol-phenothalein + methylene green 1-2</td>
<td>(8.8)</td>
<td>green</td>
</tr>
<tr>
<td>thymolphenothalein</td>
<td>9.0-10.5</td>
<td>colorless</td>
</tr>
<tr>
<td>eriochrome black T</td>
<td>7.10</td>
<td>blue</td>
</tr>
<tr>
<td>alizarin yellow</td>
<td>10.1-12</td>
<td>yellow</td>
</tr>
</tbody>
</table>

*Screened indicator, neutral gray at stated pH

*Based on addition of 1 or 2 drops of 0.1% indicator solution to 10 ml of aqueous solution.

*Screened indicator, pale blue at stated pH

Example 7.14
0.02N sulfuric acid is used to titrate 110 ml of water. 3.3 ml of titrant is needed to reach the phenolphthalein point, and 13.2 ml is needed to reach the methyl orange point. What are the total and phenolphthalein alkalinitities?

From equation 7.19, the phenolphthalein alkalinity is

$$\frac{(3.3)(0.02)(50.000)}{(33.3)} = 30 \text{ mg/l as CaCO}_3$$

The total alkalinity is

$$\frac{(13.2 + 3.3)(0.02)(50.000)}{(33.3)} = 150 \text{ mg/l as CaCO}_3$$

The alkalinity of 150 mg/l is caused by carbonates ($2 \times 30 = 60 \text{ mg/l}$) and bicarbonates ($150 - 60 = 90 \text{ mg/l}$).

B. HARDNESS

Water hardness is caused by multi-valent (doubly-charged, triply-charged, etc., but not singly-charged) positive metallic ions such as calcium, magnesium, iron, and manganese. (Iron and manganese are not as common, however.) Hardness reacts with soap to reduce its cleansing effectiveness, and to form scum on the water surface and ring around the bathtub.

Water containing bicarbonate ($\text{HCO}_3^-$) ions can be heated to precipitate a carbonate molecule. This is known as temporary hardness or carbonate hardness. When heated, the reaction is:

$$\text{Ca}^{++} + 2\text{HCO}_3^- + \text{heat} \rightarrow \text{CaCO}_3 + \text{CO}_2 + \text{H}_2\text{O} \quad 7.20$$

$$\text{Mg}^{++} + 2\text{HCO}_3^- + \text{heat} \rightarrow \text{MgCO}_3 + \text{CO}_2 + \text{H}_2\text{O} \quad 7.21$$

Remaining hardness due to sulfates, chlorides, and nitrates is known as permanent hardness or non-carbonate hardness because it cannot be removed by heating. Permanent hardness can be calculated numerically by causing precipitation, drying, and then weighing the precipitate.

$$\text{Ca}^{++} + \text{SO}_4^{2-} + \text{Na}_2\text{CO}_3 \rightarrow 2\text{Na}^+ + \text{SO}_4^{2-} + \text{CaCO}_3 \quad 7.22$$

$$\text{Mg}^{++} + 2\text{Cl}^- + 2\text{NaOH} \rightarrow 2\text{Na}^+ + 2\text{Cl}^- + \text{Mg(OH)}_2 \quad 7.23$$

Total hardness is the sum of temporary and permanent hardnesses, both expressed in mg/l as CaCO$_3$.

Hardness can also be measured by the titration method using a titrant (complexion, versene, EDTA, or BDH).

---

3 Hard water forms scale when heated. This scale, if it forms in pipes, eventually restricts water flow. Even in small quantities, the scale insulates boiler tubes. Therefore, water used in steam-producing equipment must be essentially hardness-free.

4 The hardness is known as carbonate hardness even though it is caused by bicarbonate radicals, not carbonate radicals.
and an indicator (such as eriochrome black T). The standard hardness reagent used for titration has an equivalent hardness of 1 mg/l per ml used.

The hardness of water can be classified according to table 7.4. Although high values of hardness are not organically dangerous, public acceptance of the water supply requires a hardness of well below 150 mg/l. Except for special industrial uses, potable water should have the carbonate hardness reduced to at least 40 mg/l, and the total hardness should be below 75 mg/l. Where it is economically feasible, the carbonate hardness should be reduced to 25 mg/l.

Table 7.4
Hardness Classifications

<table>
<thead>
<tr>
<th>class</th>
<th>type</th>
<th>hardness</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>soft</td>
<td>below 60 mg/l</td>
</tr>
<tr>
<td>B</td>
<td>medium hard</td>
<td>60-120 mg/l</td>
</tr>
<tr>
<td>C</td>
<td>hard</td>
<td>120-180 mg/l</td>
</tr>
<tr>
<td>D</td>
<td>very hard</td>
<td>180-350 mg/l</td>
</tr>
<tr>
<td>E</td>
<td>saline, brackish</td>
<td>above 350 mg/l</td>
</tr>
</tbody>
</table>

Example 7.15

A 75 ml water sample required 8.1 ml of EDTA. What is the hardness?

\[
\text{hardness} = \frac{(8.1 \text{ ml})(1 \text{ mg/l})}{(75/1000)} = 108 \text{ mg/l}
\]

Example 7.16

Water is found to contain sodium (Na⁺, 15 mg/l), magnesium (Mg²⁺, 70 mg/l), and calcium (Ca²⁺, 40 mg/l). What is the hardness?

Sodium is singly-charged, so it does not contribute to hardness. The approximate equivalent weights of the relevant compounds and elements are:

\[\text{Mg} : 12 \quad \text{Ca} : 20 \quad \text{CaCO}_3 : 50\]

The equivalent hardness is

\[
(60) \left(\frac{50}{12}\right) + (40) \left(\frac{50}{20}\right) = 392 \text{ mg/l as CaCO}_3
\]

Alternatively, appendix A could have been used to convert the ionic concentrations to CaCO₃ equivalents.

\[
(40)(10) + (40)(2.50) = 387 \text{ mg/l as CaCO}_3
\]

C. IRON CONTENT

Even in low concentrations, iron is objectionable because it stains bathroom fixtures, causes a brown color in laundered clothing, and affects taste. Water originally pumped from anaerobic sources may contain (Fe²⁺) ferrous ions which are invisible and soluble when exposed to oxygen. Insoluble (Fe³⁺) ferric oxides form which give water the rust coloration.

Iron is measured optically by comparing the color of a sample with standard colors. The comparison can be made by eye or with a photoelectric colorimeter. Iron concentrations greater than 0.3 mg/l are undesirable.

D. MANGANESE CONTENT

Manganese ions are similar in effect, detection, and measurement to iron ions. Manganous manganese (Mn⁺⁺) oxidizes to manganic manganese (Mn⁴⁺⁺) to give water a rust color. An undesirable concentration is 0.05 mg/l.

E. FLUORIDE CONTENT

An optimum concentration of fluoride in the form of a fluoride ion, F⁻, is between 0.8 mg/l for hot climates (80°F-90°F average) to 1.2 mg/l for cold climates (50°F average). These amounts reduce the population cavity rate to a minimum without producing significant fluorosis (staining) of the teeth. The actual amount of fluoridation depends on the average outside temperature since the temperature affects the amount of water that is ingested by the population.

Table 7.5
Maximum Fluoride Concentrations

(Note that the 1974 Safe Drinking Water Act and its 1986 amendments set the maximum fluoride concentration at 4 mg/l for all temperatures.)

<table>
<thead>
<tr>
<th>5-year average of maximum daily air temperatures</th>
<th>fluoride concentrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>(deg F)</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>50.0-53.7</td>
<td>2.4</td>
</tr>
<tr>
<td>53.8-58.3</td>
<td>2.2</td>
</tr>
<tr>
<td>58.4-63.8</td>
<td>2.0</td>
</tr>
<tr>
<td>63.9-70.6</td>
<td>1.8</td>
</tr>
<tr>
<td>70.7-79.2</td>
<td>1.6</td>
</tr>
<tr>
<td>79.3-90.5</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Fluoridation can be obtained by the readily dissociating compounds in table 7.6.
Table 7.6
Fluoridation Chemicals

\[(NH_4)_2SiF_6\] (ammonium silicofluoride)
CaF_2 (calcium fluoride)
H_2SiF_6 (fluosilicic acid)
NaF (sodium fluoride)
Na_2SiF_6 (sodium silicofluoride)

Fluoride content is measured by colorimetric and electrical methods.

F. CHLORIDE CONTENT

Chlorine is used as a disinfectant for water, but its strong oxidation potential allows it also to be used to remove iron and manganese ions. Chlorine gas in water forms hypochlorous and hydrochloric acids.

\[\text{Cl}_2 + \text{H}_2\text{O} \xrightarrow{pH<4} \text{HCl} + \text{HOCl} \xrightarrow{pH>9} \text{H}^+ + \text{OCl}^- \]

7.24

Free chlorine, hypochlorous acid, and hypochlorite ions are known as free chlorine residuals. Hypochlorous acid reacts with ammonia (if it is present) to form mono-, di-, and trichloramines. Chloramines are known as combined residuals. Chloramines are more stable than free residuals, but their disinfecting ability is weaker. Their action may extend for a considerable distance into the distribution system.

The amount of chlorine to be added depends on the organic and inorganic matter present in the water. However, most waters are effectively treated within 10 minutes if a free residual of 0.2 mg/l is maintained. Larger residual concentrations may cause objectional odor and taste.

If the water contains phenol, it and the chlorine will form chlorophenol compounds which produce an objectionable taste. This may be stopped by adding ammonia to the water before chlorination.

Both free and combined residual chlorine can be detected by color comparison. However, organic matter in waste water makes it necessary to use a test based on water conductivity. The color comparison test with supply water, however, is adequate.

G. PHOSPHORUS CONTENT

Orthophosphates (H_2PO_4^-, HPO_4^{2-}, and PO_4^{3-}) and polyphosphates (such as Na_2(P_2)O_7) result from the use of synthetic detergents (synets). Phosphate content is more of a concern in waste water than in supply water.

Excessive phosphate discharge contributes to aquatic plant growth and subsequent eutrophication.

Phosphates are measured by a variety of means, including colorimetry and filtered precipitation analysis. Care should be taken not to confuse phosphates with phosphorus. Multiply mg/l of phosphate by 0.326 to obtain mg/l of phosphorus.

H. NITROGEN CONTENT

Nitrogen is present in water in many forms, including organic (protein), ammonia, nitrate, and gaseous ammonia. As with phosphates, nitrogen contamination is more of a problem with waste water than with supply water. Nitrogen pollution promotes algae growth. Ammonia is toxic to fish.

Drinking water is typically tested only for nitrates. The following tests are used:

Table 7.7
Tests for Nitrogen

<table>
<thead>
<tr>
<th>to test for</th>
<th>procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>ammonia</td>
<td>distillation</td>
</tr>
<tr>
<td>organic nitrogen</td>
<td>digestion with distillation</td>
</tr>
<tr>
<td>nitrate, nitrite</td>
<td>colorimetry</td>
</tr>
</tbody>
</table>

Gaseous nitrogen is of little concern since it is not normally metabolized by plants and it is of no danger to animal or human life.

I. COLOR

Color in domestic water is undesirable aesthetically, and it may dull the color of clothes and stain bathroom fixtures. Some industries (such as beverage production, dairy, food processing, paper manufacturing, and textile production) also have strict water color standards.

Water color is measured with a colorimeter or colorimetrically with tubes containing standard platinum/cobalt solutions. Color is graded on a scale of 0 (clear) to 70 color units.

I. TURBIDITY

Turbidity is a measure of the insoluble solids (soil, organics, and microorganisms) in water which impede light passage. Completely clean water measures 0 turbidity units (NTU). 5 NTU is noticeable to an average consumer, and this is a practical upper limit for drinking water. Muddy water exceeds 100 NTU. A TU is equivalent to 1 mg/l of silica in suspension.
K. SUSPENDED AND DISSOLVED SOLIDS

Solids present in a sample of drinking water can be divided into several categories, not all of which are mutually exclusive:

- **Suspended solids**: Suspended solids, the same as filterable solids, are measured by filtering a sample of water and weighing the residue.

- **Dissolved solids**: Dissolved solids, same as non-filterable solids, are measured as the difference between total solids and suspended solids.

- **Total solids**: Total solids are made up of suspended and dissolved solids. They are measured by drying a sample of water and weighing the residue.

- **Volatile solids**: Volatile solids are measured as the decrease in weight of total solids which have been ignited in an electric furnace.

- **Fixed solids**: The fixed solids can be found as the difference between total solids and volatile solids.

- **Settleable solids**: The volume (ml/l) of settleable solids is measured by allowing a sample to stand for one hour in a graduated conical container (Imhoff cone).

An upper limit of 500 mg/l of total solids is recommended.

---

**Table 7.8**

Water-Borne Organisms

<table>
<thead>
<tr>
<th>this organism</th>
<th>causes this disease</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BACTERIA</strong></td>
<td></td>
</tr>
<tr>
<td>Salmonella typhosa</td>
<td>typhoid fever</td>
</tr>
<tr>
<td>Vibrio comma</td>
<td>cholera</td>
</tr>
<tr>
<td>Shigella dysenteriae</td>
<td>dysentery</td>
</tr>
<tr>
<td>Escherichia coli</td>
<td>enteric problems</td>
</tr>
<tr>
<td>fecal streptococci</td>
<td>enteric problems</td>
</tr>
<tr>
<td><strong>VIRUSES</strong></td>
<td></td>
</tr>
<tr>
<td>Poliomyelitis</td>
<td>polio</td>
</tr>
<tr>
<td>Infectious hepatitis</td>
<td>hepatitis</td>
</tr>
<tr>
<td><strong>PROTOZOA</strong></td>
<td></td>
</tr>
<tr>
<td>Entamoeba histolytica</td>
<td>dysentery</td>
</tr>
<tr>
<td><strong>PARASITES</strong></td>
<td></td>
</tr>
<tr>
<td>flatworms</td>
<td>Schistosomiasis</td>
</tr>
<tr>
<td>flatworms</td>
<td>Bilharziasis</td>
</tr>
</tbody>
</table>

L. WATER-BORNE DISEASES

Organisms that are present in water consist of bacteria, fungi, viruses, algae, protozoa, and multicellular animals. Not all of these are dangerous, but some important organisms are listed in Table 7.8.

5 TRIHALOMETHANES

Trihalomethanes (THM’s) are organic chemicals produced during the disinfection of water. The chemically active elements of chlorine, iodine, and bromine react with various organic precursors to produce THM’s. However, iodine is seldom used in disinfection. Therefore, only four THM’s are found in significant quantities.5

- CHCl₃ - trichloromethane (chloroform)
- CHBr₃ - tribromomethane (bromoform)
- CHBrCl₂ - bromodichloromethane
- CHBr₂Cl - dibromochloromethane

The organic precursors which react with chlorine to produce THM’s tend to be naturally occurring. For example, decaying vegetation produces humic and fulvic acids which are natural precursors. The precursors themselves are not harmful, but the THM’s produced from them have been shown to be carcinogenic.6

Table 7.9 lists the maximum contaminant level (MCL) for THM’s in drinking water. Communities with populations of 10,000 and above are covered by the MCL if they add a disinfectant to their drinking water supply. The MCL reported in Table 7.9 is for total THM (TTHM), not just chloroform. Precursors are not limited or monitored.

When THM levels need to be reduced, several options are available. These options fall into two categories, depending on whether the precursors are removed prior to chlorination, or a disinfectant is chosen that does not produce THM’s. The first category includes the following options:

- Using granular activated carbon (GAC), other adsorbents, or filters, including weak-base resins, to remove precursors
- Selecting a water source with fewer precursors

---

5 Bromine can be present in gaseous chlorine as an impurity. Bromine also results from reacting chlorine with the bromide present in high-salinity water.

6 Actually, tests have shown that only chloroform in high doses is carcinogenic to rats and mice. The other THM’s are considered carcinogenic by association.
- Moving the chlorination point to the end of the treatment process so that most precursors are removed prior to disinfection (70-75% TTHM reduction)
- Optimizing coagulation and settling processes to improve precursor removal

Within the second category are the following options:
- Using ozone, chlorine dioxide, or potassium permanganate to disinfect without THM formation (60-90% TTHM reduction)
- Dechlorination using sodium metabisulfate or other methods after chlorination to prevent the reaction of chlorine with precursors
- Adding ammonia to water prior to discharge to induce chloramine formation, since chloramines suppress the formation of THM's

25-60% TTHM's can also be removed after formation by contacting with granular activated carbon.

Caution is required when changing to alternate disinfectants. The disinfectant and its byproducts should be evaluated to determine disinfecting power, residual power, toxicity, and other health effects.\(^7\)

Costs of operation will increase when alternate disinfectants are used. Moving the point of application may not result in any significant operating costs after a modest capital expenditure is made. Cost of using ozone as an alternative disinfectant is often less than using chlorine dioxide, but it is more than using chloramines or changing the points of chlorine application.

6 COMPARISON OF ALKALINITY AND HARDNESS

Hardness measures the presence of Mg\(^{++}\), Ca\(^{++}\), Fe\(^{++}\), and other multi-valent ions. Alkalinity measures the presence of HCO\(_3^-\), SO\(_4^{2-}\), Cl\(^-\), NO\(_3^-\), and OH\(^-\) ions. Both positive and negative ions can exist side by side, so an alkaline water can also be hard.

If certain assumptions are made, then it is possible to draw conclusions about the water composition. For example, Fe\(^{+++}\) is an unlikely ion in most water supplies, and it is often neglected in comparing alkalinity and hardness.

Figure 7.1 gives an easy method of comparing hardness and alkalinity, and using the comparison to deduce other compounds in the water.

If hardness (as CaCO\(_3\)) and alkalinity (also as CaCO\(_3\)) are the same, then there are no SO\(_4^{2-}\), Cl\(^-\), or NO\(_3^-\) ions present. (That is, there is no non-carbonate, permanent hardness.) If hardness is greater than alkalinity, however, then non-carbonate, permanent hardness is present, and the temporary carbonate hardness is equal to the alkalinity. If hardness is less than alkalinity, then all hardness is carbonate, temporary hardness, and the extra HCO\(_3^-\) comes from other sources (such as NaHCO\(_3\)).

\[ M = \text{alkalinity} \]
\[ H = \text{total hardness} \]
\[ Ca = \text{calcium} \]
\[ O = \text{hydroxides} \]
\[ S = \text{sulfate hardness} \]
\[ L = \text{free lime} \]

\[ \text{magnesium carbonate} = H - Ca \]
\[ \text{calcium carbonate} = Ca \]
\[ \text{sodium hydrate} = O \]
\[ \text{sodium carbonate} = S - O \]

\[ \text{magnesium bicarbonate} = M - H \]
\[ \text{calcium bicarbonate} = Ca \]
\[ \text{calcium bicarbonate} = Ca \]
\[ \text{sodium carbonate} = S - O \]

\[ \text{calcium carbonate} = H - O + S \]
\[ \text{calcium hydrate} = S \]
\[ \text{calcium hydrate} = H - M \]
\[ \text{calcium carbonate} = H - O + S \]
\[ \text{calcium hydrate} = S \]

\[ \text{calcium carbonate} = H - O + S \]
\[ \text{calcium carbonate} = H - O + S \]
\[ \text{calcium carbonate} = H - O + S \]

\[ \text{magnesium sulfate} = H - Ca \]
\[ \text{calcium sulfate} = Ca - M \]
\[ \text{calcium carbonate} = Ca \]
\[ \text{calcium bicarbonate} = Ca \]
\[ \text{calcium carbonate} = Ca \]

\[ \text{magnesium sulfate} = H - M \]
\[ \text{magnesium bicarbonate} = M - Ca \]
\[ \text{calcium bicarbonate} = Ca \]
\[ \text{calcium carbonate} = Ca \]
\[ \text{magnesium carbonate} = H - M \]

Figure 7.1 Hardness and Alkalinity
(All results expressed as CaCO\(_3\))

43
Example 7.17

A sample of water has been found to contain the following:

- alkalinity 220 mg/l as CaCO₃
- hardness 180 mg/l as CaCO₃
- calcium (Ca⁺⁺) 140 mg/l as CaCO₃

(a) What is the non-carbonate hardness?

(b) What is the Mg⁺⁺ content in mg/l as substance?

To use figure 7.1, the absence of any significant hydroxides must be assumed. Since the alkalinity is greater than the hardness, the figure indicates the following compounds in the water:

\[ \text{NaHCO}_3 = 220 - 180 = 40 \text{ mg/l as CaCO}_3 \]
\[ \text{Mg(HCO}_3)_2 = 180 - 140 = 40 \text{ mg/l as CaCO}_3 \]
\[ \text{Ca(HCO}_3)_2 = 140 \text{ mg/l} \]

There is no non-carbonate hardness in the water.

The Mg⁺⁺ ion content in CaCO₃ is equal to the Mg(HCO₃)₂ content as CaCO₃. Appendix A can be used to convert CaCO₃ equivalents to amounts as substance.

\[ \text{Mg}^{++} = \frac{30}{4.1} = 9.6 \text{ mg/l as substance} \]

### 7 WATER QUALITY STANDARDS

Minimum drinking water quality standards have been set by the Safe Drinking Water Act. Typical minimum standards are given in Table 7.9 as Maximum Contaminant Levels (MCL’s). Values in Table 7.9 are subject to change as new legislation is enacted.

### 8 WATER DEMAND

Water demand comes from a number of sources, including residential, commercial, industrial, and public consumers, as well unavoidable loss and waste. In project planning, a minimum of about 165 gallons per capita day should be considered. This 165 gpd is a total of all demands, as given in Table 7.10. If large industries are present (such as canning, steel making, automobile production, electronics, etc.), then these industries’ special needs must also be considered.

For ordinary domestic use, the water pressure should be 25 to 40 psi. A minimum of 60 psi at the fire hydrant is usually adequate, since that allows for up to 20 psi pressure drop in fire hoses. 75 psi and higher is common in commercial and industrial districts.

### Table 7.9
Typical Water Quality Standards
(subject to change)

<table>
<thead>
<tr>
<th>contaminant/quality</th>
<th>MCL</th>
</tr>
</thead>
<tbody>
<tr>
<td>inorganic compounds</td>
<td></td>
</tr>
<tr>
<td>arsenic</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>barium</td>
<td>1.0 mg/l</td>
</tr>
<tr>
<td>cadmium</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>chloride</td>
<td>250 mg/l</td>
</tr>
<tr>
<td>chromium</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>copper</td>
<td>1.0 mg/l</td>
</tr>
<tr>
<td>cyanide</td>
<td>0.005 mg/l</td>
</tr>
<tr>
<td>iron</td>
<td>0.3 mg/l</td>
</tr>
<tr>
<td>lead</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>manganese</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>mercury</td>
<td>0.002 mg/l</td>
</tr>
<tr>
<td>nitrate</td>
<td>10 mg/l</td>
</tr>
<tr>
<td>selenium</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>silver</td>
<td>0.05 mg/l</td>
</tr>
<tr>
<td>sulfate</td>
<td>250 mg/l</td>
</tr>
<tr>
<td>zinc</td>
<td>5.0 mg/l</td>
</tr>
<tr>
<td>organic compounds</td>
<td></td>
</tr>
<tr>
<td>trihalomethanes (total)</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>organic pesticides</td>
<td></td>
</tr>
<tr>
<td>endrin</td>
<td>0.0002 mg/l</td>
</tr>
<tr>
<td>lindane</td>
<td>0.004 mg/l</td>
</tr>
<tr>
<td>methoxychlor</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>toxaphene</td>
<td>0.005 mg/l</td>
</tr>
<tr>
<td>2.4-D</td>
<td>0.1 mg/l</td>
</tr>
<tr>
<td>2.4,5-TP(silvex)</td>
<td>0.01 mg/l</td>
</tr>
<tr>
<td>miscellaneous regulations</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>6.5-8.5</td>
</tr>
<tr>
<td>turbidity</td>
<td>1 NTU</td>
</tr>
<tr>
<td>color</td>
<td>15 units</td>
</tr>
<tr>
<td>microbiological</td>
<td>1 coliform/100 ml</td>
</tr>
<tr>
<td>total dissolved solids</td>
<td>500 mg/l</td>
</tr>
<tr>
<td>odor (in threshold)</td>
<td>3 T.O.N.</td>
</tr>
<tr>
<td>odor numbers</td>
<td></td>
</tr>
</tbody>
</table>

### Table 7.10
Annual Average Water Requirements (gpd)

(Excluding fire fighting)

<table>
<thead>
<tr>
<th>demand</th>
<th>gpd</th>
</tr>
</thead>
<tbody>
<tr>
<td>residential</td>
<td>75-130</td>
</tr>
<tr>
<td>commercial &amp;</td>
<td></td>
</tr>
<tr>
<td>industrial</td>
<td>70-100</td>
</tr>
<tr>
<td>public</td>
<td>10-20</td>
</tr>
<tr>
<td>loss &amp; waste</td>
<td>10-20</td>
</tr>
<tr>
<td></td>
<td>165-270 total</td>
</tr>
</tbody>
</table>
Table 7.18
Characteristics of Aerators

<table>
<thead>
<tr>
<th>type</th>
<th>operating head (ft)</th>
<th>loading capacity</th>
<th>efficiency in CO₂ removal</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>spray</td>
<td>8-28</td>
<td>4 180 gpm nozzle</td>
<td>high</td>
<td>requires protection from loss of water by wind, ice hazard in cold climates</td>
</tr>
<tr>
<td>cascade</td>
<td>3-10</td>
<td>20 50 gpm/sq ft</td>
<td>low to fair</td>
<td>requires large space</td>
</tr>
<tr>
<td>perforated tray</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>coke tray</td>
<td>6 10</td>
<td>&lt;35 gpm sq ft</td>
<td>high</td>
<td>requires larger space and higher head than coke tray</td>
</tr>
<tr>
<td>forced draft</td>
<td>10-25</td>
<td>16 18 gpm sq ft</td>
<td>high</td>
<td>compact but more complex than above types</td>
</tr>
<tr>
<td>diffused air</td>
<td>5-10 psi¹</td>
<td>0.02 0.2 cfm/gpm²</td>
<td>high</td>
<td>requires compressed air, most complex</td>
</tr>
</tbody>
</table>

¹ Air pressure depends upon water depth and pipe friction losses.
² Air requirement

Transfer efficiencies of diffused air systems vary with depth and bubble size. If coarse bubbles are produced, only 4% to 8% of the available oxygen will be transferred to the water. With medium-sized bubbles, the transfer efficiency varies between 6% and 15%. Fine bubble systems are capable of transferring 10% to 30% of the supplied oxygen to the water.

B. PLAIN SEDIMENTATION (CLARIFICATION)
Water contaminated with sand, dirt, mud, etc., can be treated in a sedimentation basin or tank. Up to 80% of the incoming sediment can be removed in this manner. Sedimentation basins are usually concrete, rectangular or circular in plan, and equipped with scrapers or raking arms to periodically remove accumulated sludge.

Settlement of water-borne particles depends on the water temperature (which affects viscosity), particle size, and particle specific gravity. Typical specific gravities are given in table 7.19.

If it is necessary to calculate the settling velocity of a particle of diameter \( d \) (in feet), the following procedure can be used.

**step 1**: Assume \( v_s \).

**step 2**: Calculate the Reynolds number

\[
N_{Re} = \frac{v_s d}{\nu}
\]

**step 3**: If \( N_{Re} < 1 \), use Stoke’s law.

\[
v_s = \frac{(\rho_{\text{particle}} - \rho_{\text{water}})d^2g}{18\mu_c} = \frac{(SG_{\text{particle}} - 1)d^2g}{18\nu}
\]

If \( 1 < N_{Re} < 2000 \), use figure 7.6.

If \( N_{Re} > 2000 \), use Newton’s law.

\[
v_s = \sqrt{\frac{4g(\rho_{\text{particle}} - \rho_{\text{water}})d}{3(\rho_{\text{water}})C_D}}
\]

Values of \( C_D \) are given in table 7.20.
Figure 7.6  Settling Velocities
(spherical particles, 68°F water)

Table 7.20
Approximate Drag Coefficients for Spheres

<table>
<thead>
<tr>
<th>$N_{Re}$</th>
<th>$C_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>0.4</td>
</tr>
<tr>
<td>10,000</td>
<td>0.4</td>
</tr>
<tr>
<td>50,000</td>
<td>0.5</td>
</tr>
<tr>
<td>100,000</td>
<td>0.5</td>
</tr>
<tr>
<td>200,000</td>
<td>0.4</td>
</tr>
</tbody>
</table>

If it is assumed that the water velocity is a uniform $v_f$, then all particles with $v_p > v^*$ will be removed. $v^*$ is known as the overflow rate (surface loading or critical velocity) and typically has a value of 600 to 1000 gpd/ft$^2$ for rectangular basins. For square and circular basins, the surface loading should be within 500–750 gpd/ft$^2$. $B$ is the tank width, typically 30 to 40 feet, and $L$ is the length, typically 100 to 200 feet. (For radial flow circular basins, a typical diameter is 100 feet.)

$$v^* = \frac{Q}{A_{surface}} = \frac{Q}{BL} \quad 7.40$$

If water enters at some level other than the surface, such as at level $h$ in figure 7.7, all particles will be removed that have

$$v_s > \frac{hQ}{HBL} \quad 7.41$$

Rectangular basins are preferred, even if sludge removal means emptying and taking them out of service. Rectangular basins should be constructed with aspect ratios of greater than 3:1, and preferably in excess of 4:1. Slope the bottom toward the drain at no less than 1 percent. Use multiple inlets along the entire inlet wall, if possible. If fewer than four ports are used, an inlet baffle should be provided.

In the case of square or circular basins, the slope toward the drain should be greater, typically on the order of 1:12. A baffled center inlet should be provided. Square and circular basins are appropriate when space is limited. Otherwise, rectangular basins or solid contact units are preferred.

In using equation 7.40, divide the total flow to be treated into at least two basins. That way, one basin can be out of service for cleaning without interruption of operation.

Basins should be constructed from concrete for all permanent installations. Steel should be used only for small or temporary installations. Where steel parts are unavoidable, as in the case of internal parts of rotors, adequate corrosion resistance is necessary.

The time spent by water in the basin is known as the detention (or retention) time (or period). The detention time is given by equation 7.42. A minimum time recommended is 3–4 hours, although periods from 1 to 10 hours are used.$^{12}$

$$t = \frac{\text{tank volume}}{Q} \quad 7.42$$

$^{12}$ Long detention times, up to 12 hours, are required to remove fine particles.
Optimum filter operation will occur when the top layer of sand is slightly more coarse than the rest of the sand. During backwashing, however, the finest sand rises to the top. Various designs using coal and garnet layers in conjunction with sand layers (i.e., dual- and triple-media filters) overcome this difficulty due to the differences in specific gravity.

Historically, the flow rate has been 2 gpm/ft² in rapid sand filter design, although some current filters operate at 8 gpm/ft². 4 gpm/ft² is a reasonable rate for modern designs.

A water treatment plant should have at least three filters so that two can be in operation when one is being cleaned. Typical total throughputs per filter range from 350 gpm for small plants to 3500 gpm for large plants.

Optimum design of a filtering system includes discharge into a clearwell. Clearwells are storage reservoirs with capacities of 30% to 60% of the daily output with a minimum of 12 hours of maximum daily consumption. Demand can be satisfied by the clearwell if one or more of the filters is serviced.

The most common type of service needed by filters is backwashing. Filters require backwashing when the pores between sand particles clog up. Typically, this occurs after 1 to 3 days of operation. Backwashing is done when the head loss through the filter bed reaches approximately 8 feet. Backwashing with filtered water expands the sand layer up to 50%, which dislodges the trapped material. Backwashing for 3 to 5 minutes at 8-15 gpm/ft² is a reasonable design standard. The head loss is reduced to 1 1/4 foot after washing.

Water is pumped through the filter from the bottom during backwashing. The rate at which the water rises in the filter housing varies between 12 and 36 inches per minute. This rise rate should not exceed the settling velocity of the smallest particle which is to be retained in the filter. Backwashing usually takes between 3 and 5 minutes. Water which is collected in troughs for disposal and used for backwashing, constitutes between 1% and 5% of the total processed water (approximately 75-100 gal/ft² total).

The actual amount of backwash water can be found from equation 7.52. Be sure to use consistent units.

\[
\text{water volume} = \left( \frac{\text{backwash time}}{\text{time}} \right) \left( \frac{\text{filter area}}{\text{area}} \right) \left( \frac{\text{rise rate}}{\text{rate}} \right) \quad 7.52
\]

Slow sand filters are similar in design to rapid sand filters, except that the sand layer is thicker (24" to 48"), the gravel layer is thinner (6" to 12"), and the flow rate is much lower (0.05 to 0.1 gpm/ft²). Slow sand filters are limited to low-turbidity applications not requiring chemical treatment. Cleaning is usually accomplished by removing a few inches of sand. Slow sand filters operate with a 0.2 to 4.0 foot head loss.

Other types of filters are seeing limited use:

- **Pressure filters**: Similar to rapid sand filters except incoming water is pressurized up to 25 feet (hydraulic). Filter rates of 2 to 4 gpm/ft². Not used in large installations.
- **Diatomaceous earth filters**: 1 to 3 gpm/ft². Short (20-hour) cycle life.
- **Microstrainers**: Woven stainless steel fabric usually mounted on a rotating drum.

F. DISINFECTION

Chlorination is used for disinfection and oxidation. As a disinfectant, chlorine destroys bacteria and microorganisms. As an oxidant, it removes iron, manganese, and ammonia nitrogen.

Chlorine can be added as a gas or a solid. (If it is added to the water as a gas, it is stored as a liquid which vaporizes around -35°C.) Liquid chlorine is the predominant form since it is cheaper than hypochlorite solid (Ca(OCl)₂). If chlorine liquid or gas is added to water, the following reaction occurs to form hypochlorous acid, which itself ionizes to hypochlorite and hydrogen ions.

\[
\text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{HCl} + \text{HOCl} \quad \text{pH > 8} \quad \text{H}^+ + \text{OCl}^- + \text{HCl}. \quad \text{pH < 7} \quad 7.53
\]

If calcium hypochlorite solid is added to water, the ionization follows immediately.

\[
\text{Ca(OCl)}_2 + \text{H}_2\text{O} \rightarrow \text{Ca}^{2+} + 2\text{OCl}^- + \text{H}_2\text{O} \quad 7.54
\]

Chlorine existing in water as hypochlorous acid and hypochlorite ions is known as free available chlorine (free residuals). Chlorine in combination with ammonia is known as combined available chlorine (combined residuals).

The average chlorine dose is in the 1 to 2 mg/l range. Minimum chlorine residuals for 70°F water are given in table 7.21. Reliable chlorination requires a pH of water below 9.0. However, inactivated viruses (such as might be present in surface water) require a heavier chlorine concentration. Since treatment of water is by both free and combined residuals, ammonia can be added to the water to produce chloramines.

Excess chlorine can be removed with a reducing agent, usually called a dechlor. Sulfur dioxide and sodium bisulfate (sodium metabisulfate) are used in this manner. Aeration also reduces chlorine content, as does passing the water through an activated charcoal filter.
Table 7.21
Minimum Chlorine Residuals (mg/l)

<table>
<thead>
<tr>
<th>pH Value</th>
<th>Free residuals after 10-20 minutes</th>
<th>Combined residuals after 1-2 hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0</td>
<td>0.2</td>
<td>1.0</td>
</tr>
<tr>
<td>7.0</td>
<td>0.7</td>
<td>1.5</td>
</tr>
<tr>
<td>8.0</td>
<td>0.4</td>
<td>1.8</td>
</tr>
<tr>
<td>9.0</td>
<td>0.8</td>
<td>3.0*</td>
</tr>
<tr>
<td>10.0</td>
<td>0.8</td>
<td>3.0*</td>
</tr>
</tbody>
</table>

* Not recommended

Alternatives to chlorination have become popular since THM's were traced to the chlorination process. Chlorine dioxide can be used in place of chlorine, but it is expensive. In high dosages, chlorine dioxide is also thought to produce its own toxic by-products. Ozone is a more powerful disinfectant than chlorine, but it is expensive to generate and requires costly contact chambers. Ozone, which is used extensively in Western Europe, Canada, the USSR, and Japan, is generated on-site by running high voltage electrical currents through dry air or pure oxygen.

G. FLUORIDATION

Fluoridation can occur any time after filtering. Smaller utilities almost always choose liquid solution and a volumetric feeding mechanism, with solutions being manually prepared. Larger utilities use gravimetric dry feeders with sodium silicofluoride or solution feeders with fluorosilic acid. The characteristics and dose rates of common fluorine compounds are given in Table 7.22.

Table 7.22
Dose Rates for Fluorine Compounds

<table>
<thead>
<tr>
<th>Formula</th>
<th>$\text{H}_2\text{SiF}_6$</th>
<th>NaF</th>
<th>$\text{Na}_2\text{SiF}_6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Form</td>
<td>liquid</td>
<td>solid</td>
<td>solid</td>
</tr>
<tr>
<td>Typical purity</td>
<td>72–30%</td>
<td>90–98%</td>
<td>98–99%</td>
</tr>
<tr>
<td>Dose to obtain 1.0 mg/l (in pounds per million gallons)</td>
<td>33.2</td>
<td>18.8</td>
<td>14.0</td>
</tr>
<tr>
<td>(with 30% (with 98% (with 98.5% purity) purity) purity) purity)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Defluoridation (required if the fluoride exceeds 1.5 mg/l) can be achieved with calcined alumina or bone char (tricalcium phosphate). Softening using lime can also be used when waters contain smaller amounts of fluoride. Each 45 to 65 mg/l reduction in magnesium will result in a 1.0 mg/l reduction in fluoride.

H. IRON AND MANGANESE REMOVAL

Several methods of removing iron exist. Manganese is not easily removed by aeration alone. However, the remaining methods work.

- Aeration, followed by sedimentation and filtration:
  \[ \text{Fe}^{++} + \text{O}_2 \rightarrow \text{FeO}_2 \downarrow \]

- Aeration, followed by chemical oxidation, sedimentation, and filtration. Chlorine or potassium permanganate may be used as an oxidizer.

- Manganese zeolite process: Manganese dioxide removes soluble iron ions.

- Lime water softening

Table 7.23 lists the characteristics of iron and manganese removal processes.

I. WATER SOFTENING

- Lime and Soda Ash Softening

Water softening can be accomplished with lime and soda ash to precipitate calcium and magnesium ions from the solution. Lime treatment has added benefits of disinfection, iron removal, and clarification.

Lime (CaO) is available as granular quicklime (minimum purity: 90% CaO) or hydrated lime, Ca(OH)$_2$. Quicklime is slaked prior to use, which means that water is added to form a lime slurry in an exothermic reaction:

\[ \text{CaO} + \text{H}_2\text{O} \rightarrow \text{Ca(OH)}_2 + \text{heat} \]

The hydrated lime is delivered to the water supply as a suspension (i.e., milk of lime).

Soda ash is usually available as 98% pure sodium carbonate (Na$_2$CO$_3$).

FIRST STAGE TREATMENT: In the first stage treatment, lime added to water reacts with free carbon dioxide to form calcium carbonate precipitate.

\[ \text{CO}_2 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{H}_2\text{O} \]

Next, the lime reacts with calcium bicarbonate.

\[ \text{Ca(HCO}_3\text{)}_2 + \text{Ca(OH)}_2 \rightarrow 2\text{CaCO}_3 \downarrow + 2\text{H}_2\text{O} \]

Any magnesium hardness is also removed at this time.

\[ \text{Mg(HCO}_3\text{)}_2 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 \downarrow + 2\text{H}_2\text{O} + \text{MgCO}_3 \]

To remove the soluble MgCO$_3$, the pH must be above 10.8. This is accomplished by adding an excess of approximately 35 mg/l of CaO or 50 mg/l of Ca(OH)$_2$ plus lime to satisfy equation 7.60.

\[ \text{MgCO}_3 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{Mg(OH)}_2 \downarrow \]

48
Table 7.23
Iron/Manganese Removal Processes

<table>
<thead>
<tr>
<th>processes</th>
<th>iron and/or manganese removed</th>
<th>pH required</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>aeration, settling, and filtration</td>
<td>ferrous bicarbonate</td>
<td>7.5</td>
<td>provide aeration unless incoming water contains adequate dissolved oxygen</td>
</tr>
<tr>
<td></td>
<td>ferrous sulfate</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>manganous bicarbonate</td>
<td>10.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>manganous sulfate</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ferrous bicarbonate</td>
<td>5.0</td>
<td>provide aeration unless incoming water contains adequate dissolved oxygen</td>
</tr>
<tr>
<td></td>
<td>manganous bicarbonate</td>
<td>9.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ferrous bicarbonate colloidal or organic iron</td>
<td>8.5-9.6</td>
<td>require lime and alum or iron coagulant</td>
</tr>
<tr>
<td></td>
<td>manganous bicarbonate colloidal or organic iron</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ferrous bicarbonate manganese</td>
<td>6.5±</td>
<td>water must be devoid of oxygen</td>
</tr>
<tr>
<td></td>
<td>manganese bicarbonate</td>
<td></td>
<td>iron and manganese in raw water not to exceed 2.0 mg/l</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>consult manufacturers for type of ion exchange resin to be used</td>
</tr>
</tbody>
</table>

FIRST STAGE RECARBONATION: Lime added to precipitate hardness removes itself. This is desirable because any calcium that remains in the water has the potential for forming scale. Further stabilization can be achieved by recarbonation (treatment with carbon dioxide).

\[
\text{Ca(OH)}_2 + \text{CO}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{H}_2\text{O}\quad 7.61
\]

Excess recarbonation should be avoided. If the pH is allowed to drop below 9.5, then carbonate hardness reappears.

\[
\text{CaCO}_3 + \text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{Ca(HCO}_3)_2\quad 7.62
\]

At this time, any unsettled Mg(OH)_2 can be returned to a soluble state.

\[
2\text{Mg(OH)}_2 + 2\text{CO}_2 \rightarrow 2\text{MgCO}_3 + 2\text{H}_2\text{O}\quad 7.63
\]

SECOND STAGE TREATMENT: The second stage treatment removes calcium noncarbonate hardness (sulfates and chlorides) which needs soda ash for precipitation.

\[
\text{CaSO}_4 + \text{Na}_2\text{CO}_3 \rightarrow \text{CaCO}_3 \downarrow + \text{Na}_2\text{SO}_4\quad 7.64
\]

Magnesium noncarbonate hardness needs both lime and soda ash.

\[
\begin{align*}
\text{MgSO}_4 + \text{Ca(OH)}_2 & \rightarrow \text{Mg(OH)}_2 \downarrow + \text{CaSO}_4 \quad 7.65 \\
\text{CaSO}_4 + \text{Na}_2\text{CO}_3 & \rightarrow \text{CaCO}_3 \downarrow + \text{Na}_2\text{SO}_4 \quad 7.66
\end{align*}
\]

Excess soda ash leaves sodium ions in the water. However, noncarbonate hardness is a small part of total hardness. Soda ash is also costly, so the actual dose might be slightly reduced from what is needed.

SECOND STAGE RECARBONATION: Second stage recarbonation is needed to remove CaCO₃. CO₂ is added until the pH is about 8.6, at which time no further precipitation will occur because [Ca⁺⁺][CO₃⁻⁻] < Ksp of CaCO₃.

Sodium polyphosphate can be added at this time to inhibit the formation of scale on filter sand and inside pipes.

A split process can be used to reduce the amount of lime that is neutralized by recarbonation (and is wasted). Excess lime is added in the first stage. This forces precipitation of magnesium in the first stage instead of in the second stage. Excess lime reacts with calcium hardness in the second stage. The amount of bypass depends on the allowable hardness of water leaving the plant. A typical split process is shown in figure 7.9.
Practical limits of precipitation softening by the lime process are 30 to 35 mg/l of CaCO₃ and 8 to 10 mg/l of Mg(OH)₂ as CaCO₃ due to intrinsic solubilities. Water softened with this process usually leaves the apparatus with a hardness of between 50 and 80.

**Example 7.18**

Water contains 130 mg/l as CaCO₃ of Ca(HCO₃)₂. How much slaked lime (Ca(OH)₂) is required to remove the hardness?

Since the Ca(HCO₃)₂ is given in CaCO₃ equivalents, 130 mg/l of lime (as CaCO₃) is implicitly required. It only remains to convert the CaCO₃ equivalent to a substance measurement using appendix A.

\[ \text{Ca(OH)}₂ = \frac{130}{1.35} = 96.3 \text{ mg/l as substance} \]

**Example 7.19**

How much slaked lime (90% pure), soda ash, and carbon dioxide are required to reduce the hardness of the water evaluated below to zero using the lime-soda ash process? Neglect the fact that this process cannot really produce zero hardness, and base your answer on stoichiometric considerations.

- Total hardness: 250 mg/l as CaCO₃
- Alkalinity: 150 mg/l as CaCO₃
- Carbon dioxide: 5 mg/l

Using appendix A, the CO₂ is first converted to its CaCO₃ equivalent.

\[ \frac{(2.27)(5)}{2} = 11.35 \text{ mg/l as CaCO₃} \]

The alkalinity of 150 mg/l is already in CaCO₃ equivalent form. Therefore, the total CaCO₃ equivalent from substances requiring lime for neutralization is 11.35 + 150 = 161.35 mg/l as CaCO₃.

From appendix A, the amount of 90% pure slaked lime (Ca(OH)₂) is

\[ \frac{161.35}{1.35}(0.90) = 132.8 \text{ mg/l as substance} \]

50 mg/l of lime is arbitrarily added to raise the pH above 10.8. The total lime requirement is then

\[ 132.8 + \frac{50}{0.90} = 188.4 \text{ mg/l as substance} \]

The noncarbonate hardness is 250 - 150 = 100 mg/l as CaCO₃. The soda ash (Na₂CO₃, 98% pure) requirement is

\[ \frac{100}{(0.94)(0.98)} = 108.6 \text{ mg/l as substance} \]

The first stage recarbonation CO₂ requirement depends on the excess lime added.

\[ \frac{(50)(1.35)}{(2.27)} = 29.7 \text{ mg/l as substance} \]

- **Ion Exchange Method**

In the ion exchange process (also known as zeolite process, resin exchange process, or ion exchange method), water is passed through a filter bed of exchange material. This exchange material is known as zeolite. Ions in the insoluble exchange material are displaced by ions in the water. When the exchange material is spent, it is regenerated with a rejuvenating solution such as sodium chloride (salt), or, in the case of common cationic resins, sulfuric and hydrochloric acids are used as regenerants. Soda ash is used as a regenerant in weakly-basic exchangers.

The processed water will have a zero hardness. However, since there is no need for water with zero hardness, some water is usually bypassed around the unit.

There are three types of ion exchange materials. Greensand (glauconite) is a natural substance that is mined and treated with manganese dioxide. Silicious-gel zeolite is an artificial solid used in small volume deionizer columns. Polystyrene resins are also synthetic. Polystyrene resins currently dominate the softening field.¹⁵

¹⁵ Differences in the polymerization step can result in polymers with gel or macroporous structures. Gel polymers have low cross linking, high capacity, and fast reaction kinetics. Macroporous polymers have high cross linking, reduced capacity, and lower kinetics. Gel resins have historically been used in water softening. However, the chemical resistance of macroporous forms is advantageous in special applications.
During operation, the calcium and magnesium ions are removed in reactions similar to the following reaction. Z is the zeolite anion. The resulting sodium compounds are soluble.

\[
\begin{align*}
\left\{ \text{Ca} \right\} & \left\{ \text{Mg} \right\} \left\{ \text{HCO}_3 \right\}_2 \text{SO}_4 \text{Cl}_2 + \text{Na}_2Z \\
\rightarrow & \left\{ \text{2NaHCO}_3 \right\} \left\{ \text{Na}_2\text{SO}_4 \right\} + \left\{ \text{Ca} \right\} \left\{ \text{Mg} \right\} \text{Z} = 7.67
\end{align*}
\]

Typical characteristics of an ion exchange unit are expressed per 1000 grains of hardness removed.\(^\text{16}\)

- exchange capacity: 3000 grains hardness/ft\(^3\) zeolite for natural; 5000–30,000 (20,000 typical) for synthetic.
- flow rate: 2 to 6 gpm/ft\(^3\) (2 gpm/ft\(^3\) standard)
  6 gpm/ft\(^2\) of filter bed
- backwash flow: 5 to 6 gpm/ft\(^2\)
- salt dosage: 5 to 20 pounds/ft\(^3\). Alternatively, 0.3 to 0.7 pound of salt per 1000 grains of hardness removed

\(^{16}\) 1000 grains of hardness is also known as a kilogram.

### Example 7.20

A municipal plant receives water with a total hardness of 200 mg/l. The designed discharge hardness is 50 mg/l. If an ion exchange unit is used, what is the bypass factor?

Let \(x\) be the bypass factor. Since the water passing through the ion exchange unit is reduced to zero hardness,

\[(1 - x)0 + x(200) = 50\]

\[x = 0.25\]

### J. TURBIDITY REMOVAL

Coagulants can be based on aluminum (e.g., aluminum sulfate, sodium aluminate, potash alum, or ammonia alum) or iron (e.g., ferric sulfate, ferrous sulfate, chlorinated copperas, or ferric chloride). If significant hydrolysis of iron and aluminum salts is ignored, the re-

### Table 7.24

<table>
<thead>
<tr>
<th>type of resin</th>
<th>drained density (lbm/ft(^3))</th>
<th>operating pH range</th>
<th>regeneration</th>
<th>characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>strong acid</td>
<td>49–53</td>
<td>0–14</td>
<td>excess</td>
<td>high exchange rates; are stable; low swelling; long life, up to 20 years or more; can split strong and weak salts</td>
</tr>
<tr>
<td>weak acid</td>
<td>45</td>
<td>7–14</td>
<td>weak or strong acid</td>
<td>capacities double of strong acid; resistant to chlorine and other oxidants; high (90%) swell; not effective for electrolytic salt cations</td>
</tr>
<tr>
<td>strong base</td>
<td>45</td>
<td>0–14</td>
<td>excess strong base</td>
<td>irreversibly fouled by humic acids from decaying vegetation; can split strong or weak salts; less stable than cation resins (life less than 3 years); can remove silica; often used with food processing</td>
</tr>
<tr>
<td>weak base</td>
<td>32</td>
<td>0–6</td>
<td>weak or strong base</td>
<td>resistant to organic fouling; does not remove CO(_2) or silica; capacity double of strong base; can remove color</td>
</tr>
<tr>
<td>intermediate base</td>
<td>43</td>
<td>0–14</td>
<td>strong base</td>
<td>can absorb CO(_2) silica and phenol. Useful as substitutes for weak base resins in multiple-bed processes</td>
</tr>
</tbody>
</table>
relationships in this section can be used to calculate the approximate stoichiometric quantities.

The most-used coagulant is aluminum sulfate \((\text{Al}_2\text{(SO}_4\text{)}_3 \cdot 14\text{H}_2\text{O})\). Filter alum is about 17% soluble material. The hydrolysis of the aluminum ion is complex. Assuming that the aluminum floc is \(\text{Al(OH)}_3\) and the water pH is near neutral, then 1 mg/l of alum with a molecular weight of 600 removes the following quantities:

- 0.5 mg/l \((\text{CaCO}_3)\) of natural alkalinity
- 0.39 mg/l of 95% hydrated lime \((\text{Ca(OH)}_2)\)
- 0.33 mg/l of 85% quicklime \((\text{CaO})\)
- 0.53 mg/l of soda ash \((\text{Na}_2\text{CO}_3)\)

If the alum has a molecular weight that is different than 600 (due to the variation in the number of waters of hydration), multiply the above quantities by \((600/\text{actual molecular weight})\).

Typical doses of alum are 5 to 50 mg/l, depending on turbidity. Alum flocculation is effective within pH limits of 5.5 to 8.0.

Ferrous sulfate \((\text{FeSO}_4 \cdot 7\text{H}_2\text{O})\), also known as copperas, reacts with lime \((\text{Ca(OH)}_2)\) to flocculate ferric hydroxide \((\text{Fe(OH)}_3)\). This is an effective method of clarifying turbid waters at higher pH, as in lime softening. 1 mg/l of ferrous sulfate with a molecular weight of 278 will react with 0.27 mg/l of lime.

Ferrous sulfate \((\text{Fe}_2\text{(SO}_4\text{)}_3)\) reacts with natural alkalinity or lime to create floc. 1 mg/l of ferric sulfate will react with

- 1.22 mg/l of \(\text{Ca(HCO}_3\text{)}_2\)
- 0.56 mg/l of \(\text{Ca(OH)}_2\)
- 0.62 mg/l of natural alkalinity (as \(\text{CaCO}_3\))

Ferric sulfate can be used for color removal at low pH; at high pH, it is useful for iron and manganese removal, as well as a coagulant with precipitation softening.

**K. TASTE AND ODOR CONTROL**

- **Copper Sulfate Treatment.** This treatment is used in impounding reservoirs, lakes, storage reservoirs, and occasionally in settling basins or treated water, to prevent biological growths. Dosages may vary from 0.5 to 2.0 milligrams per liter; the lower dosage ordinarily suffices for soft water. For very hard water, a dosage above 2.0 milligrams per liter may be used after laboratory tests to determine the necessary algicidal dose. Effects on fish life should be monitored.

- **Aeration.** This process can be used to improve tastes and odors in water where the cause is hydrogen sulfide or the absence of dissolved oxygen. This method has little effect on most tastes and odors.

- **Activated Carbon.** This material removes most tastes and odors. Dosages may vary from 0.5 to 200 milligrams per liter, ordinarily ranging from 2 to 10 milligrams per liter.

- **Superchlorination and Dechlorination.** This treatment will improve tastes and odors caused by organic matter and industrial wastes, especially phenolic wastes. Normally, the dosage required will be several times greater than those for ordinary disinfection (as determined by testing). Provide chlorinating equipment capable of dosing at these high values; allow a minimum of 20 minutes contact time; furnish equipment for dechlorinating with sulfur dioxide or other reducing agent.

- **Chlorine-Ammonia Treatment.** Where chloro-substitution products cause tastes and odors, the chlorine-ammonia treatment can be used to prevent them. It can also be used for maintaining the combined residual chlorine for an extended period as, for example, in reservoirs or distribution systems.

  - Chloramines are less active disinfectants than free chlorine and, therefore, may not be substituted where adequate disinfection requires free residual chlorine.
  - The ratio of chlorine to ammonia required for disinfection varies from 3:1 to 7:1.
  - Periodic laboratory tests should be conducted to determine the proper dosage. Apply chlorine after ammonia has been properly dispersed in the water.

- **Free Residual Chlorination.** Use this method before filtration to reduce tastes and odors caused by organic matter at locations where experience shows it to be effective and acceptable. Increase the chlorine dosage until the residual consists solely of free available chlorine.

- **Chlorine Dioxide.** In some cases, this chemical can be used to destroy phenolic and other organic tastes and odors in raw water. The dosage varies from 0.2 to 0.3 milligram per liter, as determined by testing.

- **Microstraining.** This method is used as a means of reducing the number of algae and other organisms in the water, and thus reduces the subsequent production of tastes and odors. The microstrainer
removes no dissolved or colloidal organic matter. It utilizes monel metal cloth with 35 micron (0.0014 inch) openings. Finer mesh can be obtained.

I. DEMINERALIZATION/DESALINATION

If dissolved salts are to be removed, one of the following methods must be used:

- **Distillation**: The water is vaporized, leaving the salt behind. The vapor is reclaimed by condensation.

- **Electrodialysis**: Positive and negative ions flow through selective membranes under the influence of an induced electrical current.

- **Ion exchange**: This is the same process as described for water softening.

- **Reverse osmosis**: This is the least expensive method of demineralization. In operation, a thin membrane of cellulose acetate plastic separates two salt solutions of different concentrations. Although ions would normally flow through the membrane into the solution with the lower concentration, the migration direction can be reversed by applying pressure to the low concentration fluid. Typical reverse osmosis units operate at 400 psi and produce about 2 gallons per day of fresh water for each square foot of surface.

14 TYPICAL MUNICIPAL SYSTEMS

The processes employed in treating incoming water will depend on the characteristics of the water. However, some sequences work better than others due to the physical and chemical nature of the processes. Listed in this section are some typical sequences. Not present in the lists are the usual system hardware items such as intake screens, pumps, pipes, hydrants, reservoirs, and holding basins.

Table 7.25 provides guidelines for choosing processes required to achieve satisfactory water quality. This table bases the required processes on the incoming water quality.

Additives and chemicals can be applied to the water supply at various points along the treatment path. Figure 7.10 indicates typical application points.

- **For Well Ground Water** (typically cleaner than surface water)

  sequence #1: intake
  chlorination
  fluoridation

- **For Lake or Surface Water** (typically turbid, and carrying odor and color)

  sequence #1: intake
  chlorination
  coagulation
  rapid mixing
  flocculation
  optional chlorination
  addition of activated carbon
  settling
  addition of activated carbon
  filtering
  chlorination
  fluoridation

- **For River Surface Water** (very turbid)

  sequence #1: intake
  presedimentation (holding basin)
  chlorination
  coagulation
  rapid mix
  flocculation
  settling
  coagulation
  rapid mix
  flocculation
  addition of activated carbon
  settling
  addition of activated carbon
  filtering
  chlorination
  fluoridation
<table>
<thead>
<tr>
<th>Constituents</th>
<th>Concentration, mg/l</th>
<th>Screening</th>
<th>Pretreatment</th>
<th>Treatment</th>
<th>Special Treatments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coliform monthly avg. MPN/100 ml²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0–20</td>
<td>E</td>
<td>O</td>
<td>O</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>20–100</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>100–500</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>&gt;5,000</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>Suspended solids</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0–100</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>100–200</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>&gt;200</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>Color, mg/l</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20–70</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>&gt;70</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>Taste and odor</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CaCO₃, mg/l</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;200</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>&lt;5.0–9.0 pH</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>Iron and manganese mg/l</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;0.3</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>0.3–1.0</td>
<td>E</td>
<td>O³</td>
<td>E E E E</td>
<td>O E</td>
<td></td>
</tr>
<tr>
<td>Chloride, mg/l</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0–250</td>
<td>E</td>
<td>E E</td>
<td>E E E O</td>
<td>O</td>
<td></td>
</tr>
<tr>
<td>250–500</td>
<td>E</td>
<td>E E</td>
<td>E E E O</td>
<td>O</td>
<td></td>
</tr>
<tr>
<td>&gt;500</td>
<td>E</td>
<td>E E</td>
<td>E E E O</td>
<td>O</td>
<td></td>
</tr>
<tr>
<td>Phenolic compounds, mg/l</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;0.005</td>
<td>E</td>
<td>E E</td>
<td>E E E O</td>
<td>O</td>
<td></td>
</tr>
<tr>
<td>Toxic chemicals</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Less critical chemicals</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: E = essential, O = optional

1 Superchlorination shall be followed by dechlorination.
2 As alternate, dilute with low chloride water.
3 Double settling shall be provided for coliform exceeding 20,000 MPN/100 ml².
4 For extremely muddy water, presedimentation by plain settling may be provided.
5 MPN = most probable number

- For Hard Water

Sequence #1: intake (bypass to second flocculator)
- lime addition
- alum addition
- rapid mixing
- flocculation
- sedimentation
- oxidation (chlorine or potassium permanganate)
- second flocculation
- sedimentation
- filtering
- fluoridation
- chlorination

Sequence #2: intake
- presedimentation (in a basin)
- chlorination
- mixing
- addition of activated carbon
- lime addition
- alum addition
- flocculation
- sedimentation
- addition of activated carbon
- mixing
- filtering
- fluoridation
- chlorination
- soda ash addition
sequence #3: intake
lime addition
alum addition
addition of activated carbon
mixing
flocculation
chlorination
sedimentation
recarbonation
filtering (bypass
to discharge)
zeolite treatment

Typical flow diagram of water treatment plant

<table>
<thead>
<tr>
<th>category of chemicals</th>
<th>possible points of application</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>algicide</td>
<td></td>
</tr>
<tr>
<td>disinfectant</td>
<td></td>
</tr>
<tr>
<td>activated carbon</td>
<td></td>
</tr>
<tr>
<td>coagulants</td>
<td></td>
</tr>
<tr>
<td>coagulation aids</td>
<td></td>
</tr>
<tr>
<td>alkali:</td>
<td></td>
</tr>
<tr>
<td>for flocculation</td>
<td></td>
</tr>
<tr>
<td>for corrosion control</td>
<td></td>
</tr>
<tr>
<td>for softening</td>
<td></td>
</tr>
<tr>
<td>acidifier</td>
<td></td>
</tr>
<tr>
<td>fluoride</td>
<td></td>
</tr>
<tr>
<td>cupric-chloramine</td>
<td></td>
</tr>
<tr>
<td>dechlorinating agent</td>
<td></td>
</tr>
</tbody>
</table>

Note: With solids contact reactors, point C is same as point D.

Figure 7.10 Application Points for Chemicals
Trickling Filter

A trickling filter may be used following a primary settling tank, a septic tank, or an Imhoff tank to provide secondary treatment of the sewage. Habitation should not be closer than 400 ft. Some odors and filter flies can be expected with a standard rate filter. Filter flies can be controlled by weekly chlorination (1 mg/l in effluent for 4 to 8 hours), flooding (24 hours), increased hydraulic loading, and insecticide treatment. A receiving stream providing adequate dilution and supervision over operation is required. Seeding of the filter stone and development of a gelatinous film of aerobic microorganisms is necessary before good results can be produced. High BOD reduction is obtained within 7 days of starting a trickling filter, but as long as 3 months may be required to obtain equilibrium, including high nitrification. Nitrification is an aerobic process in which the ammonia from sewage is acted upon by the oxygen in the air to form nitrate and carbon dioxide. Continuous operation, particularly during cold months of the year, is necessary to maintain nitrification efficiency. High nitrification is important in reducing the nitrogenous oxygen demand downstream in a body of receiving water.

Small standard-rate trickling filters are usually 6 ft deep and designed for a dosage of 200,000 to 300,000 gpd/acre-ft, or not more than 1,800,000 gal for a 6-ft deep filter. Filter loading is also expressed, with greater accuracy, in terms of 5-day BOD in the sewage applied to the filter. It is usually assumed that 35 percent of the BOD in a raw sewage is removed by the primary settling unit. Standard-rate trickling filters are dosed at 200 to 600 lb of BOD/acre-ft/day. Average loading are 400 lb in northern states and 600 lb in southern states. Since dosage must be controlled, dosing siphons or tipping trays may be used for very small filter and dosing tanks with siphons or pumps containing revolving distributors or stationary spray nozzles on the usual filter. Periodic dosing with interim resting usually produces a better effluent than continuous dosing. Continuous dosage at a higher rate, with recirculation of part of the effluent, may be suitable where good supervision is available and operation can be controlled to produce the intended results. Lower-rate dosage results in a higher quality effluent. Filter flies are reduced with recirculation. Good natural or forced ventilation of the filter is necessary.

A trickling filter should be followed by a secondary settling or humus tank to remove the biological growths sloughed off the filter stone; this unit will require the removal of sludge at least twice a day. The sludge is removed by pumping or by gravity flow if possible, usually to the sludge digester or Imhoff tank, depending on the plant design. The discharge of the raw sludge to a sand drying bed is not advisable, as sludge drying will be slow and odors will result.

For odor control or disinfection of the sewage effluent for bacterial reduction, chlorination of the final effluent is an additional and often required treatment. Trickling-filter treatment can be supplemented by sand filtration, oxidation pond, solids contact basin, flocculator-clarifiers, or chemical coagulation and settling where a higher quality effluent is necessary. Variations of the standard-rate trickling filter include the high-rate filter with recirculation; the biological tower (20 to 30 ft), which uses a plastic media, biological aerated filter, which uses a submerged media and forced air; and rotating biological contractors.

Flow diagrams including trickling filters are illustrated in Figure 4-34. A typical design of an Imhoff tank standard-rate trickling filter plant is shown under Typical Designs of Small Plants, and Design for a Small Community, this chapter. Other combinations are used.

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126 G R Grantham and J G Seeger, Jr., “Progress of Purification During the Starting of a Trickling Filter,” Sewage Ind. Wastes, December 1951, pp 1486-1492
Extended Aeration

Extended aeration plants, also referred to as aerobic digestion plants, have particular application for relatively small installation serving subdivisions, trailer parks, motels, shopping centers, and the like. Some basic design data are given below and in Table 4-21 for conventional activated sludge and other aerobic processes. A three month adjustment period is needed to produce an acceptable effluent. Therefore, extended aeration plants are not recommended for seasonal operations such as camps and schools.

<table>
<thead>
<tr>
<th>Process</th>
<th>Aeration Tank Organic Loading (lb BOD/day) per 1000 ft³</th>
<th>F/M Ratio (lb BOD/day per lb MLVSS)</th>
<th>MLSS'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Step aeration</td>
<td>40</td>
<td>0.2 - 0.5</td>
<td>1000 - 3000</td>
</tr>
<tr>
<td>Complete mix</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contact stabilization</td>
<td>50º</td>
<td>0.2 - 0.6</td>
<td>1000 - 3000</td>
</tr>
<tr>
<td>Extended aeration</td>
<td>15</td>
<td>0.05 - 0.1</td>
<td>3000 - 3000</td>
</tr>
<tr>
<td>Oxidation ditch</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


* Food to microorganism ratio (F/M)
* Mixed liquor volatile suspended solids (MLVSS)
* MLVSS values are dependent upon the surface area provided for sedimentation and the rate of sludge return as well as the aeration process.
* Total aeration capacity, includes both contact and reaeration capacities. Normally the contact zone equals 30 to 35 percent of the total aeration capacity.
**Average Sewage Flow.** 400 gal/dwelling or 100 gpd/capita. See Table 3-14 for other unit flows.

**Screening and Comminator.** Recommended, bar screen minimum.

**Aeration Tanks.** At least two to treat flows greater than 40,000 gpd, 24- to 36-hour detention period at average daily flow, not including recirculation, and 1000 ft² per 7½ to 15 lb of BOD, whichever is greater. Raw sewage goes directly to aeration tank, primary tank is omitted. Provide 18 in. freeboard.

**Air Requirements.** 3 cfm/ft of length of aeration tank, or 2000 to 4000 ft³/lb of BOD entering the tank daily, whichever is larger. Additional air is required if air is needed for air-lift pumping of return sludge from settling tank.

**Settling Tanks.** At least two to treat flows greater than 40,000 gpd, 4-hour detention period based on average daily sewage flow, not including recirculation. For tanks with hopper bottoms, upper third of depth of hopper may be considered as effective settling capacity.

**Rate of Recirculation.** At least 1:1 return activated sludge based on average daily flow.

**Measurement of Sewage Flow.** By V-notch weir or other appropriate device. Recording devices required for larger installations.

**Sludge Holding Tanks.** Provide 8 ft³/capita. Sludge holding tanks should be required for all plants. A minimum of 1000 gal capacity per 15,000 gal design flow and 20- to 40-day retention. Tanks should be aerated.

Daily operation control is essential. Air blowers must be operated continuously and sludge removed. Clogging of the air lift for return sludge is a common cause of difficulty. Grease that accumulates on the surface of settling tanks should be skimmed off and disposed of separately, not to the aeration tank. Aeration tubes or orifices require periodic cleaning. Dissolved oxygen level in the aeration tank and the mixed liquor suspended solids concentration must be watched. Odors should be minimal. A 90 to 97 percent BOD and suspended solids removal and good nitrification of ammonia nitrogen can be expected with proper control.
Table 4-26 Conventional Sewage Treatment Plant Design Factors

<table>
<thead>
<tr>
<th>Preliminary Treatment</th>
<th>Coagulation and Sedimentation Treatment</th>
<th>Sedimentation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Racks</strong></td>
<td><strong>Surface settling rates at peak flows:</strong> primary and intermediate set—tanks 1500 gpd/ft²; final set—tanks 1200 gpd/ft² after trickling filters or rotating biological contactors, and for activated sludge for conventional step aeration, contact stabilization, and the carbonaceous stage of separate stage nitrification, following extended aeration 1000 gpd/ft²; for physical-chemical treatment using lime: 1400 gpd/ft².</td>
<td><strong>Wet weather: 10,000 gpd per linear foot for average flows to 10 mgd and up to 15,000 for larger flows.</strong></td>
</tr>
<tr>
<td>Area: 200% plus sanitary sewer, 300% plus combined sewer. Bar space: 1&quot; to 1 1/2&quot;, dual channels.</td>
<td><strong>Sludge hopper: 1 hor. to 1 1/2 vert.</strong></td>
<td><strong>Sludge pipe: 6 in./min.</strong></td>
</tr>
<tr>
<td><strong>Screen</strong></td>
<td><strong>Chemical Precipitation</strong></td>
<td><strong>Impuff Tank</strong></td>
</tr>
<tr>
<td>Net submerged area: 3 ft² per mgd for sanitary sewer, 3 ft² per mgd for combined sewer. Slot opening 2&quot; min. Dual units, preceded by racks.</td>
<td>Rapid mix, coagulation, sedimentation, ferrous chloride, ferrous sulfate, ferrous sulfate alum, lime, or a polymer.</td>
<td><strong>Detention period: 2 to 2 1/2 hours. Gas vent 20 percent total area of tank from Bot- tom slope: 1 in. vert. to 1 hor., sludge compartment 3 to 4 ft² per capita 18 in. below slot; 6 to 10 ft² per capita secondary treatment Bottom slope: 1 in. vert. to 1 hor. 2 Slot and overlap 8 in. Sludge pipe: 8 in. min. under 6 ft hold. Velocity 1 fps. Surface settling rate: 600 gpd/ft².</strong></td>
</tr>
<tr>
<td><strong>Grit Chamber</strong></td>
<td><strong>Flow Basis</strong></td>
<td><strong>Tube and Inclined Plate Settlers</strong></td>
</tr>
<tr>
<td>Sewage velocity: 1 fps mean, 1 fps, min. Detention: 45 to 60 sec., floor 1 ft below outlet. Min. of 2 channels.</td>
<td>100 gal per capita plus industrial wastes. Usual to assume total flow reaches small plants in 16 hours.</td>
<td>PVC or metal tubes: at 45 to 60 deg. from hor., 2 in. X 2 to 6 in. 4 ft long. May be installed in existing basin.</td>
</tr>
<tr>
<td><strong>Skimming Tank</strong></td>
<td><strong>Flow Equalization</strong></td>
<td></td>
</tr>
<tr>
<td>Air or mechanical agitation with or without chemicals. Detention: 20 min for grease removal, 5 to 15 min for aeration, 30 min for flocculation.</td>
<td>Based on 24 hour plot to smooth out hydraulic and organic loading.</td>
<td></td>
</tr>
<tr>
<td><strong>Comminutors</strong></td>
<td><strong>Chemical Treatment</strong></td>
<td></td>
</tr>
<tr>
<td>Duplicate or bypass. downstream from grit chamber.</td>
<td>For odor control, oxidation, corrosion control, neutralization.</td>
<td></td>
</tr>
</tbody>
</table>

**Note:** Surface settling rate = gpd/ft² = \[ \frac{180 \times \text{tank depth in ft}}{\text{detention, hr}} \]

"Anaerobic sludge digestion will require approximately 65 days at 55°F, 56 days at 60°F, 42 days at 71°F, 27 days at 80°F. 24 days at 95°F, 20 days at 133°F. The optimum temperature is 80 to 95°F. Mixing of sludge can reduce digestion time up to 50 percent. In large plants, sludge is usually digested in two stages. Temperature of 140°F causes caking on pipes."

"Gallons per acre per day = gpad
Million gallons per acre per day = mgad
For multimedia, see state standards."
Biological Treatment

**Intermittent Sand**
- Filter rate: 50,000 to 100,000 gpd/ft² with plain settling and 400,000 gpd/ft² with trickling filter or activated sludge. Sand 24 in. all passing 1/4 in. sieve. E.S. 0.25 to 0.6 mm. Unif. Coef. < 3.5.

**Contact Bed**
- Filter rate: 75,000 to 100,000 gpd per ft².

**Trickling Filter**
- Standard rate: 400 to 600 lb BOD/acre-foot/day, or 2 to 4 mgd, 6 ft depth. High rate: 3000 + lb BOD/acre-foot/day, or 30 mgd for 6 ft depth. Min. filter depth 5 ft, max. 10 ft.

**Activated Sludge**
- See Table 4.21. Normally 2-hour retention in primary and final sed. and 6 to 8-hour aeration.

**Rapid Filtration—Tertiary Treatment**
- 2 to 5 gpm/ft². 1 to 4 mm sand, 48 in. back wash 15 to 25 gpm/ft².

**Land Treatment**
- See text.

**Stabilization Pond—Foulwater**
- 15 to 35 lb BOD/acre-foot/day, 3 to 5 ft liquid depth, center inlet, variable withdrawal depth, 3 ft freeboard, detention 90 to 180 days, multiple units, winter flow retention. Use up to 50 lb BOD loading in mild climate and 15 to 20 in cold areas. See Table 4.15.

**Rotating Biological Contactors**
- See text.

**Disinfection**
- Chlorine, ozone. See text.

Sludge Treatment

**Digester**
- Capacity: with plain sedimentation 2 to 3 ft³ per cap. heated, or 4 to 6 unheated. With standard trickling filter 3 to 4 ft³ heated and 6 to 8 unheated, 4 to 5 ft³ heated and 8 to 10 ft³ unheated with a high rate filter. With activated sludge 4 to 6 ft³ per cap. heated and 8 to 12 ft³ unheated. Bottom slope: 1 on 4, gravity.

**Sludge Drying Bed**
- Open 1 ft³ per capita with plain sedimentation. 1 ft³ with trickling filter. 1.5 ft³ with activated sludge. 2 ft³ with chemical coagulation. Glass covered. Reduce area by 25 percent.

**Vacuum Filtration**
- Lb/ft² dry solids. Prim. 6 to 10, trickling filter 1.5 to 2.0, activated sludge 1 to 2.

**Centrifuge**
- Flow rate based on gpm per HP.

**Wet Combustion**
- Sludge thickener loading of 10 lb/day/ft².

**Land Disposal**
- Stabilized sludge only. See text.

**Incineration**
- Ton/hr. depending on moisture and solids content. Temperature 1250 to 1400°F. Pyrolysis temperature higher.

**Gas Production**
- A properly operated heated digester should produce about 1 ft³ of gas per capita per day from a secondary treatment plant and about 0.8 ft³ from a primary plant. The fuel value of the gas (methane) is about 640 Btu/ft³.
<table>
<thead>
<tr>
<th>Treatment plant</th>
<th>Suspended Solids</th>
<th>Biochemical Oxygen Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedimentation plus sand filter</td>
<td>90-98</td>
<td>85-95</td>
</tr>
<tr>
<td>Sedimentation plus standard trickling filter, 600 lb BOD/acre-ft maximum loading</td>
<td>75-90</td>
<td>80-95</td>
</tr>
<tr>
<td>Sedimentation plus single-stage high-rate trickling filter</td>
<td>50-80</td>
<td>35-65&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>Sedimentation plus two-stage high-rate trickling filter</td>
<td>70-90</td>
<td>80-95&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>Activated sludge</td>
<td>85-95</td>
<td>85-95</td>
</tr>
<tr>
<td>Chemical treatment</td>
<td>65-90</td>
<td>45-80</td>
</tr>
<tr>
<td>Preaeration (1 hour) plus sedimentation</td>
<td>60-80</td>
<td>40-60</td>
</tr>
<tr>
<td>Plain sedimentation</td>
<td>40-70</td>
<td>25-40</td>
</tr>
<tr>
<td>Fine screening</td>
<td>2-20</td>
<td>5-10</td>
</tr>
<tr>
<td>Stabilization (aerobic) pond</td>
<td>—</td>
<td>70-90</td>
</tr>
<tr>
<td>Anaerobic lagoon</td>
<td>70</td>
<td>40-70</td>
</tr>
</tbody>
</table>

<sup>a</sup>No recirculation. Efficiencies can be increased within limits by controlling organic loading, efficiencies of settling tanks, volume of recirculation, and the number of stages; however, effluent will be less nitrified than from standard rate filter, but will usually contain dissolved oxygen. Filter flies and odors are reduced. Study first cost plus operation and maintenance.
12.15 EFFICIENCY EQUATIONS FOR STONE-MEDIA TRICKLING FILTERS

The BOD load on a trickling filter is calculated using the raw BOD in the primary effluent applied to the filter, without regard to the BOD in the recirculated flow. BOD loadings are expressed in terms of pounds of BOD applied per unit of volume per day. Common values are 25–45 lb/1000 ft³/day (400–720 g/m³·d) for single-stage filters and 45–65 lb/1000 ft³/day (720–1040 g/m³·d) for two-stage filters based on the total media volume of both filters.

The hydraulic load is computed from the raw wastewater flow plus recirculated flow. Hydraulic loadings are expressed in terms of average flow in gpm applied per square foot of surface area per day. Recirculation flow is required to maintain an open, well-aerated bed by preventing excessive accumulation of biological growth in the voids and impeding the passage of water and air. The minimum recommended hydraulic loading is 0.16 gpm/ft² (9.4 m³/m²·d). The maximum recommended is 0.48 gpm/ft² (28 m³/m²·d). Above this, the flushing action is excessive and contact time of the wastewater with the filter media becomes too short. The recirculation ratio for this range of hydraulic loadings is usually between 0.5 and 3.0.

Trickling filters have a bed depth of 5–7 ft (1.5–2.1 m) for most efficient BOD removal per unit volume of stone or slag media. In the early development of filters, they were constructed as deep as 8 ft and as shallow as 3 ft. Experience showed that two 3-ft-deep filters in series were no more efficient than one 6-ft-deep filter, and that the filter media below 6 ft in a bed did not result in significant increased BOD removal.

Figure 12.23 Typical recirculation patterns for two-stage trickling filter plants without and with intermediate sedimentation.
General practice in trickling-filter design has been to use empirical relationships to find the required filter volume for a desired degree of wastewater treatment. Several of these associations have been developed from operational data collected at existing treatment plants. One of the first evolved was the National Research Council (NRC) formula, based on data collected from filter plants at military installations in the United States in the early 1940s [9].

The NRC formula for a single-stage trickling filter is

\[ E = \frac{100}{1 + 0.0561 \left( \frac{w}{VF} \right)^{0.5}} \] (12.42)

where
- \( E \) = BOD removal at 20°C, %
- \( w \) = BOD load applied, lb/day
- \( V \) = volume of filter media, \( \text{ft}^3 \times 10^{-3} \)
- \( F \) = recirculation factor
- \( w/V \) = BOD loading, lb/1000 ft³/day

The recirculation factor is calculated from the formula

\[ F = \frac{1 + R}{(1 + 0.1R)^2} \] (12.43)

where \( R \) is the recirculation ratio (ratio of recirculation flow to raw-wastewater flow).

The NRC formula for the second stage of a two-stage filter is

\[ E_s = \frac{100}{1 + \left( 0.0561/\left(1 - E_1 \right) \right) \left( \frac{w_2}{VF} \right)^{0.5}} \] (12.44)

where
- \( E_s \) = BOD removal of the second stage at 20°C, percent
- \( E_1 \) = fraction of BOD removed in the first stage
- \( w_2 \) = BOD load applied to the second stage, lb/day
- \( w_2/V \) = BOD loading, lb/1000 ft³/day

The effect of wastewater temperature on stone-filled trickling-filter efficiency may be expressed as follows:

\[ E = E_{20} \cdot 1.035^{T-20} \] (12.45)

where
- \( E \) = BOD removal efficiency at temperature \( T \) in °C
- \( E_{20} \) = BOD removal efficiency at 20°C

The BOD removal efficiencies computed by the NRC formulas include final settling of the filter effluent. In the empirical development of these formulas, the field procedure used in collecting data sampled the filter influent and final clarifier effluent. Therefore, in evaluating the efficiency of a trickling-filter secondary, the overflow rate and detention time of the final clarifier should be examined for adequacy of design.

For a two-stage filter secondary without an intermediate settling tank (Fig. 12.23), the NRC formulas cannot be used to determine the efficiency of the first stage. In this case it is common to assume that the first-stage efficiency is 50% and find the efficiency of the second stage from Eq. 12.44.
EXAMPLE 12.4
Calculate the BOD and hydraulic loadings and BOD removal efficiency of a single-stage high-rate trickling filter based on the following data:

wastewater flow pattern = as shown in Fig. 12.16a
recirculation rate = as shown in Fig. 12.16a
settled wastewater BOD (primary effluent) = 130 mg/l
diameter of filter = 12.5 m
depth of media = 2.1 m

Solution

raw-wastewater flow = 280 gpm = 1530 m³/d
recirculation flow = 0.50 x 1530 = 765 m³/d

BOD load = 1530 m³/d x 130 mg/l x kg/m³/1000 mg/l = 200 kg/d

surface area of filter = π(12.5)²/4 = 122 m²
volume of media = 122 x 2.1 = 256 m³

BOD loading = 700,000 g / 256 m³ = 781 g/m³-d = 48.8 lb/1000 ft²/day

hydraulic loading = 1530 m³ / 122 m² = 18.8 m³/m²-d = 0.32 gpm/ft²

By Eqs. 12.43 and 12.42,

\[ F = \frac{1 + 0.5}{(1 + 0.1 \times 0.5)} = 1.36 \]

\[ E = \frac{100}{1 + 0.0561(48.8/1.36)^{0.3}} = 75\% \]

\[ \text{T} = -3 \]
EXAMPLE 12.5

The design flow for a new two-stage trickling filter plant is 1.2 mgd with an average BOD concentration of 450 mg/l. Determine the dimensions of the sedimentation tanks and trickling filters (surface areas and depths) for the flow scheme shown in Fig. 12.24. Calculate the volume of filter media based on a loading of 50 lb of BOD/1000 ft²/day, and divide the resulting volume equally between the primary and secondary filters. Estimate the BOD concentration in the plant effluent.

Solution

**Primary Tank**  Criteria: (1) 500-gpd/ft² overflow rate based on raw Q or 750 gpd/ft² based on Q plus recirculation flow; (2) minimum depth of 7 ft. However, if accumulated sludge is to be retained in the bottom of the tank, increase the depth to accommodate the necessary sludge storage volume:

\[
\text{area required} = \frac{1,200,000}{900} = 2400 \text{ ft}^2
\]

or

\[
\text{area required} = \frac{1.5 \times 1,200,000}{750} = 2400 \text{ ft}^2 \quad \text{(Use)}
\]

Estimate the daily sludge accumulation at 4% solids, assuming a sludge solids accumulation equal to 90% of the BOD load:

\[
\text{volume} = \frac{0.9 \times 450 \times 1.2 \times 8.34}{0.04 \times 62.4} = 1620 \text{ ft}^3
\]

\[
\text{depth of sludge} = \frac{1620}{2400} = 0.7 \text{ ft}
\]

Provide a side-wall depth of 8 ft plus freeboard.

**Primary BOD removal** = 35%

---

**Figure 12.24** Flow scheme of the two-stage trickling filter plant for Example 12.5.
Trickling Filters  Criteria: (1) 50 lb/1000 ft²/day BOD loading; (2) 0.16–0.48 gpm/ft² hydraulic loading.

\[
\text{volume required} = \frac{0.65 \times 450 \times 1.2 \times 8.34}{0.050} = 58,500 \text{ ft}^3
\]

volume of each filter = 29,300 ft³

Try 6-ft depth.

Area = \frac{29,300}{6} = 4880 \text{ ft}²

Check the hydraulic loading:

\[
\frac{1.5 + 0.75}{1,200,000} \times \frac{1,200,000}{4880 \times 1440} = 0.38 \text{ gpm/ft}² \quad \text{(OK)}
\]

Use 6-ft-deep filters with a 4880-ft² area.

Intermediate Settling Tank  Criteria: (1) 1000-gpd/ft² overflow rate; (2) minimum depth of 7 ft.

Area required = \frac{1.25 \times 1,200,000}{1000} = 1500 \text{ ft}²

Use a side-wall depth of 7 ft plus freeboard.

Final Settling Tank  Criteria: (1) 800-gpd/ft² overflow rate; (2) minimum depth of 7 ft.

Area required = \frac{1,200,000}{800} = 1200 \text{ ft}²

Use a side-wall depth of 7 ft plus freeboard.

Calculation of BOD removal efficiency

Primary tank = 35%

First-stage filter:

BOD loading = \frac{0.65 \times 450 \times 1.2 \times 8.34}{29.3} = 100 \text{ lb/1000 ft}²/day

\[
R = \frac{0.50Q + 0.75Q}{Q} = 1.25
\]

\[E = 70\%\]

Second-stage filter:

BOD loading = 0.30 \times 100 = 30 \text{ lb/1000 ft}²/day

\[
R = \frac{0.25Q + 0.75Q}{Q} = 1.0
\]

\[F = \frac{1 + 1}{(1 + 0.1 \times 1.0)} = 1.65
\]

By Eq. 12.44,

\[E_1 = \frac{100}{1 - [(0.0561/11 - 0.70)(10/1.65)0.5]} = 56\%
\]

The plant efficiency is

\[E = 100 - 100[(1 - 0.35)(1 - 0.70)(1 - 0.56)] = 92\%
\]

The estimated effluent BOD is \[0.08 \times 450 = 38 \text{ mg/l}\].
Lamp holes: Sewer inspection holes large enough to lower a lamp into but too small for a man.

Lateral: A sewer line which goes off at right angles to another.

Main: A large sewer at which all other branches terminate.

Malodorous: Offensive smelling.

Mesophilic bacteria: Bacteria growing between 10 and 40°C, with an optimum temperature of 37°C. 40°C is, therefore, the upper limit for most wastewater processes.

Mohlman index: Same as the 'Sludge volume index.'

Nitrogenous demand: Oxygen demand from nitrogen-consuming bacteria.

Outfall: The pipe which discharges completely treated wastewater into a lake, stream, or ocean.

Partial treatment: Primary treatment only.

Post-chlorination: Addition of chlorine after all other processes have been completed.

Pre-chlorination: Addition of chlorine prior to sedimentation to help control odors and to aid in grease removal.


Refractory: Dissolved organic materials which are biologically resistant and difficult to remove.

Regulator: A device or weir which deflects large volume flows into a special high-capacity sewer.

Sag pipe: See 'Inverted siphon.'

Second stage demand: See 'Nitrogenous demand.'

Seed: The activated sludge initially taken from the secondary settling tank and returned to the aeration tank to start the activated sludge process.

Separate system: Separate sewers for domestic and storm waste water.

Septic: Produced by putrefaction.

Sludge bulking: Failure of suspended solids to completely settle out.

Split chlorination: Addition of chlorine prior to sedimentation and after final processing.

Submain: See 'Branch.'

Supernatant: The clarified liquid floating on top of a digesting sludge layer.

Thermophilic bacteria: Bacteria which thrive in the 45°C to 75°C range (optimum near 55°C).

Volatile solid: Solid material in a water sample or in sludge which can be burned or vaporized at high temperature.

Wet well: A short-term storage tank containing a pump or pump entrance, and into which the raw influent is brought.

Zooglea: The gelatinous film of aerobic organisms which cover the rocks in a trickling filter.

3 WASTEWATER QUALITY CHARACTERISTICS

A. DISSOLVED OXYGEN

Fish and most aquatic life require oxygen. The biological decomposition of organic solids is also dependent on oxygen. If the dissolved oxygen content of water is less than the saturated values given in Appendix B, there is good reason to believe that the water is organically polluted. Other reasons for measuring the dissolved oxygen concentration are for aerobic treatment monitoring, aeration process monitoring, BOD testing, and pipe corrosion studies.

The difference between the saturated and actual dissolved oxygen concentrations is known as the oxygen deficit.

\[ D = DO_{sat} - DO \]  \hspace{1cm} 8.1

The oxygen deficit is reduced by aerating the water (i.e., the dissolved oxygen concentration is increased). An exponential decay is traditionally used to predict the oxygen deficit as a function of time. Equation 8.2 assumes that oxygen is not being depleted during the reoxygenation process.

\[ D_t = D_0 10^{-K_R t} \]  \hspace{1cm} 8.2

\( K_R \) is the reoxygenation (reo aireation) coefficient, which depends on the type of flow and temperature. Reoxygenation coefficients are also given for use with a different logarithmic base.

\[ D_t = D_0 e^{-K_R t} \]  \hspace{1cm} 8.3

1 4-6 mg/l is the generally accepted range of dissolved oxygen required to support fish populations. 5 mg/l is adequate, as is verifiable from high-altitude trout lakes. However, 6 mg/l is preferable, particularly for large fish populations.

2 \( K_R \) may be written as \( K_s \) in the literature.
The constants \( K_R \) and \( K'_R \) are not the same, but they are related.\(^3\)
\[
K'_R = 2.3 K_R
\]
\[8.4\]

Table 8.12 lists representative values of \( K_R \).

**B. BIOCHEMICAL OXYGEN DEMAND**

When oxidizing organic waste material in water, biological organisms remove oxygen from the water. Therefore, oxygen use is an indication of the organic waste content. The biochemical oxygen demand (BOD) of a biologically active sample is given by equation 8.5:

\[
\text{BOD}_s = \frac{DO_i - DO_f}{V_{\text{sample}} + V_{\text{dilution}}}
\]

\[8.5\]

BOD is determined by adding a measured amount of wastewater (which supplies the organic material) to a measured amount of dilution water (which reduces toxicity and supplies dissolved oxygen). An oxygen use curve similar to that in figure 8.1 will result. (More than one identical sample must be prepared in order to determine initial and final concentrations of dissolved oxygen.)

![Figure 8.1 BOD Time Curve](image)

The deviation from the expected exponential growth curve in figure 8.1 is due to nitrification or nitorgenous demand. Nitrification is the use of oxygen by autotrophic bacteria.\(^4\) Such bacteria use fixed carbon as food. (For example, the carbon in carbon dioxide is used by autotrophic bacteria.) Autotrophic bacteria oxidize ammonia to nitrites and nitrates. However, the number of autotrophic bacteria is small. Generally, six to ten days are required for the autotrophic population to become sufficiently large enough to affect a BOD test. Therefore, the standard BOD test is terminated before the autotrophic contribution to BOD becomes significant.

The standard BOD test typically calls for a 5-day incubation period at 20\(^°\)C. The BOD at any time can be found from equation 8.6.

\[
\text{BOD}_t = \text{BOD}_u (1 - 10^{-K_{D}t})
\]

\[8.6\]

\( K_D \) is the deoxygenation rate constant, typically taken as 0.1. The ultimate BOD cannot be found from long term studies due to the effect of nitrogen-consuming bacteria in the sample. However, if \( K_D \) is 0.1, the ultimate BOD can be found from equation 8.7.

\[
\text{BOD}_u \approx 1.47 \text{BOD}_s
\]

\[8.7\]

\( K_D \) for other temperatures can be found from equation 8.8. (The 1.047 constant is often quoted in literature. Recent research suggests 1.135 for 4\(^°\)C to 20\(^°\)C, and 1.056 for 20\(^°\)C to 30\(^°\)C.)

\[
K_{D,T} = (1.047)^{T-20} K_{D,20°C}
\]

\[8.8\]

The variation in BOD with temperature is given by equation 8.9.

\[
\text{BOD}_T = \text{BOD}_{20°C}(0.02T + 0.6)
\]

\[8.9\]

**Table 8.1**

Typical Values of \( K_D \)

<table>
<thead>
<tr>
<th>treatment plant effluents</th>
<th>highly polluted shallow streams</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05 0.10</td>
<td>0.25</td>
</tr>
</tbody>
</table>

**Example 8.1**

Ten 5-ml samples of wastewater are placed in 300 ml BOD bottles. Half of the bottles are titrated immediately with an average initial concentration of dissolved oxygen of 7.9 mg/l. The remaining bottles are incubated for 5 days, after which the average dissolved oxygen is determined to be 4.5 mg/l. What is the standard BOD and ultimate carbonaceous BOD assuming \( K_D = 0.13? \)

From equation 8.5:

\[
\text{BOD}_s = \frac{7.9 - 4.5}{5} = 204 \text{ mg/l}
\]

\[\text{68}\]
From equation 8.6, the ultimate BOD is

$$\text{BOD}_u = \frac{204}{1 - 10^{-0.13(5)}} = 263 \text{ mg/l}$$

If a sample of industrial wastewater is taken, it will probably lack sufficient microorganisms to metabolize the organic matter. In such a case, seed organisms must be added. The BOD for seeded experiments is found by measuring dissolved oxygen in the seeded sample after 15 minutes ($DO_1$) and after 5 days ($DO'_5$), as well as the dissolved oxygen of the seed material itself after 15 minutes ($DO^*_{15}$) and after 5 days ($DO^*_{5}$).

$$\text{BOD}_s = \frac{DO_1 - DO'_5 - x(\text{DO}^*_{15} - \text{DO}^*_{5})}{V_{\text{sample}} + V_{\text{dilution}}}$$  \hspace{1cm} \text{Eq. 8.10}

$$x = \frac{\text{volume of seed added to sample}}{\text{volume of seed used to find } \text{DO}^*_{15}}$$  \hspace{1cm} \text{Eq. 8.11}

The BOD of domestic waste is typically taken as 0.17 to 0.20 pounds per capita-day, excluding industrial wastes. This makes it possible to calculate the population equivalent of any BOD loading.

$$P_e = \left(\frac{\text{BOD mg/l}}{1,000,000 \text{ gal/MG}}\right) \left(\frac{Q \text{ gal/day}}{1000}\right) \left(8.345 \frac{\text{lbm-l}}{\text{MG-mg}}\right) \left(0.17 \frac{\text{lbm}}{\text{person-day}}\right)$$  \hspace{1cm} \text{Eq. 8.12}

Values of BOD for various industrial wastewaters are given in table 8.2.

BOD of 100 mg/l is considered a weak wastewater, BOD of 200 to 250 mg/l is considered a medium strength wastewater, above 300 mg/l, it is considered to be a strong wastewater.

C. RELATIVE STABILITY

The relative stability test is much easier to perform than the BOD test, although it is much less accurate. The relative stability of an effluent is defined as the percent of initial BOD that has been satisfied. The test consists of taking a sample of effluent and adding a small amount of methylene blue dye. When all oxygen has been removed from the water, anaerobic bacteria start to remove the dye. The time for the color to start degrading is known as the stabilization time or decoloration time.

The relative stability can be found from the stabilization time by using table 8.3.

### Table 8.3
Relative Stability (at 20°C)

<table>
<thead>
<tr>
<th>stabilization time (days)</th>
<th>relative stability %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>11</td>
</tr>
<tr>
<td>1</td>
<td>21</td>
</tr>
<tr>
<td>1 1/2</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>37</td>
</tr>
<tr>
<td>2 1/2</td>
<td>44</td>
</tr>
<tr>
<td>3</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>68</td>
</tr>
<tr>
<td>6</td>
<td>75</td>
</tr>
<tr>
<td>7</td>
<td>80</td>
</tr>
</tbody>
</table>

### Table 8.2
Typical BOD and COD of Industrial Wastewaters

<table>
<thead>
<tr>
<th>industry/type of waste</th>
<th>BOD</th>
<th>COD</th>
</tr>
</thead>
<tbody>
<tr>
<td>canning</td>
<td>19.5 lbm/ton corn</td>
<td></td>
</tr>
<tr>
<td>corn</td>
<td>8.4 lbm/ton tomatoes</td>
<td></td>
</tr>
<tr>
<td>tomatoes</td>
<td>1150 lbm/ton raw milk</td>
<td></td>
</tr>
<tr>
<td>dairy milk processing</td>
<td>1000 mg/l</td>
<td></td>
</tr>
<tr>
<td>beer brewing</td>
<td>1.2 lbm/barrel beer</td>
<td></td>
</tr>
<tr>
<td>commercial laundry</td>
<td>1250 lbm/1000 pounds dry</td>
<td></td>
</tr>
<tr>
<td>slaughterhouse</td>
<td>7.7 lbm/animal</td>
<td></td>
</tr>
<tr>
<td>(meat packing)</td>
<td>1400 mg/l</td>
<td></td>
</tr>
<tr>
<td>papermill</td>
<td>121 lbm/ton pulp</td>
<td></td>
</tr>
<tr>
<td>synthetic textile</td>
<td>1500 mg/l</td>
<td></td>
</tr>
<tr>
<td>chlorophenolic</td>
<td>4300 mg/l</td>
<td></td>
</tr>
<tr>
<td>manufacturing</td>
<td>230 mg/l</td>
<td></td>
</tr>
<tr>
<td>milk bottling</td>
<td>3200 mg/l</td>
<td></td>
</tr>
<tr>
<td>cheese production</td>
<td>4300 mg/l</td>
<td></td>
</tr>
<tr>
<td>candy production</td>
<td>1600 mg/l</td>
<td></td>
</tr>
</tbody>
</table>
Example 8.2

A sample treatment plant effluent begins to clarify after 13 days. What percent of the original BOD remains unsatisfied?

From table 8.3, the relative stability is 95%. Therefore, only 5% of the initial BOD remains unsatisfied.

D. CHEMICAL OXYGEN DEMAND

Unlike BOD, which is a measure of oxygen removed by biological organisms, chemical oxygen demand (COD) is a measure of maximum oxidizable substances. Therefore, COD is an excellent measure of effluent strength.

COD testing is required in environments of chemical pollution. In such environments, the organisms necessary to metabolize organic compounds may not exist. Furthermore, the toxicity of the water may make the standard BOD test impossible to carry out. The COD test also produces results faster than the BOD test. COD test results are usually available in a matter of hours.

If the toxicity is low, BOD and COD test results can be correlated. The BOD/COD ratio typically varies from 0.4 to 0.8. This is a wide range, but for any given treatment plant and waste type, the correlation is essentially constant. The correlation can, however, vary along the treatment path.

E. CHLORINE DEMAND

Chlorination destroys bacteria, hydrogen sulfide, and other noxious substances. For example, hydrogen sulfide is oxidized according to equation 8.13.

\[ \text{H}_2\text{S} + 4\text{H}_2\text{O} + 4\text{Cl}_2 \rightarrow \text{H}_2\text{SO}_4 + 8\text{HCl} \] 8.13

Chlorine demand is the amount of chlorine (or its chloramine or hypochlorite equivalent) required to give a 0.5 mg/l residual after 15 minutes of contact time. 15 minutes is the recommended contact and mixing time prior to discharge since this period will kill nearly all pathogenic bacteria in the water. Typical doses for wastewater effluent are given in table 8.4.

<table>
<thead>
<tr>
<th>final process</th>
<th>dose (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>no treatment (straight discharge)</td>
<td>10-30</td>
</tr>
<tr>
<td>secondary sand filter</td>
<td>2-6</td>
</tr>
<tr>
<td>secondary activated sludge</td>
<td>2-8</td>
</tr>
<tr>
<td>secondary trickling filter</td>
<td>3-15</td>
</tr>
<tr>
<td>primary sedimentation</td>
<td>5-25</td>
</tr>
</tbody>
</table>

Table 8.4

Typical Chlorine Doses

In actuality, the chlorine dose needs to be determined by careful monitoring of coliform counts and free residuals since there are several ways that chlorine can be used up without producing significant disinfection. Only after uncombined (free) chlorine starts showing up is it assumed that all chemical reactions and disinfection are complete.\footnote{Chlorine kills most bacteria, but many viruses are resistant.}

Because of their reactivity, chlorine is initially used up in the neutralization of hydrogen sulfide and the rare ferrous and manganous (Fe<sup>2+</sup> and Mn<sup>2+</sup>) ions. The resulting HCl, FeCl<sub>2</sub>, and MnCl<sub>2</sub> ions do not contribute to disinfection. They are known as unavailable combined residuals.

Plants and animals use nitrogen. Bacterial decomposition and the hydrolysis of urea produces ammonia, NH<sub>3</sub>. This ammonia, once it enters the wastewater stream, forms ammonium ion, NH<sub>4</sub><sup>+</sup>, also known as ammonia nitrogen.

Ammonia nitrogen combines with chlorine to form the family of chloramines. Depending on the water pH, monochloramines (NH<sub>2</sub>Cl), dichloramines (NHCl<sub>2</sub>), or trichloramines (nitrogen trichloride, NCl<sub>3</sub>) may form.\footnote{Lower pH favors the formation of di- and trichloramines}

Chloramines have long-term disinfection capabilities, and chloramines are therefore known as available combined residuals. Equation 8.14 is a typical chloramine formation reaction.

\[ \text{NH}_4^+ + \text{HOCI} \rightarrow \text{NH}_2\text{Cl} + \text{H}_2\text{O} + \text{H}^+ \] 8.14

The continued addition of chlorine after chloramine formation changes the pH, and chloramine destruction begins. Chloramines are converted to nitrogen gas (N<sub>2</sub>) and nitrous oxide (N<sub>2</sub>O). The destruction of chloramines continues with the continued application of chlorine, until no ammonia remains in the water. The point at which all ammonia has been removed is known as the breakpoint. Equation 8.15 is a typical chloramine destruction reaction.

\[ 2\text{NH}_2\text{Cl} + \text{HOCI} \rightarrow \text{N}_2 + 3\text{HCl} + \text{H}_2\text{O} \] 8.15

In the breakpoint chlorination method, additional chlorine is added after the breakpoint in order to obtain free chlorine residuals. The free residuals have a high disinfection capacity. Typical free residuals are free chlorine (Cl<sub>2</sub>), hypochlorous acid (HOCI), and hypochlorite ions. Equations 8.16 and 8.17 illustrate the formation of these free residuals.

\[ \text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{HCl} + \text{HOCl} \] 8.16

\[ \text{HOCl} \rightarrow \text{H}^+ + \text{ClO}^- \] 8.17
There are several problems associated with breakpoint chlorination.

- It may not be economical to use breakpoint chlorination unless the ammonia nitrogen has been reduced.
- Free chlorine residuals favor the formation of trihalomethanes. Where free residuals are not permitted, the water may need to be dechlorinated using sulfur dioxide gas or sodium bisulfate. Where small concentrations of free residuals are permitted, dechlorination may be needed only during the dry months. During winter storm months, the chlorine residuals may be adequately diluted with rain water.)

F. GREASE

Greases are organic substances including fats, vegetable and mineral oils, waxes, fatty acids from soaps, and other hydrocarbons. Grease's low solubility causes adhesion problems in pipes and tanks, reduces contact area during various filtering processes, and produces sludge which is difficult to dispose of.

G. VOLATILE ACIDS

Volatile acids (acetic, propionic, and butyric) occur in anaerobically digested sludge. These acids can be used to indicate the completion of a sludge digestion process. Acid content is given in mg/l as acetic acid.

II. SUSPENDED SOLIDS

Suspended solids, as in water supply engineering, can be categorized in several ways. Generally, suspended solids constitute only a small amount of the incoming flow, less than 1/10%. Together with dissolved solids, suspended solids constitute total solids. Figure 8.3 illustrates the relationships between the various solids categories. A further division of each category into organic and inorganic solids is possible.

The term volatile solids can be used as a measure of organic pollutants capable of affecting the oxygen content of the flow. Volatile solids, as in water supply testing, are measured by igniting filtered solids, and measuring the decrease in mass.
Refractory pollutants are solids which are difficult to remove by common processes. In this case, the term "refractory" is used to mean "stubborn."

4 DISINFECTION

Chlorine gas is the least expensive, and therefore the most common, method of disinfecting wastewater. However, chlorine gas is toxic, corrosive, and displaces oxygen since it is heavier than air. Chlorine gas also lowers the pH of the water, favoring the formation of combined residuals.

Because of these disadvantages, alternatives to chlorine gas need to be considered when recommending the disinfection method.

- **Hypochlorites**: Both sodium and calcium hypochlorite are solids that dissolve in water. They have a limited shelf life, and are susceptible to photodecomposition. Hypochlorites are less effective and slightly more expensive than chlorine gas.

- **Chlorine Dioxide Gas**: Cl₂O is explosive and must be generated on-site. It reacts with many compounds, requiring larger doses than chlorine gas. However, it combines with organics without combining with ammonia.

- **Ozone Gas**: Ozone is one of the most effective oxidizing agents. In addition to its disinfection capabilities, ozone also increases the dissolved oxygen content of water. Ozone is toxic and corrosive. It must be generated at the point of application. Because of its very short half-life, step feeding is required to obtain the necessary contact period.

- **Exotics**: Other methods exist, but are typically high in cost. These exotic alternatives include bromine (bromine chloride), iodine, silver oxide, gamma radiation, and ultraviolet radiation.

5 TYPICAL COMPOSITION OF DOMESTIC SEWAGE

Not all sewage flows are the same. Some sewages are stronger than others. Table 8.5 lists typical values for strong and weak domestic sewages. A medium classification would be approximately midway between the values for strong and weak.

6 WASTEWATER QUALITY STANDARDS

Applicable wastewater quality standards have been set by both the Water Pollution Control Act and the Environmental Protection Agency. General standards set by the Water Pollution Control Act are given in Table 8.6. These standards must be met by facilities that receive federal funding.

The EPA's standards for secondary treatment are given in terms of 5-day BOD, suspended solids, coliform count, and pH. Table 8.7 presents typical values.

### Table 8.5

**Strong and Weak Domestic Sewages**

(All concentrations in mg/l unless noted.)

<table>
<thead>
<tr>
<th>constituent</th>
<th>strong</th>
<th>weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>solids, total</td>
<td>1200</td>
<td>350</td>
</tr>
<tr>
<td>dissolved, total</td>
<td>850</td>
<td>250</td>
</tr>
<tr>
<td>fixed</td>
<td>525</td>
<td>145</td>
</tr>
<tr>
<td>volatile</td>
<td>325</td>
<td>105</td>
</tr>
<tr>
<td>suspended, total</td>
<td>350</td>
<td>100</td>
</tr>
<tr>
<td>fixed</td>
<td>75</td>
<td>30</td>
</tr>
<tr>
<td>volatile</td>
<td>275</td>
<td>70</td>
</tr>
<tr>
<td>settleable solids. (ml/liter)</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>biochemical oxygen demand</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>5-day, 20°C</td>
<td></td>
<td></td>
</tr>
<tr>
<td>total organic carbon</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>chemical oxygen demand</td>
<td>1000</td>
<td>250</td>
</tr>
<tr>
<td>nitrogen. (total as N)</td>
<td>85</td>
<td>20</td>
</tr>
<tr>
<td>organic</td>
<td>35</td>
<td>8</td>
</tr>
<tr>
<td>free ammonia</td>
<td>50</td>
<td>12</td>
</tr>
<tr>
<td>nitrites</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>nitrates</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>phosphorus (total as P)</td>
<td>20</td>
<td>6</td>
</tr>
<tr>
<td>organic</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>inorganic</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>chlorides</td>
<td>100</td>
<td>30</td>
</tr>
<tr>
<td>alkalinity (as CaCO₃)</td>
<td>200</td>
<td>50</td>
</tr>
<tr>
<td>grease</td>
<td>150</td>
<td>50</td>
</tr>
</tbody>
</table>

7 DESIGN FLOW QUANTITY

Approximately 70 to 80% of a community's domestic and industrial water use will return as wastewater. This water is discharged into the sewer systems, which may be different or the same as storm drains. Therefore, the nature of the return system must be known before sizing can occur.

Sanitary sewer sizing can often be based on an average of 100-125 gpcd. There will be variations with time in the flow, although the variations are not as pronounced as they are for water supply. Table 8.8 lists peak multipliers for treatment plant influent volume. Due to storage in ponds, clarifiers, and sedimentation basins, these multipliers may not be applicable throughout all processes in the treatment plant.
Table 8.6
Typical Surface Water Standards

<table>
<thead>
<tr>
<th>water use</th>
<th>minimum dissolved oxygen (mg/l)</th>
<th>maximum dissolved solids</th>
<th>maximum coliforms per 100 ml</th>
</tr>
</thead>
<tbody>
<tr>
<td>domestic use (food preparation)</td>
<td>6</td>
<td>*</td>
<td>none</td>
</tr>
<tr>
<td>water contact recreation</td>
<td>4 to 6</td>
<td>*</td>
<td>1000 total ave.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200 fecal ave.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>not more than</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10% exceeding</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>400 fecal (2000 total)</td>
</tr>
<tr>
<td>fisheries</td>
<td>4 to 6</td>
<td>*</td>
<td>5000 ave.</td>
</tr>
<tr>
<td>industrial supply</td>
<td>3 to 5</td>
<td>750 to 1500 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>agricultural irrigation</td>
<td>3 to 5</td>
<td>750 to 1500 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>shellfish harvesting</td>
<td>4 to 6</td>
<td>*</td>
<td>70 total ave.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>not more than</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10% exceeding 230</td>
</tr>
</tbody>
</table>

* No floating solids or settling solids that form deposits.

n codes established by state boards of health fre-e' specify a design loading of 400 gpcd (lateral
s and 250 gpcd (mains, trunks, and out-
Both of these include the effect of infiltration. ation due to cracks and poor joints is limited by
municipal codes to 500 gallons per day per mile
per inch of diameter. Modern piping mate-
and joints should be able to reduce this quantity
1 gpd/inch-mile. Infiltration may also be roughly
ated at 3%-5% of the peak hourly domestic rate, 10%
of the average rate.

8 COLLECTION SYSTEMS

A. STORM DRAINS AND INLETS

Curb inlets to storm drains should be placed no more
than 600 feet apart, and a limit of 300 feet is advisable.
Inlets are required at all low points where pondage could
occur. A common practice is to install 3 inlets in a sag
vertical curve—one at the lowest point and one on each
side with an elevation of 0.2 feet above the center inlet.
Openings may be of the covered grate type or the curb
inlet type.

Table 8.7
Typical Secondary Effluent Standards

<table>
<thead>
<tr>
<th>identity</th>
<th>average discharge (mg/l)</th>
<th>maximum discharge (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 (5-day)</td>
<td>30 days</td>
<td>30 mg/l</td>
</tr>
<tr>
<td></td>
<td>7 days</td>
<td>45 mg/l</td>
</tr>
<tr>
<td></td>
<td>30 days</td>
<td>15% of incoming BOD</td>
</tr>
<tr>
<td>ended solids</td>
<td>30 days</td>
<td>30 mg/l</td>
</tr>
<tr>
<td></td>
<td>7 days</td>
<td>45 mg/l</td>
</tr>
<tr>
<td></td>
<td>30 days</td>
<td>15% of incoming SS</td>
</tr>
<tr>
<td>seasonals*</td>
<td>30 days</td>
<td>200 per 100 ml</td>
</tr>
<tr>
<td></td>
<td>7 days</td>
<td>400 per 100 ml</td>
</tr>
<tr>
<td>at all times</td>
<td>within 6 to 9</td>
<td></td>
</tr>
</tbody>
</table>

* Arithmetic mean is used, not arithmetic.

Table 8.8
Variations in Wastewater Flow
(based on the average daily flow)

<table>
<thead>
<tr>
<th>description</th>
<th>when/where</th>
<th>variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>daily peak</td>
<td>10-12 a.m. (residential)</td>
<td>225%</td>
</tr>
<tr>
<td></td>
<td>constant during day</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(commercial)</td>
<td>150%</td>
</tr>
<tr>
<td></td>
<td>12 noon at the outfall</td>
<td>150%</td>
</tr>
<tr>
<td>daily minimum</td>
<td>4-5 a.m.</td>
<td>40%</td>
</tr>
<tr>
<td>seasonal peak</td>
<td>late summer</td>
<td>125%</td>
</tr>
<tr>
<td>seasonal minimum winter's end</td>
<td></td>
<td>90%</td>
</tr>
<tr>
<td>seasonal average</td>
<td>May, June</td>
<td>100%</td>
</tr>
<tr>
<td>maximum peaks</td>
<td>in laterals</td>
<td>300%</td>
</tr>
<tr>
<td></td>
<td>treatment plant influent</td>
<td>200%</td>
</tr>
</tbody>
</table>
11 DILUTION PURIFICATION

Dilution purification (also known as self purification) refers to discharge of partially treated sewage into a body of water such as a stream or river. If the body is large and is adequately oxygenated, the sewage’s BOD may be satisfied without putrefaction. Other conditions which must be monitored besides BOD are oxygen content and suspended solids.

Equation 8.20 can be used to calculate the final concentration of BOD, oxygen, and sediment when the two flows are mixed. Dilution requirements may be expressed in terms of ratios (e.g., 23 stream volumes per discharge volume) or absolute flow quantities (e.g., 4 to 7 cfs per 1000 population).

\[ C_1 Q_1 + C_2 Q_2 = C_f (Q_1 + Q_2) \quad 8.20 \]

Example 8.3

Wastewater (DO = 0.9 mg/l, 6 MGD) is discharged into a 50°F stream flowing at 40 cfs. Assuming the stream is saturated with oxygen, what is the oxygen content of the stream immediately after mixing?

From appendix B, the saturated oxygen content at 50°F (10°C) is 11.3 mg/l.

\[ (6 \text{ MGD}) \left( \frac{1.547 \text{ cfs}}{\text{MGD}} \right) = 9.28 \text{ cfs} \]

\[ C = \frac{(0.9)(9.28) + (11.3)(40)}{9.28 + 40} = 9.34 \text{ mg/l} \]

The oxygen deficit is the difference between actual and saturated oxygen concentrations. Since reoxygenation and deoxygenation of a polluted river occur simultaneously, an oxygen deficit will occur only if the reoxygenation rate is less than the deoxygenation rate. If the oxygen content goes to zero, anaerobic decomposition and putrefaction will occur.

The oxygen deficit at any time t is given by the Streeter-Pheils equation:

\[ D_t = DO_{sat} - DO_t \]

\[ = \frac{K_D BOD_u}{K_R - K_D} \left( 10^{-K_R t} - 10^{-K_D t} \right) \]

\[ + D_o \left( 10^{K_R t} \right) \quad 8.21 \]

\[ D_t \] is the dissolved oxygen deficit, \( t \) is in days, and \( BOD_u \) is the ultimate carbonaceous BOD of the stream immediately after mixing. \( K_D \) and \( K_R \) are the deoxygenation and reoxygenation rate constants respectively, and \( D_o \) is the dissolved oxygen deficit immediately after mixing.\(^9\)

\( K_R \) can be approximated by equation 8.22 if field test data is not available.\(^10\)

\[ K_{R, 20^°C} \approx \frac{3.3v}{d_1 x_3} \quad 8.22 \]

\( K_R \) for different temperatures is given by equation 8.23. Typical values of \( K_R \) are given in table 8.12.

\[ K_{RT} = (1.016)T^{-20}K_{R, 20^°C} \quad 8.23 \]

Table 8.12

<table>
<thead>
<tr>
<th>Condition</th>
<th>Typical Reoxygenation Constants (base 10, 1/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>white water</td>
<td>0.5 and above</td>
</tr>
<tr>
<td>swiftly flowing</td>
<td>0.3 to 0.5</td>
</tr>
<tr>
<td>large streams</td>
<td>0.15 to 0.3</td>
</tr>
<tr>
<td>large lakes</td>
<td>0.10 to 0.15</td>
</tr>
<tr>
<td>sluggish streams</td>
<td>0.10 to 0.15</td>
</tr>
<tr>
<td>small ponds</td>
<td>0.05 to 0.10</td>
</tr>
</tbody>
</table>

Equations 8.6 and 8.21 can be plotted simultaneously as shown in figure 8.5. The plot of equation 8.21 is known as the oxygen sag curve. The difference between the two curves is the effect of reoxygenation.

Figure 8.5  The Oxygen Sag Curve

\(^9\) \( K_R \) and \( K_D \) may be written as \( K_1 \) and \( K_2 \) by other authors.

\(^{10}\) Equation 8.22 is the O’Connor and Dobbins formula for natural streams.
The time to the minimum or critical point on the sag curve is given by equation 8.24.

\[ t_c = \frac{1}{K_R - K_D} \times 
\log_{10} \left[ \left( \frac{K_D BOD_u - K_R D_o + K_D D_o}{K_D BOD_u} \right) \left( \frac{K_R}{K_D} \right) \right] \]

8.24

The ratio of \( K_R/K_D \) is known as the self-purification coefficient. The critical oxygen deficit is given by equation 8.25.

\[ D_c = \left( \frac{K_D BOD_u}{K_R} \right) 10^{-K_D t_c} \]

8.25

Knowing \( t_c \) and the stream flow velocity will locate the point where the oxygen level is the lowest.

**Example 8.4**

A treatment plant discharge has the following characteristics:

- 15 cfs
- 45 mg/l BOD (5 day, 20°C)
- 2.9 mg/l DO
- 24°C
- \( K_D,20°C = 0.1 \) per day (when mixed with river water)

The outfall is located in a river with the following characteristics:

- 0.55 ft/sec velocity
- 4.0 feet average depth
- 120 cfs
- 4 mg/l BOD (5 day, 20°C)
- 8.3 mg/l DO
- 16°C

Determine the distance downstream where the oxygen level is minimum, and predict if the river can support fish life at that point.

**Step 1:** Find the river conditions immediately after mixing. Use equation 8.20 three times.

\[
(BOD)_{20°C} = \frac{(15)(45) + (120)(4)}{15 + 120} = 8.56 \text{ mg/l}
\]

\[
DO = \frac{(15)(2.9) + (120)(8.3)}{135} = 7.7 \text{ mg/l}
\]

\[
T = \frac{(15)(24) + (120)(16)}{135} = 16.89°C
\]

**Step 2:** Calculate the rate constants. From equation 8.8,

\[
K_D,16.89°C = (0.1)(1.047)^{16.89-20} = 0.0867
\]

From equation 8.22,

\[
K_R,20°C = 3.3(0.55/4)^{1.33} = 0.287
\]

From equation 8.23,

\[
K_D,16.89°C = (0.287)(1.016)^{16.80-20} = 0.275
\]

**Step 9:** Estimate BOD\(_u\). Using equation 8.6,

\[
(BOD_u)_{20°C} = \frac{8.56}{1 - (10)^{-0.0867(15)}} = 13.56
\]

From equation 8.9,

\[
(BOD_u)_{16.89°C} = (13.56)(0.02)(16.89 + 0.6) = 12.72
\]

**Step 4:** Calculate \( D_o \). From appendix B, the saturated oxygen concentration at 16.89°C is approximately 9.7 mg/l. So, \( D_o = 9.7 - 7.7 = 2.0 \).

**Step 5:** Calculate \( t_c \) from equation 8.24.

\[
t_c = \frac{1}{0.275 - 0.0867} \times 
\log \left[ \frac{(0.0867)(12.72) - (0.275)(2) + (0.0867)(2)}{(0.0867)(12.72)} \times \left( \frac{0.275}{0.0867} \right) \right]
\]

\[
= 1.70 \text{ days}
\]

**Step 6:** The distance downstream is

\[
\frac{(1.70 \text{ days})(0.55 \text{ ft/sec})(86,400 \text{ sec/day})}{(5280 \text{ ft/mile})} = 15.3 \text{ miles}
\]

**Step 7:** The critical oxygen deficit is found from equation 8.25.

\[
D_c = \frac{(0.0867)(13.56)}{0.275}(10)^{-0.0867(1.77)} = 3.0
\]

**Step 8:** If the temperature 15.9 miles downstream is 16°C, the saturated oxygen content is 10 mg/l. Since the critical deficit is 3, the minimum oxygen content is 7 mg/l. This is adequate for fish life.
3 WASTEWATER PLANT SITING CONSIDERATIONS

Wastewater plants should be located as far as possible from inhabited areas. A minimum distance of 1000 feet for uncovered plants is desired. Uncovered plants should be located downwind when a definite wind direction prevails. Foundation conditions need to be evaluated, as does the elevation of the water table. Elevation in relationship to the need for sewage pumping (and for dikes around the site) is relevant. Furthermore, the plant must be protected against flooding. 100-year storms are typically chosen as the design flood when designing dikes and similar facilities. Distance to the outfall and possible effluent pumping need to be considered.

Table 8.13 lists the approximate acreage for preliminary engineering estimates. Of course, an estimate of expansion is proper when evaluating acreage requirements.

<table>
<thead>
<tr>
<th>type of treatment</th>
<th>acres per MGD</th>
</tr>
</thead>
<tbody>
<tr>
<td>activated sludge plants</td>
<td>2</td>
</tr>
<tr>
<td>trickling filter plants</td>
<td>3</td>
</tr>
<tr>
<td>aerated lagoons</td>
<td>10</td>
</tr>
<tr>
<td>stabilization basins</td>
<td>20</td>
</tr>
<tr>
<td>physical-chemical plants</td>
<td>1.5</td>
</tr>
</tbody>
</table>

15 WASTEWATER PROCESSES

A. PRELIMINARY TREATMENT

Preliminary preparation of the wastewater stream is essentially a mechanical process. It removes large objects, rags, and wood from the flow. Heavy solids and excessive oils and grease are also eliminated. Damage to pumps and other equipment would be expected without preliminary treatment.

Screens: Trash racks or coarse screens with openings 2 inches or larger should precede pumps to prevent clogging. Screenings usually consist of paper, wood, and rags. Medium screens (1/2" to 1 1/2" openings) and fine screens (1/16" to 1/8") are also used to relieve the load on grit chambers and sedimentation basins. Screens are cleaned by automatic scraping arms. Screen capacities and head losses are specified by the manufacturer. In general, however, flow through screens should be limited to 3 fps or less.

Grit Chambers: Grit is an abrasive that wears pumps, clogs pipes, and accumulates in excessive volumes. A grit chamber (also known as a grit clarifier or detritus tank) slows the wastewater down to approximately 1 ft/sec. This velocity allows the grit to settle out but moves the organic matter through. The grit can be manually or mechanically removed with buckets or screw conveyors.

11 Fine screens are rare except when used with some industrial waste processing plants.
in holding basins can also be loosely categorized as pre-treatment processes.

B. PRIMARY TREATMENT

Primary treatment is a mechanical (settling) process used to remove most of the settleable solids. A 25 to 35% reduction in BOD is also achieved, but BOD reduction is not the goal of primary treatment.

Plain Sedimentation: Plain sedimentation basins are described in chapter 7. Design characteristics for wastewater treatment are:

- **BOD reduction**
  - 20% to 40%
- **total suspended solids reduction**
  - 35% to 65%
- **bacteria reduction**
  - 50% to 60%
- **organic content of settled solids**
  - 50% to 75%
- **specific gravity of settled solids**
  - 1.2 or less
- **typical settling velocity**
  - above 4 feet/hr
- **plan shape**
  - rectangular or circular
- **basin depth**
  - 6 to 15 feet
  - (12 typical)
- **basin width**
  - 10 to 50 feet
- **minimum freeboard**
  - 18 inches
- **minimum hopper wall angle**
  - 60°
- **aspect ratio (rectangular)**
  - 3.1 to 5:1
- **detention time**
  - 1.5 to 2.5 hours
- **circular diameter**
  - 30 to 150 feet
  - (100 common)
- **flow-through velocity**
  - 0.005 ft/sec
- **flow-through time**
  - at least 30% of detention time
- **overflow rate**
  - 400 to 2000 gpd/ft²
  - (800 to 1200 typical)
- **bottom**
  - slight slope (8%)
  - towards hopper
- **inlet**
  - baffled for uniform velocity
- **scum removal**
  - mechanical or manual
- **weir loading**
  - 10,000 to 20,000 gpd/ft²

C. SECONDARY TREATMENT

Secondary treatment is a biological treatment. It became mandatory for all publicly owned wastewater treatment plants as of July 1977 under the Federal Water Pollution Control Act amendments of 1972.

**Trickling Filters**: Trickling filters (also known as biological beds) consist of beds of 2" to 5" rocks up to 9 feet thick (6 feet typical) over which influent is sprayed. The biological and microbial slime growth attached to the rocks purify the wastewater as it trickles through the rocks. The water is introduced into the filter by rotating arms which move by virtue of the spray reaction. The clarified water is collected by an underdrain system. Some water may be returned to the filter for a longer contact time.

On the average, one acre of standard filter area is needed for each 20,000 people served. Trickling filters can remove 70% to 90% of the suspended solids, 65% to 85% of the BOD, and 70% to 95% of the bacteria. Most of the reduction occurs in the first few feet of bed, and organisms in the lower part of the bed may be in a near-starvation condition. The bed will periodically slough off (unload) parts of its slime coating, and sedimentation after filtering is necessary.

Since there are limits to the heights of trickling filters, longer contact times can be achieved by returning some of the collected filter water to the filter. This is known as recirculation. Recirculation is also used to keep the filter medium from drying out and to smooth out fluctuations in the hydraulic loading.

**High rate filters** are now in use by most modern facilities. The higher hydraulic loading flushes the rockpile and inhibits excess biological growth. High rate filters may be only 3 to 4 feet deep. The high rate is possible because much of the filter discharge is recirculated.

The **hydraulic loading** of a trickling filter is the water flow divided by the plan area. Typical values of hydraulic loading are 25 to 100 gpd/ft² for standard filters, and up to 1000 gpd/ft² for high-rate filters.

\[
L_H = \frac{Q_w + Q_R}{A_{filter}} = \frac{Q_w + R_R Q_w}{A_{filter}} = \frac{Q_w (1 + R_R)}{A_{filter}} \quad (Q \text{ in gpd})
\]

The **recirculation ratio** is given by equation 8.28. It can be as high as 3 for high rate filters, although it is zero for standard low-rate filters.

\[
R_R = \frac{Q_R}{Q_w} = \frac{L_H A_{filter}}{Q_w} - 1 \quad (8.28)
\]

The **BOD loading** (same as organic loading) is calculated without considering any recirculated flow. BOD
loading for the filter/clarifier combination is essentially the BOD of the applied wastewater divided by the filter volume.

\[ L_{BOD} = \frac{(Q_{\text{MCY}})(BOD_{\text{mg/L}})(8.345)(1000)}{\text{filter volume in ft}^3} \quad 8.29 \]

BOD loading is given in pounds per 1000 cubic feet per day. Typical values are 5 to 25 lbm/1000 cubic feet-day (low rate) and 30 to 90 lbm/1000 cubic feet-day (high rate.)

Significant reduction in BOD occurs in a trickling filter. Standard rate filters produce an 80%–85% reduction. Because they offer less contact area and time, high rate filters only remove 65%–80% of the BOD.

If it is assumed that the biological layer and hydraulic loading are uniform, the water is at 20°C, and the filter is single-stage rock followed by a settling tank, then the following equation developed by the National Research Council can be used to calculate the BOD removal efficiency of the filter/clarifier combination.\(^\text{14}\) Equation 8.30 is easily solved from figure 8.10.\(^\text{15}\)

\[ \eta = \frac{\text{BOD}_{\text{removed}}}{\text{BOD}_{\text{entering}}} = \frac{1}{1 + 0.0561\sqrt{L_{BOD}/F}} \quad 8.30 \]

\(^\text{14}\) The National Research Council did studies in 1946 on sewage treatment plants at military installations. It concluded that the organic loading had a greater effect on removal efficiency than did volumetric loading.

\(^\text{15}\) The constant 0.0561 in equation 8.30 is also reported as 0.0065 in the literature. However, that value is for use with media volumes expressed in acre-feet, not 1000's of ft\(^3\).

\[ \eta_T = \eta_{20^\circ C}(1.01)^{T-20} \quad 8.31 \]
Other common design characteristics of a mechanically aerated lagoon are:

- aspect ratio: less than 3:1
- depth: 10 to 12 feet
- detention time: 4 to 10 days
- BOD loading: 20 to 400 lbm/day-acre (220 typical)
- temperature range: 32° to 100°F (70°F optimum)
- typical effluent BOD: 20 to 70 mg/l
- oxygen requirements: 0.7 to 1.4 times BOD removed

**Activated Sludge Processes:** Sludge produced during the oxidation process has an extremely high concentration of active aerobic bacteria. For this reason, partially oxidized sludge is called *activated sludge.* Purification of raw sewage can be speeded up considerably if the raw sewage is mixed (seeded) with activated sludge. The mixture of raw sewage and activated sludge is known as *mixed liquor (ML).* The biological systems in the mixed liquor are known as *mixed liquor suspended solids (MLSS).*

In operation, an activated sludge process takes raw water and allows it to settle. The settled effluent is mixed with activated sludge in the approximate ratio of 1 part sludge per 3 or 4 parts effluent. Mechanical aeration is used. The effluent is then settled in a second sedimentation tank, chlorinated, and discharged. Settled sludge from this last tank supplies the continuous seed for the activation.

Activated sludge processes are highly efficient, with the following typical characteristics for a conventional system. (Also, see table 8.14).

- BOD reduction: 90 to 95%
- BOD loading: 0.25 to 1 lbm/lbm MLSS
- maximum aeration chamber volume: 5000 ft³
- aeration chamber depth: 10 to 15 feet
- aeration chamber width: 20 ft
- air rate: 1/2 to 2 ft³/gal raw sewage
- minimum dissolved oxygen: 2 mg/l
- biological mass density: 1000 to 4000 mg/l
- sedimentation basin depth: 15 ft
- sedimentation basin detention time: 2 hours
- basin overflow rate: 400 to 2000 gpd/ft² (1000 typical)
- % sludge returned: 20 to 30%
- frequency of sludge transfer: once each hour
- activated sludge volume index: 50 to 150
- weir loading: 10,000 gpd/ft
- maximum tank volume: 2500 ft²

The *rate of oxygen transfer (ROT)* from the air to the mixed liquor during aeration is given by equation 8.42.

\[ \text{ROT} = K_iD \]  

8.42

---

**Figure 8.14** A Typical Activated Sludge Plant

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**High Purity Oxygen Aeration:** This method requires the use of bottled or manufactured oxygen which is introduced to closed aerating tanks in place of atmospheric air. Mechanical mixing is needed to take full advantage of the oxygen.

**Intermediate Clarifiers:** Sedimentation tanks located between trickling filter stages (see figure 8.12) or between a filter and subsequent aeration are known as intermediate clarifiers. Recommended standards are:

- maximum overflow rate: 1000 gpd/ft²
- minimum water depth: 7 feet
- maximum weir loading: 10,000 gpd/ft² (plants 1 MGD or less)
  15,000 gpd/ft² (plants over 1 MGD)

**Final Clarifiers:** Final sedimentation in secondary treatment is done in final clarifiers. The purpose of final clarifiers is to collect sloughed off filter material (trickling filter processes) or to collect sludge and return it for aeration (activated sludge processes).

General characteristics for clarifiers following trickling filters are:

- BOD removal: See equation 8.30
- minimum depth: 7 feet
- maximum overflow rate: 800 gpd/ft²
- maximum weir loading: Same as for intermediate clarifiers, but lower preferred.

If the final clarifier follows an activated sludge process, the sludge should be removed rapidly from the entire bottom of the clarifier. Characteristics of clarifiers following an activated sludge process are given in table 8.15.

**D. ADVANCED TERTIARY TREATMENT**

**Suspended Solids:** Suspended solids are removed by microstrainers or polishing filter beds.

**Phosphorus Removal:** Phosphorus can be removed by chemical precipitation. Aluminum and iron coagulants, as well as lime, are effective in removing phosphates.

**Nitrogen Conversion and Removal:** In the ammonia stripping (air stripping) method, lime is added to water to increase its pH to above 10. The water is then passed through a packed tower into which air is blown. The air (at the rate of approximately 400 ft³/gallon) strips the ammonia out of the water. Recarbonation follows to remove the excess lime.

\[
NH_4 + OH^- \xrightarrow{pH > 11} NH_4OH \xrightarrow{pH < 10} H_2O + NH_3
\]

8.54

In the nitrification and denitrification process, bacteria oxidize ammonium ions to nitrate and nitrite in an aeration tank kept at low BOD. Nitrate and nitrite ions do not absorb further oxygen and may be discharged.

\[
NH_4^+ \xrightarrow{\text{bacteria}} NO_2^- + NO_3^-
\]

8.55

**Table 8.14**

Representative Operating Conditions for Aeration

<table>
<thead>
<tr>
<th>Type of Aeration</th>
<th>Plant Flow Rate (MGD)</th>
<th>( t_A ) (hrs)</th>
<th>Oxygen Required (lbm/lbm BOD removed)</th>
<th>Waste Sludge BOD (lbm/lbm BOD removed)</th>
<th>Total Plant BOD Load (lbm/1000 ft³/day)</th>
<th>Aerator BOD-load, ( L_{BOD} ) (lbm/lbm)</th>
<th>( R_F/M )</th>
<th>MLSS (mg/l)</th>
<th>( R_R ), ( η_{BOD} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional</td>
<td>0.5 - 1.5</td>
<td>7.5</td>
<td>0.8 - 1.1</td>
<td>0.4 - 0.6</td>
<td>0 - 1000</td>
<td>30</td>
<td>0.2 - 0.5</td>
<td>1500 - 3000</td>
<td>30, 90 - 95</td>
</tr>
<tr>
<td></td>
<td>0.5 - 1.5</td>
<td>7.5 - 6.0</td>
<td></td>
<td></td>
<td>1000 - 3000</td>
<td>30 - 40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5 up</td>
<td>6.0</td>
<td></td>
<td></td>
<td>3000 up</td>
<td>40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contact Stabilization</td>
<td>0 - 0.5</td>
<td>3.0*</td>
<td>0.8 - 1.1</td>
<td>0.4 - 0.6</td>
<td>0 - 1000</td>
<td>30</td>
<td>0.2 - 0.5</td>
<td>1000 - 3000*</td>
<td>100, 85 - 90</td>
</tr>
<tr>
<td></td>
<td>0.5 - 1.5</td>
<td>3.0 - 2.0</td>
<td></td>
<td></td>
<td>1000 - 3000</td>
<td>30 - 40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5 up</td>
<td>1.5 - 2.0</td>
<td></td>
<td></td>
<td>3000 up</td>
<td>50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extended</td>
<td>0 - 0.05</td>
<td>24</td>
<td>1.4 - 1.6</td>
<td>0.15 - 0.3</td>
<td>all</td>
<td>10.0</td>
<td>0.05 - 0.1</td>
<td>3000 - 6000</td>
<td>100, 85 - 95</td>
</tr>
<tr>
<td></td>
<td>0.05 - 0.13</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.15 up</td>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High Rate</td>
<td>0 - 0.5</td>
<td>4.0</td>
<td>0.7 - 0.9</td>
<td>0.5 - 0.7</td>
<td>2000 up</td>
<td>100</td>
<td>1.0 or less</td>
<td>4000 - 10.000</td>
<td>100, 80 - 85</td>
</tr>
<tr>
<td></td>
<td>0.5 - 1.5</td>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5 up</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step Aeration</td>
<td>0 - 0.5</td>
<td>7.5</td>
<td></td>
<td></td>
<td>0 - 1000</td>
<td>30</td>
<td>0.2 - 0.5</td>
<td>2000 - 3500</td>
<td>50, 85 - 95</td>
</tr>
<tr>
<td></td>
<td>0.5 - 1.5</td>
<td>7.5 - 5.0</td>
<td></td>
<td></td>
<td>1000 - 3000</td>
<td>30 - 40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5 up</td>
<td>5.0</td>
<td></td>
<td></td>
<td>3000 up</td>
<td>50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High Purity Oxygen</td>
<td>1.0 - 3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>above 120</td>
</tr>
</tbody>
</table>

* in contact unit only
Table 8.15
Final Clarifiers for Activated Sludge Processes

<table>
<thead>
<tr>
<th>type of aeration</th>
<th>design flow (MGD)</th>
<th>minimum detention time (hr)</th>
<th>maximum overflow rate (gpd/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>conventional, high rate, and step</td>
<td>&lt; 0.5</td>
<td>3.0</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>0.5 to 1.5</td>
<td>2.5</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td>&gt; 1.5</td>
<td>2.0</td>
<td>800</td>
</tr>
<tr>
<td>contact stabilization</td>
<td>&lt; 0.5</td>
<td>3.6</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>0.5 to 1.5</td>
<td>3.0</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>&gt; 1.5</td>
<td>2.5</td>
<td>700</td>
</tr>
<tr>
<td>extended aeration</td>
<td>&lt; 0.05</td>
<td>4.0</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>0.05 to 0.15</td>
<td>3.6</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>&gt; 0.15</td>
<td>3.0</td>
<td>600</td>
</tr>
</tbody>
</table>

Following sedimentation and sludge recirculation, the effluent may be treated to convert nitrates and nitrites to nitrogen gas. Methanol supplies the energy required by the denitrification bacteria.

With secondary treatment, the total sludge load will be about 0.2 pounds per capita day (when dried). This amounts to approximately 2 quarts of sludge per 100 gallons of wastewater processed.

Table 8.16
Typical Characteristics of Domestic Sewage Sludge
(also, see table 8.18)

<table>
<thead>
<tr>
<th>origin of sludge</th>
<th>solids content of wet sludge (lbs/dry solids)</th>
</tr>
</thead>
<tbody>
<tr>
<td>primary settling tank</td>
<td>6 (0.12 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>trickling filter secondary</td>
<td>4 (0.04 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>mixed primary and trickling filter secondary</td>
<td>5 (0.16 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>high rate activated sludge secondary</td>
<td>2.5–5 (0.06 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>mixed primary and high rate activated sludge secondary</td>
<td>5 (0.18 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>conventional activated sludge secondary</td>
<td>0.5–1 (0.07 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>mixed primary and conventional activated sludge secondary</td>
<td>2–3 (0.19 lbs/dry solids/capita)</td>
</tr>
<tr>
<td>extended aeration secondary</td>
<td>2 (0.02 lbs/dry solids/capita)</td>
</tr>
</tbody>
</table>

The dry weight of solids from primary settling basins is:

\[ W_p = (\text{decrease in SS})(Q_{gpd})(8.345 \times 10^{-6}) \]

The dry weight of solids from secondary aeration lagoons and biological filters is:

\[ W_s = K(\text{BOD})_{\text{removed}}(Q_{gpd})(8.345 \times 10^{-6}) \]
Equation 8.57 depends on the food-to-microorganism ratio as given in Table 8.17. \( K \) is the fraction of BOD that appears as excess biological solids. Trickling filters and extended aeration. \( K \) ranges from 0.2 to 0.33. For conventional and step aeration, \( K \) is from 0.33 to 0.42. \( K \) is known as the cell yield coefficient.

### Table 8.17
Cell Yield (Yield Coefficient)

<table>
<thead>
<tr>
<th>( R_{F,M} )</th>
<th>( K )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>0.2</td>
</tr>
<tr>
<td>0.07</td>
<td>0.21</td>
</tr>
<tr>
<td>0.1</td>
<td>0.24</td>
</tr>
<tr>
<td>0.15</td>
<td>0.28</td>
</tr>
<tr>
<td>0.2</td>
<td>0.33</td>
</tr>
<tr>
<td>0.3</td>
<td>0.37</td>
</tr>
<tr>
<td>0.4</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Equations 8.56 and 8.57 give dry weight of sludge. Assuming the sludge specific gravity is near 1, the vol of wet sludge with a solids concentration, \( s \), can be found from the dry weight by using equation 8.58.

\[
\text{tons of dried weight per day, \( \text{lbm} \)} \times (s)(8.345) = 8.58
\]

Total values of \( s \) are given in Table 8.18.

### Table 8.18
Total Sludge Solids, \( s \)
(also, see Table 8.16)

<table>
<thead>
<tr>
<th>Source or type of sludge</th>
<th>( s ), as fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary settling tank sludge</td>
<td>0.06 to 0.08</td>
</tr>
<tr>
<td>Primary settling tank sludge mixed with filter sludge</td>
<td>0.04 to 0.06</td>
</tr>
<tr>
<td>Primary settling tank sludge mixed with activated sludge from aeration lagoons</td>
<td>0.03 to 0.04</td>
</tr>
<tr>
<td>Excess activated sludge</td>
<td>0.005 to 0.02</td>
</tr>
<tr>
<td>Filter backwashing water</td>
<td>0.01 to 0.1</td>
</tr>
<tr>
<td>Softening sludge</td>
<td>0.02 to 0.15</td>
</tr>
</tbody>
</table>

Specific gravity of the wet sludge is:

\[
\frac{1}{(G)_{\text{total}}} = \frac{\text{fraction moisture}}{10} + \frac{\text{fraction solids}}{(sG)_{\text{solids}}} \quad 8.59
\]

\( V \), the volume of sludge is:

\[
\frac{W}{(sG)_{\text{total}}(62.4)} \quad \text{m}^3/\text{day} \quad 8.60
\]

### Example 8.8
A trickling filter plant processes domestic waste with the following characteristics. 190 mg/l BOD, 230 mg/l SS, and 4,000,000 gpd.

(a) What is the wet sludge volume from the primary sedimentation tank and trickling filter? Assume the combined sludge solids content is 5%.

(b) What is the approximate weight of dry solids produced per person-day?

#### step 1
Find the weight of the dry solids obtained from the primary settling basin. From equation 8.56, assuming 50% of solids can be removed.

\[
W_p = (0.5)(230)(4.345\times10^6) = 3839 \text{ pounds/day}
\]

Assume a 30% BOD reduction, so the BOD leaving the basin is \( (0.7)(190) = 133 \text{ mg/l} \).

#### step 2
Assume a cell yield value of 0.25. Then, the weight of dry solids from the filter is given by equation 8.57.

\[
W_s = (0.25)(133)(4.345\times10^6) = 1110 \text{ pounds/day}
\]

#### step 3
The wet sludge volume can be found from equation 8.58.

\[
\frac{3839 + 1110}{(0.05)(8.345)} = 11,860 \text{ gallons/day}
\]

#### step 4
The equivalent population is given by equation 8.12.

\[
P_e = \frac{(190)(8.345)(4)}{0.17} = 37,310 \text{ people}
\]

#### step 5
The per capita dried solids rate is

\[
\frac{3839 + 1110}{37,310} = 0.13 \text{ lbm dry solids per person-day}
\]

### B. SLUDGE THICKENING
Since the volume of wet sludge is inversely proportional to its solids content (equation 8.58), thickening of sludge is desirable. Thickening is required to at least 4% solids if dewatering is to be feasible. Gravity thickening uses a stirred sedimentation tank into which sludge is fed. A doubling of solids content is usually possible with a gravity thickener.
Appendix A: Selected “10-States’ Standards”

The following selected standards are derived from “Recommended Standards for Sewage Works”, originally developed by the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers. Since there are ten states in this board, the publication is commonly referred to as “10-States’ Standards”.

The standards are subject to change and this summary is not complete. The items listed here are merely meant to be representative of good wastewater plant design in colder areas of the U.S.

- **Hydraulic Load**: 100 gallons per capita-day for new systems in undeveloped areas unless other information is available.

- **Pumps**: At least 2 pumps are required, and 3 are required when the design flow exceeds 1 MGD. Both pumps should have the same capacity if only 2 pumps are used. This capacity must exceed the design flow for each pump. If 3 or more pumps are used, the capacities may vary, but capacity pumping must be possible with one pump out of service.

- **Racks and Bar Screens**: All racks and screens shall have openings less than 1.75” wide. The smallest opening for manually-cleaned screens is 1.0”. The smallest opening for automatically-cleaned screens is 0.625”. Flow velocity should be 1.25 to 3.0 fps.

- **Grinders and Shredders**: Grinder/shredders are required if there is no primary sedimentation or fine screens. Gravel traps or grit-removal equipment should precede comminutors.

- **Grit Chambers**: Grit chambers are required when combined storm and sanitary sewers are used. A minimum of two grit chambers in parallel should be used, with a provision for bypassing. The optimum velocity is 1.0 fps throughout. The detention time is dependent on particle sizes to be removed.

- **Plain Sedimentation Tanks**: Multiple units are desirable and must be provided if the flow exceeds 100,000 gpd. For primary settling, the depth should be 7.0’ or greater. The maximum peak overflow rate is 1500 gpd/ft².²² 15,000 gpd/ft is the maximum weir loading. If the flow rate is less than 1.0 MGD, the weir loading should be reduced to 10,000 gpd/ft.

- **Trickle Filters**: Rock media should have a depth of 5’ to 10’. Manufactured media should have a depth of 10’ to 30’. The rock media should be 1” to 4 1/2” in size, with no fines. Freeboard of 4’ or more is required. The drain should slope at 1% or more.

- **Activated Sludge Processes**: For sedimentation basins, the following hydraulic loading maximums are specified: 1200 gpd/ft² for conventional, step, and contact units; 1000 gpd/ft² for extended aeration units; 800 gpd/ft² for separate nitrification units. The maximum BOD loading shall be: 40 lbm/day-1000 ft³ for conventional, step, and complete mix units; 50 lbm/day-1000 ft³ for contact stabilization units; 15 lbm/day-1000 ft³ for extended aeration units.

Aeration tank depths should be between 10’ and 30’. At least two aeration tanks should be used. The dissolved oxygen content should not be allowed to drop below 2.0 mg/l at any time. The aeration rate should be 1500 ft³ oxygen per pound of BOD₅. For extended aeration, the rate should be 2000 ft³ oxygen per pound of BOD₅.

- **Final Clarifiers**: Maximum surface settling rate is 800 gpd/ft² for separate nitrification stages, 1000 gpd/ft² for extended aeration, and 1200 gpd/ft² for all other cases, including fixed film biological processes.

- **Lagoons**: Maximum BOD application is 15 to 35 lbm BOD₅ per acre-day for both controlled-discharge and flow-through stabilization ponds.

- **Chlorination**: Requires a 15 minute contact period at peak flow.

- **Anaerobic Digesters**: For completely mixed digesters, up to 80 lbm of volatile solids per day per 1000 ft³ of digester. For moderately mixed digesters, the limit is 40 lbm/day-1000 ft³. Multiple units. Minimum 20 feet sidewater depth.

- **Sludge Drying Beds**: Requires 2 ft²/capita-day, if drying beds are the primary dewatering method, and 1 ft²/capita-day if beds are a back-up dewatering method.

---

²² The basin size shall also be calculated based on the average design flow rate and a maximum overflow rate of 1000 gpd/ft².

The larger of the two sizes shall be used.
Appendix C: Typical Sequences Used in Wastewater Plants

The following partial sequences are used to construct a complete treatment plant:

I  intake and preconditioning
P  primary treatment
S  secondary treatment
T  tertiary treatment
D  discharge
SP sludge processing
SD sludge disposal

Intake and Preconditioning

S: Secondary Treatment
T: Tertiary Treatment

sand filter
(for removal of organics)

T

D

 tertiary settling
(for removal of organics)

T

SP

D

 tertiary settling

S

SP

flocculants
(for phosphate removal)

T

D

 air tower
(for ammonia removal)

lime
Carbon dioxide

T

D

 anerobic filter
(for nitrates removal)

methanol

C

D

 ion exchange
(for phosphorus/nitrogen removal)

SP: Sludge Processing

chemical floculants

optional thickening

vacuum filtering

SP

P

P

liquid

SP

P

anaerobic digestion

solid

liquid

SP

P

aerated lagoon

SP

SD

SD: Solids Disposal

drying bed

landfill or burial

SD

P

solid

liquid

SD

incineration

ash landfill

D: Discharge

D

to outfall (rural only)

D

chlorination

in outfall
Figure 5.17 Multimedia cap. A minimum 4-percent surface slope is usually preferred. "Loamy topsoil 2 ft and to deepest frost "Sand permeability 10⁻³ cm/sec or greater "Combined permeability of synthetic liner and soil liner 10⁻³ cm/sec or less "Soil liner or soil admix: bentonite, clay-blend, soil-cement, or equal (Source: Handbook Remedial Action at Waste Disposal Sites, EPA/625/6-85/006, Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. EPA, Cincinnati, Ohio, October 1985, p. 3-11.)

Figure 5.20 Industrial/commercial and ash monofill waste landfills. (Source: New York State Dept of Environmental Conservation, Division of Solid Waste, Albany, N.Y., 1990, 6 NYCRR Part 360 2.14.)
Methane Recovery

Methane is produced in a landfill when anaerobic methane-producing bacteria are active. This condition may be reached in 6 months to 5 years depending on the landfill. Acidic conditions inhibit growth of methane-producing bacteria; alkaline conditions have the opposite effect. Methane production is quite variable depending on the amount and type of decomposable material in the landfill, moisture content, temperature, and resulting rate of microbial decomposition under anaerobic conditions. One estimate is a maximum gas production of 0.18 l/kg per day, or 5.41 l/kg dry weight per day; the effective life of a landfill for gas extraction at a rate of 20 ml/kg dry weight per day is approximately 17 years. In another study, a rate of gas generation (CO₂ and CH₄) of 3.1 to 37 l/kg per year during the more active period of methane production, on the order of 10 years, was considered reasonable. This involved municipal refuse as received, with methane production at about 53 percent of the total gas.

Methane is odorless, has a heat value of about 500 Btu/ft³ compared to 1000 Btu for commercial gas, has a specific gravity less than air, and is nearly insoluble in water. The gases from landfills, after anaerobic conditions have been established, are quite variable, ranging from 50 to 60 percent methane and 40 to 50 percent carbon dioxide. Included are small amounts of nitrogen, oxygen, water, mercaptans (very odorous), and hydrocarbons. Hydrogen sulfide may also be released if large amounts of sulfates are in the landfill. Vinyl chloride, benzene, and other toxics in trace amounts may also be produced by the action of bacteria on chlorinated solvents deposited in the fill. The presence of oxygen and nitrogen with methane gas would indicate the entrance of air into the landfill. This may be due to the rate at which methane is being withdrawn which, if not controlled to reduce or eliminate the entrance of oxygen and nitrogen, would slow down or stop methane production. In the early stages the landfill gases are primarily carbon dioxide with some methane. The carbon dioxide is heavier than air and can dissolve in water to form carbonic acid, which is corrosive to minerals with which it comes into contact. Mercaptans, carbon dioxide, and water are usually extracted to upgrade the methane to "pipeline quality." Removal of carbon dioxide may improve Btu content to 900 or 1000 Btu/ft³. Methane as it comes from a landfill is often very corrosive. Deep landfills, 30 ft or deeper, and 30 acres or more in area with a good cover are better methane producers. The life of such a landfill in California is estimated at 12 to 20 years. Actually, gas will be generated as long as biodegradable material remains and is primarily dependent on precipitation, infiltration, and moisture content. Gas can be extracted using plastic tube wells in each cell with perforations or well screens toward the bottom connected to a controlled vacuum pump or a series of covered horizontal gravel trenches connected to a pipe collection system. See Figures 5-8 and 5-9. The gas may be used for heating and generating electricity.
It must be kept in mind that methane in the presence of air is explosive at concentrations between 5 and 15 percent. However, a concentration above 15 percent is potentially hazardous as the gas may be quickly diluted by air movement to below 15 percent. Therefore, gas monitoring and control must be included in the planning, design, and operation of a sanitary landfill. This is necessary to prevent the lateral migration of methane to buildings, tunnels, manholes, sewers, or other enclosed spaces, where it would be an explosive and fire hazard, particularly if the surrounding soil is dry, well drained, channeled or cracked and the landfill cover material is a tight soil that does not permit venting to the atmosphere. The control of methane migration is discussed under Sanitary Landfill this chapter.

### Table 5-1 Approximate Composition of Residential Solid Wastes in 1977 and 1989

<table>
<thead>
<tr>
<th>Component</th>
<th>1977</th>
<th>1989</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food waste</td>
<td>17.0</td>
<td>7.9</td>
</tr>
<tr>
<td>Paper products</td>
<td>33.5</td>
<td>41.0</td>
</tr>
<tr>
<td>Rubber, leather, textiles, wood</td>
<td>7.8</td>
<td>8.1</td>
</tr>
<tr>
<td>Plastics</td>
<td>3.6</td>
<td>6.5</td>
</tr>
<tr>
<td>Metals</td>
<td>9.2</td>
<td>8.7</td>
</tr>
<tr>
<td>Glass and ceramics</td>
<td>9.9</td>
<td>8.2</td>
</tr>
<tr>
<td>Yard wastes</td>
<td>17.5</td>
<td>17.9</td>
</tr>
<tr>
<td>Rock, dirt, miscellaneous</td>
<td>1.5</td>
<td>1.6</td>
</tr>
</tbody>
</table>


*Weights and percentages will vary with community, time of the year, and geography. For design purposes, make actual weighings. Figures do not reflect recycling.

The above figures do not include junked vehicles, water and sewage treatment plant sludges, waste oil, pathological wastes, agricultural wastes, industrial wastes, mining or milling wastes.
TABLE 8-3
Projection of average composition of U.S. solid waste by year

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Paper</td>
<td>37.4</td>
<td>39.2</td>
<td>40.1</td>
<td>43.4</td>
<td>48.0</td>
</tr>
<tr>
<td>Yard wastes</td>
<td>13.9</td>
<td>13.3</td>
<td>12.9</td>
<td>12.3</td>
<td>11.9</td>
</tr>
<tr>
<td>Food wastes</td>
<td>20.0</td>
<td>17.5</td>
<td>16.1</td>
<td>14.0</td>
<td>12.1</td>
</tr>
<tr>
<td>Glass</td>
<td>9.0</td>
<td>9.9</td>
<td>10.2</td>
<td>9.5</td>
<td>8.1</td>
</tr>
<tr>
<td>Metal</td>
<td>8.4</td>
<td>8.6</td>
<td>8.9</td>
<td>8.6</td>
<td>7.1</td>
</tr>
<tr>
<td>Wood</td>
<td>3.1</td>
<td>2.7</td>
<td>2.4</td>
<td>2.0</td>
<td>1.6</td>
</tr>
<tr>
<td>Textiles</td>
<td>2.2</td>
<td>2.3</td>
<td>2.3</td>
<td>2.7</td>
<td>3.1</td>
</tr>
<tr>
<td>Leather and rubber</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>Plastics</td>
<td>1.4</td>
<td>2.1</td>
<td>3.0</td>
<td>3.9</td>
<td>4.7</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>3.4</td>
<td>3.0</td>
<td>2.7</td>
<td>2.4</td>
<td>2.1</td>
</tr>
</tbody>
</table>

*Percentage of waste stream as discarded on a dry mass basis.


---

TABLE 8-6
Typical properties of uncompacted solid waste as discarded in Davis, California

<table>
<thead>
<tr>
<th>Component</th>
<th>Mass (kg)</th>
<th>Density (kg/m³)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food wastes</td>
<td>4.3</td>
<td>288</td>
<td>0.0149</td>
</tr>
<tr>
<td>Paper</td>
<td>19.6</td>
<td>81.7</td>
<td>0.240</td>
</tr>
<tr>
<td>Cardboard*</td>
<td>2.92</td>
<td>99.3</td>
<td>0.0297</td>
</tr>
<tr>
<td>Plastics</td>
<td>0.82</td>
<td>64</td>
<td>0.013</td>
</tr>
<tr>
<td>Textiles</td>
<td>0.091</td>
<td>64</td>
<td>0.0014</td>
</tr>
<tr>
<td>Rubber</td>
<td></td>
<td>128</td>
<td></td>
</tr>
<tr>
<td>Leather</td>
<td>0.68</td>
<td>160</td>
<td>0.0043</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>6.5</td>
<td>104</td>
<td>0.063</td>
</tr>
<tr>
<td>Wood</td>
<td>1.59</td>
<td>240</td>
<td>0.00663</td>
</tr>
<tr>
<td>Glass</td>
<td>3.4</td>
<td>194</td>
<td>0.018</td>
</tr>
<tr>
<td>Tin cans</td>
<td>2.36</td>
<td>88.1</td>
<td>0.0268</td>
</tr>
<tr>
<td>Nonferrous metals</td>
<td>0.68</td>
<td>160</td>
<td>0.0043</td>
</tr>
<tr>
<td>Ferrous metals</td>
<td>1.95</td>
<td>320</td>
<td>0.00609</td>
</tr>
<tr>
<td>Dirt, ashes, brick</td>
<td>0.50</td>
<td>480</td>
<td>0.0010</td>
</tr>
<tr>
<td>Total</td>
<td>45.4</td>
<td></td>
<td>0.429</td>
</tr>
</tbody>
</table>

*Cardboard partially compressed by hand before being placed in container.

### Table 10.3 Typical composition of municipal solid wastes

<table>
<thead>
<tr>
<th>Component</th>
<th>Range</th>
<th>Typical</th>
<th>Davis California*</th>
<th>Merida, Venezuela†</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food wastes</td>
<td>6-26</td>
<td>14</td>
<td>8.3</td>
<td>27.4</td>
</tr>
<tr>
<td>Paper</td>
<td>15-45</td>
<td>34</td>
<td>35.8</td>
<td>15.5</td>
</tr>
<tr>
<td>Cardboard</td>
<td>3-15</td>
<td>7</td>
<td>10.9</td>
<td>13.0</td>
</tr>
<tr>
<td>Plastics</td>
<td>3-8</td>
<td>5</td>
<td>6.9</td>
<td>4.6</td>
</tr>
<tr>
<td>Textiles</td>
<td>0-4</td>
<td>2</td>
<td>2.5</td>
<td>2.3</td>
</tr>
<tr>
<td>Rubber</td>
<td>0-2</td>
<td>0.5</td>
<td>2.5</td>
<td>0.4</td>
</tr>
<tr>
<td>Leather</td>
<td>0-2</td>
<td>0.5</td>
<td>0.7</td>
<td>1.3</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>0-20</td>
<td>12</td>
<td>10.8</td>
<td>5.8</td>
</tr>
<tr>
<td>Wood</td>
<td>1-4</td>
<td>2</td>
<td>1.9</td>
<td>3.6</td>
</tr>
<tr>
<td>Misc. organics</td>
<td>0-5</td>
<td>2</td>
<td>2.0</td>
<td>0.6</td>
</tr>
<tr>
<td>Glass</td>
<td>4-16</td>
<td>8</td>
<td>7.5</td>
<td>10.3</td>
</tr>
<tr>
<td>Tin cans</td>
<td>2-8</td>
<td>6</td>
<td>5.1</td>
<td>8.3</td>
</tr>
<tr>
<td>Nonferrous metals</td>
<td>0-1</td>
<td>1</td>
<td>1.6</td>
<td>0.1</td>
</tr>
<tr>
<td>Ferrous metals</td>
<td>1-4</td>
<td>2</td>
<td>2.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Dirt, ashes, brick, etc</td>
<td>0-10</td>
<td>4</td>
<td>1.3</td>
<td>5.6</td>
</tr>
</tbody>
</table>

* Based on measurements made during the month of October over a 5-year period (1979 through 1982).
† Based on measurements made during the month of July over a 3-year period (1979 through 1980).
Table 10-4 Typical data on moisture content of municipal solid waste components

<table>
<thead>
<tr>
<th>Component</th>
<th>Moisture, percent</th>
<th>Range</th>
<th>Typical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food wastes</td>
<td>30-80</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>Paper</td>
<td>4-10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Cardboard</td>
<td>4-8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Plastics</td>
<td>1-4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Textiles</td>
<td>6-15</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Rubber</td>
<td>1-4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Leather</td>
<td>8-12</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Garden trammings</td>
<td>30-80</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>Wood</td>
<td>15-40</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Misc organics</td>
<td>10-60</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Glass</td>
<td>1-4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Tin cans</td>
<td>2-4</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Nonferrous metals</td>
<td>2-4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Ferrous metals</td>
<td>2-6</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Durt, ashes, brick, etc</td>
<td>6-15</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Municipal solid wastes</td>
<td>15-40</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

Source: From Efchenobolou et al. [10-5]

To obtain the dry mass, the solid-waste material is dried in an oven at 77 °C (170 °F) for 24 h. This temperature and time is used to dehydrate the material completely and to limit the vaporization of volatile materials.

Typical data on the moisture content for the solid-waste components are given in Table 10-4. For most industrial solid wastes, the moisture content will vary from 10 to 35 percent. The use of Eq. (10-1) is illustrated in Example 10-1.

Example 10-1: Estimating the moisture content of a solid-waste sample Estimate the moisture content of a solid-waste sample with the following composition

<table>
<thead>
<tr>
<th>Component</th>
<th>Percent by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food wastes</td>
<td>15</td>
</tr>
<tr>
<td>Paper</td>
<td>45</td>
</tr>
<tr>
<td>Cardboard</td>
<td>10</td>
</tr>
<tr>
<td>Plastics</td>
<td>10</td>
</tr>
<tr>
<td>Garden trammings</td>
<td>10</td>
</tr>
<tr>
<td>Wood</td>
<td>5</td>
</tr>
<tr>
<td>Tin cans</td>
<td>5</td>
</tr>
</tbody>
</table>
SOLUTION

1. Set up a computation table to determine the dry mass of the solid-waste sample using the data given in Table 10.4.

<table>
<thead>
<tr>
<th>Component</th>
<th>Percent by mass</th>
<th>Moisture content, %</th>
<th>Dry mass,* kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food waste</td>
<td>15</td>
<td>70</td>
<td>4.5</td>
</tr>
<tr>
<td>Paper</td>
<td>45</td>
<td>6</td>
<td>42.3</td>
</tr>
<tr>
<td>Cardboard</td>
<td>10</td>
<td>5</td>
<td>9.5</td>
</tr>
<tr>
<td>Plastics</td>
<td>10</td>
<td>2</td>
<td>9.8</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>10</td>
<td>60</td>
<td>4.0</td>
</tr>
<tr>
<td>Wood</td>
<td>5</td>
<td>20</td>
<td>4.0</td>
</tr>
<tr>
<td>Tin cans</td>
<td>5</td>
<td>3</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>79.0</td>
</tr>
</tbody>
</table>

* Based on 100 kg sample of waste

2. Determine the moisture content using Eq. (10.1) and the data from step 1.

\[
\text{Moisture content} = \left( \frac{100 - 79.0}{100} \right) \times 100 = 21.0\%.
\]

COMMENT: The composition of the solid-waste sample used in this example will be used in all of the examples in this chapter. By using the same composition throughout, the interrelationship of the various parameters can be established more clearly.

Density

Typical densities for various wastes as found in containers are reported by source in Table 10.5. Because the densities of solid wastes vary markedly with geographic location, season of the year, and length of time in storage, great care should be used in selecting typical values. Estimation of the density of a waste sample is illustrated in Example 10.9.


SOLUTION

1. Set up a computation table to determine the as-discarded volume of the solid waste sample using the data reported in Table 10.5.
<table>
<thead>
<tr>
<th>Component</th>
<th>Percent by mass</th>
<th>Typical density, kg/m³</th>
<th>Volume, m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food waste</td>
<td>15</td>
<td>290</td>
<td>0.52</td>
</tr>
<tr>
<td>Paper</td>
<td>45</td>
<td>85</td>
<td>5.29</td>
</tr>
<tr>
<td>Cardboard</td>
<td>10</td>
<td>50</td>
<td>2.00</td>
</tr>
<tr>
<td>Plastics</td>
<td>10</td>
<td>65</td>
<td>1.54</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>10</td>
<td>105</td>
<td>0.95</td>
</tr>
<tr>
<td>Wood</td>
<td>5</td>
<td>240</td>
<td>0.21</td>
</tr>
<tr>
<td>Tin cans</td>
<td>5</td>
<td>90</td>
<td>0.36</td>
</tr>
</tbody>
</table>

* Based on a 1000-kg sample of waste.

2. Compute the density of a waste sample using the data from step 1.

\[
\text{Density} = \frac{1000 \text{ kg}}{11.07 \text{ m}^3} = 90.33 \text{ kg/m}^3
\]

Table 10-5 Typical densities for solid wastes components and mixtures

<table>
<thead>
<tr>
<th>Item</th>
<th>Density, kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
</tr>
<tr>
<td>Component*</td>
<td></td>
</tr>
<tr>
<td>Food wastes</td>
<td>120–480</td>
</tr>
<tr>
<td>Paper</td>
<td>30–130</td>
</tr>
<tr>
<td>Cardboard</td>
<td>30–80</td>
</tr>
<tr>
<td>Plastics</td>
<td>30–130</td>
</tr>
<tr>
<td>Textiles</td>
<td>30–160</td>
</tr>
<tr>
<td>Rubber</td>
<td>90–200</td>
</tr>
<tr>
<td>Leather</td>
<td>90–260</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>60–225</td>
</tr>
<tr>
<td>Wood</td>
<td>120–320</td>
</tr>
<tr>
<td>Misc. organics</td>
<td>90–360</td>
</tr>
<tr>
<td>Glass</td>
<td>160–480</td>
</tr>
<tr>
<td>Tin cans</td>
<td>45–160</td>
</tr>
<tr>
<td>Nonferrous metals</td>
<td>60–240</td>
</tr>
<tr>
<td>Ferrous metals</td>
<td>120–1200</td>
</tr>
<tr>
<td>Dirt, ashes, brick, etc</td>
<td>320–960</td>
</tr>
<tr>
<td>Municipal solid wastes</td>
<td></td>
</tr>
<tr>
<td>Uncompacted</td>
<td>90–180</td>
</tr>
<tr>
<td>Compacted</td>
<td>180–450</td>
</tr>
<tr>
<td>(in compactor truck)</td>
<td></td>
</tr>
<tr>
<td>In landfill</td>
<td>350–550</td>
</tr>
<tr>
<td>(compacted normally)</td>
<td></td>
</tr>
<tr>
<td>In landfill</td>
<td>600–750</td>
</tr>
</tbody>
</table>

* Data for components is on an as-discarded basis.
Table 10-7 Typical data on inert residue and energy content of municipal solid wastes

<table>
<thead>
<tr>
<th>Component</th>
<th>Inert residue,* percent</th>
<th>Energy,† kJ/kg</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Typical</td>
</tr>
<tr>
<td>Food wastes</td>
<td>2-8</td>
<td>5</td>
</tr>
<tr>
<td>Paper</td>
<td>4-8</td>
<td>6</td>
</tr>
<tr>
<td>Cardboard</td>
<td>3-6</td>
<td>5</td>
</tr>
<tr>
<td>Plastics</td>
<td>6-20</td>
<td>10</td>
</tr>
<tr>
<td>Textiles</td>
<td>7-4</td>
<td>2.5</td>
</tr>
<tr>
<td>Rubber</td>
<td>8-20</td>
<td>10</td>
</tr>
<tr>
<td>Leather</td>
<td>8-20</td>
<td>10</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>0.6-6</td>
<td>4.5</td>
</tr>
<tr>
<td>Wood</td>
<td>0.6-2</td>
<td>1.5</td>
</tr>
<tr>
<td>Misc. organics</td>
<td>2-8</td>
<td>6</td>
</tr>
<tr>
<td>Glass</td>
<td>96-99</td>
<td>98</td>
</tr>
<tr>
<td>Tin cans</td>
<td>96-99</td>
<td>98</td>
</tr>
<tr>
<td>Nonferrous metals</td>
<td>90-99</td>
<td>96</td>
</tr>
<tr>
<td>Ferrous metals</td>
<td>96-99</td>
<td>96</td>
</tr>
<tr>
<td>Drt., ashes, brick, etc.</td>
<td>60-80</td>
<td>70</td>
</tr>
<tr>
<td>Municipal solid wastes</td>
<td>9,900-12,300</td>
<td>10,500</td>
</tr>
</tbody>
</table>

* After combustion.
† As-discarded basis.

Example 10-3: Estimating the energy content of a solid-waste sample

Estimate the energy content of a solid-waste sample with the composition given in Example 10-1. What is the content on a dry basis and on an ash-free dry basis?

Solution

1. Set up a computation table to determine the total as-discarded energy content of the solid-waste sample using the data in Table 10-7.

<table>
<thead>
<tr>
<th>Component</th>
<th>Percent by mass</th>
<th>Energy,* kJ/kg</th>
<th>Total energy,† kJ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food waste</td>
<td>15</td>
<td>4,650</td>
<td>69,750</td>
</tr>
<tr>
<td>Paper</td>
<td>45</td>
<td>16,750</td>
<td>753,750</td>
</tr>
<tr>
<td>Cardboard</td>
<td>10</td>
<td>16,300</td>
<td>163,000</td>
</tr>
<tr>
<td>Plastics</td>
<td>10</td>
<td>32,600</td>
<td>326,000</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>10</td>
<td>6,500</td>
<td>65,000</td>
</tr>
<tr>
<td>Wood</td>
<td>5</td>
<td>18,600</td>
<td>93,000</td>
</tr>
<tr>
<td>Tin cans</td>
<td>5</td>
<td>700</td>
<td>3,500</td>
</tr>
</tbody>
</table>

1,474,000

* From Table 10-7, as discarded basis.
† Based on 100-kg sample of waste.
2. Compute the unit energy content

\[
\text{Energy content} = \frac{1,474,000 \text{ kJ}}{100 \text{ kg}} = 14.74 \text{ kJ/kg}
\]

3. Determine the energy content on a dry basis.
   a. From Example 10-1, the moisture content of the waste is 21.0 percent.
   b. Using Eq. (10-2), the energy on a dry basis is

\[
\text{kJ/kg (dry basis)} = 14.74 \cdot \frac{100}{100 - 21} = 18.658
\]

4. Determine the energy content on an ash-free dry basis.
   a. Assume the ash content is equal to 5.0 percent.
   b. Using Eq. (10-3), the energy content on an ash-free dry basis is

\[
\text{kJ/kg (ash-free dry basis)} = 14.74 \cdot \frac{100}{100 - 5 - 21} = 19.919
\]