Chapter 7

Tertiary Treatment of Municipal Wastewater

Secondary treatment processes remove 85-95% BOD and solids from wastewater; however, such mechanisms often exhibit insignificant removal of nutrients (i.e. nitrogen, phosphorus) and heavy metals. If the nutrients are discharged (with wastewater) into natural channels without any treatment, they can cause: (a) depletion of dissolved oxygen; (b) eutrophication; and (c) methemoglobinemia.

In order to protect the natural channels from the adverse impact of nutrients, wastewater is often required to be treated through tertiary treatment process. Tertiary treatment includes removal of nutrients, dissolved solids, metals, toxic compounds and refractory organic compounds from wastewater, thereby further improving wastewater quality for safe discharge.

Tertiary treatment includes a combination of aerobic-anoxic-anaerobic tanks, followed by a secondary settler, to allow biological removal of nitrogen (N) and phosphorus (P); in addition, chemical precipitation is also employed for the removal of phosphorus. Figure 7.1 depicts a typical biological tertiary treatment process, to assist nutrient removal.

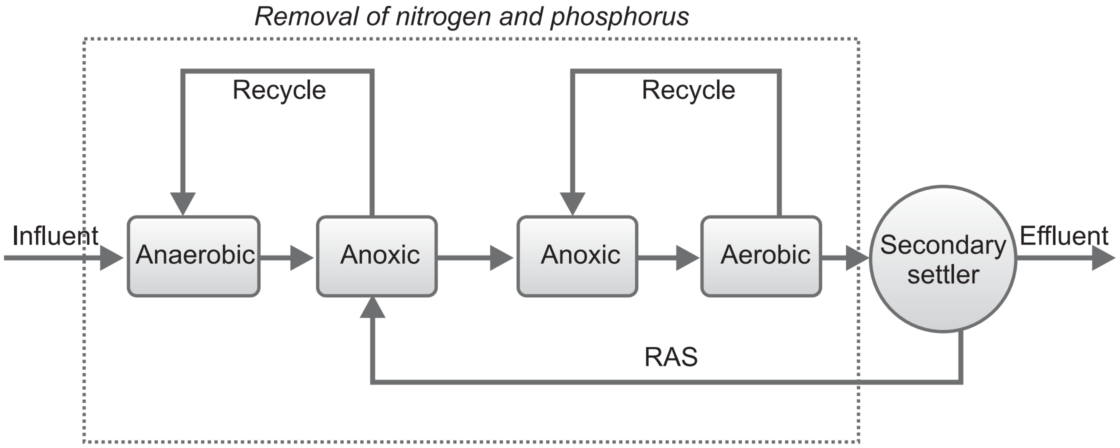


Figure 7.1 Tertiary treatment process for the removal of nutrients.

This chapter provides a brief description of nutrient removal mechanisms in tertiary treatment processes. Sections 7.1-7.5 present common and newly discovered nitrogen removal routes and mechanisms in treatment plants. The chemical and biological removal phosphorus have been discussed in sections 7.6-7.9. Section 7.10 provides a design example, to assist the readers, in order to understand the design procedures for achieving simultaneous nitrification and organics removal (in treatment plants).

7.1 Removal of Nitrogen

Nitrogen (N) is one of the principal pollutants in wastewater that can cause eutrophication, affect the dissolved oxygen level of receiving water, and impart toxicity to the aquatic organisms. Nitrogen exists in wastewater in both organic and inorganic forms. Organic nitrogen includes amino acids (forms peptide chain that make proteins), urea (disposed of ammonia by mammals when amino acids are used for energy production), uric acids (produced by birds and insects) and purine, pyrimidines (involved in DNA making) (Kadlec and Knight, 1996). The inorganic forms of nitrogen are ammonia (NH_4^+), nitrite

(NO^{2-}), nitrate (NO_3^-), nitrous oxide (N_2O), and dissolved elemental nitrogen or nitrogen gas (N_2).

Major pathways of nitrogen transformations, which ultimately remove N from wastewater in activated sludge include: ammonification, biological nitrification, and denitrification, as illustrated in Figure 7.2.

Ammonification. It is the process where organic nitrogen is converted into inorganic NH_4-N, as shown in equation (7.1).

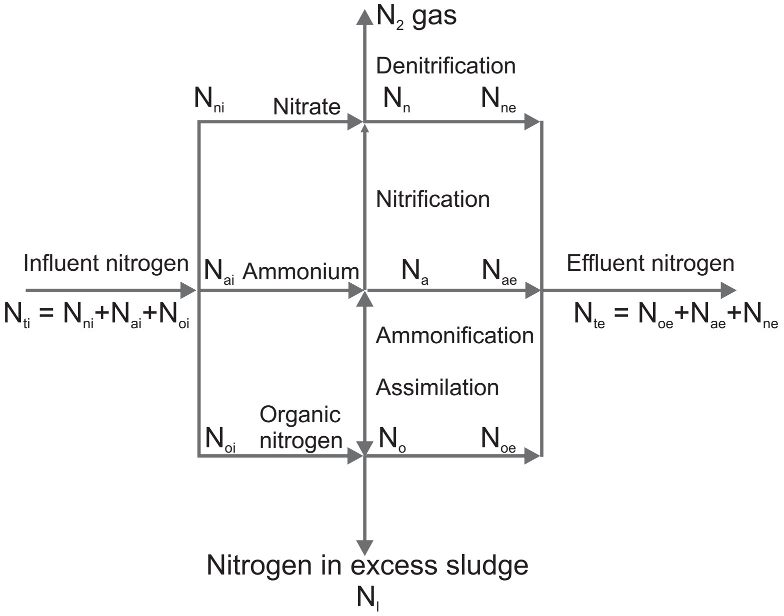


Figure 7.2 Traditional nitrogen removal pathways in activated sludge process. The notations N_{ti}, N_{ai}, N_{ni}, N_{oi} denote influent total nitrogen, ammonium, nitrate, organic nitrogen; N_{te}, N_{ae}, N_{ne}, N_{oe} denote effluent total nitrogen, ammonium, nitrate, organic nitrogen.

$$Organic\ Nitrogen \xrightarrow{heterotrophic} NH_4^+\text{-}N \tag{7.1}$$

Ammonification is faster in aerobic environments; the process is slower in facultative and anaerobic conditions (Reddy and Patrick, 1984). The ideal pH range for ammonification is 6.5-8.5 (Patrick and Wyatt, 1964; Vymazal, 1995). Ammonification proceeds faster at higher temperature, doubling rate with temperature increase of 10°C (Kadlec and Knight, 1996).

Nitrification. Nitrification is a two step process where ammonium nitrogen is first converted to nitrite nitrogen by *Nitrosomonas* bacteria, then to nitrate nitrogen by *Nitrobacter* bacteria. This can be expressed in the following equations proposed by Reddy and Patrick (1984):

$$NH_4^+ + 1.5O_2 \xrightarrow{Nitrosomonas} H^+ + H_2O + NO_2^- \tag{7.2}$$

$$NO_2^- + 0.5O_2 \xrightarrow{Nitrobacter} NO_3^- \tag{7.3}$$

The overall nitrification reaction is given below:

$$NH_4^+ + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O \tag{7.4}$$

Taking into account of the nitrogen assimilated by bacteria for their own cell synthesis, the first nitrification bioreaction can be written as (U.S. EPA, 1993):

$$55NH_4^+ + 76O + 109HCO_3^- \rightarrow C_5H_7NO_2 + 54NO_2^- + 57H_2O + 104H_2CO_3 \tag{7.5}$$

And, the second nitrification reaction can be written as:

$$400NO_2^- + NH_4^+ + 4H_2CO_3 + HCO_3^- + 195O_2 \rightarrow C_5H_7NO_2 + 3H_2O + 400NO_3^- \tag{7.6}$$

Equations (7.5) and (7.6) can be combined to determine the overall oxidation of ammonia and cell synthesis of biomass (Kadlec and Knight 1996):

$$\begin{aligned} NH_4^+ + 1.83O_2 + 0.98HCO_3^- \rightarrow 0.021C_5H_7NO_2 + 1.04H_2O \\ + 0.98NO_3^- + 1.88H_2CO_3 \end{aligned} \tag{7.7}$$

Several conclusions can be made from the above equation, regarding the compulsory environmental conditions required for nitrification:

(i) 1 mol of ammonia nitrogen produces 0.021 mol of microbial biomass (0.17 g of dry weight biomass per gram of ammonia nitrogen consumed), and 3.22 g O_2 are consumed for per gram NH_4^+ oxidation, 1.11 g of O_2 consumed per g NO_2^- oxidation (Kadlec and Knight 1996). Complete NH_4^+ oxidation requires 4.2-4.5 mg O_2 mg^{-1} N (Brix, 1987).

(ii) 7.14 mg/L (as $CaCO_3$) of alkalinity are consumed for each mg/L nitrification of ammonia nitrogen, and 1.98 mol of H^+ are released for each mole of ammonia nitrogen consumed (Kadlec and Knight 1996).

Denitrification. Denitrification, the main mechanism of total nitrogen (TN) removal in activated sludge processes, is a bacterial process when N oxides (ionic and gaseous forms) serve as terminal electron acceptors for respiratory electron transport, and organic compounds serve as electron donors (Vymazal, 1995). The conversion process of nitrate to nitrite, and then to nitrogen gas (via denitrification) is illustrated in Equations (7.8)-(7.9):

$$2NO_3^- + 4e^- + 4H^+ \rightarrow 2NO_2^- + 2H_2O \tag{7.8}$$

$$2NO_2^- + 6e^- + 8H^+ \rightarrow N_2 + 4H_2O \tag{7.9}$$

The overall transformation can be expressed by Equation (7.10):

$$2NO_3^- + 10e^- + 12H^+ \rightarrow N_2 + 6H_2O \tag{7.10}$$

Equation (7.10) indicates that each mole of nitrate reduction is associated with the acceptance of five electrons. When an external carbon source is being employed (for example methanol-CH_3OH), the stoichiometry of denitrification reaction becomes (U.S. EPA, 1993; Kadlec and Knight, 1996):

$$NO_3^- + 0.833CH_3OH \rightarrow 0.5N_2 + 0.833CO_2 + 1.167H_2O + OH^- \tag{7.11}$$

$$NO_3^- + 1.08CH_3OH + 0.24H_2CO_3 \rightarrow 0.056C_5H_7NO_2 + 0.47N_2 + 1.68H_2O + HCO_3^- \quad 7.12$$

From equation (7.12) it can be seen that denitrification process produces alkalinity, and approximately 3 g bicarbonate (as $CaCO_3$) is produced for per g NO_3-N reduction.

The facultative bacterial groups involved for denitrification include: *Bacillus, Enterobacter, Micrococcus, Pseudomonas, Spirillum* (Kadlec and Knight 1996). Although it is uncommon for denitrification to occur in the presence of DO, the process has been observed in suspended and attached bacteria growth environment with low DO content and anoxic zones (Kadlec and Knight, 1996; Lee et al., 2009), including places containing finest sediments and in DO saturated conditions (Cerezo et al., 2001).

7.2 Nitrification Theory

Kinetics. The growth of Nitrosomonas bacteria (that converts NH_4-N to NO_2-N-Equation 7.5) in activated sludge process can be described, employing Monod equation:

$$\frac{dX_n}{dt} = \left(\frac{dX_n}{dt}\right)_g + \left(\frac{dX_n}{dt}\right)_d \quad 7.13$$

$$= \frac{\mu_m X_n S_a}{K_s + S_a} - k_d X_n$$

where X_n = Nitrosomonas concentration, mg VSS/L

$\frac{dX_n}{dt}$ = net growth rate of *Nitrosomonas*, mg VSS/L

$\left(\frac{dX_n}{dt}\right)_g$ = *Nitrosomonas* growth rate, mg VSS/L

$\left(\frac{dX_n}{dt}\right)_d$ = *Nitrosomonas* decay rate, mg VSS/L

μ = specific growth rate for *Nitrosomonas*, d^{-1}; for example μ=0.4 indicates that the daily rate of microorganism synthesis is equal to 40% of the mass originally present.

μ_m = maximum specific growth rate for *Nitrosomonas*, d^{-1}

k_d = Nitrosomonas decay rate, d^{-1}

K_s = Monod half saturation constant, mg N/L

S_a = residual substrate concentration (i.e. ammonium), mg/L

Equation (7.13) can be used in completely mixed steady state activated sludge process, for calculating the residual ammonium concentration (S_a, units in mg/L). However, under such conditions, there is no mass variation of *Nitrosomonas* in the system; the net growth rate (growth rate minus decay rate) is equal to the discharge rate (due to discharge through excess sludge). Considering these boundaries, Equation 7.13 can be written as:

$$\frac{dX_n}{dt} = 0 = \left(\frac{dX_n}{dt}\right)_g + \left(\frac{dX_n}{dt}\right)_d + \left(\frac{dX_n}{dt}\right)_e \quad 7.14$$

The term $\left(\frac{dX_n}{dt}\right)_e$ refers to change of Nitrosomonas concentration as a result of excess sludge discharge, as expressed in Equation (7.15).

$$\left(\frac{dX_n}{dt}\right)_e = -\frac{X_n}{\theta_c} \qquad 7.15$$

where c = sludge age, d

As such, Equations (7.13) and (7.14) can be written as:

$$\frac{dX_n}{dt} = 0 = \frac{\mu_m X_n S_a}{K_s + S_a} - k_d X_n - \frac{X_n}{\theta_c} \qquad 7.16$$

And the residual ammonium concentration (S_a) can be calculated by Equation (7.17):

$$S_a = \frac{K_s\left(k_d + \frac{1}{\theta_c}\right)}{\mu_m - \left(k_d + \frac{1}{\theta_c}\right)} \qquad 7.17$$

The residual ammonium concentration depends on four parameters: μ_m, K_s, k_d and θ_c ; residual concentration does not depend on initial concentration, and can never be superior to the available ammonium concentration for nitrification. As such, the minimum sludge age (cm) for nitrification is defined by Equation (7.18), after rearranging Equation (7.17):

$$\theta_{cm} = \frac{1 + \frac{K_s}{S_a}}{\mu_m - k_d\left(1 + \frac{K_s}{S_a}\right)} \qquad 7.18$$

At minimum nitrification sludge age, the available ammonium concentration for nitrification will be greater than half saturation value K_s; in such case, $K_s/S_a <<1$, and Equation (7.18) can be simplified to:

$$\theta_{cm} = \frac{1}{\mu_m - k_d} \qquad 7.19$$

Ammonium oxidation rate. The ammonium oxidation rate (conversion of NH_4-N to NO_2-N) is related to the Nitrosomonas growth rate, and can be expressed as:

$$r_N = \frac{\mu}{Y_N} = \frac{\frac{\mu_m S_a}{K_s + S_a}}{Y_N} = \hat{r}_N \frac{S_a}{S_a + K_s} \qquad 7.20$$

where r_N = ammonia oxidation rate, lb NH_4^+-N oxidized/lb VSS.d
μ = Nitrosomonas growth rate, d^{-1}
Y_N = organism yield coefficient, lb Nitrosomonas
$\hat{r}_N = \frac{\mu_m}{Y_N}$ = peak ammonia oxidation rate, lb NH_4^+-N oxidized/lb VSS.d
μ_m = peak Nitrosomonas growth rate, d^{-1}

Dependency on temperature. The kinetics of nitrifying bacteria is inherently dependent on the change of temperature. In case of rapid temperature rise, their growth rate is lower; in contrast, higher growth decline rate is often associated with sudden temperature drop. The growth rate of nitrifiers is lower in colder regions, compared with tropical regions. For such reasons, in European countries, the activated sludge processes are operated at a sludge age >15 days for nutrient removal (Haandel and Lubbe, 2007).

The critical dependency between the metabolism of nitrifiers, and temperature change can be illustrated by Arrhenius dependency coefficient (θ), as illustrated below:

$$\mu_{m(T)} = \mu_{m(20^\circ C)}\theta^{T-20} \qquad 7.21$$

where T = temperature in °C

Different values have been proposed for the coefficient θ, as illustrated in Table 7.1.

Table 7.1 Arrhenius coefficient for Nitrosomonas bacteria.

Arrhenius coefficient θ	**Temperature interval °C**	**Source**
1.116	19-21	Gujer (1977)
1.123	15-20	Downing et al., (1964)
1.130	20-30	Lijklema (1973)

Downing and Hopwood (1964) provided kinetic equations on the inter-relationship between maximum growth (μ_m), half saturation constant (K_s), and temperature for both *Nitrosomonas* and *Nitrobacter* in suspended growth system. Such relationships have been illustrated in Equations (7.22)-(7.23).

$$\mu_m = 0.47e^{0.098(T-15)} \qquad 7.22$$

$$K_s = 10^{0.051T-1.158} \qquad 7.23$$

Dependency on oxygen. The kinetics of nitrifiers is critically dependent on oxygen concentration; such dependency can be described by Monod type equation, as illustrated in Equation (7.24).

$$\mu = \frac{\mu_m S_a}{S_a + K_s}\frac{DO}{DO + K_{s,O_2}} \qquad 7.24$$

where DO= dissolved oxygen concentration, mg O_2/ L
$K_{s,O2}$= half saturation oxygen constant, mg O_2/L

Typical half saturation values of O_2 vary between 0.5-1.5 mg/L. It should be noted that these values have been determined from the bulk phases of the liquid. However, in the deeper portion of the sludge flocs, the concentration of DO is lower due to oxygen consumption (Figure 7.3). As such, anaerobic portion may develop in deeper sludge flocs, promoting denitrification.

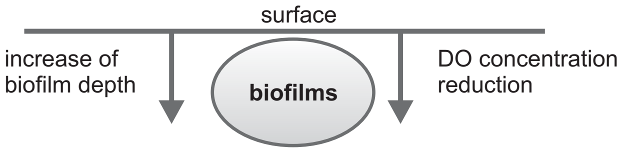


Figure 7.3 Reduction of oxygen concentration at deeper portion of the sludge flocs.

Dependency on pH. Since nitrification process consumes alkalinity (Equation 7.7), it is enhanced between pH values 7.0-8.5, and is hindered at a pH value <7 (Figure 7.4). In activated sludge processes, the consumption of alkalinity due to nitrification decreases pH; as such, the pH of the mixed liquor is often observed to be less than 8.

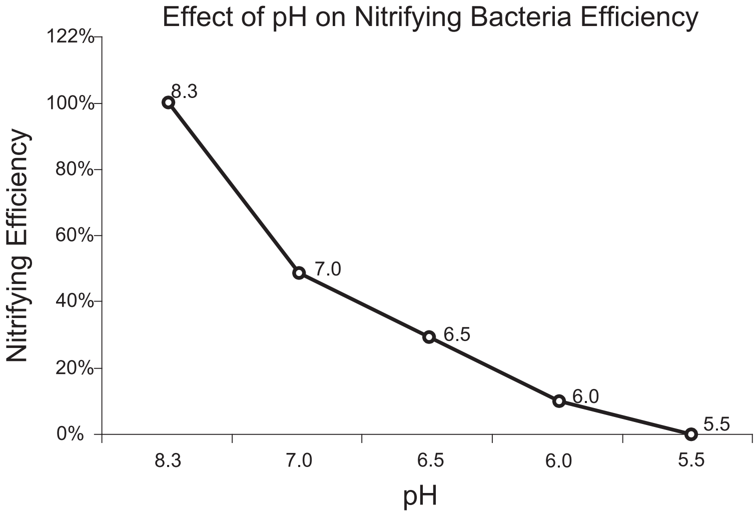


Figure 7.4 Dependency of nitrification process on alkalinity.

The impact of pH on the growth rate of nitrifiers can be illustrated by the kinetic model (Equation 7.25) developed by Downing and Hopwood (1964), for pH values up to 7.2.

$$\mu = \mu_m [1 - 0.833\,(6.2 - \text{pH})] \qquad 7.25$$

Multiple Monod kinetics: effect of substrate, DO, pH and temperature. Since nitrification process is critically dependent on four factors: (a) presence of ammoniacal nitrogen; (b) DO; (c) pH; and (d) temperature change, the impact of these parameters on the growth rate of nitrifiers can be simultaneously expressed by multiple Monod equation, illustrated in Equation (7.26) (US EPA, 1975):

$$\mu = \mu_m \left(\frac{S_a}{K_s + S_a}\right)\left(\frac{DO}{DO + K_{s,O_2}}\right)[1 - 0.833\,(6.2 - \text{pH})] \qquad 7.26$$

Substituting effect of temperature (Equation 7.22), Ks (Equation 7.23), $K_{s,O2}$= 1.3 mg/L, Equation (7.26) can be developed for *Nitrosomonas* kinetics (Equation 7.27), when pH <7.2, and wastewater temperature ranges between 8-30°C. For pH≥7.2, the term $[1 - 0.833(6.2 - \text{pH})]$ in Equation (7.27) will be equal to unity (Lin, 2007).

$$\mu = 0.47[e^{0.098(T-15)}]\left[\frac{S_a}{10^{(0.051T-1.158)} + S_a}\right]\left[\frac{DO}{1.3 + DO}\right][1 - 0.833\,(6.2 - \text{pH})] \qquad 7.27$$

Table 7.2 enlists typical reaction rate constants of nitrifying bacteria.

Table 7.2 Reaction rate constants for nitrifying bacteria at 20°C.[a]

	Symbol	Unit	Ammonia oxidation	Nitrite oxidation
Maximum specific growth	μ_{max}	d^{-1}	0.6-0.8	0.6-1.0
Saturation constant	$K_{s, NH4}$	g NH_4-N/m^3	0.3-0.7	0.8-1.2
Saturation constant	$K_{S, O2}$	g O_2/m^3	0.5-1.0	0.5-1.5
Maximum yield constant	Y_{max}	g VSS/g N	0.1-0.12	0.05-0.07
Decay constant	k_d	d^{-1}	0.03-0.06	0.03-0.06

[a] *After Henze et al., (2002).*

7.3 Denitrification Theory

Denitrification process is dependent on several environmental and operating factors, such as: presence of facultative biomass, presence of nitrate, lack of oxygen, pH, temperature and electron donor availability. A brief description of these factors has been illustrated in the following sections.

Presence of facultative biomass. The denitrification process is often performed by facultative biomass, which can utilize both oxygen and nitrate as an oxidant for organic matter. The aerobic sludge (of activated sludge process) usually consume nitrate rapidly in anoxic environment, as significant portion of such biomass is facultative.

Presence of nitrate. The presence of nitrate is a prerequisite to foster denitrification in treatment plants. However, nitrate concentration > 0.5 mg/L (in wastewater) has insignificant impact on denitrification rate; in such cases, nitrogen removal becomes independent of nitrate concentration.

Absence of oxygen. The denitrification process proceeds in a zone that lacks of dissolved oxygen. Dissolved oxygen concentration > 0.2-0.5 mg/L reduces the denitrification rate substantially. The deeper portion of the microbiological films often provides an ideal environment for the reduction of nitrate, due to oxygen deprived conditions.

pH. Literature reports show that denitrification process can be hampered at pH <6.0, and pH>8.0, and the highest rate is observed at pH ranges 7.0-7.5 (U.S. EPA, 1975). It may occur at a slower rate when pH < 5 (Vymazal, 2007).

Temperature: Denitrification has been found to proceed slowly at low temperatures (e.g. 5°C), and its reaction rate increases exponentially with higher temperature, reaching a plateau between 20-25°C, as long as other environmental factors do not restrict the rate (U.S. EPA, 1975).

Presence of an electron donor. Denitrification process is critically dependent on the presence of electron donor materials (i.e. biodegradable organics), for the reduction of nitrate. In treatment plants, this requirement can be accomplished by adding organic matter externally (for example: methanol, ethanol, acetone, acetic acid etc.), or providing organic matter internally.

Monod kinetics for denitrification. The above sections illustrate that the main limiting factors for continuing denitrification metabolism are: (a) availability of nitrate; (b) availability of organic carbon, and (c) dominancy of anoxic/anaerobic conditions. These three limiting factors can be expressed through multiple Monod kinetics by: (a) taking into account of the NO_3-N level; (b) using COD as an indicator of organic carbon for denitrification; and (c) taking into account of the inhibitory oxygen constant S_{DOi} (denitrification ceases above this constant value as this process occurs in anoxic/anaerobic environment). The interrelation of these factors can be expressed through the following equation.

$$\mu = \mu_m \frac{S_{NO3}}{S_{NO3} + K_{s,NO3}} \frac{S_{DOi}}{S_{DO_i} + DO} \frac{S_{COD}}{S_{COD} + K_{s,COD}} \qquad 7.28$$

where, S_{NO3} = effluent NO_3-N concentration, mg/L
S_{DOi} = inhibitory oxygen constant, mg/L
S_{COD} = effluent COD concentration, mg/L
$K_{S,COD}$ = half saturation COD constant, mg/L
$K_{S,NO3}$ = half saturation NO_3-N constant, mg/L

Typical denitrification kinetic coefficients have been illustrated in Table 7.3.

Table 7.3 Denitrification rate constants (Henze et al., 2002).

	Symbol	Unit	Value
Maximum growth rate for organic matter in wastewater	μ_{max}	d^{-1}	3-6
Maximum growth rate for methanol	μ_{max}	d^{-1}	5-10
Decay constant	k_d	d^{-1}	0.05-0.10
Saturation constant, nitrate	$K_{S, NO3}$	g N/m^3	0.2-0.5
Saturation constant, methanol	$K_{s, MeOH}$	g COD/m^3	5-10
Saturation constant, COD in wastewater	$K_{S, COD}$	g COD/m^3	10-20
Inhibition constant, oxygen	S_{DOi}	g O_2/m^3	0.1-0.5

7.4 Aerobic-Anoxic Reactors for Nitrification-Denitrification

In a typical activated sludge process, if the MLSS is aerated in the aeration tank for a longer time period, the NH_4-N of wastewater is nitrified. When this nitrified liquor is transferred to the following secondary clarifier, an ideal environment is created for denitrification after the consumption of oxygen. The dentrification in the secondary clarifiers produces nitrogen gas bubbles, which rises towards the water surface. These nitrogen bubbles may be attached to sludge flocs, resulting lifting (from the bottom of the clarifier) and floatation of sludge in the water surface. Such floating sludge layer escapes with the effluent, which often fails to fulfill discharge criteria (due to presence of solids). In addition, excess discharge of sludge reduces the sludge age and mass in the system, diminishing system efficiency. To overcome such problems, an anoxic reactor is added to the system, where the MLSS is remained in suspended form through moderate stirring without aeration. The amalgamation of aerobic and anoxic reactors accomplishes nitrification-denitrification of the MLSS, prior to its entrance into the secondary clarifier.

The arrangement of aerobic and anoxic reactors for achieving nitrification-denitrification can be accomplished by three sludge systems (where organic carbon is added externally), or by single sludge system (where organic carbon is provided internally). The single sludge system can further be classified into pre D, post D or Bardenpho systems, depending on the arrangement of the anoxic reactors. A brief description of each of these systems has been provided below.

Three sludge systems. The system comprises of three reactors; each reactor is followed by a separate settler. Influent wastewater is passed through the first stage aerobic reactor, to facilitate the removal of organics; the MLSS is transferred from the first settler to the second aerobic reactor for nitrification. The nitrified effluent is then passed through the following anoxic reactor. Since such nitrified effluent lacks biodegradable organics, external source of organic carbon (for example methanol) is mixed with wastewater (to foster denitrification), prior to anoxic reactor entrance. The presence of three stages increases the construction cost, which is a major drawback of these systems. Figure 7.5 gives a schematic diagram of the three sludge systems.

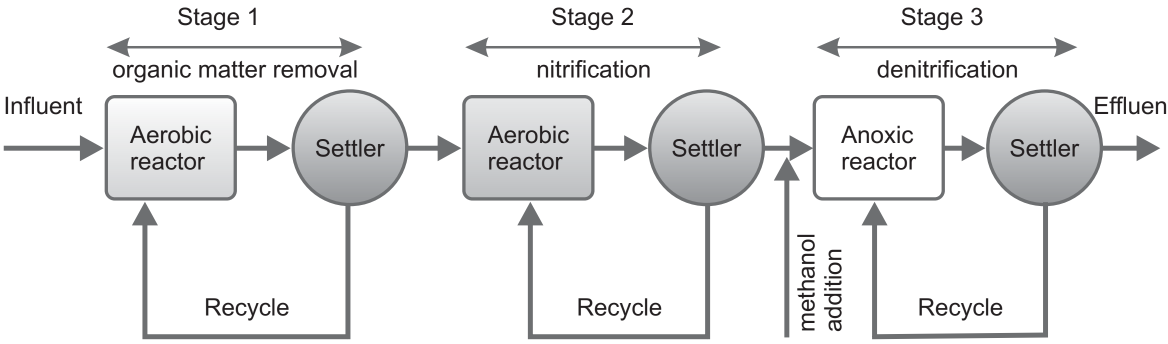


Figure 7.5 Three sludge systems for the removal of organics and nitrogen.

Pre denitrification (Pre D systems). This system comprises of an anoxic reactor, followed by an aerobic reactor and a settler (Figure 7.6). The nitrate produced in the second aerobic reactor is recirculated to the first anoxic tank, along with return sludge from the settler for denitrification. This system exhibits higher denitrification rates due to presence of substantial amount of biodegradable organics in the anoxic reactor. However, this system cannot achieve complete nitrate removal, as a portion of the produced nitrate (from aerobic reactor) is discharged directly from the settler without being recirculated to the anoxic reactor

Post denitrification (Post D systems). In post D systems, the aerobic reactor is followed by an anoxic reactor and a settler. Wastewater is firstly treated in the aerobic reactor, where complete biodegradable organic matter removal and nitrification occurs. The mixed liquor is then transferred to the following anoxic reactor. Non-metabolized organic matter (not removed in the aerobic reactor), and organic matter released during the decay of sludge acts as a source of carbon (for denitrification) in the anoxic tank. The mixed liquor is then

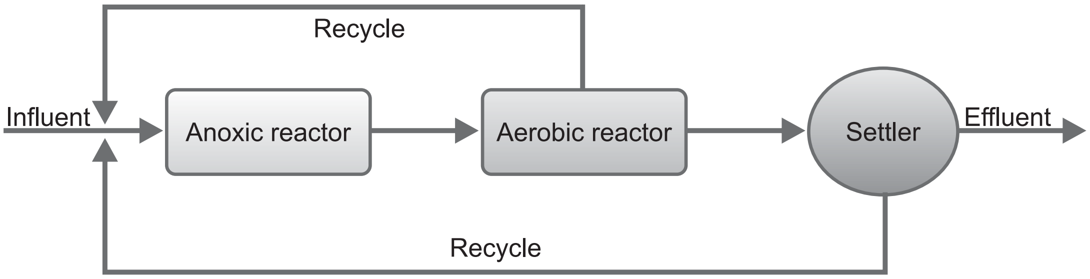


Figure 7.6 Pre D systems for nitrogen removal.

transferred to the following settler, which is again recirculated to the aerobic reactor. The main disadvantage of this system can be attributed to lower biodegradable organic matter availability, for denitrification, thereby diminishing denitrification rate. Figure 7.7 illustrates an operational diagram of the post D systems, for the removal of nitrogen (from wastewater).

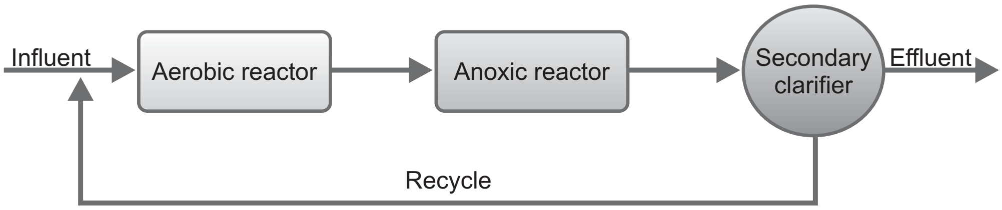


Figure 7.7 Post D systems for nitrogen removal.

Bardenpho system. This system includes the principle of both pre and post D systems for denitrification. The system comprises of three reactors; the first reactor is anoxic followed by an aerobic and an anoxic reactor (Figure 7.8). The nitrate produced in the aerobic reactor is recirculated directly to the first anoxic reactor for denitrification. The remaining nitrate is reduced in the third anoxic reactor. The MLSS is then transferred to a settler. A smaller fourth reactor can be provided in these systems, for reaeration of the anoxic MLSS. Such reaeration removes the nitrogen bubbles formed in the post D reactor, thereby eliminating the possibilities of sludge floc floatation in the settler.

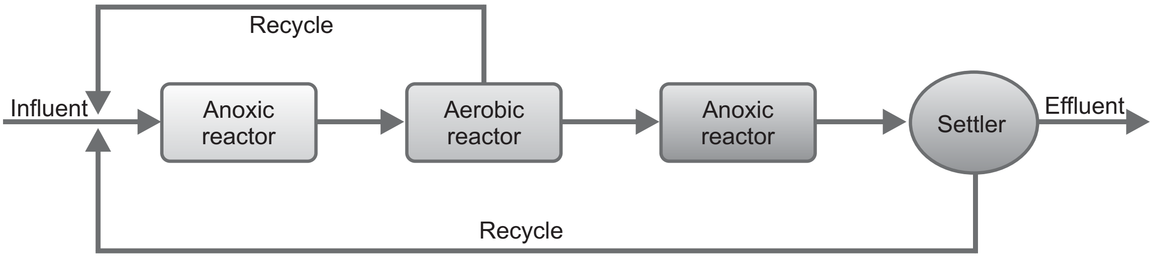


Figure 7.8 Configuration of Bardenpho systems.

7.5 New Nitrogen Removal Routes

The above sections have been focused on nitrification-dentirification process in activated sludge processes, for the removal of nitrogen from wastewater. Apart from the traditional nitrogen removal mechanisms, several new nitrogen biodegradation routes have been discovered to date. These include: Sharon (Single reactor for high activity ammonium removal over nitrite), Babe (Bio-augmentation batch enhanced), Anammox (Anaerobic ammonium oxidation), Canon (Completely autotrophic nitrogen removal over nitrite), and Deammox (Denitrifying ammonium oxidation). This section provides a brief description on these newly implemented nitrogen removal routes.

Sharon process. The nitrogen removal by Sharon process depends on the metabolism of ammonium and nitrite oxidizers, with increase and decrease of temperature. When temperature is less than 20°C, the kinetics of nitrite oxidation proceeds at a higher rate, compared to ammonium oxidizers. As such, when temperature falls down below 20°C, it is almost impossible to limit the oxidation of nitrite, thereby hindering the application of Sharon process.

However, above 20°C the growth rate of ammonium oxidizers becomes higher than nitrite oxidation. Such growth rate of ammonium oxidizers allows accumulation of nitrite, and limits the oxidation of nitrite to nitrate, which fulfills the basic principle of Sharon process. The aerobic sludge age is also controlled at lower ranges to prevent the growth of nitrite oxidizers (Figure 7.9a), resulting conversion of ammonium to nitrite only. Since a portion of ammonium is converted to nitrite, a typical Sharon reactor effluent comprises of 10-50 mg/L of NH_4-N. If aerobic sludge age is increased, NH_4-N concentration is reduced; this increase also re-establishes nitrite oxidizers again in the reactor, thereby converting the nitrite to nitrate. It should be noted that the bacteria in the Sharon effluent is present in free form; as such, the effluent from Sharon reactor consists of significant biomass and organic nitrogen.

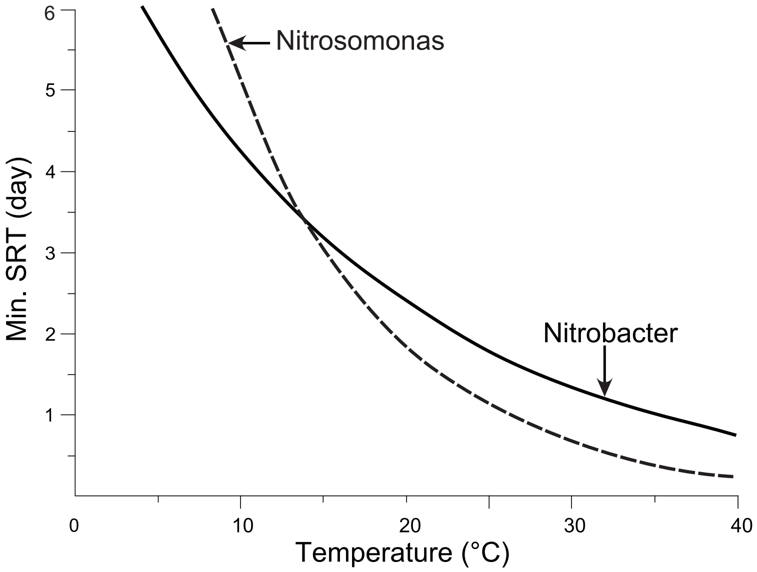


Figure 7.9 (a) Growth rates of nitrifying bacteria depending on temperature.

The Sharon reactor can be constructed either as a single tank with alternate aerobic-anoxic phase (Figure 7.9b), or as series of two tanks: an aerobic tank followed by an anoxic reactor. The aerobic phase is usually maintained for 80 minutes, followed by an anoxic period of 40 minutes (Have and Kempen, 2004). In a single stage Sharon reactor, the alkalinity produced in the anoxic phase can be immediately utilized by the following aerobic phase for nitrification. However, one disadvantage of a single stage Sharon reactor is the possibilitiy of

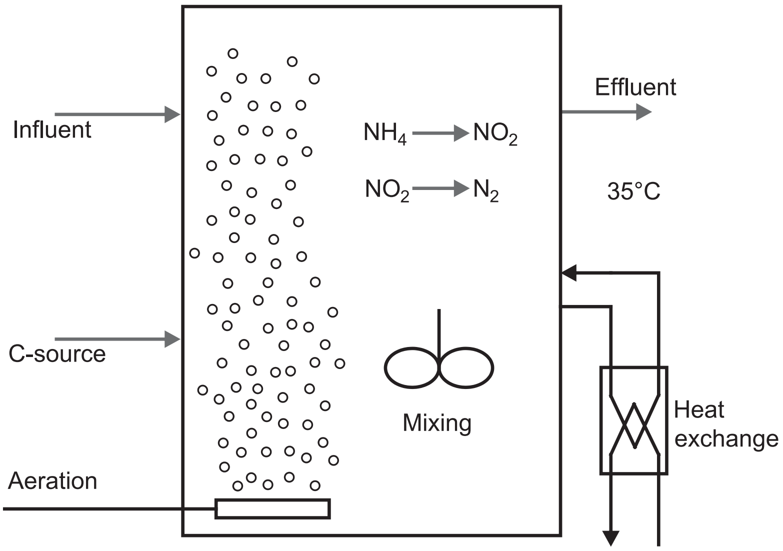


Figure 7.9 (b) Single stage Sharon reactor.

direct discharge of a portion of ammonium with effluent, due to the transition towards anoxic phase. On the other hand, in two stage Sharon reactors, the influent is nitrified first in the aerobic reactor, followed by denitrification of the produced nitrite in the anoxic reactor. For these reasons, the effluent ammonium concentration of two stage tanks is often lower, compared with single stage Sharon reactor. A recirculation pump is required in two stage systems, to transfer the produced alkalinity into the anoxic reactor (due to denitrification) to the aerobic reactor (for nitrification). The preferable temperature ranges for Sharon reactors fall within 30-35°C.

The Sharon process has certain advantages over traditional nitrification process. According to Equations (7.2), and (7.3), oxidation to nitrite requires 3/4th of the demand for oxidation to nitrate, thereby reducing the aeration cost. Subsequently, the denitrification of the produced NO_2 requires only $3/5^{th}$ of the organic material required for nitrification (Equations 7.29-7.30).

$$NO_2^- + 0.5CH_3OH + H^+ \rightarrow 0.5N_2 + 0.5CO_2 + 1.5H_2O \qquad 7.29$$

$$NO_3^- + 0.83CH_3OH + H^+ \rightarrow 0.5N_2 + 0.83CO_2 + 2.16H_2O \qquad 7.30$$

Babe process. The Babe process includes a small reactor, where a part of the return sludge from the secondary settler is contacted with sludge reject water (after sludge dewatering) (Figure 7.11 a). The Babe process has two-fold objectives in an activated sludge system: (a) it reduces the nitrogen load from the sludge treatment recycle streams; and (b) it seeds (augments) the main activated sludge systems with nitrifiers integrated in sludge flocs, thereby increasing nitrification capacity. Such augmentation is extremely important in systems, where effluent ammonium levels are higher due to lower sludge age.

The Babe reactor is operated in five steps: filling, aeration, mixing, settling and discharging (Figure 7.11 b); these steps are usually completed within three hours. Filling is continued, as long as the centrifuge is in operation. Aeration and mixing are alternated in order to achieve nitrification and denitrification in the Babe reactor. A portion of return activated sludge

Box 7.1

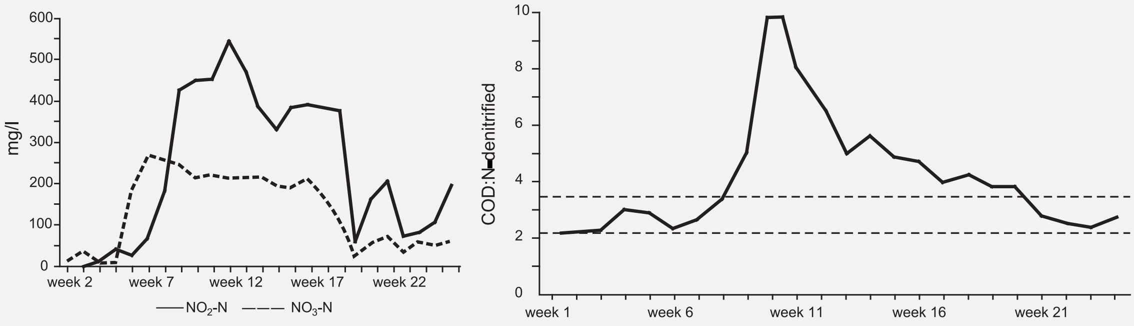
Performance of the Sharon reactor in Netherlands (Mulder et al., 2001).

Table 7.4, provides the dimensional values of a Sharon reactor, constructed in the Rotterdam, Dorkhaven, Netherlands at 1999 (Mulder et al., 2001), for the removal of nitrogen from wastewater.

Table 7.4 Design parameters of Dokhaven Sharon reactor, Netherlandsa (Mulder et al., 2001).

Design Parameter	Dimension	Value
Tank Volume	m^3	1800
Flow, design	m^3/h	31.5
Flow, maximum	m^3/h	50
N-load, design	kg/d	540
N-load, maximum	kg/d	830
NH_4, influent	mg/L	1000-1500
Retention Time aerobic	d	1
Retention Time anoxic	d	0.5-1.5

The nitrogen removal performances by the Dokhaven Sharon reactor are illustrated in Figure 7.10, indicating that NH_4-N was initially converted to both NO_2-N and NO_3-N, due to higher oxic detention times. After controlling the cyclic times, the NH_4-N was converted to NO_2-N, and then to N_2 gas, as indicated by decrease of C/N ratio to 2.4 (Mulder et al., 2001).



(a) (b)

Figure 7.10 (a) Effluent NO_2-N and NO_3-N concentration across the Dokhaven Sharon reactor; and (b) COD/N ratio of the reactor effluent (Mulder et al., 2001).

(RAS) is introduced in the Babe reactor during denitrification; the carbon in the RAS reduces methanol requirements.

Anammox process. Anaerobic ammonium oxidation (Anammox) is a newly discovered nitrogen removal process, where ammonium is directly oxidized to nitrogen gas (by nitrite) in presence of planctomycete bacteria group under anaerobic conditions. Compared with conventional nitrification and denitrification processes, the advantages of Anammox process are: (a) no requirement of external carbon sources; (b) lower oxygen demand; and (c) lower energy consumption. The overall reaction of the Anammox process has been provided in Equation (7.31), according to Jetten et al., (2000):

$$NH_4^+ + 1.32NO_2^- + 0.066HCO_3^- + 0.13H^+ \rightarrow 1.02N_2 + 0.066CH_2O_{0.5}N_{0.15} + 0.26NO_3^- + 2.03H_2O \qquad 7.31$$

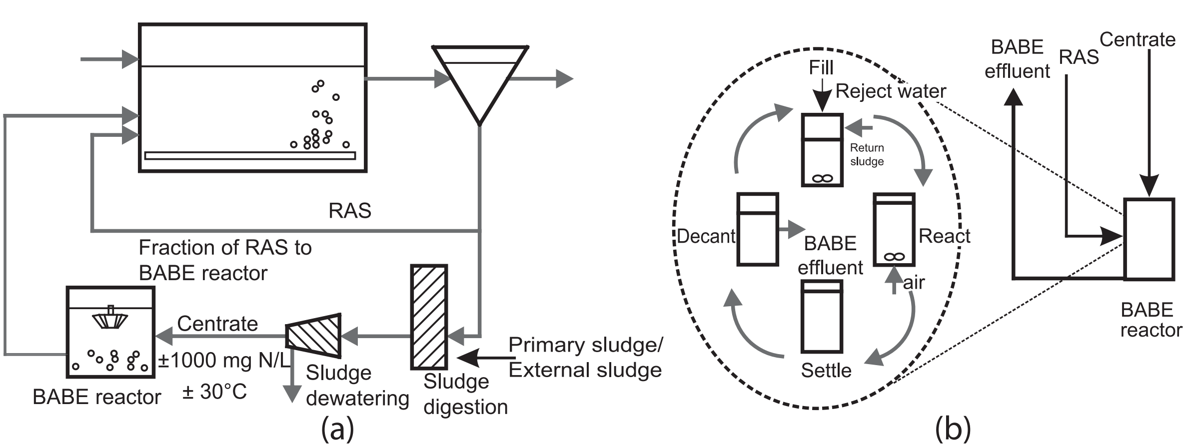


Figure. 7.11 (a) Schematic operation of the Babe process; and (b) operational steps of the Babe process.

Complete ammonium removal by Anammox process requires ammonium and nitrite ratio of 1/1.32 (Jetten et al., 2000). If different ammonium and nitrite ratios are present in wastewater, complete ammonium and nitrite removal are critical to achieve by Anammox process.

The growth rate of Anammox bacteria is extremely low (0.04-0.06 d^{-1} at 35°C). Such lower growth rate often results slow start up of the Anammox reactor. The presence of various substrates in wastewater (eg. sulphide, ammonium, nitrite) can also inhibit the growth of Anammox bacteria. If sulphate is present in wastewater, it is converted to sulphide in anaerobic conditions (by sulphate reducing bacteria), that is toxic to Anammox bacteria. However, under anoxic conditions, the growth of sulphate reducers is inhibited. Higher ammonium concentrations (>several hundred mg/L NH_4-N), and nitrite concentration >70 mg/L can also cause growth toxicity; in addition, the Anammox reactor must be operated under nitrite limiting conditions (<50 mg/L NO_2-N) (Haandel and Lubbe, 2007).

The Anammox process is optimized at a pH range of 6.7-8.3, and a temperature range 30-37 °C. When temperature rises above 37 °C, ammonium conversion decreases drastically; in contrast, the conversion of nitrite increases up to a temperature of 42°C, which indicates the possibilities of different metabolic pathway under these conditions.

Combination of Sharon-Anammox process. In a combined Sharon and Anammox process, as illustrated in Figure 7.12, the first Sharon reactor nitrifies 50% influent ammonium. The produced nitrite is transferred to the following Anammox reactor (operated under anaerobic conditions), where nitrite is reduced to nitrogen gas.

When a Sharon reactor is operated without pH control, nitrification proceeds at a rate of approximately 50-60%. At this point, the majority portion of available biocarbonate is consumed, hindering nitrite oxidation with a nitrite-ammonium ratio closer to 1:1 in the effluent. This ratio is suitable for ammonium oxidation in following the Anammox reactor. The operating variables for the Sharon reactor, in order to produce an effluent suitable for the following Anammox reactor are: pH 6.6–7.0, T = 30–40°C, HRT = 1 day, and no sludge retention (Ahn, 2006). The combination of Sharon-Anammox process often saves 40% aeration, and 100% carbon costs, when compared with the conventional nitrification–denitrification processes (Loosdrecht and Jetten, 1998; Dongen et al., 2001).

Partial nitrification and Anammox in a single reactor: Canon process. Canon process is an autotrophic nitrogen removal process, that provides treatment of highly loaded nitrogen streams with an unfavorable carbon to nitrogen ratio. The main principle of this process is dependent on the co-existence of aerobic ammonium oxidizers, and anaerobic Anammox bacteria in a single reactor. This can be established under oxygen-limited conditions to avoid inhibition of Anammox bacteria by oxygen, and to achieve appropriate conditions to obtain partial nitrification.

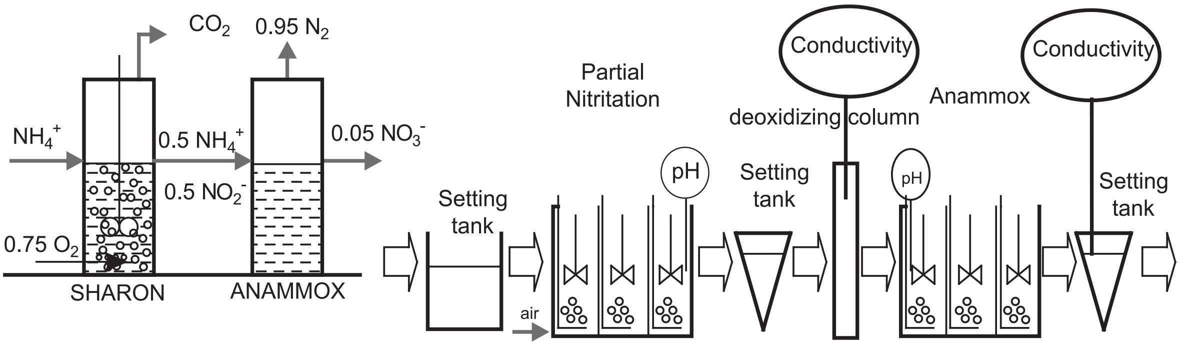


Figure 7.12 Stoichiometric and operating conditions of the Sharon-Anammox process.

Box 7.2

Performance evaluation of Sharon-Anammox process (Jetten et al., 2000).

Jetten et al. (2000) conducted an experiment with combined Sharon-Anammox system, to treat reject water from sludge digestion plant in The Netherlands. The system showed 94% ammonium removal efficacy (Table 7.5), whereas total nitrogen removal performance was 91%. Such nitrogen removal performances illustrate the possibilities of the combined system, for achieving higher nitrogen removal (from wastewater) without carbon addition.

Table 7.5 Nitrogen removal performances by the combined Sharon-Anammox process.[a]

	Unit	Sharon			Anammox		
		Influent	Effluent	Removal	Influent	Effluent	Removal
NH$_4^+$	mg/L	1180	600	580	600	70	530
NO$_2^-$	mg/L	0	550	550	550	0	550
NO$_3^-$	mg/L	0	0	0	0	12	12
Organic N	mg/L	0	30	30	30	30	0
Total N	mg/L	1180	1180	0	1180	112	1068

[a] Jetten et al., (2000).

In Canon process, ammonium oxidizers convert a part of the ammonium into nitrite; the remaining ammonium is converted to nitrogen gas in presence of the produced nitrite, by Anammox bacteria. The growth of nitrite oxidizing bacteria is usually lower in Canon reactor under low concentration of dissolved oxygen, due to their lower affinity to oxygen compared to ammonium oxidizing bacteria. As such, nitrate is primarily produced by the Anammox bacteria metabolism (Equation 7.31) in the Canon reactor. Since the entire nitrogen removal process occurs in a single reactor, it is difficult to control the overall biodegradation routes. The important parameters that can affect nitrogen removal in a Canon reactor are: dissolved oxygen (DO), temperature, pH, free ammonia and sludge retention time.

The overall Canon reaction can be expressed through Equation (7.32) (Third et al., 2001):

$$NH_4^+ + 0.85O_2 \rightarrow 0.435N_2 + 0.13NO_3^- + 1.3H_2O + 1.4H^+ \qquad 7.32$$

Equation (7.32) indicates production of H^+ during Canon process which reduces alkalinity of wastewater, allowing the treatment of higher alkaline wastewaters (i.e. reject water from anaerobic digester).

Deammox process. Deammox process is the integration of denitrification and Anammox processes to remove nitrogen from wastewater that contains higher nitrogen concentration, and slower biodegradable organics. Under these conditions, the growth of heterotrophic denitrifiers is limited, due to lack of easily biodegradable organics. Such limited growth promotes the formation of Anammox bacteria, which converts ammonium to nitrogen gas; the nitrate produced from Anammox process is reduced by heterotrophic denitrifiers. In addition, the intermediate nitrite product (from denitrification process) can be utilized by the Anammox bacteria, for the conversion of ammonium to nitrogen gas.

It should be noted that Anammox bacteria cannot compete with heterotrophic denitrifying bacteria at COD/N ratio greater than 2 (Chamchoi et al, 2008). Consequently, growth of Anammox bacteria is irreversibly inhibited by lower methanol (15 mg/L), and ethanol concentrations (Güven et al, 2005).

Table 7.6 provides a comparative evaluation between the novel nitrogen removal routes, and conventional nitrification-denitrification process.

7.6 Removal of Phosphorus

Phosphorus is the main limiting nutrient, that accelerates eutrophication (see chapter 11) of surface waters. The forms of phosphorus found in wastewater include orthophosphates, polyphosphates (molecularly dehydrated phosphates), and organic phosphates. Orthophosphates for example: PO_4^{3-}, HPO_4^{2-}, $H_2PO_4^-$, H_3PO_4 can be converted biologically. The polyphosphates are subjected to hydrolysis, which is a slow process. Organic phosphates are dominant in industrial wastewaters, and are not significant in municipal wastewater.

Phosphates can be removed from wastewater by (a) chemical precipitation; and (b) biological processes. The following sections describe each of these removal processes.

7.7 Chemical Precipitation of Phosphorus

Principles. Chemical phosphorus removal comprises of chemical precipitation of phosphorus via metal salt. The most commonly employed chemicals, to achieve chemical precipitation are: alum, sodium aluminate, ferric chloride and lime, shown in Equations (7.33)-(7.36).

Table 7.6 Comparison of the innovative processes for nitrogen removal (Jetten et al 2002; Ahn, 2006).

Characteristic	Conventional nitrification/ denitrification	Nitritation/ denitritation (Sharon)	Partial nitritation (50%) and Anammox in two reactors	Partial nitritation and Anammox in one single reactor
Number of reactors	2	2	2	1
Conditions	oxic/anoxic	oxic/anoxic	oxic/anoxic	oxygen limited
Oxygen requirement [gO_2/gN]	4.57 / 0	3.43 / 0	1.71/ 0	1.94
% O_2 saving	-	24.9 %	62.6 %	57.5 %
Alkalinity consumption [$gCaCO_3/gN$]	7.07 / -3.57	7.07 / -3.57	3.57 / 0.24	3.68
pH control	yes	none	none	none
Carbon source requirement [gCOD/gN]	3.7	2.3	0	0
% reduction in carbon source requirement	-	37.8 %	100 %	100 %
Main bacteria involved	Nitrifiers / denitrifiers	Ammonium oxidizers / denitrifiers	Ammonium oxidizers / Anammox	Ammonium oxidizers / Anammox
Biomass retention	none / none	none / none	none / yes	yes
Sludge production	high	low	low	low

Alum:

$$Al_2(SO_4)_3.18H_2O + 2HPO_4^{\;2-} \rightarrow 2AlPO_4 \downarrow + 2H^+ + 3SO_4^{\;2-} + 18H_2O \qquad 7.33$$

Sodium-aluminate:

$$Na_2O.Al_2O_3 + 2HPO_4^{\;2-} + 4H_2O \rightarrow 2AlPO_4 \downarrow + 2NaOH + 6OH^- + 2H^+ \qquad 7.34$$

Ferric chloride:

$$FeCl_3 + HPO_4^{\;2-} \rightarrow FePO_4 \downarrow + H^+ + 3Cl^- \qquad 7.35$$

Lime:

$$5Ca(OH)_2 + 3HPO_4^{\;2-} \rightarrow Ca_5(PO_4)_3OH \downarrow + 3H_2O + 6OH^- \qquad 7.36$$

The pH values (of wastewater) decrease if alum, sodium aluminate, and ferric chloride are employed (Equation 7.33-7.35), whereas, addition of lime increases the pH values (Equation 7.36). The optimum pH for alum and ferric chloride is 5.5-6; for lime the optimum pH values are above 10.

Chemical precipitation configurations. The chemical precipitation (of phosphorus) in activated sludge is achieved by four processes: pre-precipitation, simultaneous precipitation, side stream precipitation and post precipitation.

(a) Pre-precipitation: In this process, chemicals are added in raw influent, or in the primary clarifier. The metal phosphate precipitates are removed in the primary tank. Although this process enhances the removal of solids, and COD/BOD in the primary tank, it can also lead to the depletion of organics, thereby hindering biological nitrogen removal in the following reactors. In addition, this process demands huge amount of chemicals, along with significant production of chemical sludge. Figure 7.13 indicates a typical configuration of pre-precipitation phosphorus removal process.

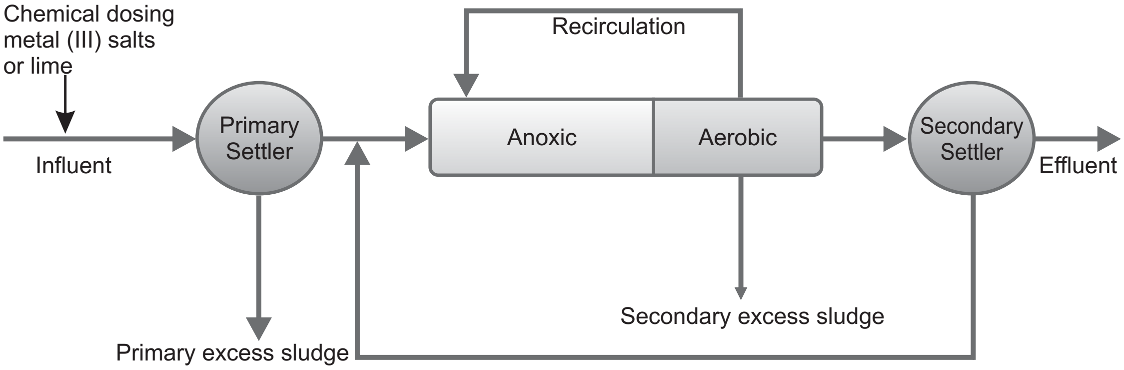


Figure 7.13 Pre-precipitation process.

(b) Simultaneous precipitation: Chemicals are added at aerobic tank, and in the overflow of the aeration tank to the final settler (Figure 7.14). The metal salts Fe(II) are added in the aerobic tank where Fe(II) is oxidized to Fe (III), allowing precipitation of phosphorus. Metal (III) salts are added in the overflow of the aeration tank to the final settler, in order to prevent phosphorus limitation to the biomass. Lime can also be added at this point, as lime dosing in the aeration tank can elevate wastewater pH, which in turn can cause severe reduction of nitrification/ denitrification process.

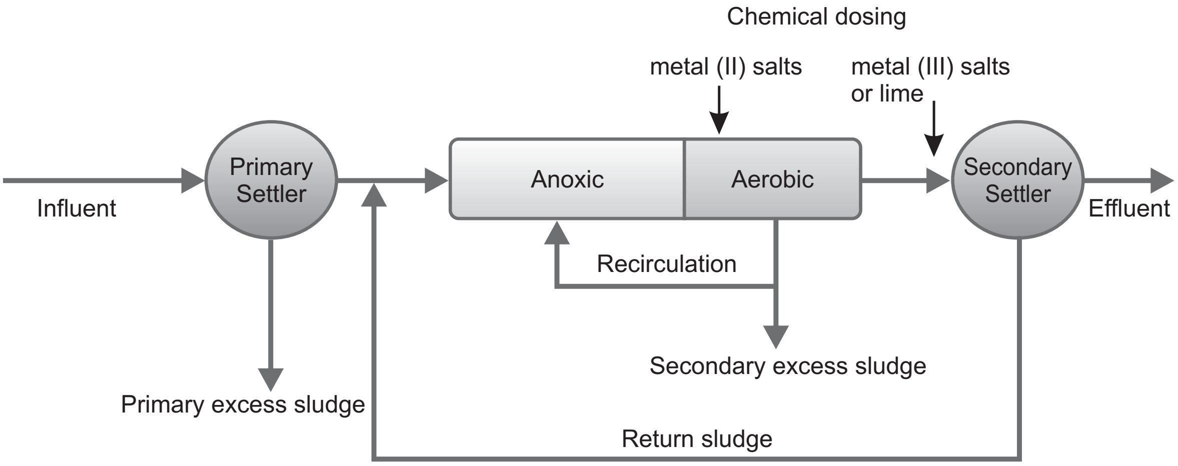


Figure 7.14 Simultaneous precipitation process.

(c) Post precipitation: Post precipitation is the process, where chemicals are added to the clarifier effluent. However, this process is associated with higher capital cost, and is not employed widely for the removal of phosphorus. Figure 7.15 illustrates typical post precipitation phosphorus removal configurations.

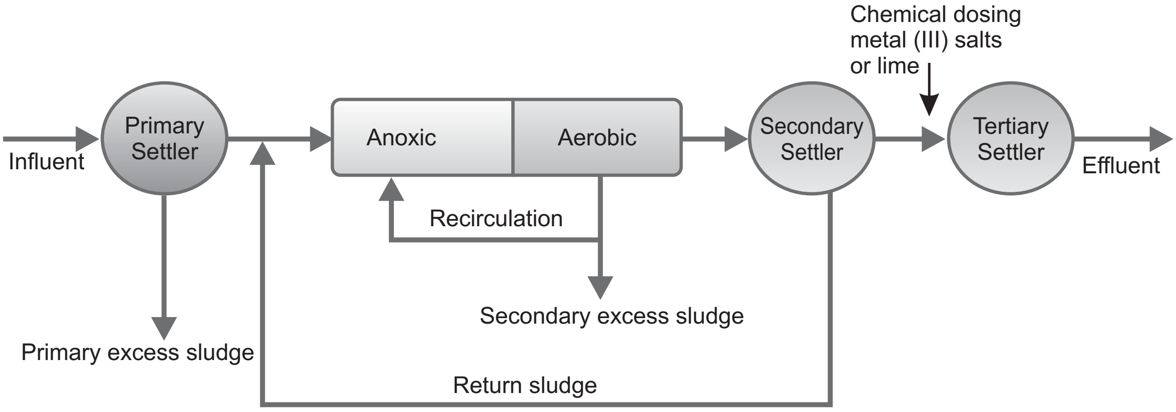


Figure 7.15 **Post precipitation process for phosphorus removal.**

(d) Sidestream precipitation: Phosphorus is concentrated into a low volume sidestream with higher phosphorus content. This sidestream is subjected to chemical removal of phosphorus. Sidestream process is often employed as a supplementary process to biological phosphorus removal. One of the common sidestream precipitation mechanism is BCFS (biological chemical phosphate nitrogen removal), as shown in Figure 7.16. Such process is operated at longer sludge age (45 days), to optimise nitrification. Lower sludge production limits the accumulation of polyphosphate in the sludge, and is removed chemically. An in-line "stripper" is provided by placing baffles at the end of the anaerobic reactor compartment, which produces a quiescent zone in which the sludge partly settles. The phosphate-rich supernatant is pumped from the activated sludge tank, and precipitated in the sludge thickener (Loosdrecht et al., 1997).

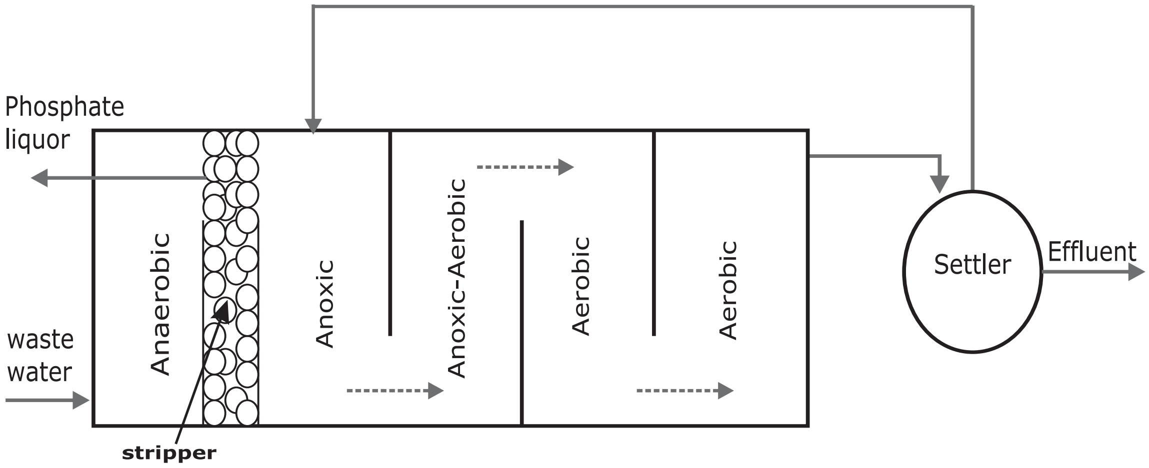


Figure 7.16 **BCFS process for phosphorus removal.**

7.8 Biological Phosphorus Removal

Theory. The chemical precipitation of phosphorus employs substantial amount of chemicals, which increases operating costs, and produces excess sludge. In addition, ions accumulation (from salts) can also restrict the reuse possibilities of the effluent. Due to these disadvantages, phosphorus removal in modern wastewater treatment plants is often achieved through biological routes, utilizing phosphate accumulating organisms (PAO) in sequential anaerobic and aerobic environments.

In anaerobic environments, PAO organisms absorb the volatile fatty acids in their cell in the form of internal polymers, such as polyhydroxy-butyrate (PHB). The energy required for this process is obtained (by PAO organisms) through splitting previously stored polyphosphate into orthophosphate (PO_4^{3-}); the produced orthophosphate is then released from the cell. The anaerobic reactor must be free of nitrate, as nitrate reduction depletes availability of biodegradable organic compounds, which is critical for the metabolism of PAO organisms. In the following anoxic and oxic reactors, the PHB is used by the PAO organisms as a source of energy, and bacterial growth. A portion of the energy released is employed, to store the phosphate (from the liquid phase), released in the previous anaerobic reactors.

Configurations of biological phosphorus removal. Biological phosphorus removal includes numerous configurations; the main difference of these systems is the inclusion and maintenance of the anaerobic reactors to prevent nitrate accumulation. The following sections provide a brief description of such systems.

(a) Phoredox (A/O) and three stage Phoredox (A^2/O) configuration: The Phoredox (A/O) process includes an anaerobic zone, at the head of the aeration reactor (Figure 7.17a). The RAS is pumped from the clarifier to the anaerobic zone. The process has

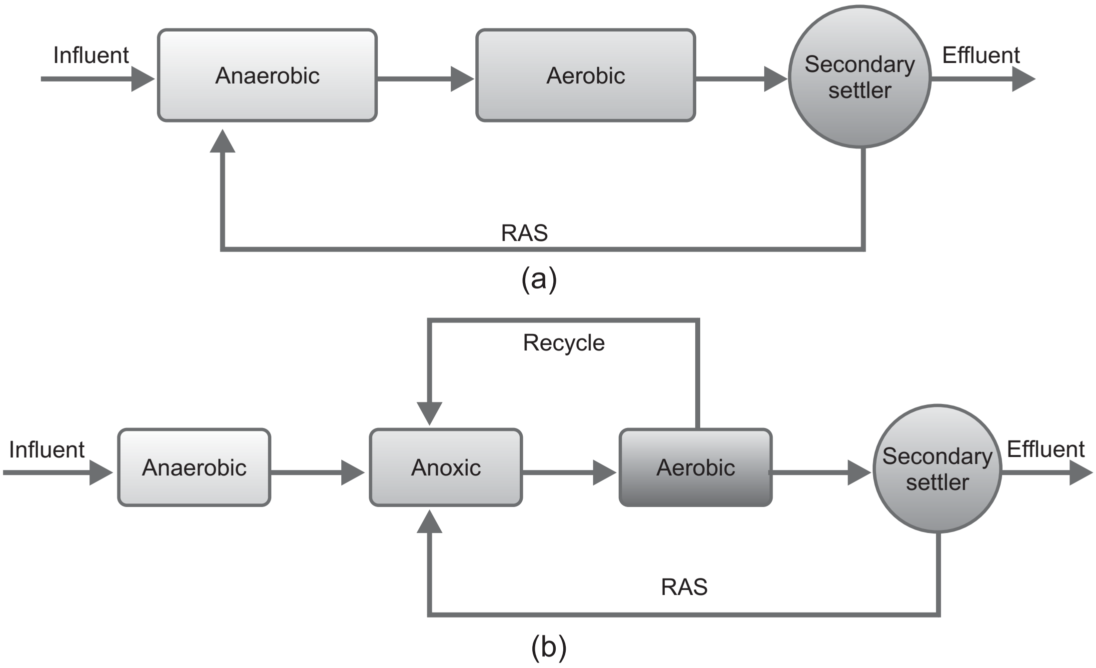


Figure 7.17 Biological phosphorus removal configurations: (a) Phoredox (A/O); and (b) three stage Phoredox (A^2/O) configurations.

lower sludge age, which hinders nitrification. However, this system is difficult to operate at temperatures above 25°C, where nitrification is unavoidable.

The three stage Phoredox (A^2/O) configuration includes an anoxic zone, between the anaerobic and aerobic reactors (Figure 7.17b), to achieve nitrate removal. The nitrate rich liquor is recycled from the aerobic reactor to the head of the anoxic zone, for promoting denitrification. A disadvantage of this process is the presence of nitrate in the RAS, which can hinder biological phosphorus removal.

(b) Modified Bardenpho process: Figure 7.18 illustrates configurations of modified Bardenpho process, for phosphorus removal. Such process consists of 5 stages; an anaerobic zone at the head of the process, followed by alternating anoxic and aerobic stages. The nitrate rich liquor is recycled from the first aerobic stage; the RAS is recycled from the settler to the first anaerobic stage for denitrification. Since the nitrate concentration in the RAS ranges from 1 - 3 mg/L, it does not interfere with the biological phosphorus removal mechanism.

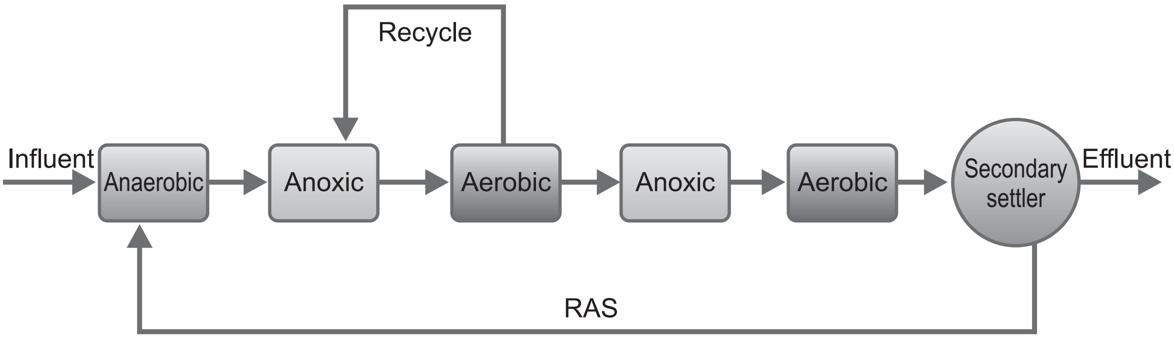


Figure 7.18 Modified Bardenpho process.

(c) University of Cape Town (UCT), and modified UCT installations: The UCT processes are modifications of three stage Phoredox (A^2/O) configurations. In a UCT configuration, RAS from the settler is transferred to the anoxic reactor. Such configuration allows denitrification of the liquor, which is then recycled to the anaerobic reactor for enhancing biological phosphorus removal (Figure 7.19a). In addition, the nitrified liquor from the aerobic reactor is recycled to the anoxic reactor to achieve denitrification, followed by recycling of the liquor to the anaerobic reactor.

Sometimes it is complicated to achieve the required denitrification rates in the anoxic zone, that receives RAS and internal recycle flows rich with nitrate. Such problems can be solved by the modified UCT process, which splits the anoxic zone into two stages. The nitrate rich liquor from the aerobic zone is recycled to previous anoxic stage. The nitrate rich RAS (from the settler) is also recycled to the first anoxic stage, for denitrification; the denitrified liquor is transferred from the anoxic stage to the anaerobic reactor (Figure 7.19b), to enhance biological P removal.

(d) Sequencing batch reactor (SBR): SBR reactors, which employ sequential fill and draw phases of wastewater (Chapter 6), can be used for the removal of nitrate and phosphorus, from nitrate rich wastewater. The SBR reactors, for simultaneous removal of nitrate and

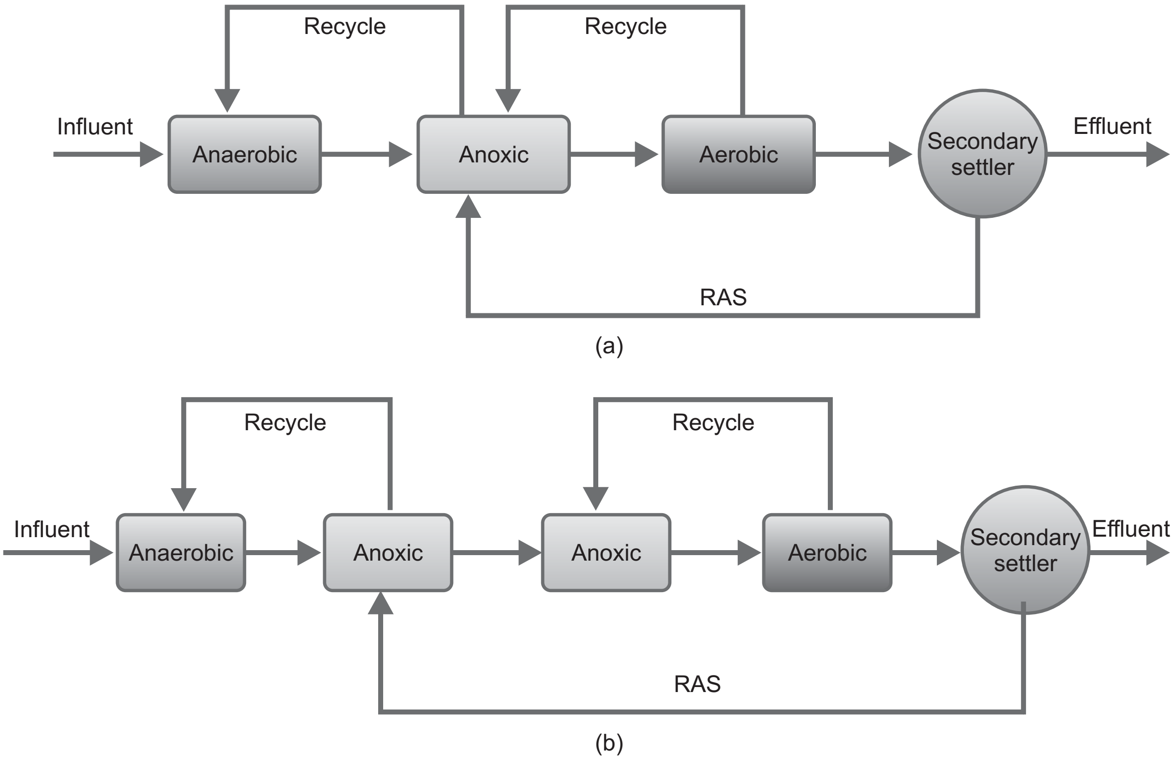


Figure 7.19 (a) University of Cape Town (UCT); and (b) modified UCT installations.

phosphorus (from wastewater), are subjected to sequential anoxic-anaerobic-aerobic phases. The anoxic phase promotes the removal of nitrates, followed by biological phosphorus removal in the subsequent anaerobic and aerobic phases. SBR reactors are usually operated without primary sedimentation, so they often have a favorable BOD_5: P ratio, to maintain effluent P concentration of less than 1.0 mg/L during the settling phase.

(e) Johannesburg (JHB) process. The JHB process includes transferring the RAS (from the settler) to the anoxic reactor. Such configuration allows the production of mixed liquor without nitrate, which is transferred to the anaerobic tank for triggering biological phosphorus removal.

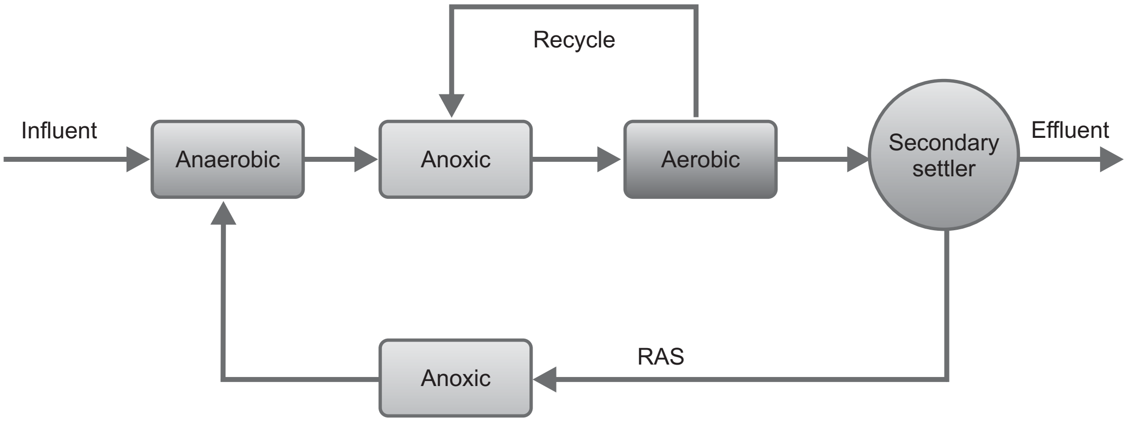


Figure 7.20 Johannesburg (JHB) process for the removal of phosphorus.

Table 7.7 illustrates the operational parameters of some of the biological phosphorus removal processes. Subsequently, the advantages and disadvantages of various biological phosphorus removal processes are summarized in Table 7.8 (Haandel and Lubbe, 2007):

Table 7.7 Flow rates and residence times of Phoredox, UCT and sequencing batch reactors, employed for the removal of phosphorus from wastewater[a].

	Phoredox	UCT	Sequencing batch
Flow rates			
Influent	0.96 l/h	0.96 l/h	4 l/6h
Sludge return line	0.25 l/h	0.25 l/h	-
Waste sludge line	0.125 l/h	0.125 l/h	200 ml/6h
Effluent	0.835 l/h	0.835 l/h	3.8 l/6h
Residence times			
Anaerobic phase	1 h 21 mins	1 h 18 mins	1 h 30 mins
Anoxic phase	1 h 36 mins	1 h 27 mins	
Aerobic phase	3 h 50 mins	3 h 50 mins	3 h 15 mins
Settler	5 h 12 mins	5 h 12 mins	+/- 30 mins

[a] *Baetens (2000).*

Table 7.8 Advantages and disadvantages of different biological phosphorus removal processes.

Configuration	Advantage	Disadvantage
Phoredox/A/O process	• Simple system • Residence time is shorter	• Nitrogen removal is not achieved • Not applicable for hot regions
A^2/O process	• Higher denitrification rate is achieved • Sludge age is shorter	• Denitrification rate is incomplete • Sludge bulking is observed
Modified Bardenpho process	• Excellent nitrogen removal is achieved	• Incomplete denitrification will affect phosphorus removal
UCT	• Recirculation of nitrate is inhibited	• Denitrification utilization capacity is inefficient
Modified UCT	• Absence of nitrate in the anaerobic reactor	• Inefficient denitrification capacity utilization
Johannesburg	• The denitrification reactor is efficiently utilized	• Incomplete denitrification

Simultaneous denitrification and phosphate uptake. Although it was believed previously that biological phosphorus removal process cannot assist denitrification process, later research studies demonstrated the denitrifying capacity of PAOs (Vlekke et al.1988; Kuba et al. 1993). As such, two different populations such as PAOs and DPAOs (denitrifying phosphate accumulating bacteria) can exist in biological phosphorus removal systems (Kern-Jespersen and Henze, 1993). The fraction of DPAOs is estimated to be in a range of 40-50% of the total PAO biomass (Kuba et al., 1994). PAOs can only utilize oxygen as a terminal electron acceptor; DPAOs can utilize both oxygen and nitrate as terminal electron acceptors. The DPAOs can exhibit simultaneous release and uptake of phosphorus, depending on the availability of external substrate, and degradation of the internally stored energy (Kuba et al. 1994; Chuang *et* al. 1996).

7.9 Optimization of Biological Phosphorus Removal

Presence of nitrate in the anaerobic reactors, and lack of biodegradable organics are two common problems, associated with performance deterioration of biological phosphorus removal reactors. Such problems can be mitigated through following ways, for enhancing biological phosphorus removal (Henze et al., 2002):

- Improvement of denitrification for reducing nitrate, in order to introduce almost nitrate free recycled sludge in the anaerobic reactor.
- External input of acetic acid for enhancing the availability of readily degradable organics.
- Increasing the retention time in aeration tanks, allowing hydrolysis/ fermentation process.

Box 7.3

Performance evaluation of Pagla treatment plant, Bangladesh (Saha et al., 2012).

A study by Saha et al., (2012) reported wastewater profile, and characteristics of Pagla treatment plant, Narayanganj, Bangladesh. Such treatment plant is a combination of conventional, and natural technologies. Influent pH, NH_4-N, NO_3-N, P, and solids (across the plant) were measured as 7.24, 57.69, 11.07, 4.06, and 124.72 mg/L respectively. The final effluent concentrations (from the treatment plant) of such parameters were recorded as 7.80, 5.24, 8.35, 1.34, and 24.48 mg/L respectively. Nitrification removal efficiencies were reported to be 91%; subsequently, lower NO_3-N removals were also recorded.

7.10 Design of Activated Sludge Processes for Simultaneous Organics Degradation and Nitrification

This section provides detailed design procedures for achieving nitrification, and organics removal in a single aeration tank. Such procedures are illustrated through design example 7.1.

Example 7.1. Design of a tertiary treatment process. Design an activated sludge system for combined nitrification and organic degradation, from the following data:

- Average flow rate = 4000 m^3/d
- Influent BOD = 200 mg/L
- Influent TKN = 50 mg/L
- Influent NH_4-N = 20 mg/L
- Half saturation NH_4-N constant, K_S = 0.3 mg/L
- DO in the aeration tank = 2.5 mg/L

- Half saturation DO constant, $K_{S,O2} = 0.5$ mg/L
- Wastewater temperature = 25°C
- Maximum growth rate of nitrifiers, $\mu_m = 0.38\ d^{-1}$
- Wastewater pH = 7.3
- MLSS = 3000 mg/L
- Safety factor, SF = 3
- Desired effluent BOD or TSS = 20 mg/L

Solution

Step 1: Determine the growth rate of nitrifiers

From Equation (7.26):

$$\mu = \mu_m \left(\frac{S_a}{K_s + S_a}\right)\left(\frac{DO}{DO + K_{S,O_2}}\right)[1 - 0.833(6.2 - \text{pH})]$$

$$= 0.38\left(\frac{20}{0.3 + 20}\right)\left(\frac{2.5}{2.5 + 0.5}\right)(1) = 0.3\ \text{d}^{-1}$$

Since the pH is greater than 7.2, the term $[1 - 0.833(6.2 - \text{pH})]$ is equal to the unity.

Step 2. Calculate maximum ammonia oxidation rate

From Equation (7.20): Ammonia oxidation rate, $r_N = \dfrac{\mu}{Y_N}$

From Table 7.2, $Y_N = 0.1$ mg VSS/mg NH_4-N

Maximum ammonia oxidation rate, $\hat{r}_N = \dfrac{\mu_m}{Y_N} = \dfrac{0.38\ \text{d}^{-1}}{0.1} = 3.8\ \text{d}^{-1}$

Step 3. Minimum and design cell residence time

Using Equation (6.29) (Chapter 6): $\dfrac{1}{\theta_c} = Yk' - k_d$

where $k' = \hat{r}_N = 3.8\ \text{d}^{-1}$ (maximum oxidation rate) and $k_d = 0.05$ (Table 7.2)

$$\frac{1}{\theta_c} = 0.1\,(3.8\ \text{d}^{-1}) - 0.05\ \text{d}^{-1} = 0.33\ \text{d}^{-1}$$

$$\text{Minimum}\ \theta_{c-\min} = 3\ \text{d}$$

The design cell residence time is calculated by incorporating safety factor; as such, the design cell residence time, $\theta_{c-design} = SF \times \theta_{c-\min} = 3 \times 3\ \text{d} = 9\ \text{d}$

Step 4. Calculate substrate utilization rate U for ammonia oxidation

From Equation (6.37) (Chapter 6): $\dfrac{1}{\theta_c} = YU - k_d$

$$U = \frac{1}{Y}\left(\frac{1}{\theta_c} + k_d\right) = \frac{1}{0.1}\left[\frac{1}{9\text{ d}} + 0.05\right] = 1.6\text{ d}^{-1}$$

where $U = r_N$ in Equation (7.20)

Step 5. Calculate steady state ammonia concentration of the effluent, S_a

From Equation (7.20):

$$r_N = \hat{r}_N \frac{S_a}{S_a + K_s}$$

$$1.6\text{ d}^{-1} = \frac{(3.8\text{ d}^{-1})\,S_a}{0.3\text{ mg/L} + S_a}$$

$$S_a = 0.22\text{ mg/L}$$

Step 6. BOD removal rate, U

From Equation (6.37) (Chapter 6): $\frac{1}{\theta_{c-design}} = YU - k_d$

From Table 6.4 (Chapter 6), heterotrophic yield coefficient, Y= 0.6 kg VSS/ kg BOD_5 and k_d= 0.06 d^{-1}

The design cell residence time ($\theta_{c\text{-design}}$) is 9 d.

The heterotrophic BOD removal rate, $U = \frac{1}{Y}\left(\frac{1}{\theta_{c-design}} + k_d\right) = 0.29$ kg BOD_5 removed/(kgMLVSS.d)

Step 7. Calculate F/M ratio

Assuming 85% BOD removal efficacy, the food to microorganism ratio can be computed as:

$$F/M = \frac{0.29}{0.85} = 0.34\text{ kg BOD}_5\text{ applied/kg MLVSS. d}$$

Step 8. Calculate hydraulic retention time for organic oxidation

From Equation (6.36) (Chapter 6): $U = \frac{Q\,(S_o - S)}{VX} = \frac{S_o - S}{\theta X}$

For organic degradation:

$$S_0 = 200\text{ mg/L};\ S = 20\text{ mg/L};\ U = 0.29\text{ d}^{-1};$$

$$X = MLVSS = 0.8\,MLSS = 0.8 \times 3000 = 2400\text{ mg/L}$$

$$\theta = \frac{(200 - 20)\text{ mg/L}}{(0.29\text{d}^{-1})\,(2400\text{ mg/L})} = 0.26\text{ d}$$

Step 9. Calculate hydraulic retention time for nitrification

$$S_0 = TKN = 50\ \text{mg/L};\ S_a = 0.22\ \text{mg/L};\ U = 1.6\ \text{d}^{-1}$$

$$X = 2400 \times 0.1 = 240\ \text{mg/L}\ \ (\text{assuming 10\% of VSS consists of nitrifiers})$$

$$\theta = \frac{(50 - 0.22)\ \text{mg/L}}{(1.6\ \text{d}^{-1})\,(240\ \text{mg/L})} = 0.12\ \text{d}$$

As HRT of organic oxidation is higher (step 8), it is the controlling HRT.

Step 10. Aeration tank volume

$$V = \theta Q = 0.26\ \text{d} \times 4000\ \text{m}^3/\text{d} = 1040\ \text{m}^3$$

Step 11. Required amount of oxygen

$$\text{Required } O_2 = Q\,(kS_0 + 4.57\ TKN)$$

$$= 4000\ \text{m}^3/\text{d}\,(1.2\,(200\ \text{mg/L}) + 4.57\,(50\ \text{mg/L})) = 1874\ \text{kg/d}$$

where k = BOD loading conversion factor for nitrification system, usually ranges between 1.1- 1.25.

Step 12. Sludge wasting

From Equation (6.24) (Chapter 6): $\theta_c = \dfrac{VX}{Q_{wa}X + Q_e X_e}$

Sludge to be wasted, $Q_{wa}X = \dfrac{VX}{\theta_c} - Q_e X_e$

$$= \frac{1040\ \text{m}^3 \times 2400\ \text{mg/L}}{9\ \text{d}} - (4000\ \text{m}^3/\text{d}) \times (20\ \text{mg/L} \times 0.8)$$

$$= 213\ \text{kg/d}$$

Step 13. Engineering diagram of the aeration tank

Figure 7.21 provides an engineered diagram of the proposed aeration tank, to be employed for simultaneous removal of organics and NH_4-N.

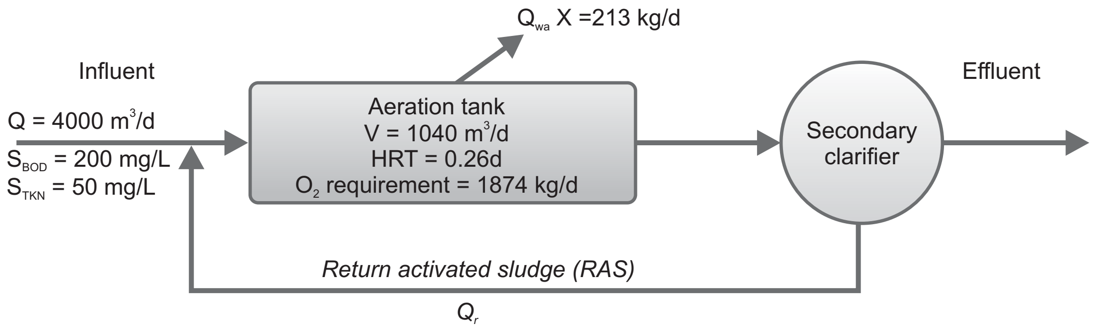


Figure 7.21 **Aeration tank diagram for achieving organics and NH_4-N removal.**

7.11 Metals Removal

Precipitation is often employed for the removal of heavy metals from wastewater. Heavy metals are generally precipitated as hydroxide through the addition of lime, or caustic (NaOH) to a pH of minimum solubility. The pH of minimum solubility varies with the types of metals to be removed. For example, the solubility of chromium and zinc are minimal at pH 7.5 and 10.2 respectively. When treating wastewater that contains metals, it is necessary to provide pretreatment of the effluents to remove substances, that will interfere with the precipitation of the metals. Cyanide and ammonia form complexes with many metals, thereby limiting removal mechanism. For many metals such as arsenic and cadmium, co-precipitation with iron or aluminium is highly effective for removal to low residual levels. In order to meet low effluent requirements, it may be necessary to provide filtration to remove flock, carried over from the precipitation process. Filtration should reduce effluent concentrations to 0.5 mg/L or less. For chromium wastes treatment, hexavalent chromium must first be reduced to the trivalent state Cr^{3+}, and then precipitated with lime. The reducing agents commonly used for chromium wastes are ferrous sulphate, sodium meta bisulphite, or sulphur dioxide.

Table 7.9 illustrates various treatment technologies, to facilitate metals removal from wastewater.

Table 7.9 Various available technologies for metal removal.

Metals	Removal technologies
Arsenic	Sulphide precipitation with filtration, carbon adsorption, ferric hydroxide co-precipitation.
Barium	Sulphate precipitation.
Cadmium	Hydroxide precipitation at pH 10-11, co-precipitation with ferric hydroxide.
Copper	Hydroxide precipitation, sulphide precipitation.
Mercury	Sulphide precipitation, alum co-precipitation, ferric hydroxide co-precipitation, ion exchange.
Nickel	Hydroxide precipitation at pH 10.
Zinc	Hydroxide precipitation at pH 11.

Questions

1. Write a short note on the factors that usually effect nitrification and denitrification process.
2. Why anoxic reactors are incorporated in activated sludge processes for the removal of nitrogen?
3. What is the main difference of pre D and post D systems employed for the removal of nitrogen?
4. Why Sharon process has certain advantages over traditional nitrification process?
5. What are the basic principles for obtaining higher effluent quality from Sharon-Anammox processes?
6. Write a short note on the theory of chemical phosphorus precipitation in wastewater treatment plants.
7. What are the necessary conditions for enhancing biological phosphorus removal?
8. Write short notes on: (a) modified Bardenpho process; and (b) modified UCT installations often employed to foster biological phosphorus removal.
9. Design an aeration tank for achieving nitrification, using the following dataset:
    - Average flow rate = 4000 m^3/d
    - Influent TKN = 80 mg/L
    - Influent NH_4-N = 50 mg/L
    - Half saturation NH_4-N constant, K_S = 0.3 mg/L
    - DO in the aeration tank = 1.5 mg/L
    - Half saturation DO constant, $K_{S,O2}$ = 0.5 mg/L
    - Wastewater temperature = 30°C
    - Maximum growth rate of nitrifiers, $\mu_m = 0.35 d^{-1}$
    - Wastewater pH = 7.1
    - MLSS= 4000 mg/L
    - Safety factor, SF = 2
    - Desired effluent TSS = 20 mg/L

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Chapter 8

Industrial Wastewater Treatment

Industries require raw materials, fluids, and energy to produce goods for consumers. These activities lead to the production of various waste forms, such as liquid wastes, gaseous emissions, energy heated fluids and solid wastes. Most of these produced wastes comprise of toxic substances. As such, these wastes must be treated prior to disposal, for ensuring environmental sustainability.

This chapter provides a brief description of different industrial effluent characteristics, and possible treatment options for safe disposal. Industrial wastewater production rates in and around Dhaka have also been elucidated in this chapter. In addition, detailed description of manufacturing processes of tannery and textile industries (two major sources of surface water pollution in Bangladesh), and possible treatment options of such effluents have also been included in this chapter. Finally, to assist the engineers in estimating preliminary operational and maintenance costs of industrial treatment plants, regression equations have been presented in this chapter.

It should be noted that, prior to the implementation of a particular treatment process into industrial sectors (for removing organic and inorganic pollutants), lab-scale and pilot-scale studies should be executed for evaluating the removal efficiencies, appropriated loading ratios across the selected process.

8.1 Changes in Industrial Management

During the post World War II era, the industries focused on product quantity management, resulting product generation without maintaining necessary qualities. In the 1960s, the Japanese and German industries introduced the concept 'product quality'. The concept allowed the industries to focus on producing quality products which eventually damaged the environment, due to the production of toxic, heavy metal rich wastewaters, and hazardous solid waste.

Owing to substantial environmental pollution, and increasing concerns by the governments and public, the industries slowly realized the necessity of controlling the quality and quantity of the produced wastes. This shift of awareness formulated 'total quality management' concept, considering the overall quality of all the elements associated in industrial production.

The shift of environmental management is eventually resulting stricter regulation on dischargeable parameter concentration. For example, in India the dischargeable limit of phenol (for petroleum industry) was 300 mg/L in 1968, which was drastically reduced to 1 mg/L in 1992. Subsequently, the number of parameters (of industrial effluents) to be monitored is also increasing day by day, as substantial elements of these effluents are toxic.

To meet the stringent discharge criteria, pollution control technologies, for example: waste exchange (between industries), and effluent treatment plants have been introduced. The concept waste exchange allows waste swap/ trade between two industries; under this criterion the generated waste by an industry is being employed by a different industry, as source of raw materials. However, such concept cannot widely be implemented between various industries, due to variation of raw materials. Considering these draw backs, effluent treatment plants are attractive options for the industries, in terms of achieving the stringent discharge criteria. However, it should be noted that the effluent treatment plants are often considered as 'dead investments' by the industries, as they demand higher operation and maintenance costs, and do not bring any profit.

8.2 Characteristics and Treatment of Industrial Wastewater

Wastewater properties. The characteristics of industrial wastewater can be classified into: (a) physical; (b) chemical; and (c) biological properties. The physical characteristics of wastewater comprises of solids, odor, temperature, color, and turbidity. The chemical properties of industrial wastewater include organic matter, nitrogen, phosphorus, phenols, sulfur, heavy metals, pH, and alkalinity. The effluent of some industries has certain pathogenic organisms. As such, biological information is also critically needed to assess the degree of wastewater treatment, prior to environmental discharge.

Table 8.1 indicates typical pollutant concentration of some industrial effluents. As observed in Table 8.1, the concentration of solids, organics is substantially higher in these effluents, that demand extensive treatment.

Table 8.1 Typical range of effluent concentration values for different industrial wastewaters .

Waste origin	pH	TSS mg/L	BOD mg/L	COD mg/L	TDS mg/L	Oil and grease mg/L
Dairy	4	12150	14000	21100	19000	320
Yeast	5.3	540	2100	3400	3500	9
Fruits & vegetable	5.5	2200	800	1400	1270	94
Textile	6.5	1800	840	1500	17000	155
Pulp & paper	8	1640	360	2300	1980	-
Beverage	9	760	620	1150	1290	-
Tannery	10	2600	2370	4950	8500	115
Fish canning	11	565	890	2350	8218	290

Wastewater treatment. Since the industrial effluents consist diverse range of pollutants, it is required to pass these wastewaters through a combination of preliminary, primary, secondary and tertiary processes (Chapters 4-7), to produce higher effluent quality. As the industrial inputs and outputs differ from one industry to another, the treatment methods also vary within the industries.

Table 8.2 illustrates the major characteristics of the effluents from different industries, along with required treatment technologies. Subsequently, Table 8.3 illustrates common advantages and disadvantages of the treatment processes, generally employed for industrial wastewater treatment.

Table 8.2 **Possible treatment options for industrial wastewater treatment.**

Industrial producing wastes	**Typical characteristics**	**Major treatment and disposal methods**
Textile	Alkalinity, color, temperature, COD and high suspended solids	Neutralization, precipitation, aeration, trickling filtration
Tannery	Hardness, salt, pH, sulfides, chromium, solids, COD and BOD	Sedimentation coupled with biological treatment
Laundry	Alkalinity, and organic solids	Screening, precipitation, flotation, and adsorption
Dairy products	Dissolved organic, protein, fat and lactose	Acidification, flotation, activated sludge, trickling filtration
Meat and poultry products	Organic matter, blood, proteins, and fats	Screening, settling and/or flotation, activated sludge, trickling filtration
Beet sugar	Higher dissolved and suspended organic matter	Coagulation aeration/ biological treatment
Pharmaceutical	Suspended and dissolved organic matter	Activated sludge
Yeast	Higher organics	Trickling filtration, anaerobic digestion
Coffee	High organics and solids	Screening, settling, and trickling filtration
Fish	Higher organics, oil-grease and odor	DAF system , activated sludge
Soft drink	Higher pH, solids and BOD	Screening, neutralization and biological treatment
Bakeries	High organics, grease, sugars, flour, and detergents	Biological treatment
Cane Sugar	Variable pH, higher BOD_5	Neutralization, biological treatment
Palm oil	High BOD_5, COD, solids, fats and lower pH	Neutralization, coagulation, flotation, and filtration
Pulp & Paper	Variable pH, color, higher solids	Settling, biological treatment, by-products recovery using flotation
Oil fields and refineries	High dissolved salts, BOD_5, odor, phenol	Recovery of salts; acidification burning of alkaline sludge
Rubber	Variable pH, suspended solid	Aeration chlorination, sulfonation, biological treatment
Petrochemical	High COD, dissolved solids, metals	Equalization-neutralization, chemical coagulation, settling/flotation, biological oxidation
Detergents	High in BOD_5 and saponified soaps	Flotation, skimming, precipitation with $CaCl_2$
Pesticides	High organic, benzenering structure, acids	Activated-carbon adsorption, alkaline chlorination

Industrial producing wastes	Typical characteristics	Major treatment and disposal methods
Coal	High-suspended solids mainly coal, low pH, high H_2SO_4 and $FeSO_4$	Settling flotation
Fertilizer	Low pH, high fluoride, phosphate, ammonia, nitrate	Neutralization, lime treatment, biological treatment
Cement	Lime, iron, solids	Neutralization followed by sedimentation, electrostatic precipitation
Poultry	Higher organics and solids	Activated sludge processes, oxidation ponds
Iron	Gases like CO, CO_2, SO_2. Fe particles and solids	Settling and reclamation of sand
Acid manufacturing	pH, highly acidic	Lime neutralization
Plywood	Higher organics, phenols	Settling ponds adsorption
Antibiotics	Higher organics, solids	Biological treatment, flocculation, sedimentation
Wood manufacturing	Acidic, high organics	Trickling filter

Table 8.3 Advantages and risks of different treatment processes for industrial wastewater treatment.

Treatment	Advantage	Risks
Screening	Remove large suspended solids	Blockage of screens; odor problems may also arise
DAF System	Remove oil and light particles	Higher consumption of chemicals
Settling basins	Shock loads are minimized	BOD overloading can cause failure of the systems; often requires larger area
Lamella separator	Higher separation efficiency; easy discharge of settled sludge	Difficult to clean the settled solids
Activated sludge	Lower land requirement	Extensive energy requirement; production of sludge
Trickling filter	Lower energy requirement and sludge production	Odor problems
Aerated ponds	Better BOD removal	Higher sludge production
Anaerobic digestor	Produces less sludge	Sensitive to temperature and shock load

Central effluent treatment plant. Higher construction and maintenance costs of treatment plants often hinder small-scaled industries, to employ these systems for wastewater treatment. Under these circumstances, central effluent treatment plant (CETP) can enhance treatment facilities (for small-scaled industries), as demonstrated in Figure 8.1. CETP process reduces the treatment cost to be borne by an individual member unit to a minimum, while ensuring maximum environmental protection. In addition, the process also reduces the burden of the supervisory authorities, responsible for evaluating water quality parameters.

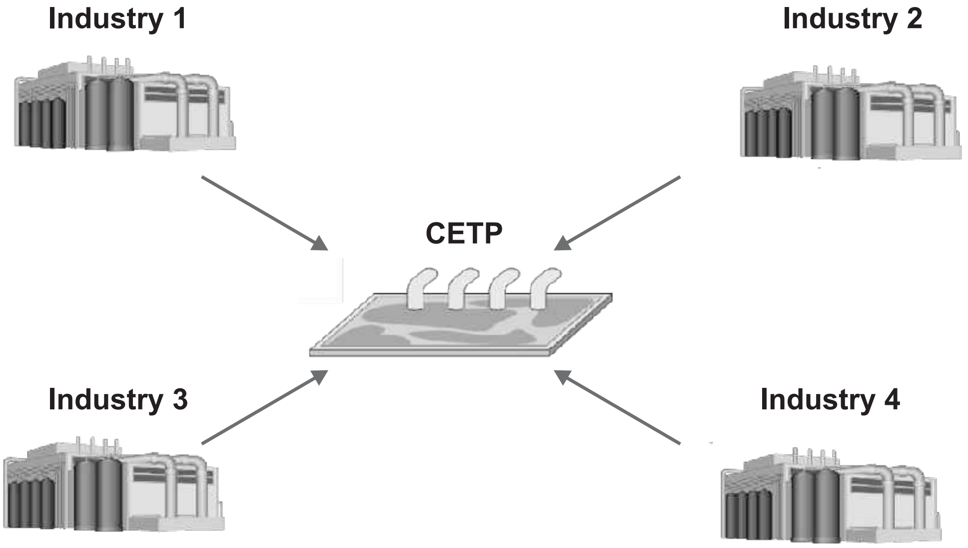


Figure 8.1 CETP processes for industrial wastewater treatment.

The advantages of CETP processes (for industrial effluent treatment) can be enlisted as:

- Capital and operational costs savings for small scale industries.
- Higher operational security.
- Enhanced treatment efficiencies at one central place.
- Efficient usage of land by the contributing industries.
- Addition of domestic waste can enhance nutrient availability, allowing a combined treatment of industrial and domestic sewage.
- Allows more organized disposal of effluent.

Despite of such unique advantages, CETP systems have some disadvantages, as illustrated below:

- Expansion of industries is not possible, if not considered during the design of CETP.
- Possibilities of revealing secrets to the competitors.

8.3 Overview of Industrial Pollution in Bangladesh

There are over 7000 industrial enterprises in Dhaka, located in nine industrial zones. Most of these industries discharge generated wastewater into surface waters, thereby disrupting aquatic ecosystem. Table 8.4 illustrates the production of domestic and industrial wastewaters in nine industrial clusters in and around Dhaka.

Table 8.5 illustrates water quality analyses at three industrial clusters: Konabari-Kasimpur (Gazipur), Fatullah and Enayatnagar (Narayangonj). At the investigated industrial clusters, organic and nutrients were found to exceed the discharge limits; dissolved oxygen did not also meet the required limits. It should be noted that wastewater sampling analyses of individual industries exhibited significant pollutant levels, when compared with the values of Table 8.5. Such discrepancy can be attributed to the possibilities of stormwater dilution (with raw wastewater), when discharged from the industries. Subsequently, Table 8.6 illustrates the main polluting industries within Dhaka with typical industrial pre-treatment processes.

Table 8.4 **Wastewater production in different industrial clusters in and around Dhaka(DWASA, 2013).**

Cluster	Total generated load (kg/d)	Total flow from outfall (m^3/d)	Total effluent load (kg/d)	Industrial effluent (m^3/d)	Industrial effluent load (kg/d)	Domestic effluent (m^3/d)	Domestic effluent load (kg/d)	Domestic load retained in situ (kg/d)
Tongi	12,555	35,158	7,159	21,708	3,797	13,450	3,362	5,396
Hazaribagh	66,664	87,184	55,773	49,489	46,349	37,695	9,424	10,891
Tejgaon	70,975	229,133	59,611	157,853	41,791	71,280	17,820	11,364
Tarabo	44,816	84,672	26,962	84,672	26,962	-	-	17,854
Narayanganj	74,957	494,946	43,025	456,225	33,344	38,721	9,681	31,932
Savar	8,291	9,114	1,757	7,738	1,413	1,376	344	6,534
Gazipur	19,965	192,845	18,922	192,845	18,922	-	-	1,043
DEPZ	48,113	314,755	31,042	314,755	31,042	-	-	17,071
Ghorashal	15,850	44,928	5,422	44,928	5,422	-	-	10,428
Total	**362,186**	**1,492,735**	**249,673**	**1,330,213**	**209,042**	**162,522**	**40,631**	**112,513**

Table 8.5 illustrates water quality analyses at three industrial clusters: Konabari-Kasimpur (Gazipur), Fatullah and Enayatnagar (Narayangonj). At the investigated industrial clusters, organic and nutrients were found to exceed the discharge limits; dissolved oxygen did not also meet the required limits. It should be noted that wastewater sampling analyses of individual industries exhibited significant pollutant levels, when compared with the values of Table 8.5. Such discrepancy can be attributed to the possibilities of stormwater dilution (with raw wastewater), when discharged from the industries. Subsequently, Table 8.6 illustrates the main polluting industries within Dhaka with typical industrial pre-treatment processes.

Table 8.5 **Water quality results for discharges from selected industrial clusters (DWASA, 2013).**

Parameter	Discharge value of Bangladesh	Konabari-Kashimpur		Fatullah		Enayatnagar	
		Range	Average	Range	Average	Range	Average
Temp (°C)	40	17-38	29	22-38	29	24-38	30
pH	6.0-9.0	7.3-12.5	9.0	7.1-13.9	9.7	7.2-12.2	9.1
TDS (mg/L)	2100	68-2550	944	447-2720	1227	362-8090	963
TSS (mg/L)	150	44-149	107	2-208	87	32-122	65
SO_4 (mg/L)	-	140-550	323	43-355	160	27-301	169
NO_3 (mg/L)	10	3.9-92	26	20-178	62	8-103	33
PO_4 (mg/L)	-	1.7-22	12.1	6.9-69.5	38	1.5-60	32
NH_3 (mg/L)	5	1-4.3	2.3	1.09-5.1	2.9	1-3	2.0
NH_4 (mg/L)	-	1.06-4.55	2.5	1.28-5.4	3.1	1.06-3.22	2.2
pH	0.1	0.0006-0.032	0.0067	0.0004-0.03	0.0104	0.0003-0.0043	0.0145
Cr (mg/L)	0.5	0.0018-0.022	0.0075	0.0017-0.0311	0.0076	0.003-0.0095	0.0064
Cd (mg/L)	0.05	0.00002-0.0071	0.0025	0.00002-0.009	0.0028	0.00002-0.008	0.0032 1.2
DO (mg/L)	4.5-8.0	0.05-8.3	1.35	0.02-5.2	0.72	0.03.6.62	80
BOD (mg/L)	50	65-129	97	78-185	122	12-147	308
COD (mg/L)	200	260-434	360	316-590	450	140-494	

Table 8.6 Major polluting industries within Dhaka (DWASA, 2013).

Industry	Wastewater quality						Major substances	Treatment method
	pH	BOD (mg/L)	SS (mg/L)	COD (mg/L)	Total N (mg/L)	Total P (mg/L)		
Textile dying	3-11	10-350	20-250	300	25	10	Colour	Ozone Chemical
Synthetic detergents	2-11	200-400	200-2500	150-2000	15-25	40-80	Phenol	Flotation Neutralisation
Pharmaceutical and chemicals manufacture	2-11	40-2000	70-600	-	80-100	10-20	Organic & Inorganic	Precipitation Neutralisation
Dry battery manufacturing	1-12	300-800	30-150	-	-	-	Cr Cu Cd Zn	Precipitation
Food processing, including poultry	6-8	300-2000	100-3500	200-1500	50-600	10-100	Organics Oils	Aerated
Tanning	7-12	500-2000	400-3000	100-2000	250-350	10-20	Cr Sulphides	Precipitation
Slaughter houses	6-8	800-2000	1200-1600	-	-	-	Organics	Aerated
Restaurants	6-8	10-900	20-800	-	-	-	Organics	Aerated
Electro-plating	1-12	-	30-150	10-200	-	-	Cr	Electrolysis

Tannery and textile industries are two major sources, responsible for aquatic environment degradation in and around Dhaka. As such, sections 8.4 and 8.5 provide a brief overview and possible treatment options of such wastewaters which can help the environmental engineers of Bangladesh, in identifying appropriate treatment technologies.

8.4 Processing and Treatment of Tannery Effluents

Manufacturing of leather produces substantial pollutants, such as: solids, organics, nitrogen, phosphorus, chromium (Cr), sulphides and alkalinity in the effluent. As such, uncontrolled release of tannery effluents (into natural water bodies) increases health risks for human beings, and fosters environmental pollution. Since the range of pollutants (in tannery effluent) is diverse, it is essential to pass the raw wastewater through a series of treatment technologies, to achieve higher effluent quality.

The removal of pollutants from tannery wastewater is heavily dependent on comprehensive understanding of the sequential tanning processes, along with the types of generated pollutants (through each step). This section provides a brief description of the tanning processes, and the by-products generated in each step, followed by possible efficient treatment approaches.

Tanning flow diagram. The production processes in a tannery industry can be divided into following categories:

- hide, skin storage and beamhouse operations
- tanyard operations
- post-tanning operations
- finishing operations

Beamhouse operations: This process consists of soaking, fleshing, trimming, deliming, bating, pickling, and degreasing of hides. In soaking process, dirt, manure, and blood are removed from raw hides. The extraneous tissue is removed in fleshing process, whereas unhairing is done by chemical dissolution of the hair. These processes are followed by deliming and bating, where acid ammonium salts are applied for the neutralization of the alkaline hides, allowing removal of hair roots and pigments. In pickling process acids and salts are added, allowing the entrance of chromium tannins into the hide; the addition of salts prevents hide swelling. Degreasing, the last stage of beamhouse operation is performed by organic solvents addition, or surfactants.

Tanyard operations: Such operations are performed through chrome tanning, for enhancing the hide resistant to bacteria and temperature. The chromium-tanned hide generally contains about 2-3% dry weight of Cr (III).

Post-Tanning: During this process, the desirable properties of more than one tanning agent are combined, and treated with dye and fat to obtain the proper filling, and colour. Splitting and shaving is done to obtain the desired thickness of the hide.

Finishing: In this process, the hide is treated with an organic solvent or water based dye and varnish. The finished end product often contains 66 -85% dry matter by weight.

Figure 8.2 indicates typical process diagram of tanning process, along with process input and output. Subsequently, Table 8.7 illustrates the chemicals required in different stages of the tanning process, whereas, Table 8.8 shows general flow rate, and production of pollutants during each stage.

Table 8.7 Chemical inputs for tanning process (Visvanathan, 2005).

Process Chemicals	Tones/annum
Sodium chloride	622
Hydrated lime	1123
Sodium sulphide	445
Sulphuric acid	160
Soda ash	74
Bate	65
Calcium formate	40
Lactic acid	35
Sodium formate	26
Bactericide	19
Ammonium chloride	9
Tanolin	760
Syntans A and B	424
Dyes	77
D-1 oil	17

Process Chemicals	Tones/annum
Other oils	295
Flour	45
Titanium dioxide	30
Methyl cellulose	9
Semi-sol glue	17

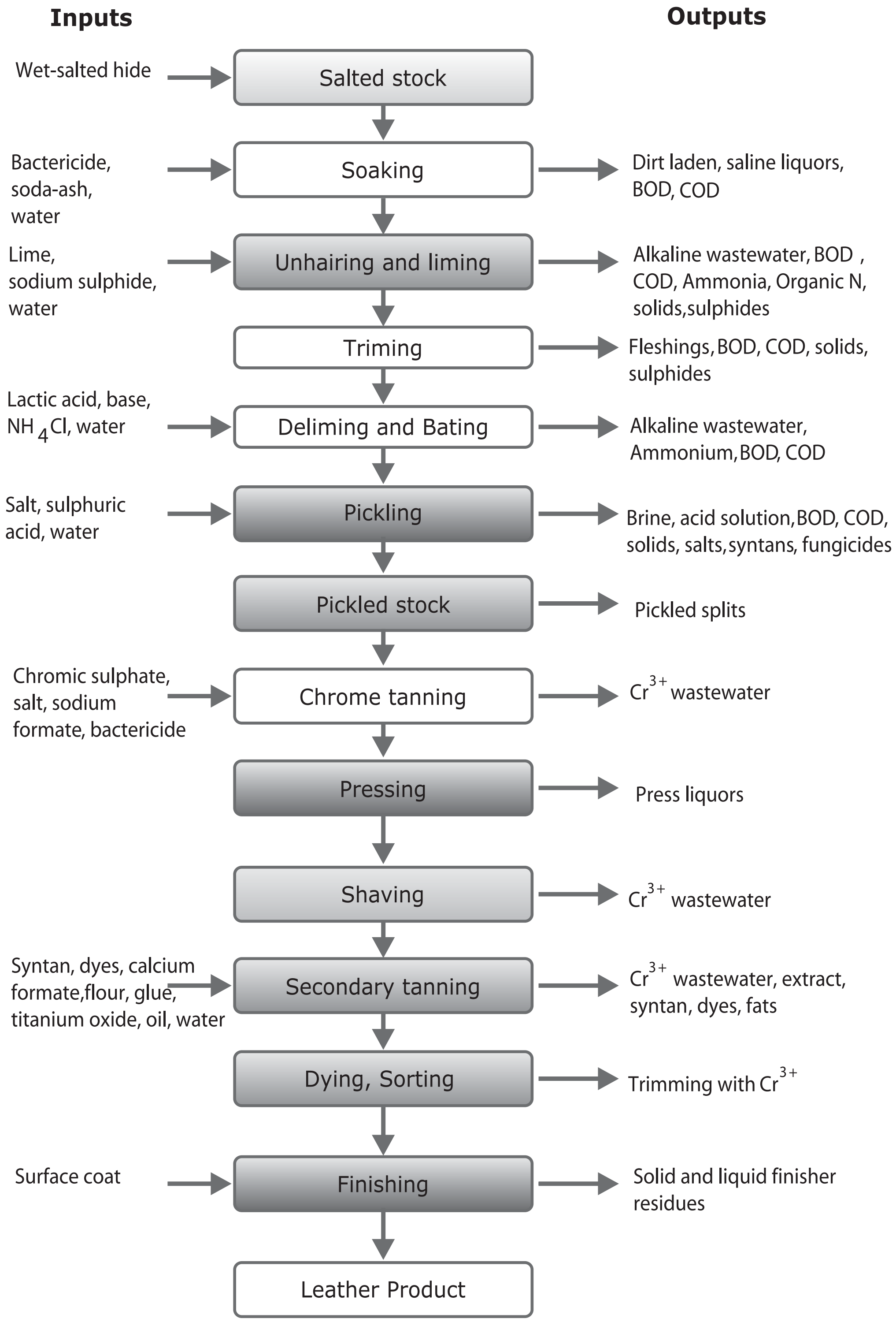


Figure 8.2 Operational diagram of tanning process.

Table 8.8 Pollutant load in tanning process (Visvanathan, 2005).

Operation	Flow	BOD		Solids	
	m^3/d	mg/L	kg/d	mg/L	kg/d
Soaking	276	2200	607	4400	1215
Unhairing	103	15500	1597	22100	2276
Reliming	103	650	67	1650	170
Delime	66	6000	396	2100	139
Pickling	37	2900	108	5200	192
Chrome tan	33	6500	215	1100	36
Secondary tanning, dyeing					
-1st dump	19	2000	38	600	11
-2nd dump	19	2200	42	850	16

Treatment of tannery effluent in Bangladesh. Substantial tannery wastewater generation, and discharge into local water bodies are causing severe environmental degradation in Bangladesh. Approximately, 18000 L/d of liquid wastes are produced, from 300 small and medium scaled tannery industries; most of such produced liquid wastes are discharged directly in river Buriganga, causing death of aquatic organisms (SEHD, 2002; Rusal et al., 2006; Islam et al., 2011). Typical influent BOD_5 and COD concentrations of untreated tannery effluent in Bangladesh are substantially higher, ranging within 2439-14675 mg/L BOD_5, and 3400-24400 mg/L COD (SEHD, 2002). Figure 8.3 depicts typical pollution of local water channels in Bangladesh, due to uncontrolled tannery wastewater discharge.

Figure 8.3 Tannery wastewater discharge into open water channels in Bangladesh.

Despite severe environmental impact of tannery wastewater (to local aquatic bodies), limited studies have been reported to date, on the efficient treatment of tannery wastewater, prior to disposal (in Bangladesh). These studies were conducted through combinations of physico-chemical, and biological process, as indicated in Boxes 8.1, and 8.2.

Box 8.1

Low cost treatment technologies to polish tannery wastewater treatment in Bangladesh (Begum and Ahmmed, 2010).

A low cost tannery effluent treatment technology had been developed by department of chemical engineering, BUET, Bangladesh. This system comprises of screening, aeration, coagulation, sedimentation, drying bed, and incineration, as illustrated in Figure 8.4.

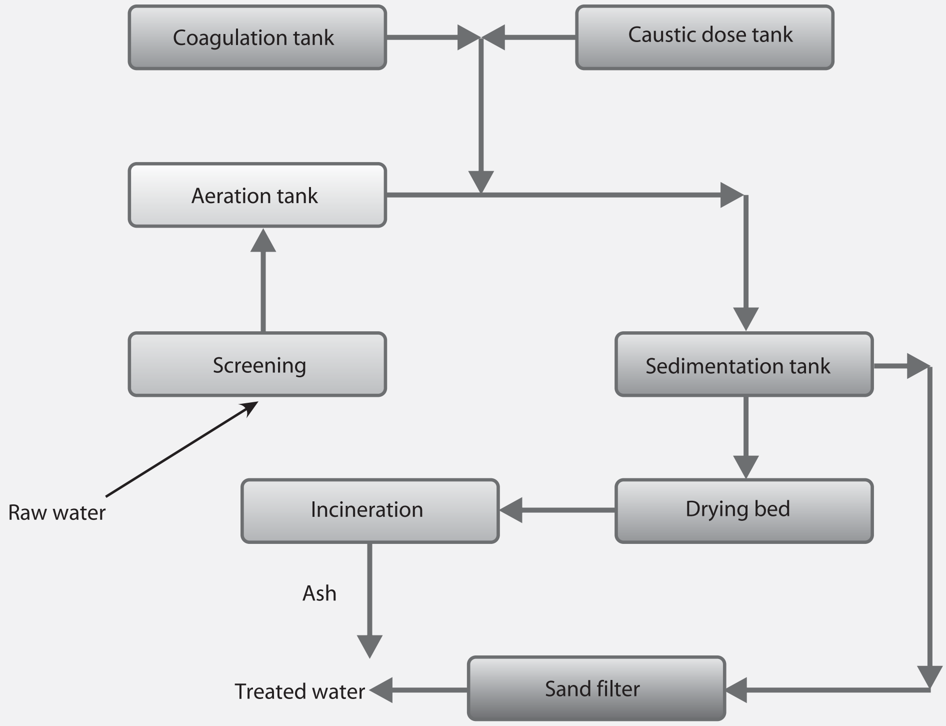


Figure 8.4 Low cost tannery effluent treatment technology, developed by department of chemical engineering, BUET.

The developed technology illustrated promising performances. Influent pH ranged between 3.45-3.96, whereas, effluent pH increased up to 6.73-6.89. COD values dropped to 18-118 mg/L in the effluent (influent concentration ranged between 2110-3335 mg/L). Total dissolved solids removal efficiencies were also recorded as 98%. In general, the system produced effluent quality, that met wastewater discharge criteria of Bangladesh, as indicated in Table 8.9.

Table 8.9 Wastewater discharge criteria for tanneries in Bangladesh (ECR, 1997).

pH	Suspended solids mg/L	Dissolved solids mg/L	Sulfide mg/L	Total chromium (Cr) mg/L	Oil and grease mg/L	BOD_5
6-9	150	2100	1	2	10	100

Box 8.2

Tannery wastewater treatment employing physico-chemical process in Bangladesh (Islam et al., 2011).

A study by Islam et al. (2011) employed coagulants, such as: alum, lime, and ferric chloride, to provide treatment of raw tannery wastewater in Bangladesh. The results of the study have been illustrated in Table 8.10.

Table 8.10 Pollutant removal profile employing coagulation process.

	Unit	Raw tannery	Treated effluents with coagulants
pH		10.43	5.73-7.3
Solids	mg/L	12333.33	3833.33-10100
Cl-	mg/L	483.33	383.33-426.67
BOD_5	mg/L	4760	420-2460
COD	mg/L	6650	1150-2632

According to Table 8.10, despite physico-chemical process reduced the influent pollutant concentrations, it could not meet wastewater discharge criteria in Bangladesh (Table 8.9). As such, employing biological treatment process could fulfill the discharge criteria of tannery wastewater in Bangladesh (Box 8.1).

As observed in Box 8.2, tannery wastewater treatment employing physico-chemical process only may not provide a sustainable solution, for tannery wastewater treatment in Bangladesh. Since a wider range of pollutants i.e. organics, nitrogen, phosphorus, solids, alkalinity, chromium, sulphides, salinity, and salts are produced during different stages of the tanning process (Figure 8.2), a combination of preliminary, primary, secondary and tertiary processes (Chapters 4-7), can produce acceptable tannery effluent quality, for safe environmental disposal.

Figure 8.5 indicates a possible approach, for the treatment of tannery effluent to discharge criteria. As observed in Figure 8.5, the treatment of wastewater through the conventional

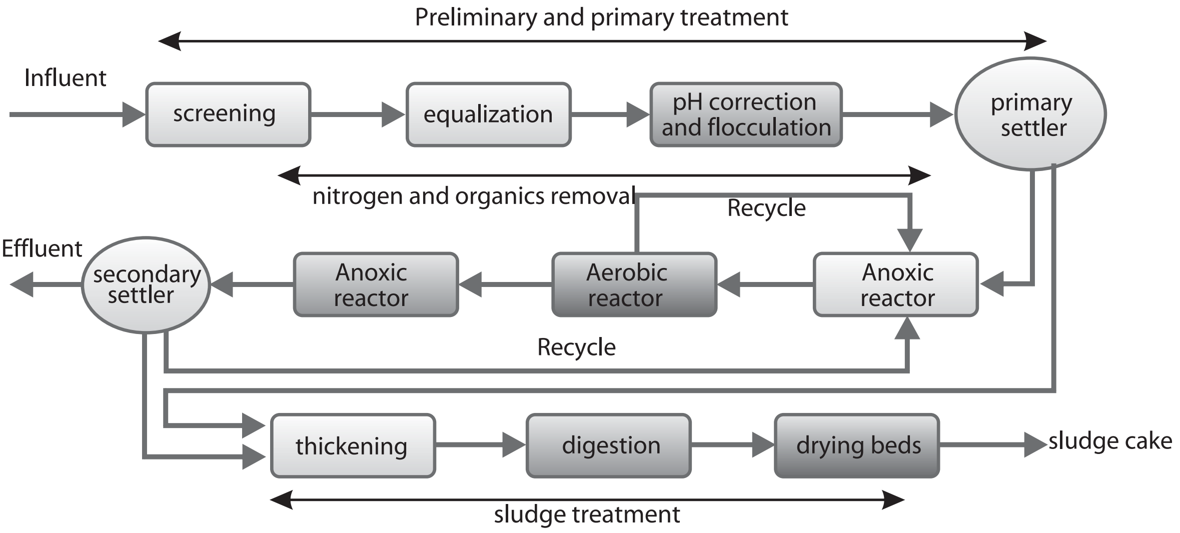


Figure 8.5 Treatment of tannery effluent employing conventional treatment process.

process is extremely complicated and demands higher operation and maintenance costs. Table 8.11 indicates the pollutant removal efficacy (in percentages), and typical effluent quality from conventional treatment processes, when employed for tannery wastewater treatment.

Table 8.11 Typical pollutant removal efficacy and effluent concentration employing different conventional treatment processes[a].

	COD		BOD		Solids		Chrome		S^{2-}		Total kjeldahl nitrogen (TKN)	
	%	mg/L	%	mg/L	%	mg/L	%	mg/L	%	mg/L	%	mg/L
Pretreatment												
Grease removal (dissolved air flotation)	20-40	-	-	-	-	-	-	-	-	-	-	-
Sulphide oxidation (liming and rinsing liquors)	10	-	-	-	-	-	-	-	-	10	-	-
Chromium precipitation	-	-	-	-	-	-	-	10	-	-	-	-
Primary treatment												
Mixing + sedimentation	25-35	-	25-35	-	50-70	-	-	20-30	-	-	25-35	-
Mixing + chemical treatment + sedimentation	50-65	-	50-65	-	80-90	-	-	2-5	-	2-10	40-50	-
Mixing + chemical treatment + flotation	55-75	-	55-75	-	80-95	-	-	2-5	-	2-5	40-50	-
Biological treatment												
Primary or chemical + extended aeration	85-95	200-400	90-97	20-60	90-98	20-50	-	<1	-	<1	50	150
Primary or chemical + extended aeration with nitrification and denitrification	85-95	200-400	90-97	20-60	90-98	20-50	-	<1	-	<1	80-90	30-60
Primary or chemical + Aerated facultative lagoons	80-90	300-500	85-95	60-100	85-90	80-120	-	<1	-	<1	50	80
Anaerobic treatment	65-75	500-700	60-70	150-200	50-80	100-200	-	<2	-	-	20-30	-

[a] *European Commission, 2001.*

Apart from conventional treatment technologies, natural treatment processes such as constructed wetlands can also be an attractive option for the treatment of tannery wastewater. Calheiros et al., (2009) employed two parallel series of horizontal subsurface flow (HSSF) constructed wetlands for tannery wastewater treatment. Each series consisted of two HSSF wetland systems. The HSSF wetlands of one series were planted with *Phragmites australis*, whereas the systems of the other series had *Typha latifolia*. The removal efficacy of BOD_5 and COD in the systems was recorded up to 88% and 92% respectively. The systems also showed better NH_4-N, NO_3-N, solids, Cr, and sulphate (SO_4^{2-}) removal performances. A brief description on constructed wetland systems is available in Chapter 10 of this book.

Box 8.3

Treatment of tannery wastewater in Bangladesh employing wetland systems (Saeed et al., 2012).

A constructed wetland system has been employed by Saeed et al., (2012) to treat tannery wastewater in Bangladesh which is the first of such application in this country. The wetland arrangements included a vertical subsurface flow (VSSF) system, followed by a HSSF and a VSSF system established in, Ahsanullah University of Science and Technology campus, Dhaka, Bangladesh(Figure 8.6). The arrangement of such hybrid systems has been illustrated in Figure 8.6. Wastewater was manually dosed into the first VSSF system; wastewater flowed vertically and was transferred to the following HSSF system under gravity action, where the flow was horizontal. The effluent of HSSF was transferred to the last stage VSSF system under gravity, producing final effluent. The effluent quality of each wetland unit was monitored (for a period of ten weeks) in the environmental engineering laboratory of the department of civil engineering, Ahsanullah University of Science and Technology. Overall, the system showed excellent pollutant removal performances. Influent COD and BOD (to the system) were 11500 mg/L and 4287 mg/L respectively, whereas effluent concentrations were 202 mg/L and 70 mg/L respectively. The system also achieved 86% NH_4-N removal efficiencies, along with 87% PO_4-P removal. In general, these results demonstrate the possibilities of the implementation of wetland systems in Bangladesh to provide necessary treatment of tannery wastewater prior to disposal.

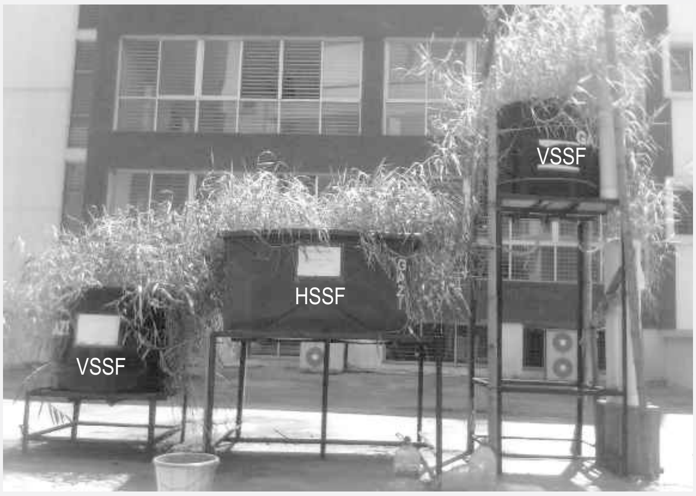


Figure 8.6 Hybrid wetland systems for tannery wastewater treatment in Bangladesh.

8.5 Textile Effluent Treatment

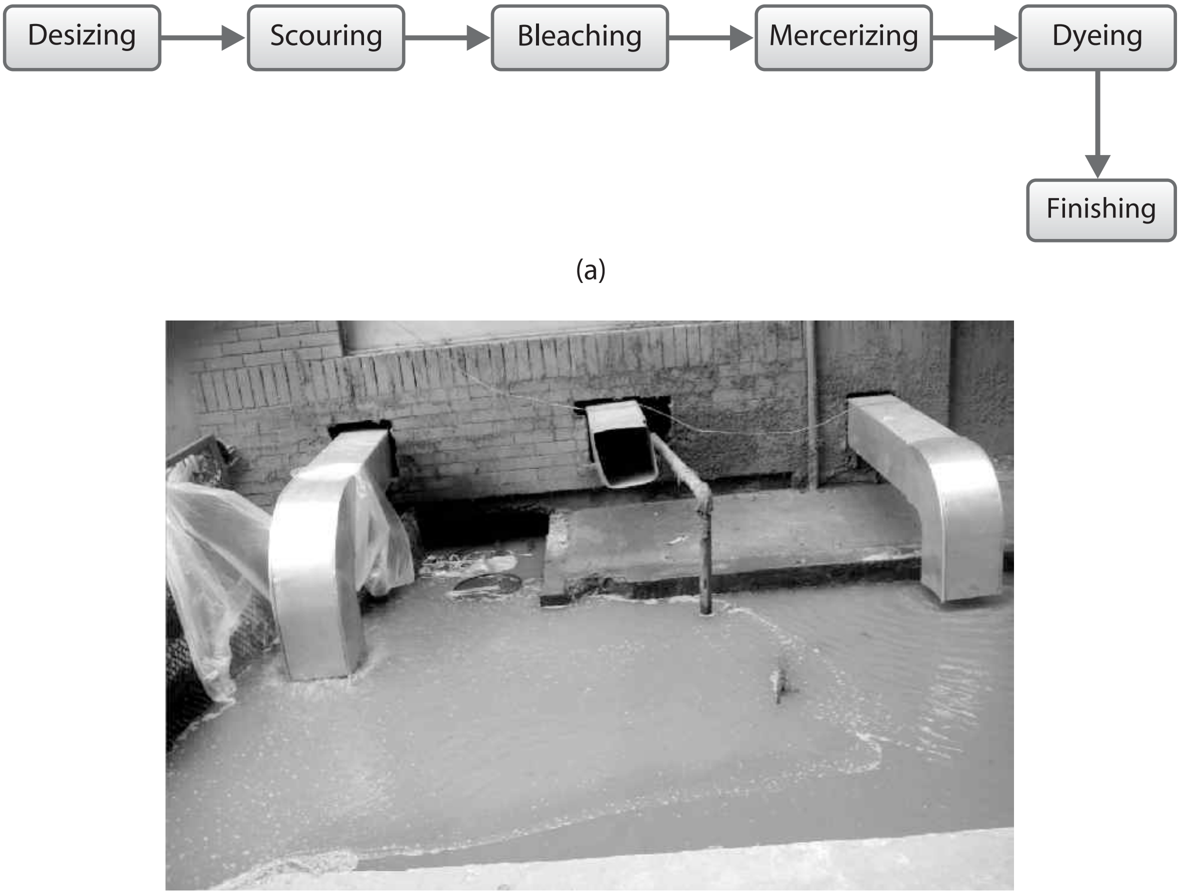
Textile industries process textile-related products for example: fiber, yarn, fabric, and dye; such processing is associated with the utilization of complex chemicals, which can create adverse impact to the receiving water channels when present in the effluents. The following section gives a brief overview of the textile manufacturing processes.

Unit operations. The different manufacturing stages, associated with textile industries can be illustrated as: desizing, scouring, bleaching, mercerizing, dyeing and finishing.

- Desizing. The presence of starch can hinder dye penetration into the fiber; as such, starch removal is essential, prior to the application of dyeing materials. In desizing process, starch is removed or transformed into simple products either by hydrolysis, or by oxidation, resulting in the production of substantial BOD in the generated wastewater.
- Scouring. Cotton, a natural fiber, contains 2-3% different impurities like waxes, fatty acids, proteins, pectines, minerals etc. Through scouring process, these natural impurities are removed from cotton fibers. The process is performed under alkaline conditions, subjected to a temperature >100°C.
- Bleaching. Since natural color renders a creamy appearance to the fabric, bleaching is performed for destroying natural coloring agents. Hypochlorite is the most popular agent to facilitate bleaching process, which also promotes the formation of chlorinated organic by-products. However, hypochlorite is currently being replaced by peracetic acid. The advantage of peracetic acid is its decomposition to oxygen and acetic acid, which is completely biodegradable.
- Mercerizing. This process includes the treatment of cotton fabric with NaOH, to allow the fiber for achieving rod-like structure. Mercerizing is performed for 40-45 seconds at 52-56°C.
- Dyeing. It is a process of imparting color to the fabric, with chemical pigments. In the dyeing process, water is used to transfer dyes. The quantity of water required is dependent on fiber types. For example, cotton requires large amount of water for processing. Once the dyeing operation is over, treatment baths are drained, resulting the production of salt and organic substances.
- Finishing. This process confirms the final chemical and mechanical properties to the fabric, according to the specific requirements. Several finishing methods are often employed to improve the drape, feel, antistatic, antisoiling, anti shrinking, water repelling, and flame retardancy of the fabric.

Figure 8.7 shows a typical flow diagram of textile dyeing process, and a pictorial plate of dyeing influent produced from these processes.

Treatment of textile pollutants. Table 8.12 indicates typical pollutant ranges in the effluents of cotton processing mills. Subsequently, Table 8.13 illustrates typical pollutant ranges in the effluent, generated from textile industries in Bangladesh (ECR, 1997).



(a)

(b)

Figure 8.7 **(a) Typical dyeing processing diagram; and (b) raw dyeing influent from a local factory in Bangladesh.**

Most of the dyes, employed during the manufacturing process of textile industries contain organic compounds with functional groups, for example: carboxylic (–COOH), amine (–NH_2), and azo (–N=N–) groups. These dyes often resist aerobic degradation (Ong et al., 2011); however, they can be converted to aromatic amines in anaerobic conditions, followed by further degradation in aerobic environments (Brown and Hamburger, 1987; Chung and Stevens, 1993). As such, a combination of anaerobic-aerobic processes can be effective for the removal of dyes from textile effluents as illustrated in Figure 8.8. Subsequently, Figure 8.9 shows a combination of anaerobic digester followed by aerobic tanks, employed to treat dyeing wastewater in Bangladesh.

Table 8.12 **Typical pollutant ranges in effluents of cotton mills. (Babu et al., 2007)**

Characteristics	Unit	Values
pH		9.8-11.8
Total alkalinity	mg/L as $CaCO_3$	17-22
BOD	mg/L	760-900
COD	mg/L	1400-1700
Solids	mg/L	6000-7000
Total Chromium	mg/L	10-13

Table 8.13 Typical pollutant ranges from textile effluents in Bangladesh.

Characteristics	Unit	Values
pH		6.5-9
Oil and grease	mg/L	10
BOD_5	mg/L	150
Suspended solids	mg/L	100
Total dissolved solids	mg/L	2100

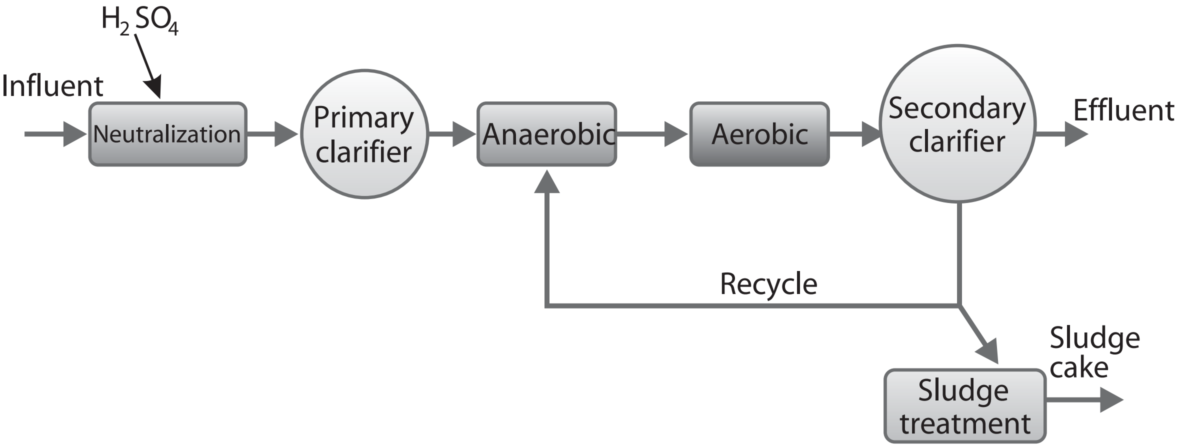


Figure 8.8 Textile wastewater treatment through anaerobic-aerobic process.

Figure 8.9 Combination of anaerobic digester-aerobic treatments for dyeing effluent treatment in Bangladesh.

The treatment of textile effluents through traditional process often demands excessive chemical requirements, thereby increasing the operational costs. In addition, lack of expertise often hinders smooth operation of these systems, to provide textile effluent treatment in Bangladesh resulting in system failure. Considering these drawbacks, natural treatment systems such as constructed wetlands can be effective (to provide treatment of textile effluent). A few previous research studies (Bulc and Ojstrˇsek, 2008; Ong et al., 2011) employed a combination of VSSF and HSSF wetlands, and upflow VSSF systems to provide treatment of textile effluents. These studies recorded higher removal efficiencies; removal percentages of COD, BOD, total nitrogen, sulfates, and dyes were reported to be 84%, 67%, 52%, 88% and 94-98% respectively. As such, wetland systems are promising technologies, to generate higher quality textile effluents (in Bangladesh) at lower operational costs.

Figure 8.10 illustrates a combination of HSSF and VSSF wetlands to provide alternative anaerobic-aerobic conditions, necessary for effective treatment of textile wastewater.

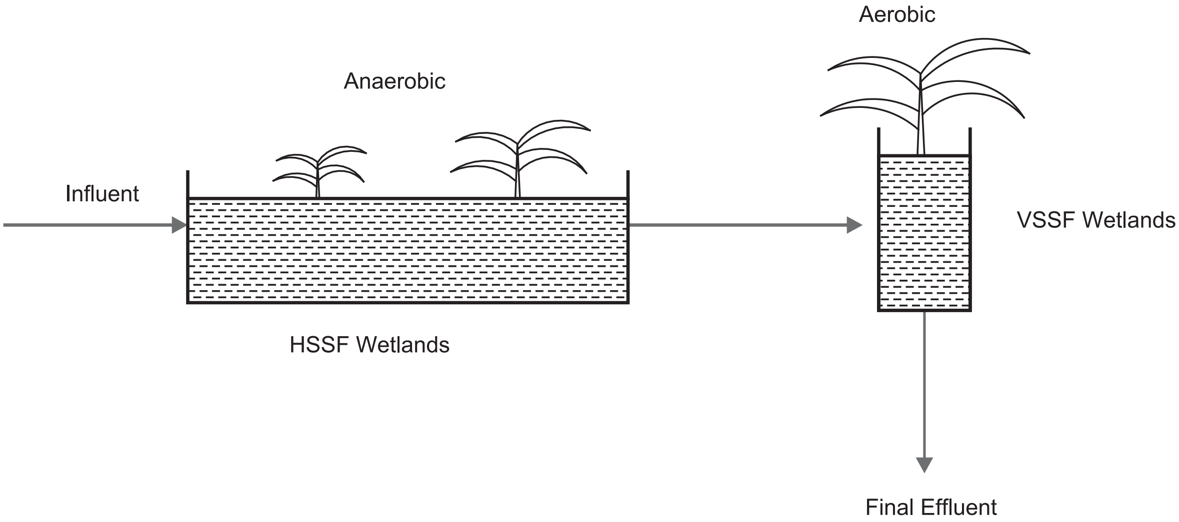


Figure 8.10 **A combination of HSSF and VSSF systems, to provide treatment of textile wastewater.**

Box 8.4

Textile wastewater treatment employing wetland systems in Bangladesh (Saeed and Sun, 2013).

Two parallel lab-scale hybrid wetland systems were employed by Saeed and Sun (2013), to provide treatment of textile wastewater in Bangladesh. These lab-scale systems were built on campus (outdoors) at Ahsanullah University of Science and Technology in Dhaka, Bangladesh. Each system consisted of two treatment stages: a VSSF wetland, followed by a HSSF wetland, as shown in Figure 8.11. Locally available media such as: organic sugarcane bagasse and sylhet sand were used as the main media in the lab-scale wetlands.

The systems were operated under high hydraulic loading (HL) ranges (566-5660 mm/d). The influent NH_4-N, NO_2-N, NO_3-N, BOD, and COD concentrations were 158, 28, 74, 2705, and 12625 mg/L respectively. Overall NH_4-N, NO_2-N, NO_3-N, BOD and COD removal efficiencies were 70, 86, 76, 97, and 89% respectively. The systems were also efficient in color removal. Effluent pH, BOD_5 and suspended solids concentration fulfilled the discharge guidelines for textile effluents in Bangladesh (Table 8.13). These findings illustrate the possibilities of employing wetland systems in Bangladesh to treat textile wastewater (prior to disposal).

Figure 8.11 Arrangement of lab-scale wetland systems for textile wastewater treatment.

8.6 Operation and Maintenance Costs for Wastewater Treatment Plants

Operation and maintenance costs are two important factors that must be considered prior to the planning of treatment plants. For industrial wastewater treatment these are the most decisive factors, as the industries have to bear such costs. As such, the engineers must estimate a rough budget for a proposed industrial treatment plant, to identify whether it meets the monetary criteria of the respective industries. To assist the engineers in estimating such operation, maintenance costs and land requirements, this section provides some regression equations in Tables 8.14-8.17 for different treatment options. These studies were carried out in Thailand by Singhirunnusorn and Stenstrom (2010). It should be noted that aerated lagoons and waste stabilization ponds are natural treatment systems, that have been discussed in details in Chapter 10 of this book.

Table 8.14 Regression equations for estimating preliminary costs.

System	Capital cost		O&M cost		Land required	
	N	Equation	N	Equation	N	Equation
Activated Sludge	6	$C_c=0.0031Q^{0.881}$	4	$C_o=0.0529 + 1.31\times10^{-5}\times F$	4	$L=1.467Q^{0.985}$
Oxidation Ditch	8	$C_c=0.0017Q^{0.910}$	10	$C_o=0.0963 + 1.02\times10^{-5}\times F$	7	$L=183.398Q^{0.513}$
Aerated Lagoons	11	$C_c=0.0143Q^{0.681}$	8	$C_o=0.0607 + 3.31\times10^{-6}\times F$	6	$L=9.876Q^{0.940}$
Waste Stabilisation Ponds	23	$C_c=0.0004Q^{1.060}$	19	$C_o=0.018 + 4.03\times10^{-6}\times F$	11	$L=127.736Q^{0.762}$

Where, C_c is capital costs (USD million); Q is design capacity (m^3/d); C_o is operations and maintenance costs (USD million/year); F is actual flow rate (m^3/d); L is land requirement (m^2); N is the number of case studies.

Tables 8.15-8.17 illustrate some examples of the use of Table 8.14, for some selected input wastewater loading.

Table 8.15 Capital cost of construction based on different influent capacity.

System	Capital cost ($M)	Treatment plant capacity				
	Equation	40MLD	60MLD	80MLD	100MLD	120MLD
Activated Sludge	$C_c=0.0031Q^{0.881}$	$35.1M	$50.2M	$64.7M	$78.8M	$92.5M
Oxidation Ditch	$C_c=0.0017Q^{0.910}$	$26.2M	$37.9M	$49.2M	$60.3M	$71.2M
Aerated Lagoons	$C_c=0.0143Q^{0.681}$	$19.5M	$25.7M	$31.2M	$36.3M	$41.1M
Stabilisation Ponds	$C_c=0.0004Q^{1.060}$	$30.2M	$46.4M	$63.0M	$79.8M	$96.8M

Table 8.16 O&M cost based on different influent capacity.

System	O&M cost ($M/year)	Treatment plant capacity				
	Equation	40MLD	60MLD	80MLD	100MLD	120MLD
Activated Sludge	$C_o=0.0529+1.31\times10^{-5}\times F$	$0.58M/y	$0.84M/y	$1.10M/y	$1.36M/y	$1.62M/y
Oxidation Ditch	$C_o=0.0963+1.02\times10^{-5}\times F$	$0.50M/y	$0.71M/y	$0.91M/y	$1.12M/y	$1.32M/y
Aerated Lagoons	$C_o=0.0607+3.31\times10^{-6}\times F$	$0.19M/y	$0.26M/y	$0.33M/y	$0.39M/y	$0.46M/y
Stabilisation Ponds	$C_o=0.018+4.03\times10^{-6}\times F$	$0.18M/y	$0.26M/y	$0.34M/y	$0.42M/y	$0.50M/y

Table 8.17 Land requirements based on different influent capacity.

System	Land area (ha)	Treatment plant capacity				
	Equation	40MLD	60MLD	80MLD	100MLD	120MLD
Activated Sludge	$L=1.467Q^{0.985}$	5.0ha	7.5ha	9.9ha	12.3ha	14.8ha
Oxidation Ditch	$L=183.398Q^{0.513}$	4.2ha	5.2ha	6.0ha	6.7ha	7.4ha
Aerated Lagoons	$L=9.876Q^{0.940}$	20.9ha	30.6ha	40.1ha	49.5ha	58.8ha
Stabilisation Ponds	$L=127.736Q^{0.762}$	41.0ha	55.9ha	69.6ha	82.5ha	94.8ha

Table 8.18 provides future plan of DWASA about operation and maintenance costs of different wastewater treatment plants to be constructed in and around Dhaka, for efficient wastewater management. The costs are based on standard unit rate of $50/capita.

Table 8.18 Comparison of costs of sewerage and wastewater treatment plants (DWASA, 2013).

No.	Catchment served	Treatment plant location	Population served	Influent flow rate		Sewage treatment plant				Sewerage	
				Daily flow (m^3/d)	Daily peak (m^3/d)	Receiving waters	Capital cost ($M)	O&M cost ($M/yr)	Treatment plant area (ha)	Capital costs ($M)	O&M cost ($M/yr)
Greater Dhaka											
1	Savar	Savar	400000	46000	69000	Dhaleswari	$20M	$0.40M/yr	5	$69M	$1.38M/yr
2	Tongi/Gazipur	Tongi Gazipur	800000 400000	92000 46000	138000 69000	Turag/Tongi Khal	$40M $20M	$0.80M/yr $0.40M/yr	10 5	$34M $86M	$0.68M/yr $1.72M/yr
3	Rupganj	Purbachal	500000	57500	86250	Lakhya	$25M	$0.50M/yr	6	$72M	$1.44M/yr
4	(Purbachal) Keraniganj	Keraniganj	400000	46000	69000	Buriganga	$20M	$0.40M/yr	5	$59M	$1.18M/yr
DWASA service area											
1	Dhaka North (Uttara)	Uttara	1600000	184000	276000	Tongi Khal	$80M	$1.60M/yr	20	$57M	$1.14M/yr
2	Dhaka West (Mirpur)	Mirpur	2800000	322000	483000	Turag	$140M	$2.80M/yr	34	$86M	$1.72M/yr
3	Dhaka East (Dasherkandi)	Dasherkandi	2400000	276000	414000	Balu River	$80M	$1.60M/yr	25	$122M	$2,44M/yr
4	Rayerbazar (Kamrangirchar)	Rayerbazar	1600000	184000	276000	Buriganga	$80M	$1.60M/yr	20	$67M	$1.34M/yr
5	Dhaka South (Pagla)	Pagla	4200000	483000	724500	Buriganga	$102M	$2.00M/yr	51	$126M	$2.52M/yr
6	DND-Demra	Pagla	900000	103500	155250	Buriganga	$45M	$0.90M/yr	12	$70M	$1.40M/yr
7	Narayanganj	Gognagar	1400000	161000	241500	Dhaleswari	$70M	$1.40M/yr	17	$116M	$2.32M/yr

Questions

1. What is the role of total quality management on achieving sustainable environment?
2. Why CETP process is preferable for small scale industries?
3. What is the main feature of wetland systems over traditional treatment technologies, when employed for industrial wastewater treatment?

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Chapter 9

Sludge Management

Although the activated sludge process removes solids, organics and nutrients efficiently, it also produces excess sludge, consisting higher fraction of biodegradable organic matter, solids, and pathogens. These impurities must be removed prior to environmental disposal, which is generally achieved through sludge treatment process (Figure 9.1).

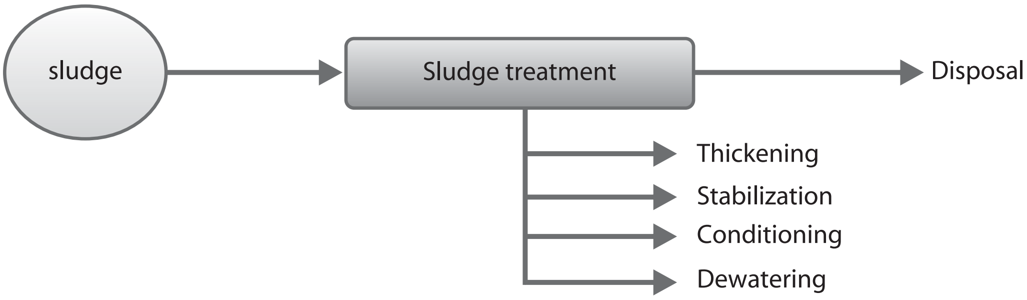


Figure 9.1 Sludge treatment process.

This chapter provides a brief description of sludge characteristics, treatment processes, and design approach, illustrated through sections 9.1-9.7.

9.1 Sludge Characteristics

Sludge properties. Sludge obtained from primary clarifier is grey in color, coarse and slimy; it contains about 3-7% solids, and 60-80% organic materials (Davis and Cornwell, 1991; Federal register, 1993). Secondary clarifier sludge is brownish, with flocculant appearance; microorganisms (70-90% organic) are the predominant part of secondary sludge (Lin, 2007). Tables 9.1 and 9.2 provide some basic properties of different sludge types (USEPA 1979; 1987; Metcalf and Eddy, 1991; Lin, 2007).

Table 9.1 Solid concentration from treatment plants.

Sludge type	Concentration, %
Primary sludge	5-8
Waste activated sludge	0.5-2
Fixed film sludge	3-10
Primary and waste activated sludge	2.5-4
Primary and fixed film sludge	3-5
Aerobic digested sludge	1-2
Anaerobic digested sludge	6-12

Table 9.2 Properties of different sludge types.

Sludge	Primary settler	Secondary settler
Amount generated, L/m^3	2.5-3.5	15-20
Solid content, %	3-7	0.5-2
Organic content, %	60-80	50-60
Treatability	easy	difficult

Sludge density. Solids mass in slurry is related to volatile suspended solids (VSS), and fixed suspended solids (FSS). The specific gravity of slurry (S_s) can be defined as:

$$S_s = \frac{m_w + m_v + m_f}{V_s} \tag{9.1}$$

where S_s = slurry specific gravity, g/cm^3
m_w = water mass, kg
m_v = VSS mass, kg
m_f = FSS mass, kg
V_s = slurry volume, m^3

Slurry volume V_s is the sum of V_w, V_v and V_f, which are the volume of water, VSS and FSS respectively (in m^3).

Therefore, Equation (9.1) can be written as:

$$\frac{m_s}{S_s} = \frac{m_w}{S_w} + \frac{m_v}{S_v} + \frac{m_f}{S_f} \tag{9.2}$$

where m_s = slurry mass, kg

Moisture content. The moisture (ρ_w) or solids (ρ_s) content of sludge is expressed on a percentage basis, as illustrated through Equation (9.3):

$$\rho_w = \frac{100 m_w}{m_w + m_s} = \frac{100 m_w}{m_w + m_v + m_f} \tag{9.3}$$

$$\rho_s = 100 - \rho_w \tag{9.4}$$

The sludge volume can be expressed as:

$$V_s = \frac{m_s}{(\rho_s / 100)\, S_s} \tag{9.5}$$

The specific gravity of VSS is 1, and for FSS the specific gravity is 2.5. The specific gravity of activated sludge is 1.01-1.10, and for chemical sludge the value is 1.5-2.5 (Droste, 1997).

Specific resistance. The sludge produced from biological treatment plants generally contains higher solids and water. Different treatment technologies are available for dewatering such sludge. Specific resistance is a parameter, that is used to express the ease of mechanical sludge. Typical values of specific resistance are 4-12 $\times 10^{13}$ m/kg for the activated sludge, 3-30 $\times$ 10^{13} m/kg for digested sludge, 3-10 $\times$ 10^{11} m/kg for conditioned primary sludge and 2-20 $\times 10^{11}$ m/kg for conditioned digested sludge (Barnes et al., 1981).

Bound water. The water content in sludge exists in either free or bound water form. Bound water is the water, that is bound physically or chemically to sludge particles. If the amount of bound water is greater in sludge particles, greater energy is required to remove it. Heukelekian and Weisberg (1956) determined bound water quantity to be 3g/g of dry solids.

Particle size. Particle size of sludge is an important factor that determines sludge dewaterability. The presence of greater number of small particles (in sludge mass) increases surface area/volume ratio. Higher surface area enhances hydration, chemical requirements, and resistance to dewatering.

Organic contents. The organic matter content is comparable to the volatile solids content (VS) of the sludge mass. If VS is higher, sludge dewatering becomes difficult. As such, higher VS contents result lower dryness (of sludge), along with higher consumption of the flocculants. When the VS contents of the sludge are high, a thickening step is required in the process, in order to achieve better dewatering.

Example 9.1. Sludge specific gravity and volume. Calculate the specific gravity of a biological sludge, containing 70% VSS and 30% FSS, when the sludge has solids concentration of 2%. Also determine the volume of 1 kg sludge.

Solution

Step 1. Sludge specific gravity

2% solids= 20000 mg/L solids=20 g/L

$$VSS = 20\text{ g/L} \times 0.7 = 14\text{ g/L} = m_v$$

$$FSS = 20\text{ g/L} \times 0.3 = 6\text{ g/L} = m_f$$

$$m_s = 1000\ g; m_w = (1000 - 14 - 6)\text{ g} = 980\text{ g}$$

From Equation (9.2): $\frac{m_s}{S_s} = \frac{m_w}{S_w} + \frac{m_v}{S_v} + \frac{m_f}{S_f}$

$$\frac{1000}{S_s} = \frac{980}{1} + \frac{14}{1} + \frac{6}{2.5} \quad \text{[assuming specific gravity of VSS and FSS 1 and 2.5 respectively]}$$

$$S_s = 1.003\text{ g/cm}^3$$

Step 2. Compute volume of 1 kg sludge

From Equation (9.5): $V_s = \frac{m_s}{(\rho_s/100)\, S_s}$

$$V_s = \frac{20\text{ g}}{(2/100)\, S_s} = \frac{20\text{ g}}{(2/100)\,(1.003\text{ g/cm}^3)\,(1000\text{ cm}^3\text{/L})} = 0.99\text{ L}$$

9.2 Sludge Treatment Mechanisms

Sludge generated from wastewater treatment plants includes suspended solids and the chemical agents added in different stages of wastewater treatment. Such impurities can be removed through thickening, stabilization, conditioning, dewatering and reduction of volume, as illustrated in Figure 9.2.

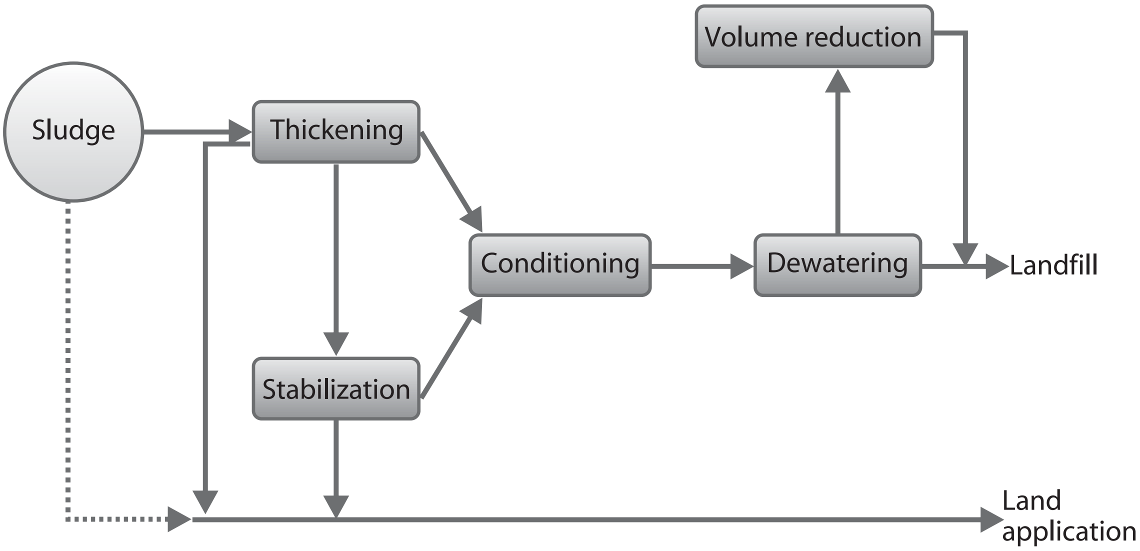


Figure 9.2 Typical sludge treatment diagram.

9.3 Sludge Thickening

This is the first step of sludge treatment process; the purpose of sludge thickening is to reduce the volume, which has to be treated in the later stages. As such, thickening process is necessary, for reducing the overall treatment cost. After the thickening process, water volume (of sludge) is reduced, and solid content is increased. Sludge thickening is achieved through gravity thickener, dissolved air floatation (DAF) system, and gravity belt thickener, as described below:

Gravity thickening. In gravity thickening, sludge from primary and secondary clarifier is mixed and stored in a sludge holding tank, and is being pumped at a constant rate to a sludge thickener. The vertical pickets on the scraper (of gravity thickeners) cause horizontal agitation, thereby allowing release of water trapped in flocculent structure of the sludge. Solids settle by gravity at bottom which is collected by mechanical scraper; the supernatant is recycled back to the equalization tank or primary sedimentation tank. Figure 9.3 indicates a schematic diagram of a gravity thickener, for sludge treatment process.

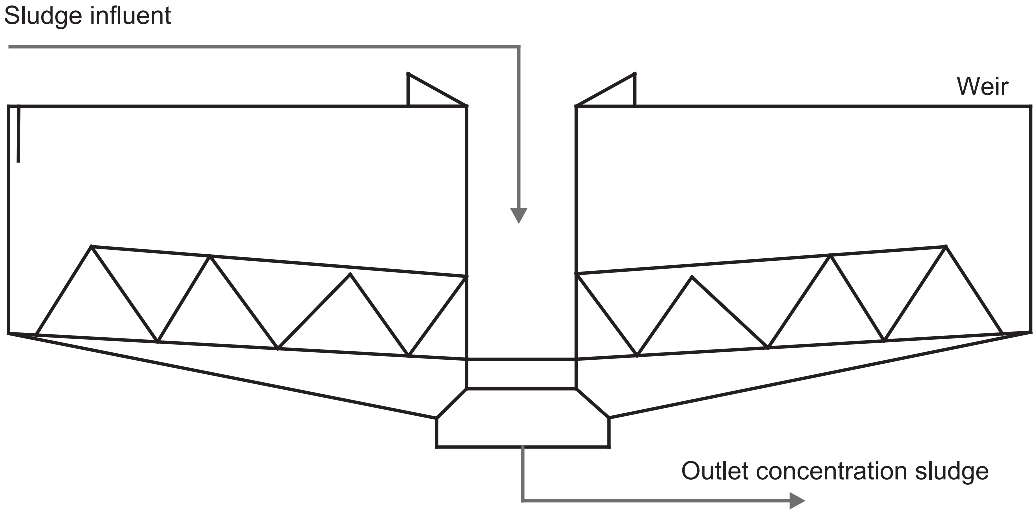


Figure 9.3 Schematic diagram of gravity thickener for sludge thickening.

Gravity thickener design usually includes circular tanks; the depth varies between 3-4 m, and the diameter can be selected up to 25 m. The slope of gravity thickener bottoms usually ranges between 1:6 and 1:3 (US EPA, 2003). The hydraulic loading varies between 16-32 $m^3/m^2.d$ for primary sludge, and 4-8 $m^3/m^2.d$ for secondary sludge (US EPA, 1979).

Sometimes, coagulants and polymers are mixed to improve sludge settling characteristics in gravity thickeners. Table 9.3 shows common chemicals, employed in gravity thickeners for improving sludge settling characteristics.

Table 9.3 Chemical dosage rates.

Solid types	**$FeCl_3$ (mg/L)**	**CaO (mg/L)**	**K_2MnO_5 (mg/L)**
Primary	1-2	6-8	10-40
Primary + tricking filter	2-3	6-8	10-40
Primary + waste activated sludge	1.5-2.5	7-9	10-40

Source: WPCF, 1987, WEF, 1996.

The performance of gravity thickener for various types of solids has been summarized in Table 9.4.

Table 9.4 Gravity thickener performance for sludge treatment.

Type of Solids	**Feed (% solids)**	**Thickened Solids (% solids)**
Primary (PR)	2-7	5-10
Trickling Filter (TF)	1-4	3-6
Rotating Biological Contactor (RBC)	1-3.5	2-5
Waste Activated Solids (WAS)	0.2-1	2-3
PR + WAS	0.5-4	4-7
PR+ TF	2-6	5-9
PR+ RBC	2-6	5-8

Source: US EPA, 2003.

Example 9.2. Sludge volume reducing by thickening. A treatment plant consists of primary treatment, followed by an activated sludge process. The primary and secondary sludge is mixed and thickened in a gravity thickener. The characteristics of the wastewater and sludge are:

	Parameters	**Values**
Wastewater characteristics	Influent solids	150 mg/L
	Solids removal efficiency	50%
	Influent BOD	300 mg/L
	BOD removal efficiency	25%
	Effluent BOD	15 mg/L
	Flow	15000 m^3/d
Sludge characteristics	Primary	6% solids
	Secondary	0.7% solids
	Thickened	5% solids

Determine (a) total mass and volume to the thickener; and (b) total volume of sludge discharged from the thickener. Assume the biomass conversion factor (Y) to be 0.30.

Solution

Step 1. Determine the mass and volume of primary solids

Primary solid mass, $M_p = \varepsilon \times solids \times Q = 0.5 \times 0.15 \text{ kg/m}^3 \times 15000 \text{ m}^3\text{/d} = 1125 \text{ kg/d}$

Primary sludge volume, $V_p = \dfrac{M_p}{1000 \times S} = \dfrac{1125 \text{ kg/d}}{1000 \text{ kg/m}^3 \times 0.06} = 18.75 \text{ m}^3\text{/d}$

Step 2. Calculate mass of secondary solids and volume

$$BOD_{in} = (300 - 300 \times 0.25) \text{ mg/L} = 225 \text{ mg/L}$$

$$BOD_{out} = \text{effluent } BOD = 15 \text{ mg/L}$$

BOD consumed in the aerator=210mg/L

Secondary solid mass, $M_s = Y \times BOD_5 \times Q = 0.3 \times 0.21 \text{ kg/m}^3 \times 15000 \text{ m}^3\text{/d} = 945 \text{ kg/d}$

Secondary sludge volume, $V_s = \dfrac{M_s}{1000 \times S} = \dfrac{945 \text{ kg/d}}{1000 \text{ kg/m}^3 \times 0.007} = 135 \text{ m}^3\text{/d}$

Step 3. Total mass and volume of solids to the thickener

Total mass, $M_T = M_p + M_s = (1125 + 945) \text{ kg/d} = 2070 \text{ kg/d}$

Total volume, $V_T = V_p + V_s = (18.75 + 135) \text{ m}^3\text{/d} = 153.75 \text{ m}^3\text{/d}$

Step 4. Total volume of sludge discharged from the thickener

$$V_{thickener} = \frac{2070 \text{ kg/d}}{1000 \times 0.05} = 41.4 \text{ m}^3\text{/d}$$

$$= \frac{153.75 - 41.4}{153.75} = 73.1\%$$

Percentage of volume reduction by the thickener

Dissolved air floatation (DAF). The activated sludge is light and flocculent in nature, which is difficult to thicken employing gravity thickeners. Such sludge type can be thickened, via dissolved air floatation (DAF) systems. DAF units are typically circular or rectangular in shape, with a float removal device (top skimmer), a bottom settling chamber (skimmer or auger), and an air saturation mechanism (i.e. whitewater system), to produce fine air bubbles for solids floatation (Figure 9.4).

In DAF systems, the influent at the tank bottom is saturated with air, and pressurized to 280-550 kPa. This supersaturated liquid is released near the tank bottom through which the sludge is passed at atmospheric pressure. Air is released in form of very small bubbles that

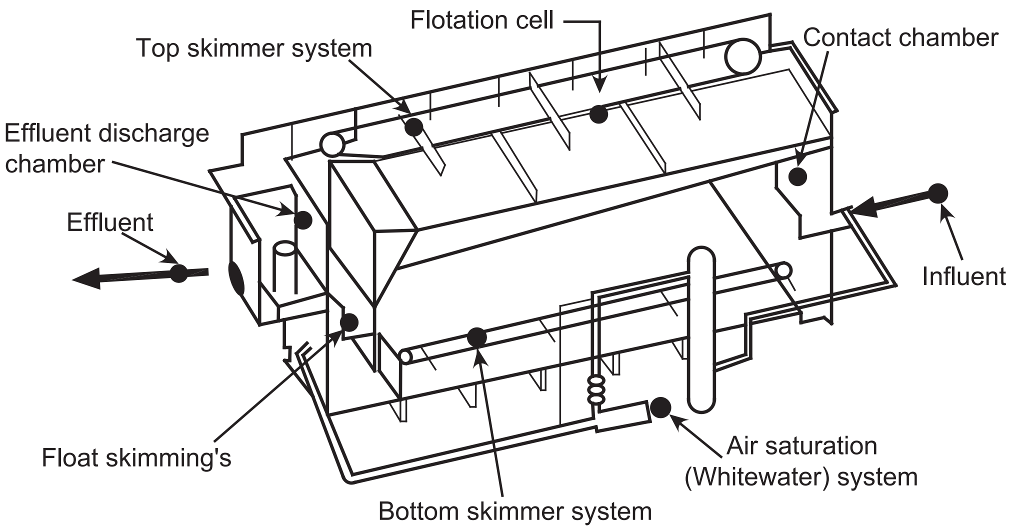


Figure 9.4 Dissolved air floatation systems (DAF) for sludge thickening.

attach themselves to, or become entrapped in sludge solids, floating the solids to the surface. The thickened sludge is skimmed off at the top of the tank; the liquid is removed near the bottom, and is returned to the aeration tank.

The air-solids ratio (A/S ratio) is the most important factor, for designing a DAF system. An A/S ratio of 0.0001 to 0.001 lb air/lb TSS is typically needed, to float flocculated biological solids. Normal loading rates for waste activated sludge range within 10-20 kg solids/m^2.d; a DAF system usually produces 4% solids (Hammer, 1986).

Box 9.1

Anaerobic effluent clarification by DAF system (Ross and Valentine, 2008).

A beverage plant was having difficulty, in terms of meeting COD and TSS discharge permit from a high-rate anaerobic process Upflow Anaerobic Sludge Blanket (UASB) (Figure 9.5). To meet the environmental criteria, the authority selected DAF system for clarification, prior to sewer discharge.

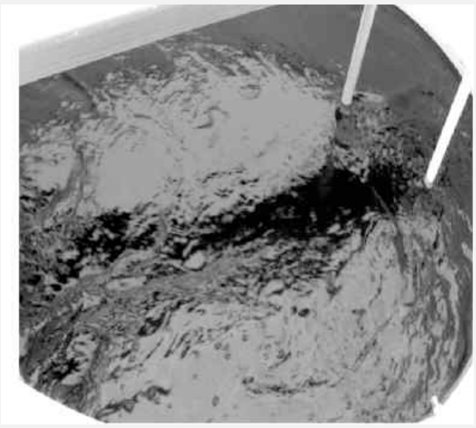


Figure 9.5 Anaerobic effluent in flocculation tank (Ross and Valentine, 2008).

Figure 9.6 DAF clarifier with flocculation treating anaerobic tank effluent (Ross and Valentine, 2008).

The treatment by DAF system involved coagulation and flocculation (with ferric chloride and anionic polymer) in a floatation tank, followed by DAF treatment (Figure 9.6). The DAF had a surface area of 100 ft^2,with an average flow of 200 gpm. The DAF successfully removed 67% of the incoming (eg. effluent of UASB) COD, 84% orthophosphate, and 82% of the biological solids. The effluent quality from the DAF system was within permit limits. Sludge from the unit was typically in 8% solids range, which was disposed through land application.

Gravity belt thickening. This process is applicable for raw and digested sludge, with solids content less than 2%. Figure 9.7 shows a diagram of gravity belt thickening process. In this process, the sludge is conditioned with polymer, prior to the entrance into the system. The conditioned sludge is squeezed by two moving belts, one at top and other at the bottom (Figure 9.7). The pressure increases from feed to discharge end, and sludge is dewatered. The liquid passes through porous cloths belt, which is collected in a trough below the belt press. The solid cake is obtained at discharge end.

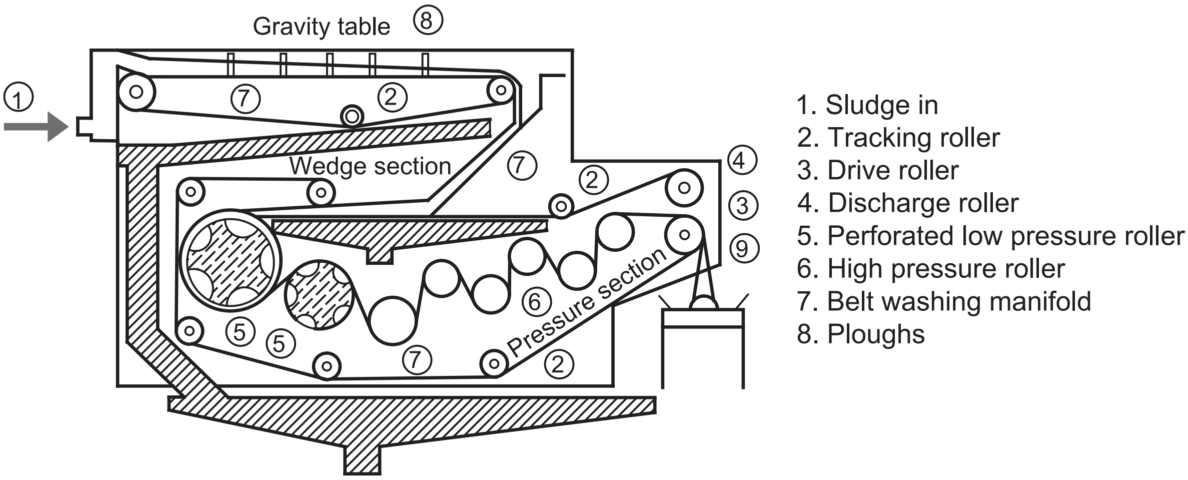


Figure 9.7 Schematic diagram of gravity belt thickening.

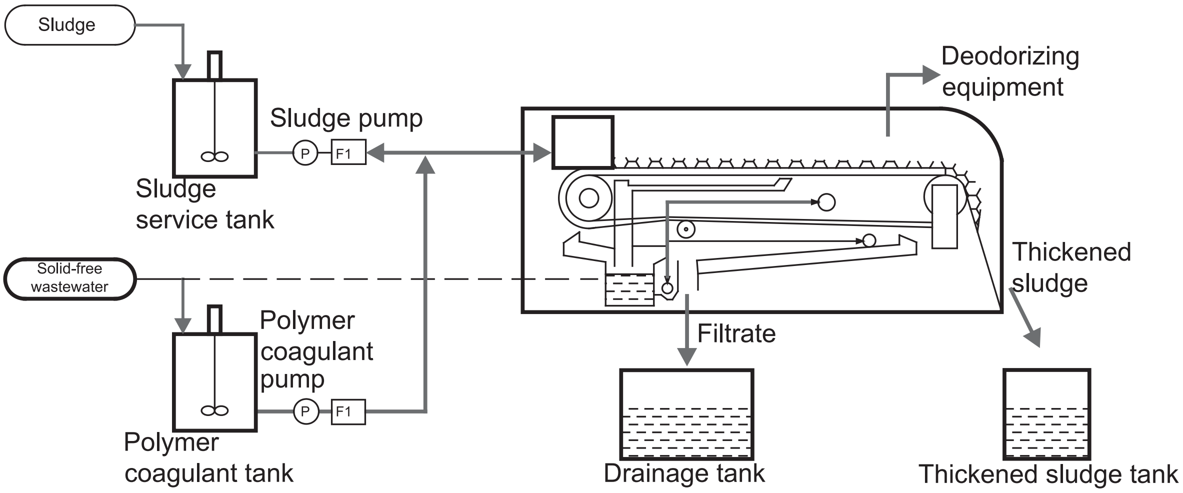


Figure 9.8 Operational arrangement of gravity belt thickening for sludge dewatering.

Figure 9.8 shows the operation diagram of gravity belt thickening, for the dewatering of sludge.

9.4 Sludge Stabilization: Anaerobic and Aerobic Digestion

After thickening process, the thickened sludge is subjected to stabilization for converting organic matter to an acceptable form, and allowing elimination of pathogens, odor causing substances. Sludge stabilization process is commonly achieved by anaerobic digestion (i.e. organic degradation in absence of oxygen), and is carried out in either a single or two stage reactors. Sometimes aerobic digestion is also employed, instead of anaerobic process.

The following sections provide the reaction phases of anaerobic metabolism, and the operational description of anaerobic digesters.

Reaction phases. When the sludge is kept in anaerobic environment, specialized bacteria employ the sludge to produce methane and carbon dioxide. Such anaerobic degradation of organic waste is carried out through four sequential steps: (a) hydrolysis; (b) acidification; (c) acetogenesis; and (d) methanogenesis.

(a) Hydrolysis: This is the first step in which macro-molecules (i.e. proteins, poly saccharides, and fats) are converted into small products (i.e. peptides, saccharides, and fatty acids) that are soluble in water. Hydrolysis process is carried out by fermentative bacteria; it is a slow process, and often limits the overall digestion rate.

(b) Acidification: It is the second process of anaerobic digestion, that converts the hydrolyzed products into simple molecules (with lower molecular weight), such as volatile fatty acids (acetic-, propionic- and butyric acid), alchohols, aldehydes and gases (i.e. CO_2, H_2, and NH_3). This process is performed by a diverse range of strictly anaerobic bacteria, sensitive to the presence of nitrate or oxygen. However, in an anaerobic digester, a certain proportion of bacteria are always present for oxygen utilization, which favors the metabolisms of acidification microbes.

(c) Acetogenesis: This is the third step of the overall anaerobic process. In this step, the products of acidification are converted into acetic acid, H_2, and CO_2, by the acetogenic bacteria. The first three steps of the anaerobic digestion are also referred as acid fermentation. If the structural formula of $C_5H_7O_2N$ is assumed to be representative for secondary sludge, the acid fermentation can be expressed as:

$$C_5H_7O_2N + 3H_2O \rightarrow 2.5CH_3COO^- + 1.5H^+ + NH_4^+ \qquad 9.6$$

(d) Methanogenesis: This is the last step of anaerobic degradation, and is carried out by a group of organisms known as methanogens, that produce methane (CH_4) gas. Two groups of methanogenic organisms are involved in CH_4 production. One group split acetate into CH_4 , and CO_2. Second group, known as hydrogen-utilizing group use H_2 as electron donor, and CO_2 as electron acceptor to produce CH_4. The production of CH_4 gas allows removal of organic through desorbing from the liquid phase. The methanogenesis reaction can be expressed by Equation 9.7:

$$2.5CH_3COO^- + 2.5H^+ \rightarrow 2.5CO_2 + 2.5CH_4 \qquad 9.7$$

The overall anaerobic digestion can be expressed by Equations 9.8-9.9.

$$C_5H_7O_2N + 3H_2O + H^+ \rightarrow 2.5CO_2 + 2.5CH_4 + NH_4^+ \qquad 9.8$$

$$C_5H_7O_2N + 4H_2O \rightarrow HCO_3^- + 1.5CO_2 + 2.5CH_4 + NH_4^+ \qquad 9.9$$

Figure 9.9 summarizes the sequential steps of anaerobic digestion.

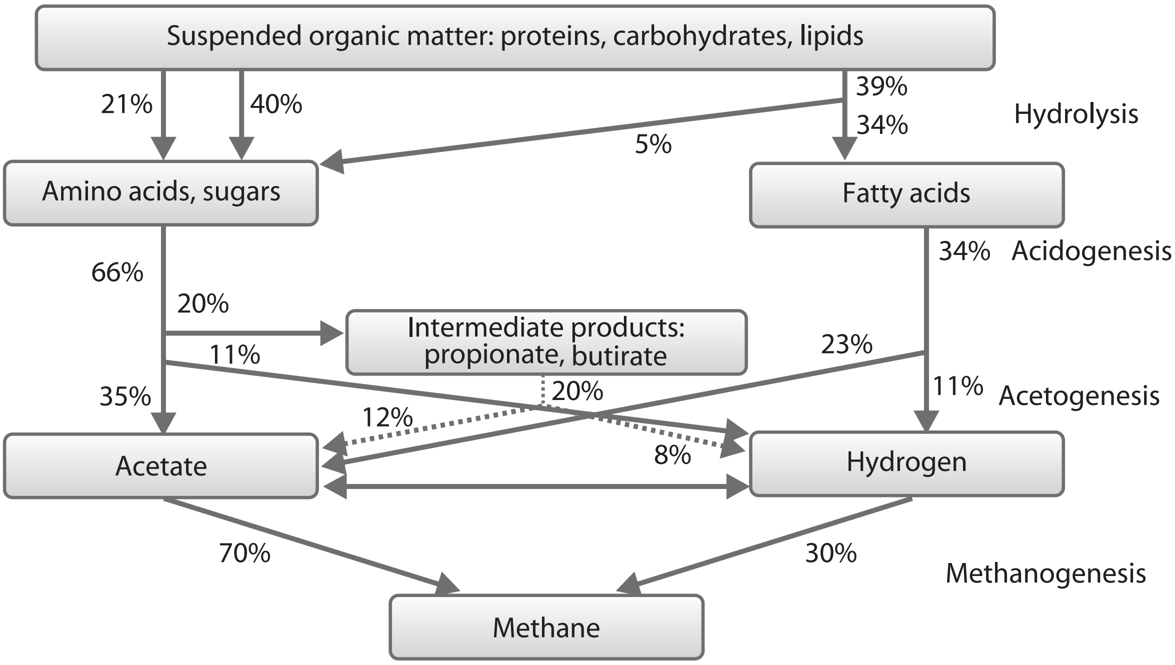


Figure 9.9 Different steps of anaerobic digestion for sludge stabilization.

Low and high rate digesters. The classic low rate anaerobic digester (to accomplish anaerobic degradation) comprises of four zones: (a) scum layer; (b) supernatant; (c) digestion; and (d) stabilized sludge zone.

- Scum layer consists of biodegradable and non-degradable floating materials, such as: leaves, hair, rags, and plastics.
- Supernatant is the liquid phase created through the sedimentation of solids.
- The digestion zone allows anaerobic transformation of organic matter into biogas.
- The digested sludge storage zone allows accumulation of the digested sludge, from where it is discharged for additional treatment.

Simultaneous operations i.e. settling of solids, and sludge digestion are difficult to carry out in a single reactor (Figure 9.10a), due to operational differences. The settling of solids requires tranquil conditions. In contrast, sludge digestion requires intensive mixing between sludge and anaerobic biomass, which is achieved through biogas recirculation or mechanical mixers. To overcome such shortcomings, high rate anaerobic digesters were developed in 1950s, where the entire digester volume is employed for digestion; the solid separation is carried out in a second unit (Figure 9.10b). In addition to solid separation, the second digester can be utilized for recirculation of accumulated methanogenic sludge into the first digester, when required. The second unit can also serve as a single digester, in case of primary digester maintenance. Figure 9.11 provides a pictorial view of anaerobic digesters, for sludge stabilization.

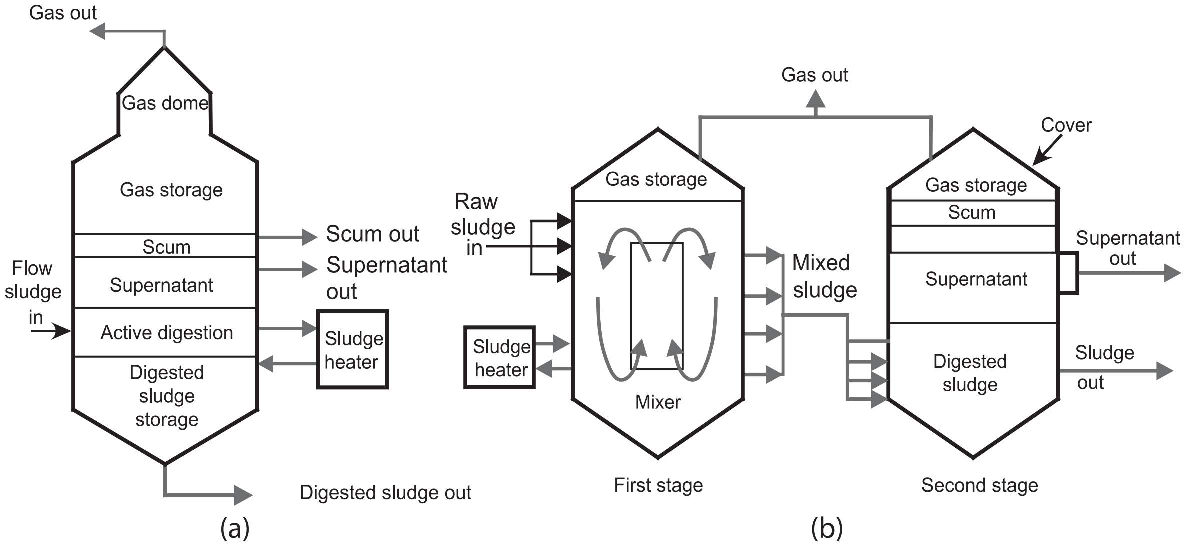


Figure 9.10 (a) Single stage anaerobic digester; and (b) two stage high rate anaerobic digesters.

Figure 9.11 Pictorial view of anaerobic digesters.

Operational factors of anaerobic digesters. The operating temperature is the most important key factor of anaerobic digester, as it influences the development of bacterial community. These reactors are usually operated under mesophilic or thermophilic conditions (Table 6.2-Chapter 6), and are defined as mesophilic or thermophilic digesters. Mesophilic digesters allow the growth of diversified bacterial group, that can be subjected to substantial environmental changes. However, such digesters require greater retention time, and produce lower amount of biogas. Thermophilic digesters allow higher sludge loading rates, which can be stabilized at a shorter period due to accelerated degradation of organic matter (as they are operated under elevated temperatures). In addition, these digesters have higher biogas yield capacity. However, the bacteria groups of these digesters are extremely sensitive to environmental changes, and toxic materials.

Substrate toxicity often governs sludge stability. Methanogenic bacteria are very sensitive to sulphide toxicity. However, sulphide concentration remains below 50-100 mg/L in anaerobic digestors, which does not inhibit methanogenic metabolism. Unionized ammonium, and industrial products for example biocides, chlorinated organics, and heavy metals (eg. Hg, Cd, Zn, Cr, Ni, Na, Ca) can cause toxic effect to the organisms. Table 9.5 provides a brief list of toxic materials, that often inhibit anaerobic digestion.

Table 9.5 **Inhibiting substances of anaerobic digestion**[a].

Substance	Moderately Inhibitive (mg/L)	Strongly Inhibitive (mg/L)
Calcium	1,500–4,500	8,000
Magnesium	1,000–1,500	3,000
Sodium	3,500–5,500	8,000
Potassium	2,500–4,500	12,000
Ammonia Nitrogen	1,500–3,000	3,000
Copper	––	50–70 (total)
Chromium VI	––	200–250 (total)
Chromium	––	180–420 (total)
Nickel	––	30 (total)
Zinc	––	1.0 (soluble)

[a] *US EPA, 2006*

Shock loading (addition of huge amount of sludge within a short period time) is harmful to anaerobic digesters. Acid formers respond quickly to increased food supply and produce increased amounts of acid. The methane formers cannot respond quickly; as such, acid accumulates in the system, causing pH decrease. Once pH reaches below the tolerance level of methane formers (i.e. 6.5-7.5), CH_4 production is ceased. However, the alkalinity of the digesting sludge is high because of CO_2 solubilization produced by biological process, and subsequent conversion to HCO_3^- (Equation 9.9). Table 9.6 provides the optimal operational ranges, for sludge digestion in anaerobic digesters.

Table 9.6 **Operational parameters of anaerobic digestors.**

Parameter	Unit	Value
Temperature	°C	34-36
pH		6.8-7.2
Alkalinity	mg/L	2000-3500
Detention time	d	60 at 20°C 15 at 35°C
Volatile solids loading	kg VS/m^3.d	1.9-2.5

Source: WEF and ASCE, 1991; Lin, 2007.

Gas production. Gas production is an important performance parameter of the anaerobic digester. The production of gas varies between 810-1120 L of digester gas, per kg volatile solids destroyed. An efficient anaerobic digester produces 65-69% CH_4 and 31-35% CO_2. The quantity of CH_4 gas production can be computed through Equation 9.10 (McCarty, 1964):

$$V = 350\left[\frac{Q\,(S_0 - S)}{1000} - 1.42\,P_x\right] \tag{9.10}$$

where V = methane volume at standard conditions (0°C, 1 atm), L/d
Q = flow rate, m^3/d
S_0 = influent ultimate BOD, mg/L
S = effluent ultimate BOD, mg/L
P_x = net mass of cell tissue produced, kg/d

For a complete-mix, high rate two stage anaerobic digesters (without recycle), the P_x can be determined using the following formula:

$$P_x = \frac{Y\,[Q(S_0 - S)]}{1 + k_d\theta_c} \tag{9.11}$$

where Y = yield coefficient, kg/kg
k_d = endogenous coefficient, d^{-1}
θ_c = mean cell residence time, d

Box 9.2

TPAD-TM anaerobic digester system in Iowa, USA (US EPA, 2006).

In order to enhance sludge treatment, volatile solids destruction and biogas production, the existing anaerobic process was upgraded in the year 2002, in Waterloo city, Iowa, USA. The upgradation included transformation of a single-stage mesophilic process, to a TPAD-TM (temperature phased anaerobic digestion- thermophilic first stage, and mesophilic second stage) system through the conversion of two of the six digesters into thermophilic digesters. The subsequent digesters were operated in the mesophilic range

In the newly upgraded system, the feed rate into the digester was slowed, followed by increase of temperature from 35°C to 53°C over a period of 3 days. Such arrangements allowed organisms stabilization. Once the first thermophilic digester was stabilized, the second was transitioned similarly. This transition from mesophilic to thermophilic was important, for limiting the number of mesophilic organisms that might survive in the thermophilic digester. During such transition, loading rate limitation was also critical, to prevent digester overloading as the thermophilic organisms flourished.

The new system improved volatile solid reduction from 47% (in old system) to 64% (in new system). Gas production increased to 0.18–0.21 m^3, per kg of VS destroyed.

Heavy metals removal by anaerobic digestion. The removal of heavy metals (from sludge) via anaerobic digestion is accomplished by four steps: solubilization of the heavy metals, separation of the water phase, precipitation, and removal from the leachate.

The solubilization of heavy metals is dependent on decrease of the sludge pH (around 1-2 units), in-conjunction with a previous increase in the redox potential of the sludge. Such combinations promote the formation of soluble metal complexes, and oxidize insoluble reduced metals into soluble forms. The solubilization step is generally accomplished by chemical, or bioleaching. In the case of chemical leaching, a previous oxidation step is performed by aeration, or by the addition of an oxidating agent in order to raise the redox potential. Then acidification step occurs through the addition of inorganic or organic acids. Bioleaching is an alternative process to chemical leaching. In bioleaching process, acidophilic *Thiobacillus* bacteria employ elemental sulfur or ferrous iron as electron donors. In aerobic conditions, these donors are oxidized to sulfuric acid by Thiobacillus bacteria.

The second step of anaerobic sludge treatment (for heavy metal removal) involves separation of the water phase, that includes mobilized heavy metals and sludge particles. Different devices such as: centrifuge, hydrocyclone, settler, or flotation tank are employed for such purpose. The clean sludge particles are removed as concentrated sludge mass which, after dewatering and pH corrections, is usually applied to landfills. Extreme pH conditions applied during bioleaching step also reduces pathogens.

In order to facilitate the removal of the solubilized heavy metals (from acidic leachate), the precipitation process followed by a separation step is generally accomplished. Alkalis (i.e. CaO, NaOH), sulfides (i.e. Na_2S, FeS) are used as precipitating agents for this purpose.

Aerobic digestion. In aerobic digestion, sludge is digested through aeration; organic materials are converted to end products such as: carbon dioxide, ammonia, and water. Volatile solids, pathogens, and odor are also reduced in aerobic digestion process.

Sludge is introduced on a batch, semi-batch, or continuous process in aerobic digesters, that are open tanks like activated aeration tank. In batch process, the digester is fed with raw sludge, and is subjected to aeration (for 2-3 weeks); the settled solids are being removed, and the supernatant is decanted. In semi-batch process, raw sludge is added every couple of days. Supernatant is being decanted periodically, and the settled solids are kept in the digester for a longer time period.

The volatile solids loading rate in aerobic digester ranges between 1.1-3.2 kg VSS /m^3.d. For activated sludge, retention time is 15-20 d, whereas, for mixture of primary and activated sludge retention time is 20-25 d. The DO concentration in the suspension should be 1-2 mg/L; diffused air requirements for waste activated sludge are 20-35 L/min.m^3.

9.5 Sludge Conditioning

Chemical conditioning is applied to the digested sludge, to improve dewater characteristics. The process is useful before sludge dewatering, and can be achieved through chemical or physical conditioning.

The mechanism of chemical conditioning consists of charge neutralization and transformation of individual sludge particles into a lattice structure; such structure achieves sufficient porosity, thereby permitting water draining. Lime, pebble quicklime, ferric chloride, alum and organic polymers can be used for chemical conditioning. Chemical conditioning can reduce sludge moisture content to 65-80% (WPCF, 1988).

The physical conditioning method includes thermal treatment of sludge. In this process, sludge is heated to a temperature 177-240°C in a reaction vessel, and subjected to a pressure of 1720-2760 KN/m^2 for 15-40 mins (US EPA, 1991). The solids are coagulated due to heat; structure of microbial cells (in waste activated sludge) is broken down, followed by release of cell bound water. However, this process is not suitable for sludge, that cannot be stabilized biologically (due to presence of toxic materials). Another disadvantage of heat conditioning process is the generation of side streams with higher concentration of organics, ammonia, nitrogen, color, and odorous gases. As such, these impurities have to be treated, incurring supplementary operational costs.

9.6 Sludge Dewatering

The objective of sludge dewatering is to remove moisture from the sludge (prior to final disposal), that passed through sludge stabilization and conditioning process. Sludge dewatering can be achieved by mechanical devices such as: rotary vacuum filters, continuous belt filter presses (CBFP), pressure filtration, centrifuges, or by natural evaporation and percolation.

Rotary vacuum filtration. Rotary vacuum filtration basically consists of a cylindrical rotating drum, covered with cloth or synthetic fabric, coil springs, or woven stainless steel mesh. The drum is partially submerged (20-40%) in a vat of sludge. The mechanisms which take place during vacuum filtration may be divided into three phases.

The first phase, known as cake pick-up or form phase, occurs when a segment of the drum rotates into the sludge. Such segment is subjected to vacuum, and the filtrate is drawn from the adhered solids, as the drum rotates out of the liquid phase, known as drying zone of the cycle. Drying zone represents 40-60% of the drum surface. The final phase, referred as cake discharge phase, occurs after an acceptable cake dryness has been achieved without vacuum. All of the above described operations are continuous in nature so that, the three phases occur simultaneously on different portions of the drum. Figure 9.12 indicates a schematic diagram of the rotary vacuum filtration.

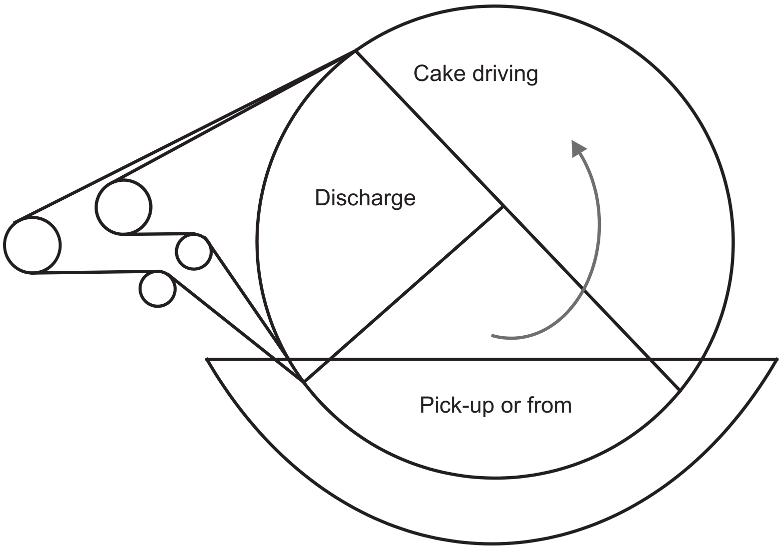


Figure 9.12 Rotary vacuum filtration for sludge dewatering.

Continuous belt filter presses (CBFP). This process includes a combination of sludge conditioning, gravity and pressure dewatering. The incoming sludge is mixed with polymer (or other chemicals), and placed onto a moving porous belt or screen. The sludge is squeezed between two belts, resulting sludge dewatering. The formed cake is discharged from the belt by a scraper.

Figure 9.13 illustrates the basic concept of continuous belt filter presses. As observed in Figure 9.13, CBFP process consists of three zones: conditioning zone, gravity drainage of conditioned sludge, and a shear –compression zone. Conditioned sludge is introduced to the gravity drainage zone, where majority of free water (from sludge) is being removed. In the shear zone, shear is applied (to the partially dewatered cake), by positioning the support and pressure rollers of the pressure belt in such a way that the belts, and the sludge between them describe an S-shape curve. After the removal of sludge cake (from CBFP process), the belts are rinsed with water spray, and the sprayed rinsed water along with the filtrate is recycled to primary or secondary clarifier.

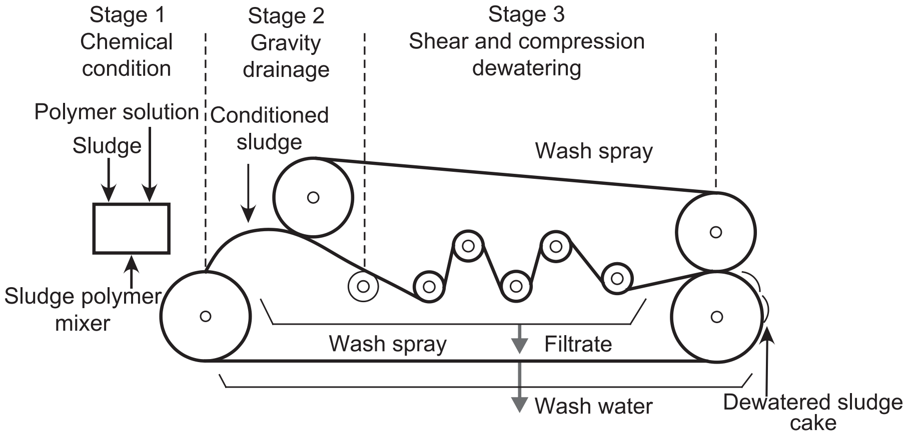


Figure 9.13 Continuous belt filter presses (CBFP).

Pressure filtration. Pressure filtration includes a series of vertically held plates, which are held rigidly in a frame, consisting of a fixed and moving end (Figure 9.14). The face of each individual plate is mounted by a filter cloth, which contains the produced cake.

In pressure filtration system, sludge is applied by batch mode through feed holes in trays (along the press length). Pressures up to 16 kg/cm^2 are applied to the sludge, to cause water movement through the cloth; the solid cakes are retained on the surface of the filter cloth. The drainage ports (at the bottom of each chamber) collect the filtrate, which is discharged. The dewatering phase is complete when the filtrate flow (through the filter cloth) is ceased. After each plate releases cake, they are pushed back for commencing the dewatering cycle.

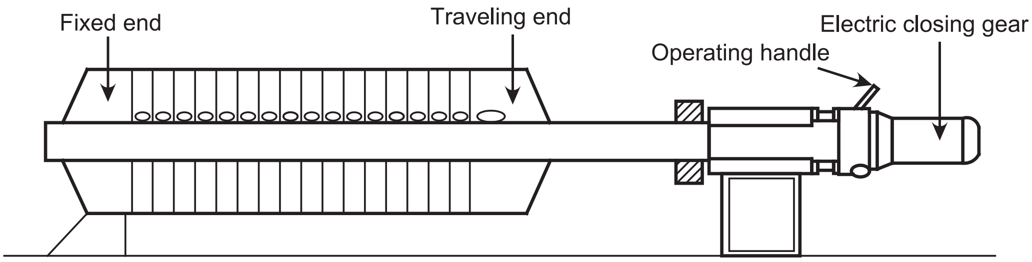


Figure 9.14 Pressure filtration system.

Continuous solid bowl centrifuge. This centrifuge consists of two principal elements: a rotating bowl having cylindrical-conical shape, and a conveyor which discharges the settled solids (Figure 9.15). Sludge is fed into the rotating bowl through a stationary pipe. The gravitational forces allow solids settling, on the inner surface of the rotating bowl. The rotating screw conveyor continuously moves the sludge solids across the bowl, towards the tapered outlet ports, for discharging. The liquid pool above the sludge layer is directed toward the concentrate outlet ports.

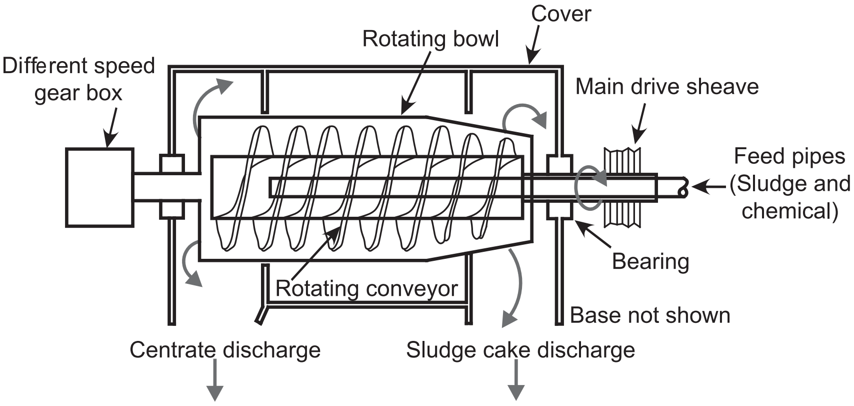
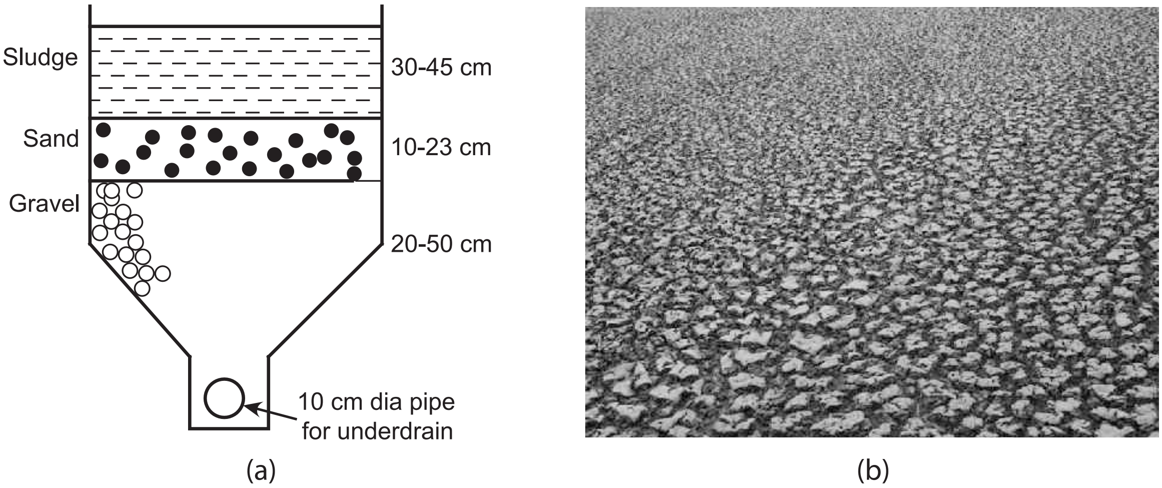


Figure 9.15 Continuous solid bowl centrifuge system.

Sludge drying bed. Sludge drying bed which is a natural dewatering process, removes moisture content of the sludge by gravity and evaporation. A typical sludge bed consists of sand (0.30-1.22 mm diameter), overlying on gravel (0.3-2.5 cm diameter) layer. The water is drained through underdrain system, consisting perforated pipe. The collected water is transferred to the main treatment plant for further polishing. The drying area is usually partitioned into 6m width by 6-30 m length. Figure 9.16 indicates the engineering diagram of sludge drying beds.



(a) (b)

Figure 9.16 (a) Engineering cross-section of sludge drying bed; and (b) pictorial view of drying bed.

In hot climate regions, large portion of sludge water (eg. 80%) percolates within a short time (1-3 days), resulting in solid cake production with TSS concentration of 150-200 g/L (Haandel and Lubbe, 2007) on drying beds. The remaining moisture is removed by evaporation, resulting in a final production with very high solids content. Dried sludge is removed manually with shovels and lost sand is refilled.

Drying beds should be covered with roofs to protect the beds from rain. The disadvantages of sludge drying beds can be an attributed to mosquitoes and rodent problems. In addition, odor can also create nuisance, which can be mitigated by spreading lime on sludge, thereby avoiding anaerobic conditions.

Table 9.7 indicates the performance of sand drying beds (in terms of sludge dewatering) in Jahra Plant, Kuwait, between time periods 1985-2010.

Example 9.3. Solids loading in drying bed. The conditioned sludge is applied to a sand drying bed, 6m by 25 m in size, at a depth of 35 cm. The solids content of the sludge is 2%.

Table 9.7 **Performance of sludge drying beds in Jahra Plant, Kuwait.**

Year	Thickened Sludge (m^3/d)	Sludge Solids (kg/d)	Area of Drying Beds (m^2)	Number of Drying Beds
1985	143	4290	12045	32
1990	190	5700	16004	43
1995	228	6840	19205	51
2000	266	7980	22405	60
2010	352	10545	29607	79

Source: Muzaini, 2003.

The drying period is 25 days, after which the sludge is removed for another application in the drying bed. Calculate the yearly solids loading rate. Assume the specific gravity of the sludge to be 1.02.

Solution

a. The volume of sludge per application, V

$$V = 6\text{ m (width)} \times 25\text{ m (length)} \times 0.35\text{ m (depth)} = 52.5\text{ m}^3\text{/app}$$

b. Yearly solids loading rate

$$\text{Solids} = 52.5\text{ m}^3\text{/app} \times 1000\text{ kg/m}^3 \times 1.02 \times 0.02 = 1071\text{ kg/app}$$

$$\text{Loading rate } \frac{\text{solids} \times 365\text{ d/yr}}{\text{area} \times \text{cycle}}$$

$$= \frac{1071\text{ kg/}app \times 365\text{ d/yr}}{(6\text{ m} \times 25\text{ m}) \times 25\text{ d/}app} = 104\text{ kg/m}^2\text{ yr}$$

9.7 Design Problem

This section provides a step-wise design procedure of anaerobic digesters (for sludge stabilization) through Example 9.4, to assist the reader in terms of understanding the key elements required during the procedure.

Example 9.4. Anaerobic digester design. Calculate the size of the required anaerobic digester for treating sludge with the following properties:

Average design flow	4000 m^3/d
Dry solids removed	0.20 kg/m^3
Ultimate BOD removed	0.18 kg/m^3
Solids content	5%
θ_c	20 d
Y	0.05 kg/kg BOD
k_d	0.03 d^{-1}
Solid specific gravity	1.01
Waste utilization efficiency	70%

Step 1. Production of sludge volume per day

$$\text{Sludge volume} = \frac{(4000\ \text{m}^3/\text{d})\,(0.20\ \text{kg/m}^3)}{1.01\,(1000\ \text{kg/m}^3)\,(0.05\ \text{kg/kg})} = 15.8\ \text{m}^3/\text{d}$$

Step 2. Total BOD loading

$$\text{Loading} = 0.18\ \text{kg/m}^3 \times 4000\ \text{m}^3/\text{d} = 720\ \text{kg/d}$$

Step 3. Required digester volume

$$V = Q\theta_c = (15.8\ \text{m}^3/\text{d}) \times (20\ \text{d}) = 316\ \text{m}^3$$

Step 4. Mass of volatile solids produced per day

$$\text{Equation (9.11)}:\ P_x = \frac{YQ\,(S_0 - S)}{1 + k_d\theta_c} = \frac{Y\,(QS_0 - QS)}{1 + k_d\theta_c}$$

$$P_x = \frac{0.05\,(720 - 720\,(1 - 0.7))\ \text{kg/d}}{1 + (0.03\ \text{d}^{-1})\,(20\ \text{d})} = 15.7\ \text{kg/d}$$

Step 5. Methane (CH_4) and total gas (CH_4+CO_2) volume production per day

Employing Equation (9.10):

$$V = (350\ \text{L/kg}) \times [Q\,(S_0 - S) - 1.42P_x]$$

$$= 350\ \text{L/kg}\,[720 - 720\,(1 - 0.7) - 1.42\,(15.7)]\ \text{kg/d} = 168597\ \text{L/d} = 168.5\ \text{m}^3/\text{d}$$

The volume of methane is 65-69% of the total gas volume. Assume 69% value.

$$\text{Total gas volume} = \frac{168.5\ \text{m}^3/\text{d}}{0.69} = 244.3\ \text{m}^3/\text{d}$$

Questions

1. Write short notes on: (a) gravity thickener; and (b) DAF systems for sludge thickening.
2. What are the advantages of two stage anaerobic digesters, over a single stage digester?
3. What is the impact of shock loading on anaerobic digester performance?
4. Differentiate between the operating mechanism of continuous belt filter presses and pressure filtration
5. Calculate the size of anaerobic digester for sludge stabilization with the following properties:

Average design flow	5000 m^3/d
Dry solids removed	0.30 kg/m^3
Ultimate BOD removed	0.14 kg/m^3
Solids content	4%
θ_c	25 d
Y	0.05 kg/kg BOD
k_d	0.04 d^{-1}
Solid specific gravity	1.01
Waste utilization efficiency	60%

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Chapter 10

Natural Treatment: Ponds and Wetlands

Chapters 4-9 provide a brief discussion on pollutant removal mechanisms (from wastewater), employing conventional treatment technologies. Apart from these technologies, the treatment of wastewater can also be carried out via natural systems, such as: wastewater stabilization ponds, and constructed wetlands (Figure 10.1). These natural technologies primarily depend on naturally occurring energies such as wind, solar radiation and biomass storage (Kadlec and Knight, 1996). Such properties of natural treatment systems provide unique advantages over conventional technologies which require significant input of fossil fuel energies.

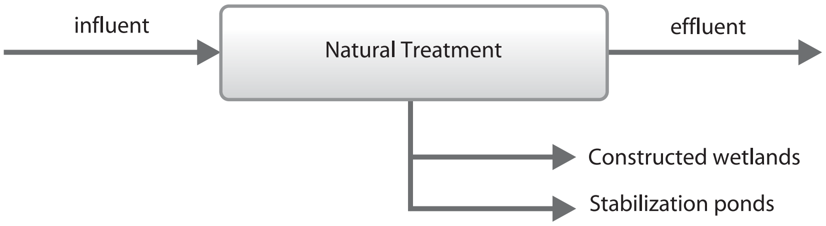


Figure 10.1 Typical natural treatment technologies for wastewater treatment.

This chapter provides a concise description of pollutant biodegradation routes in natural treatment systems (i.e. wastewater stabilization ponds, and constructed wetlands). Firstly, classification of waste stabilization ponds, corresponding pollutant removal mechanisms, along with a design example have been demonstrated in sections 10.1-10.5. Secondly, the configuration of wetland systems, along with the removal routes of pollutants have been illustrated in sections 10.6-10.11. A brief description on floating treatment wetlands have been provided in the latter sections of this chapter.

10.1 Stabilization Ponds

Wastewater stabilization ponds are shallow earthen basin where waste water is naturally treated. A part of the system can be aerobic to produce acceptable effluents. The majority portion of O_2 in ponds is provided by photosynthesis. Artificial aeration can also improve the oxygen contents of the ponds. Treatment ponds can be employed alone, or can be combined with other processes for industrial and domestic wastewater treatment. The main advantages of treatment ponds can be enlisted as (Mara, 1976):

- Ponds are simple to construct, and require lower maintenance costs.
- They often achieve higher degree of BOD, and pathogen removal (bacteria, virus, protozoa and helminths).
- Ponds can tolerate hydraulic and organic shock loads.

However, such systems generally require substantial land area, which is the major capital cost.

10.2 Classification of Stabilization Ponds

According to the metabolic operations, ponds can be classified into: anaerobic, facultative, aerobic and maturation ponds, as illustrated below.

Anaerobic ponds. These ponds are often used for the treatment of strong wastewater, such as industrial or agricultural wastewater. Sometimes they are also used as the first element of a treatment series, for reducing organic strength. Anaerobic ponds lack DO and algae; organic compounds are degraded in absence of oxygen, according to the process described in Chapter 9. Anaerobic ponds are 2-5 m deep; the incoming organic load is usually greater than 100 g BOD/m^3.d, with a typical retention time of 20-50d (US EPA, 1983). A well designed anaerobic pond often achieves 40% BOD removal at 10° C, and greater than 60% at 20° C.

The main advantages of anaerobic ponds are lower sludge production; however, generation of odorous compounds is a major disadvantage. Table 10.1 indicates typical BOD removal efficiencies in anaerobic ponds.

Table 10.1 BOD removal and loading in anaerobic ponds at various temperatures. Note that T denotes temperature.

Temperature °C	BOD removal %	Volumetric loading g/m^3.d
<10	40	100
10-20	2 T+20	20 T-100
20-25	2 T+20	10 T+100
>25	70	350

Source: Alexiou and Mara, 2003.

Facultative ponds. In facultative ponds, both aerobic and anaerobic zones co-exist (Figure 10.2). The aerobic conditions in the upper portions of the pond are usually maintained by O_2 production from algae; in addition, atmospheric O_2 penetration also provides minor contribution for maintaining aerobic conditions.

The stagnant conditions of the sludge, along the bottom portion of the ponds prevent O_2 transmission, thereby resulting anaerobic conditions. The aerobic zone is extended downwards, due to wind and sunlight penetration. In contrast, calm conditions and weak lighting cause rising of the anaerobic layer, towards the surface. As such, the middle portion between aerobic and anaerobic zone is commonly referred as facultative zone, due to fluctuating phenomena.

The products of anaerobic zones (i.e. organic acids and gases), are released and become soluble food for aerobic organisms (in the aerobic zone). The common species of aerobic zone include *Beggiatoa alba, Sphaerotilus natans, Achromobacter, Alcaligenes, Flavobacterium* etc. Subsequently, the solids produced in the aerobic zone settle to the bottom where they die, providing food for the anaerobic organisms. The aerobic and anaerobic degradation of organic matter produces nitrogen, phosphorus and carbon dioxide.

A special symbiotic relationship exists between the bacteria and algae in the aerobic zone, as shown in Figure 10.2. Bacteria use O_2 as an electron acceptor, to convert organics into stable end products such as CO_2, NO_3^-, and PO_4^{3-}. Algae, with the help of sunlight, employ these compounds as energy source, producing O_2 as end product, which is again utilized by the bacteria.

The photosynthetic activities of algae can cause diurnal variation of the DO concentration in facultative ponds. The DO concentration rises after sunrise, due to photosynthetic activity;

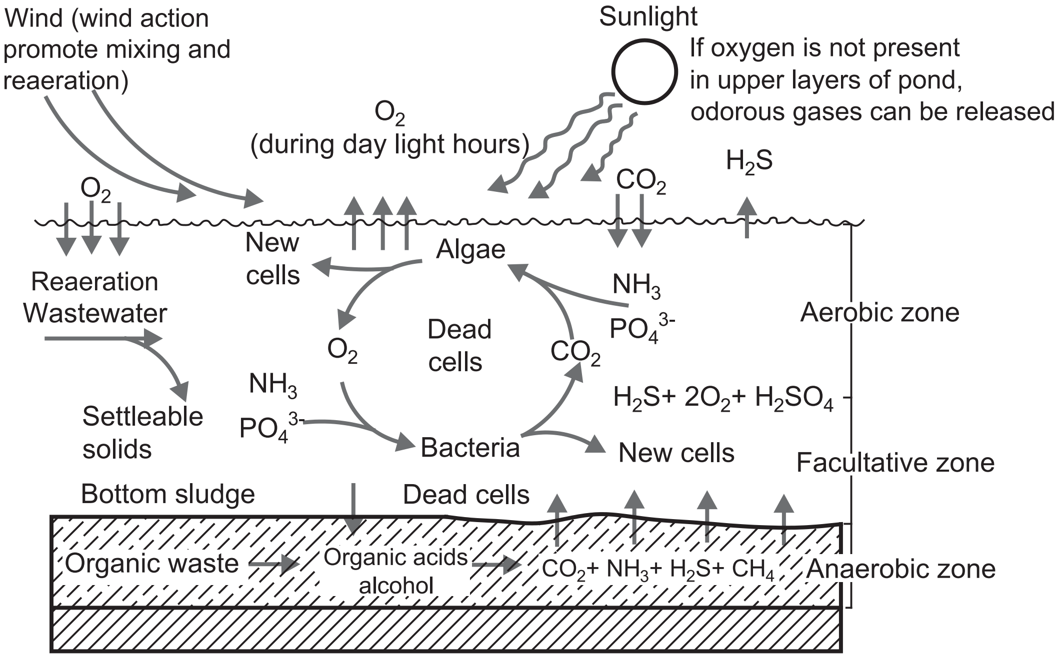


Figure 10.2 Pollutant removal routes in facultative ponds.

the concentration reaches to a maximum level in the afternoon, followed by sharp decrease during the night, when photosynthesis ceases and respiratory activities consume oxygen. During day time, the reaction of carbonate and bicarbonate provide more carbon dioxide for the algae, leaving an excess of hydroxyl ions. As a result, the pH of the water can rise to above 9.

Typical organic loading in facultative ponds ranges within 22-67 kg BOD/ha.d; the detention time varies within 25-180 d. Typical depths are within 1.2-2.5 m, with a surface area of 4-60 ha (US EPA, 1983). Table 10.2 shows the variation of organic loading rates with temperature.

Table 10.2 Loading rates for facultative ponds.

Temperature °C	BOD loading kg BOD_5/ha/d
<10	11-22
0-15	22-45
>15	45-90

Source: Reed et al.1988

Maturation ponds. The maturation ponds generally comprise of a greater length-to-width ratio (up to 10:1), to simulate plug flow regime (Mara et al., 1992). Maturation ponds receive effluent from facultative ponds. The primary function of maturation pond is to remove pathogens. The principal factors influencing faecal bacterial removal mechanisms in maturation ponds can be enlisted as: (a) time and temperature; (b) higher pH (> 9); (c) higher light intensity; and (d) dissolved oxygen concentration.

Time and temperature are the major parameters used for the design of maturation ponds, as faecal bacterial mortality is critically dependent on these parameters. Peak algal activities (during daytime) increase pH values (>9), thereby enhancing faecal bacterial mortality. Light with wavelengths between 425 – 700 nm can also damage the cells of faecal bacteria. Light mediated mortality is substantially dependent on oxygen concentration, and higher pH values.

The water depth of maturation ponds varies between 1-1.5 m, with BOD loading less than 17 kg/ha.d; the retention time is short, ranging within 4-15d (Lin, 2007).

Aerobic ponds. They are most often designed as the final cells in a multi-staged pond system. The depth of aerobic ponds is shallow ranging within 0.3-0.6 m. Such shallow depth allows sunlight penetration to the bottom of the pond, encouraging algae growth and maintaining aerobic conditions throughout the pond. Mechanical aeration can also be provided in these ponds. Sludge zone is not provided in these ponds due to lower loadings of solids. Aerobic ponds generally have shorter retention time (i.e. 3-5 d). The application of these ponds is limited in warmer climate, where higher removal of BOD is required.

10.3 Nutrient Removal in Stabilization Ponds

The stabilization ponds promote numerous nitrogen removal routes, such as: (a) assimilation of ammonia and nitrate into algae, bacteria and plants; (b) conversion of ammonia into nitrate (i.e. nitrification) in the upper aerobic zone, followed by denitrification in the lower anoxic zone; (c) ammonia volatilization (into gaseous form), due to higher pH. The nitrogen removal percentages in waste stabilization ponds range between 46-95% (on annual basis) in ponds (US EPA, 1983).

Phosphorus removal in stabilization ponds is associated with algal, plant uptake, precipitation and sedimentation. Suspended aerobic microbes remove phosphorus from bulk liquid. Some of these microbes settle in the lower anaerobic zone of the ponds, and eventually die, and are anaerobically digested, causing release of some of the stored phosphorus in the bulk water. For stringent phosphorus discharge criteria, alum addition is necessary, and is applied 1-2 weeks prior to the pond discharge. Alum is usually dosed from the boat, to allow homogeneous mixing. For domestic loads of 4-11 kgBOD_5/ha.d, required alum concentrations often vary between 80-100 mg/L to achieve an effluent phosphorus concentration of <1 mg/L.

Box 10.1

Performance of facultative ponds at Pagla sewage treatment plant.

The sewage of Dhaka city is treated at Pagla sewage treatment plant, operated by Dhaka Water Supply and Sewerage Authority (DWASA). The treatment plant includes primary sedimentation tanks and facultative ponds. The ponds were designed to receive BOD and suspended solid concentrations 120 and 80 mg/L respectively (from primary sedimentation tanks), to produce effluent concentrations 50 and 60 mg/L respectively. Water quality measurement in 2009-2010 indicated that effluent BOD and suspended solid concentrations across the effluent of the ponds were 49 and 58 mg/L respectively, when subjected to influent concentrations 127 and 119 mg/L respectively from the primary tanks. The effluent concentration of the design parameters (across the ponds) was lower than the design values, indicating higher performances of these systems at Pagla sewage treatment plant.

10.4 Design of Facultative Ponds

The modelling of pond systems assumes complete mixing of solids (i.e.; CSTR flow pattern) without recycling, in the liquid portion. The solids that fall in the bottom are not resuspended, and the settling rates of solids (in ponds) cannot be quantified. As such the mass balance of solids cannot be computed; however, mass balance of soluble food can be quantified, assuming uniform mixing of soluble food throughout the reactor liquid. Assuming first order food conversion rate, the mass balance of soluble food can be written as:

$$BOD_{in} = BOD_{out} + BOD_{consumed} \tag{10.1}$$

$$QS_0 = QS + V(KS) \tag{10.2}$$

Equation (10.2) can be arranged as:

$$\frac{S}{S_0} = \frac{1}{1 + {KV}/{Q}} = \frac{1}{1 + K\theta} \tag{10.3}$$

Where S/S_0 = fraction of soluble BOD remaining
K = reaction rate coefficient, d^{-1}
θ = hydraulic detention time, d
V = reactor volume, m^3
Q = flow rate, m^3/d

For several reactors, that are arranged in series, the substrate balance can be written across a series of n reactors, as shown in Equation (10.4):

$$\frac{S}{S_0} = \frac{1}{1 + \left(\frac{K\theta}{n}\right)^n} \tag{10.4}$$

For the treatment of municipal wastewater, at least three ponds (of same size) are arranged in series. In such cases, the first pond, referred as primary pond retains most of the incoming solids, thus are heavily loaded. As such, primary ponds are often aerated, to hinder development of anaerobic environment.

With regards to the mixing pattern in ponds, a wider range of dispersion occurs because of reactor shape, size, wind action or aerators. As such, it is not pragmatic to consider complete mixing of flow. Thirumurthi (1969) recommended that ponds be designed as dispersed flow reactors, assuming a non ideal flow pattern between CSTR and plug flow regime, by proposing the use of pond dispersion numbers (d_f). The equations proposed by Thirumurthi (1969) are demonstrated below:

$$\frac{L_e}{L_i} = \frac{4ae^{\frac{1}{2d_f}}}{(1+a)^2 e^{\frac{a}{2d_f}} - (1-a)^2 e^{-\frac{a}{2d_f}}} \tag{10.5}$$

$$a = \sqrt{(1 + 4K_{BOD_D}\theta_f d_f)} \tag{10.6}$$

$$d_f = \frac{D}{vl} \tag{10.7}$$

$$K_{BOD_D} = K_{BOD_{D20}}(1.09)^{T-20} \tag{10.8}$$

Where L_e = effluent BOD, mg/L

L_i = influent BOD, mg/L

K_{BOD_D} = dispersed flow first order reaction rate for BOD removal at any temperature, d^{-1}

$K_{BOD_{D20}}$ = dispersed flow first order reaction rate for BOD removal at 20°C, d^{-1}

θ_f = mean hydraulic retention time in facultative pond, d

d_f = dispersion numbers

D = coefficient of longitudinal dispersion, m^2h^{-1}

v = mean velocity of travel, mh^{-1}

l = mean path length of a typical particle in the pond, m

T = minimum pond temperature, °C

The value of K differs with temperature, as illustrated in Equation (10.9):

$$K_T = K_{20}\theta^{T-20} \tag{10.9}$$

Values of K_{20} range from 0.2-1.0, whereas, temperature coefficient θ varies between 1.03-1.12.

To facilitate the use of the complicated Equation (10.5), Thirumurthi developed a graphical inter-relationship (Figure 10.3) between food removal, and $K\theta$ values for dispersion factors ranging from infinity for CSTR reactors, to zero for plug-flow reactors.

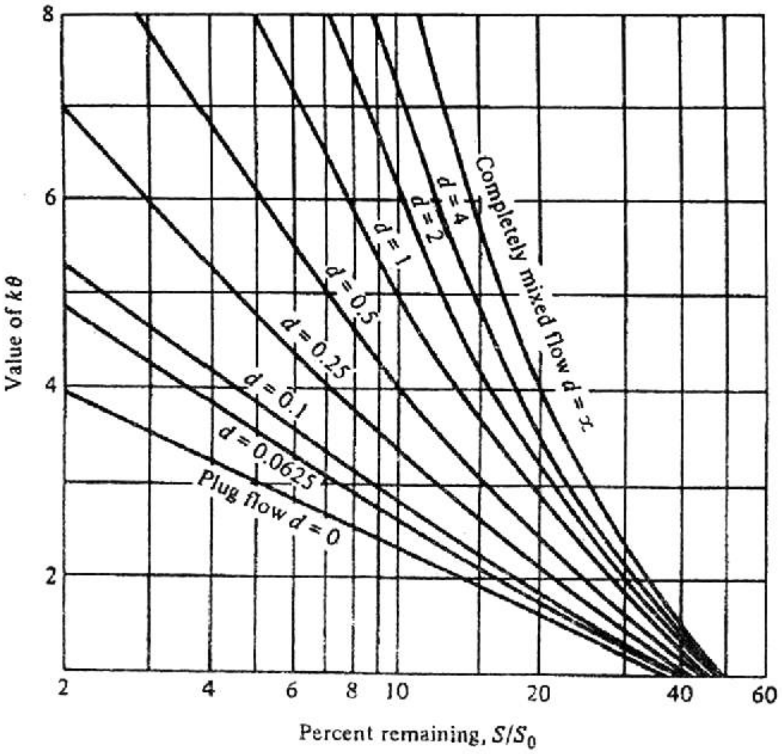


Figure 10.3 Graphical plot of the Thirumurthi equation.

Although some photosynthesis occurs in facultative lagoons, O_2 requirements are often met by aeration. As such, two kg of O_2 should be supplied for each kg of incoming BOD_5. The rate of O_2 transfer (from the aerator) often ranges between 0.3-2kg/kW.h. Table 10.3 illustrates typical design criteria of pond systems.

Table 10.3 Design criteria of pond systems.

Parameter	Design criteria
Short detention ponds	
Cell numbers (flow < 400 m^3/d)	2 minimum
(flow > 400 m^3/d)	4
Hydraulic detention time, d/cell	3 minimum
Depth, m	3 minimum
Sludge removal frequency, yr	5-10
Long detention ponds	
Cell numbers	2 minimum
Hydraulic detention time, d	60
Depth, m	1.5-2

Source: Heinke et al., 1991

10.5 Design Example

This section illustrates the design procedures of facultative ponds through a solved example.

Example 10.1. Facultative pond design. Wastewater flow from an area averages 4000 m^3/d during November (winter) and 6000m^3/d during June (summer). The average temperature of November is 10°C, and in June (summer) average temperature is 30°C. The mean concentration of influent BOD_5 is 300mg/L. Reaction coefficient K is 0.23 d^{-1} at 20°C, and θ is 1.06. Prepare a facultative pond treatment system for the area to remove 95% of the incoming BOD.

Solution

Step 1. Rate constants at summer and winter

From Equation (10.9): $K_T = K_{20}\theta^{T-20}$

Summer: $K_{30} = 0.23\,(1.06)^{30-20} = 0.41\ \mathrm{d}^{-1}$

Winter: $K_{10} = 0.23\,(1.06)^{10-20} = 0.13\ \mathrm{d}^{-1}$

Step 2. Determine $\frac{S}{S_0}$

If 95% of BOD has to be removed, then the percent remaining, $\frac{S}{S_0} = (100-95)\% = 5\%$

Step 3. Determine detention time θ

Assume dispersion factor = 0.5, and $\frac{S}{S_0} = 5$ (step 2), $K\theta$ can be measured from Figure 10.3.

From graph (Figure 10.3), $K\theta = 6$

$$\text{At summer, } \theta = \frac{6}{0.41} = 14.6 \text{ d}$$

$$\text{At winter, } \theta = \frac{6}{0.13} = 46.1 \text{ d}$$

Step 4. Calculate pond volume

Use longest detention time (i.e. in winter), θ=46.1 d

Volume of ponds, $V = \theta Q = 46.1 \text{ d} \times 4000 \text{ m}^3/\text{d} = 184400 \text{ m}^3$

Use three ponds, each with equal volume; as such, volume of each pond = 61467 m^3

Step 5. Calculate pond area

Use a pond depth of 1.5m.

$$\text{The area of each pond} = \frac{61467 \text{ m}^3}{1.5 \text{ m}} = 40978 \text{ m}^2$$

Add 1m depth for sludge storage in primary pond.

Step 6. Calculate aeration requirements

Assume, photosynthesis is not sufficient to meet O_2 requirements in the primary pond throughout the year. As such, aerators have to be provided, to meet O_2 demand.

$$\text{For summer conditions: } \theta = \frac{V}{Q} = \frac{61467 \text{ m}^3}{6000 \text{ m}^3/\text{d}} = 10.2 \text{ d}$$

$$K\theta = 0.41 \text{ d}^{-1} \times 10.2 \text{ d} = 4.18$$

Assuming dispersion factor to be 0.5, Figure 10.3 can be employed for calculating S/S_0

From Figure 10.3, with $d = 0.5$, $S/S_0 = 10\%$

BOD removed = $0.90 \times 300 \text{ mg/L} = 270 \text{ mg/L}$

O_2 supplied = $2 \times 0.27 \text{ kg/m}^3 \times 6000 \text{ m}^3/\text{d} = 3240 \text{ kg}O_2/\text{d}$

Assuming aerator transfer efficiency as 1 kg O_2/kW.h:

$$\text{Power of aerators} = 3240 \text{ kg}O_2/\text{d} \times \frac{1 \text{ d}}{24 \text{ h}} \times \frac{\text{kWh}}{1 \text{ kg } O_2} = 135 \text{ kW}$$

Use 5 aerators 27 kW each.

Step 7. Diagram of pond arrangements

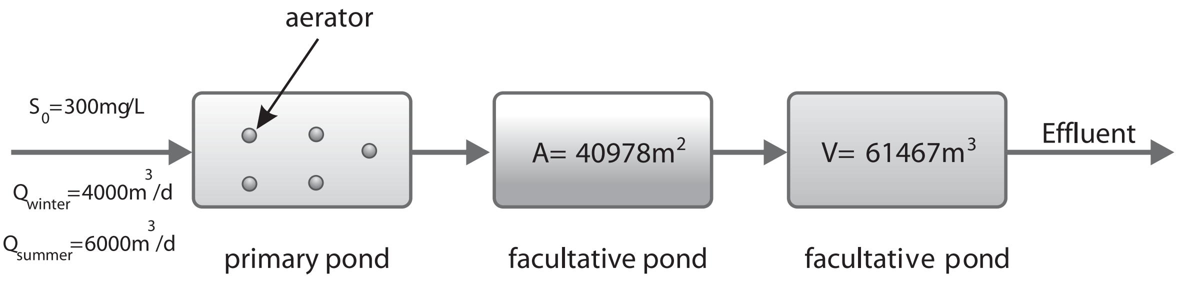


Figure 10.4 **Typical arrangement of ponds.**

10.6 Constructed Wetlands: Definition and Classification

Constructed wetlands are engineered wetlands that have saturated or unsaturated substrates, emergent/floating/submergent vegetations, and a large variety of microbial communities. The wetlands are built for water pollution control. Constructed wetlands can be classified into two groups: (a) surface flow; and (b) subsurface flow wetlands.

Surface flow wetlands are similar to natural wetlands, with shallow flow of wastewater (usually less than 60cm deep) over saturated soil substrate. Such wetlands are similar to natural wetlands, in terms of appearance. The pollutant mechanisms in surface flow systems include: sedimentation, filtration, oxidation, reduction, precipitation and adsorption. Figure 10.5 illustrates a diagram of surface flow wetland systems, employed for wastewater treatment.

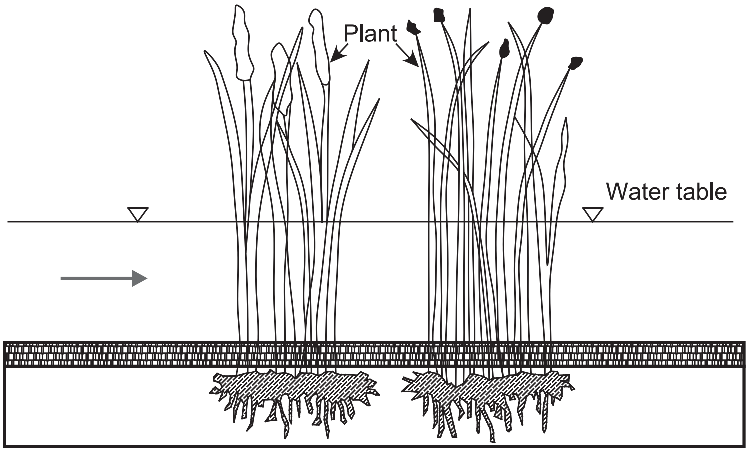


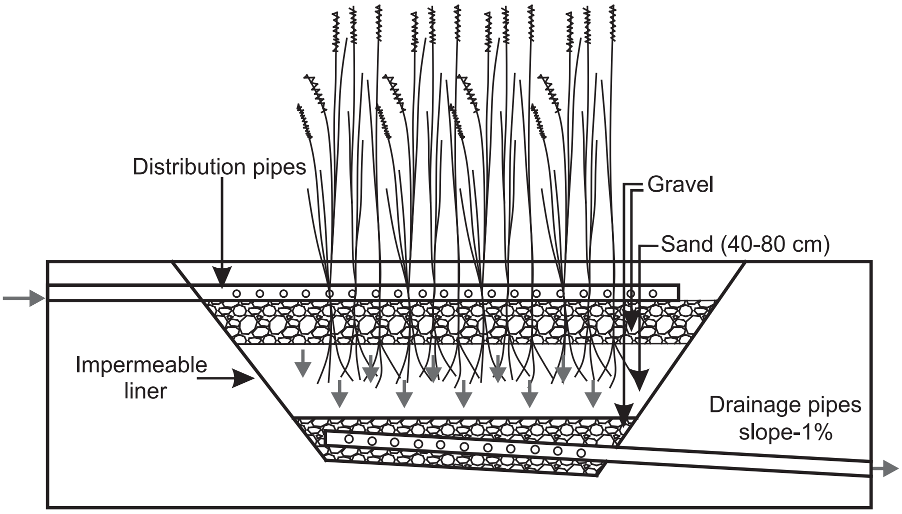
Figure 10.5 **Surface flow wetland systems for the treatment of wastewater.**

In subsurface flow wetland systems wastewater flows beneath the media surface, where it comes into contact with an inter-connected mesh of plants, media and attached biofilms. Depending on the flow pattern, subsurface flow wetlands can further be classified into: (a) vertical subsurface flow (VSSF); and (b) horizontal subsurface flow (HSSF) wetlands.

VSSF wetlands employ packed media with plants, and wastewater flows vertically downwards through the media (under the force of gravity) towards outlet. The main media is overlaid on large stones, to facilitate effluent drainage. The macrophytes (i.e. plants) are usually planted in coarse sand, which provides the top surface.

Wastewater is dosed intermittently into the bed surface of VSSF wetlands, allowing alternative wet and dry periods. During the interval between the successive doses the bed remains unsaturated, which allows diffusion of oxygen inside the media. When the wastewater is being dosed, it traps this pore air ensuring better oxygen transfer throughout the bed. As such, these systems promote higher oxygen distribution, effective for NH_4-N and BOD_5 removal from wastewater.

Figure 10.6 provides a graphical and pictorial view of VSSF systems for wastewater treatment.

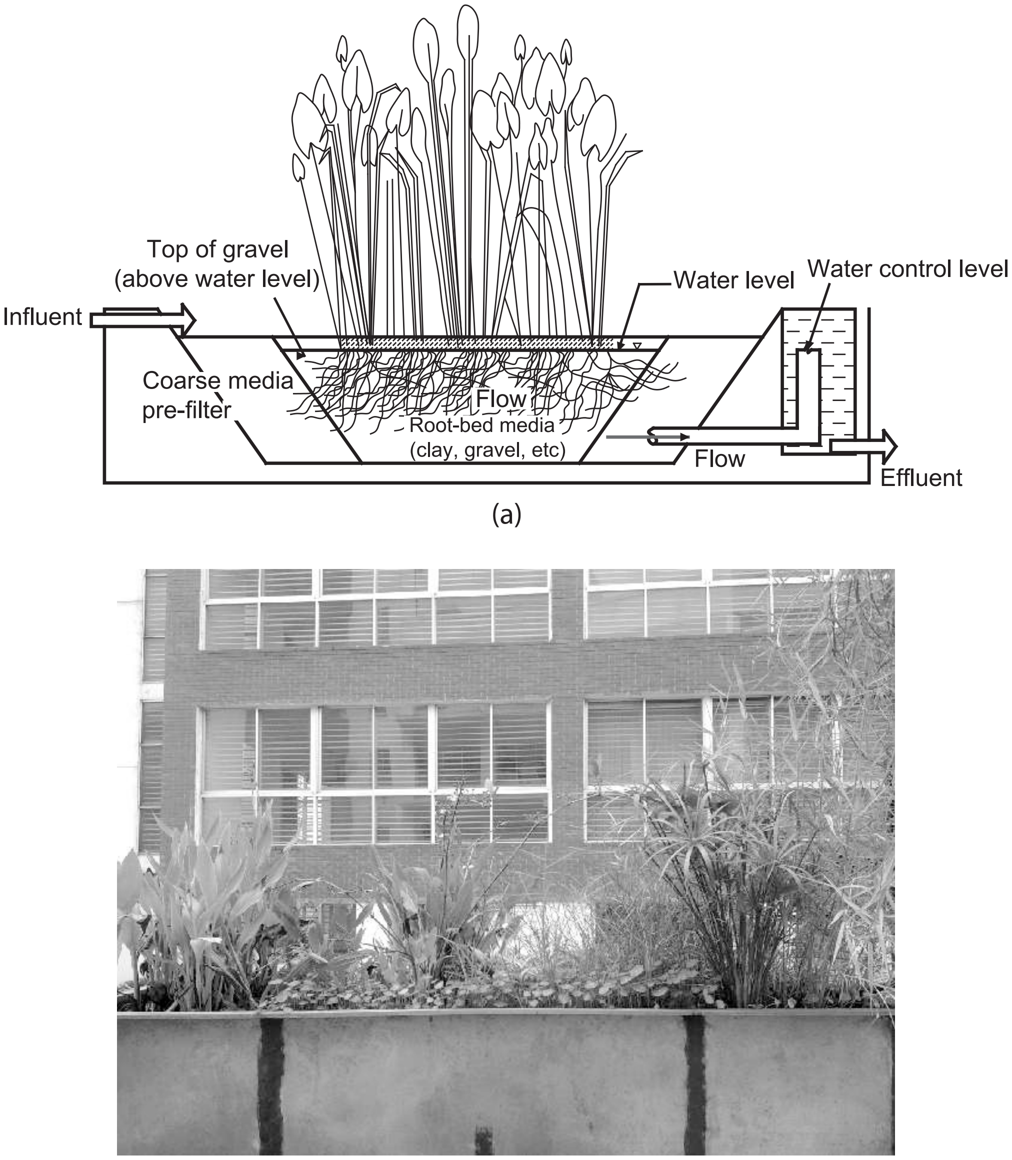


(a)

(b)

Figure 10.6 **(a) Schematic diagram of VSSF systems; and (b) photograph of VSSF systems (source: http://sustainableengineeringresearch.wordpress.com/experimental-projects/)**

In HSSF systems, the media is kept saturated (Brix 1994); the outlet is generally located at 0.6 m height from the bottom of the tank. Wastewater flows horizontally from inlet towards outlet, beneath the media surface. During this passage of water, it comes into contact with a network of media, attached microorganisms and root zones. Since these wetlands are operated under saturated conditions, anoxic-anaerobic environments are predominant throughout the media, fostering denitrification. Figure 10.7 provides the components and flow path of HSSF systems, along with a photo plate.



(a)

(b)

Figure 10.7 **(a) Schematic diagram of HSSF systems; and (b) photograph of HSSF systems.**

A combination of VSSF-HSSF or HSSF-VSSF systems, known as hybrid wetland systems can also be employed for wastewater treatment. These systems are effective, in terms of utilizing aerobic and anaerobic phases of VSSF and HSSF systems respectively, thereby achieving higher pollutant removal rates from wastewater (Vymazal, 2005).

Figure 10.8 shows three parallel hybrid wetland systems. Each system consists of a VSSF system, followed by a HSSF system, to provide treatment of wastewater.

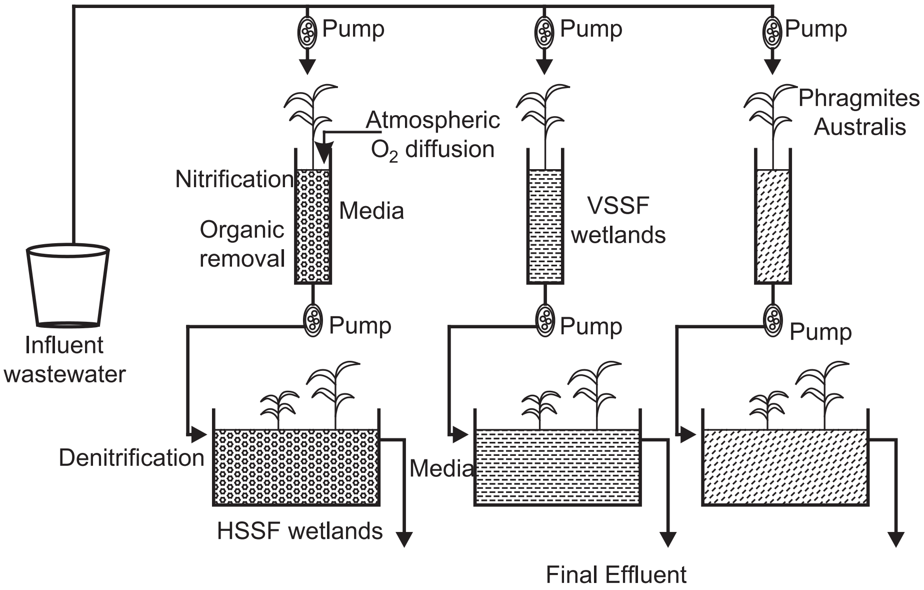


Figure 10.8 Hybrid wetland systems.

Table 10.4 indicates advantages and disadvantages of VSSF and HSSF wetland systems, when employed for wastewater treatment.

Table 10.4 Advantages and disadvantages of VSSF and HSSF wetlands.

Type	Advantages	Disadvantages
Vertical flow wetlands (VSSF)	Smaller area demand.	Short flow distances.
	Good oxygen supply, good nitrification, better organics and solids removal, simple hydraulics.	Poor denitrification, higher technical demand, low nitrate removal.
	Higher purification from the beginning, better than HF beds as water flows from surface to bottom which enhances oxygen mixing.	Loss of performance in P removal.
Horizontal flow wetlands (HSSF)	Long flowing distance, nutrients gradients can be established, efficient in the removal of solids, organics.	High area demand, clogging problem is observed, sulphur transformation can affect nitrification sensitivity.
	Denitrification possible.	Careful calculation of hydraulics necessary for optimal oxygen supply, low ammonium oxidation.
	Formation of humic acids for N, P removal.	Equal wastewater supply is complicated.

10.7 Components of Treatment Wetlands

Plants, microorganisms, and wetland media are the main components of constructed wetlands. These elements form an inter-connected network providing physical, chemical and biological treatment to the flowing wastewater. The following sections provide brief descriptions on these components.

Plants. Wetland plants (macrophytes) typically grow in water or soil media, subjected to oxygen deficiency. The macrophytes employed in the constructed wetlands can be divided into four groups (Williams, 1964):

- **Emergent macrophytes:** These are usually seen on water saturated or submerged soil; examples including: *Acorus calamus, Carex rostrata, Phragmites australis, Scirpus lacustris, Typha latifolia*. Oxygen is transferred from their roots into surrounding rhizosphere, to facilitate aerobic bacteria degradation (Moshiri, 1993).
- **Floating leaved macrophytes:** These are rooted in submersed sediments having water depth 0.5m-3.0m, and have either floating or slightly aerial leaves; examples include *Nymphaea odorata*, and *Nuphar lutea*.
- **Submerged macrophytes:** These have their photosynthetic tissue submersed by water, grow well in oxygenated water and are mainly used for polishing secondary treated wastewater (Moshiri, 1993).
- **Freely floating macrophytes:** They freely float on surface water. These plants are capable of removing N and P by incorporating those into plant biomass, by nitrogen denitrification, and they also remove solids; examples including: *Lemna minor, Spirodela polyrhiza, Eichhornia crassipes*.

Table 10.5 provides the characteristics of common aquatic plants of wetland systems.

Table 10.5 **Commonly used aquatic plants in constructed wetlands.**

Common name *Scientific name*	**Desirable temp. °C**	**Seed germination °C**	**Salinity tolerance ppt***	**Optimal pH**	**Root penetration cm**
Cattail *Typha*	10-30	12-24	30	4-10	30
Common Reed *Phragmites*	12-33	10-30	45	2-8	60
Rush *Juncus*	16-26	---	20	5-7.5	---
Bulrush *Scirpus*	16-27	---	20	4-9	76
Sedge *Carex*	14-32	---	---	5-7.5	---

**ppt= parts per thousand.*

In Bangladesh the macrophyte species that can be employed in the wetland systems are listed below (according to their types) (Saeed et al., 2014).

- **Emergent macrophytes:** Phragmites australis, Cyperus papyrus, Cyperus difformis, Dracaena sanderiana, Hydrocotyle umbellate, Echinodorus cordifolius, Colocasia esculenta, Caladium sp., Hymenocallis littoralis, Canna indica, Alternanthera philoxeroides.

- **Submerged macrophytes:** Hygrophila polysperma, Bacopa caroliniana.
- **Rooted floating macrophytes:** Nymphea pubescens, Nymphoides indica.
- **Floating leaved macrophytes:** Salvinia auriculata, Salvinia minima, Pistia stratiotes.

Microorganisms. Constructed wetlands provide an ideal environment to support the growth of microorganisms, which break down wastewater pollutants by biodegradation. Bacteria, Fungi, Algae are the predominant microorganisms of the wetlands. A brief description of the microorganism characteristics is available in Chapter 6.

Media. Wetland media play a critical role in pollutant removal from the wastewater, as the media provides attachment surfaces to microbial communities, and ingredients for bio-reactions. The nature of wetland media is also an important factor to determine environmental condition (such as redox potential), inside the wetland matrix.

Gravel is the most common media employed in subsurface flow wetlands. Typical gravel sizes used in HSSF wetlands are: 3-6 mm, 5-10 mm, and 6-12 mm. VSSF systems usually use graded gravel with a top layer of washed sharp sand. The media depth in VSSF wetlands is often maintained around 0.7 m, whereas in HSSF systems such depth is maintained around 0.6 m. According to UK specification the arrangement of gravel media (in VSSF wetlands) is given in Table 10.6 (WRc, 1996).

Table 10.6 Size of gravel beds used in VSSF wetlands.

Layer	Depth	Substrate size
Top Layer	8 cm	Sharp sand
	15cm	6 mm washed pea gravel
	10 cm	12 mm round washed gravel
Bottom Layer	15cm	30-60 mm round washed gravel

Apart from gravel, other types of media can also be used in wetland systems to enhance pollutant removal mechanisms. Table 10.7 indicates different unconventional media, along with pollutant removal efficacy (employing such media), in wetland systems.

Table 10.7 A list of unconventional media employed in wetland systems.

Literature source	Media Used	Pollutant Removal Efficiency (%)							
		NH_4-N	NO_2-N	NO_3-N	TN	TP	TSS	BOD	COD
Effert et al., 1985.	Sand	-	-	-	1-11	-	-	100	-
Wood and McAtamney 1996.	Laterite	-	-	-	-	96	-	-	-
Drizo et al., 1997.	Shale	100	-	85-95	-	99	-	-	-
Ubranc, 1997.	Mixed gravel, sand, peat, soil	41	-	98	-	63	53	4	47

Literature sourced	Media Used	Pollutant Removal Efficiency (%)							
		NH_4-N	NO_2-N	NO_3-N	TN	TP	TSS	BOD	COD
Gray et al., 2000.	Maerl	-	-	-	41-59	98	-	-	-
Brix et al., 2001.	Darup natural sand	-	-	-	-	>80	-	-	-
	NymØlle and SorØ sands	-	-	-	-	70	-	-	-
	Diatomaceous earth	-	-	-	-	60	-	-	-
	Marble	-	-	-	-	70	-	-	-
Bastviken et al., 2003.	Peat	Summer: 55 Winter: 80	-	55 20	44 20	- 70	-	-	-
Aslam et al., 2007.	Compost	-	-	-	-	-	48-73	35-77	45-78
Gunes, 2007.	Peat	-	-	-	58	40	95	-	92
Chan et al., 2008.	Coal slag	51	-	-	69	23-38	78	-	-
Yalcuk and Ugurlu, 2009.	Zeolite	71	-	-	-	56.2	-	-	23
Zhao et al., 2010.	Alum sludge	49-93	-	-	11-78	75-94	46-83	57-84	36-84
Saeed and Sun, 2011 a, d.	Wood-mulch	84-99.6	-	-	57-98	22.8-60.3	14.1-55	36-71. 6	-

10.8 Pollutant Removal Mechanisms in Wetland Systems

Wetlands are capable of removing pollutants from wastewater through a complex variety of physical, chemical and biological processes. In terms of biodegradation pathways, the wetlands are more diverse than conventional biological treatment systems (WRc, 1996). Major pollutant removal mechanisms often observed in wetlands include: a) settling and filtration of suspended particles; b) precipitation and chemical transformation to remove nutrients and metals; c) adsorption of nutrients on the surfaces of plants, substrates, sediment, etc; d) microbial transformations that remove organic matter and nutrients; and e) predation and natural die-off that remove pathogens (Jayakumar and Dandigi, 2003). The pollutants that can be removed from wastewater employing wetland systems include: nitrogen, organics, phosphorus, solids, coliforms and metals.

Nitrogen (N). Major pathways of nitrogen transformations, which ultimately remove N from the wastewater in constructed wetlands include: ammonification, biological nitrification, denitrification, and plant uptake (Vymazal, 1995), as illustrated in Figure 10.9.

Ammonification involves conversion of organic nitrogen to NH_4-N. In a constructed wetland, this process decreases with depth, indicating that ammonification is faster in the

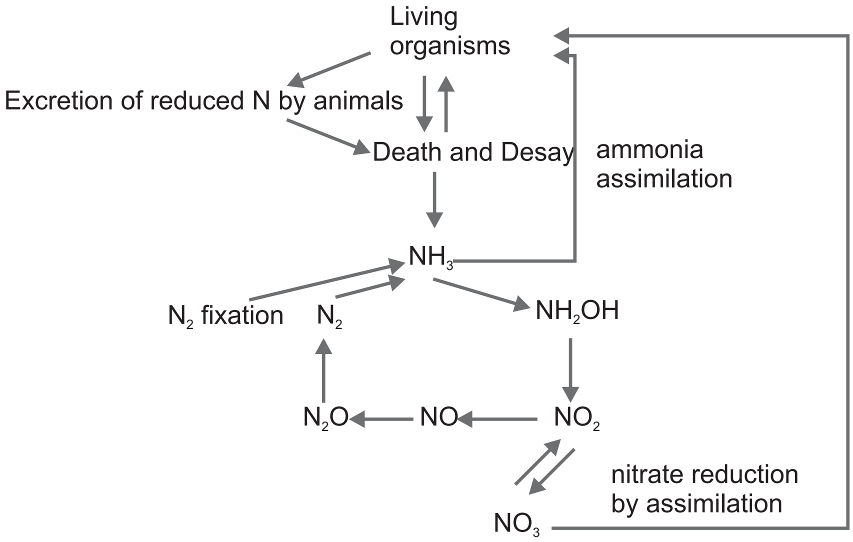


Figure 10.9 **Nitrogen cycle in wetland systems.**

upper zone of the wetlands where the condition is aerobic, and slower in the lower zone where the environment switches from facultative to obligate anaerobic conditions (Reddy and Patrick, 1984). The ideal pH range for ammonification is 6.5-8.5 (Patrick and Wyatt, 1964; Vymazal, 1995). Ammonification proceeds faster at higher temperature, doubling rate with temperature increase of 10˚C (Kadlec and Knight, 1996).

Nitrification and denitrification (Chapter 7) is the major nitrogen removal mechanisms in subsurface flow wetland systems (Bachand and Horne, 2000; Reilly et al., 2000). The VSSF systems are efficient in terms of converting NH_4-N to NO_3-N, due to predominant aerobic conditions of such systems. The HSSF systems are capable of reducing NO_3-N to N_2 gas, as a result of anoxic conditions inside the bed media. As such, a combination of VSSF, followed by HSSF system is very effective, for achieving substantial N removal.

Presence of macrophytes is essential for wetlands, in terms of improving nitrogen removal performances because they: (a) provide surfaces and oxygen for the growth of microorganisms, enhancing nitrification (Langergraber, 2005); and (b) supply carbon necessary to optimize denitrification process (Masi, 2008). Nitrogen uptake by plants differs according to the system configurations, loading ranges, wastewater types and environmental conditions; the contribution of plants, in terms of nitrogen removal had been reported within the range 0.5- 40.0% of the total nitrogen removal (Drizo et al., 1997; Shamir et al., 2001; Healy and Cawley, 2002; Meers et al., 2008; Kantawanichkul et al., 2009). In addition, plant biomass can also contribute to nitrogen removal; a research study by Shamir et al., (2001) reported 60% accumulation of total N in plant biomass. However, decaying plant materials can also contribute to increase of nutrient concentration across the effluent through leaching (Brodrick et al., 1988).

Minor nitrogen removal routes such as: ammonia volatilization and adsorption can also contribute to nitrogen removal in wetland systems. Ammonia volatilization depends on pH of wastewater; an increase of pH (above 9.3) can cause volatilization (WRc 1996). Adsorption occurs through cation exchange, with detritus and inorganic sediments in surface flow wetlands and with the media in subsurface flow wetlands. However, the adsorbed ammonia is bound loosely to the substrate and can go back to the bulk water column, if the concentration of NH_4-N is reduced and vice versa (Kadlec and Knight, 1996).

Apart from these nitrogen removal routes, a limited studies illustrated novel nitrogen removal routes in constructed wetlands, such as: Anammox (Dong and Sun, 2007) and Canon process (Sun and Austin, 2007). However, to date researchers have not been able to consistently implement these novel biodegradation routes in the wetlands, due to lack of knowledge on optimal operational conditions, and environmental parameters associated with such pathways.

Organics. Aerobic degradation of organics is often observed in VSSF wetlands, due to presence of oxygen inside the bed media. Consequently, dominancy of anaerobic condition in HSSF wetlands allows the removal of organics through anaerobic pathways (Garcia et al., 2010; Zhang et al., 2010).

The removal of organics in wetland systems is also dependent on input loading. Current literature reports increase of organics removal with loading, up to 1.14-14 $g/m^2.d$; above this loading limit, the organics removal rate can decrease (Coombes 1990; Ruan et al., 2006). In addition, temperature and hydraulic retention time can impose severe effect on organics removal in constructed wetlands.

The removal of nitrogen and organics often follows contradictory pathways in constructed wetlands. Nitrification process is usually inhibited, until BOD concentration drops below 200 mg/L (Sun et al., 1998), due to slower metabolism of nitrifiers (Grady et al., 1999). Subsequently, higher organics removal rates limit the availability of organic carbon, for NO_3-N denitrification. As such, to promote simultaneous organics removal and nitrification- denitrification, artificial aeration can be provided coupled with carbon addition, externally or internally (i.e. by providing organic media).

Box 10.2

Internal generation of carbon in VSSF wetland systems (Saeed and Sun, 2011 a; d).

A synthetic domestic wastewater was treated in five lab-scale VSSF wetland reactors, which had *Phragmites australis* planted in different types of media: organic wood mulch, and mixture of gravel-wood mulch media. The VSSF systems with organic mulch media exhibited higher nitrogen removal. The total nitrogen removal varied within 72-97.8%, where as BOD removal efficiency ranged between 24%-71%. Such higher denitrification rates could be attributed to the catering of organic carbon from the organic mulch media (i.e. internal generatrion of carbon), thereby facilitating nitrogen removal from wastewater. These findings illustrated the possibilities of simultaneous nitrification-denitrification, and organics removal in a single wetland reactor.

Phosphorus (P). Phosphorus in constructed wetlands is primarily removed via substrate retention/sorption, precipitation/ dissolution, and plant/microbial uptake (Vymazal, 2007). Sedimentation is also an important mechanism, for the removal of particulate inorganic and organic P (Liikanen et al., 2004).

Adsorption of P (by media) in constructed wetlands is observed, when the substrate is rich with Fe or Al (Luderitz and Gerlach, 2002; WRc, 1996). However, for iron rich materials, aerobic condition has to be maintained to prevent release of P into the bulk liquid (Noorvee et al., 2007). Microbial uptake of P includes rapid uptake but low storage (Vymazal, 2007).

The wetland macrophytes also contribute to P removal (from wastewater). The uptake capacity of P by macrophytes has been estimated within 30-150 kgP/ha/yr. Current literature reports removal percentages of P (by wetland plants), within a range of 1.8-39% (Healy and Cawley, 2002; Meers et al., 2008). P uptake by plants in constructed wetlands may dominate, when the wetland substrate is subjected to P saturation.

Solids. Solids from wastewater is primarily removed via filtration (by the packed media of VSSF, HSSF systems), or through settling of particles. However, if the influent wastewater comprises of higher amount of non-degradable particles, they can accumulate in the media pores (Tuszynska and Pempkowiak, 2008), resulting clogging of beds. Such phenomenon is undesirable for wetland systems, as it can cause: (a) blockage of media porosity; (b) reduction of hydraulic conductivity of the bed; and (c) decrease of flow retention time, resulting short-circuiting. Clogging also prevents oxygen transportation from atmosphere by diffusion and convection process into the substrates of wetlands, thereby enhancing the failure of the treatment systems (Langergraber et al., 2003).

To date, no mathematical formula has been formulated to prevent clogging of wetland systems. Platzer and Mauch (1997) reported to limit input COD loading rate within 25 g COD/m^2 d, for avoiding clogging. Bed resting (i.e. non-dosing) can also minimize clogging, as the resting period promotes the degradation of organic materials, accumulated inside the media.

Coliforms. The removal of coliform bacteria in constructed wetlands is usually achieved by three processes: (a) physical (filtration, sedimentation, and UV radiation); (b) chemical (oxidation, adsorption, and exposure to toxins from other microbes); and (c) biological processes (antibiosis, ingestion by nematodes and protozoa, and natural die off) (Ottova et al., 1997). However, in subsurface flow wetlands light is not the main factor of bacterial removal, and is governed by predation of bacteria (Davies and Bavor, 2000), performed by protozoa, lytic bacteria, and bacteriophages (Stottmeister et al., 2003).

Coliform removal efficiencies in wetlands depend on several factors, such as: substrate, presence of plants, temperature, flow of wastewater, and maturity of the root system. Bacterial removal efficiency in wetlands is also a function of the inflow bacterial population, which indicates that higher inflow populations fosters greater removal efficiencies (>90% for coliforms, and >80% for fecal streptococcus without disinfection) (Kadlec and Knight, 1996). However, it should be noted that the effluent concentrations of bacterial populations can never become zero (Kadlec and Knight, 1996) without disinfection, as majority of the wetlands are exposed to wildlife.

Figure 10.10 shows a conceptual diagram of pollutant removal mechanisms, in VSSF and HSSF systems. Subsequently, Table 10.8 summarises common pollutant removal routes in wetland systems.

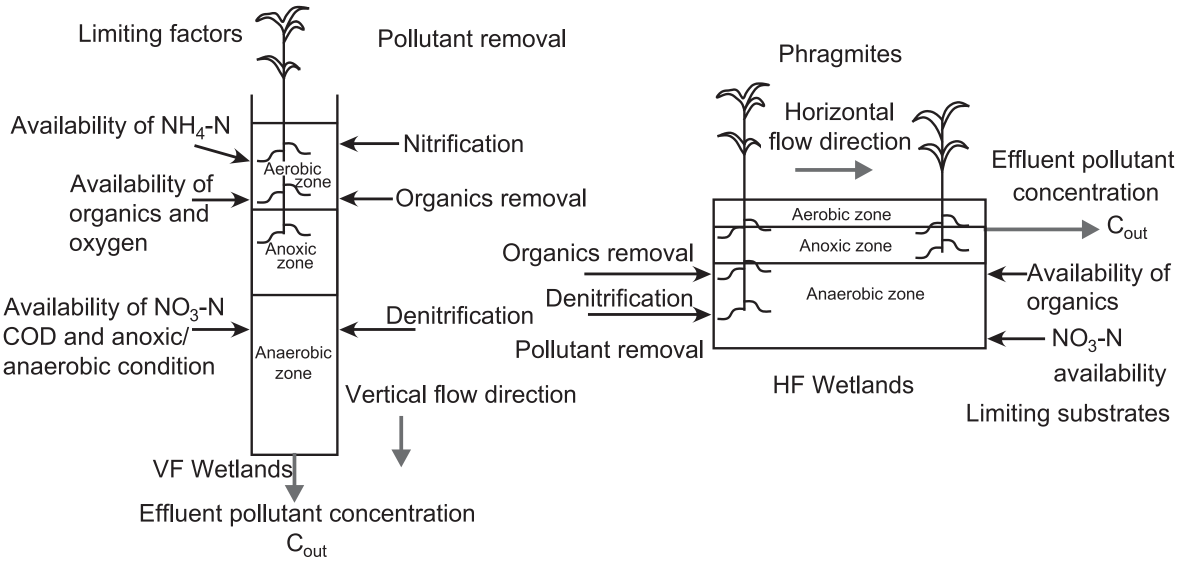


Figure 10.10 Conceptual diagram of pollutant removal routes in VSSF and HSSF systems.

Table 10.8 Common routes of pollutant removal in wetland systems.

Wastewater constituent	Removal Mechanisms
Nitrogen	Ammonification followed by nitrification-denitrification
BOD	Sedimentation, aerobic/anaerobic degradation
Phosphorus	Sorption by soil and plant uptake
Suspended solids	Sedimentation/ filtration
Pathogen	Sedimentation, filtration, natural die-off, UV radiation, toxic effect from plant roots

Tables 10.9-10.10 illustrate the pollutant removal efficacy of the wetland systems, employed in different parts of the world (Vymazal et al., 2006).

Table 10.9 Treatment efficiencies of vegetated HSSF constructed wetlands-worldwide experience.

	Concentration (mg/L)			Loading (kg/ha/d)			
	Inflow	*Outflow*	*Removal %*	*Inflow*	*Outflow*	*Removal rates*	*Efficiency %*
BOD_5	108	16	82.5	39.2	7.6	31.6	80.6
COD	284	72	74.6	120	34.6	85.4	71.2
TSS	107	18.1	83.1	53.6	11.6	42	78.4
P	8.74	5.15	41.1	1410	960	450	31.9
TN	46.6	26.9	42.3	6440	3940	2500	38.8
NH_4-N	38.9	20.1	48.3	3880	2550	1330	34.3
NO_3^--N	4.38	2.87	34.3	980	670	310	31.6

Table 10.10 **Treatment efficiencies of vegetated VSSF constructed wetlands-worldwide experience.**

	Concentration (mg/L)			Loading (kg/ha/d)			
	Inflow	*Outflow*	*Removal %*	*Inflow*	*Outflow*	*Removal rates*	*Efficiency %*
BOD_5	145	27.2	81.6	93.2	8.6	87.5	85.3
COD	303	75	71.3	158	46	112	71.2
TSS	97	18.4	77.5	92	12.7	79.3	82.8
P	8.6	4.4	52.6	1390	700	690	55.2
TN	61	35	39.7	11020	5370	5650	50.4
NH_4-N	45.6	15.8	65.1	8490	2190	6300	62.2
NO_3^--N	1.55	15.1	-16	260	3390		-8

10.9 Influence of the Environmental Factors

Since constructed wetlands are natural treatment technologies, they are critically influenced by environmental parameters, such as oxygen and temperature as illustrated below.

Presence of oxygen. Presence of oxygen is extremely critical for wetland systems for achieving nitrification and organics removal. VSSF wetlands promote more oxygen diffusion than horizontal flow constructed wetlands, and has the capability to transfer oxygen at a rate of 50-90 g O_2/m^2.d. In contrast, HSSF systems are water logged, thereby hindering the diffusion of atmospheric oxygen inside the media.

The most common wetland macrophytes (i.e. *Phragmites australis*) has the ability to pass oxygen, from its leaves through stems and rhizomes and out from its fine hair roots into the root zone or rhizosphere . The literature reports the oxygen contribution by the plants within a range 0.02- 12g O_2/m^2 (Bavor et al., 1988; Armstrong et al., 1990; Brix and Schierup, 1990).

Temperature. Theoretically, temperature has a significant effect on the intensity of microbial activities. Denitrification has been found to proceed slowly at low temperatures (e.g. 5°C), and its reaction rate increases exponentially with higher temperature and reaches a plateau between 20-25°C, as long as other environmental factors do not restrict the rate (U.S. EPA, 1975). Katayon et al., (2008) found that temperature between 30-32°C is beneficial for nitrification in constructed wetlands. Other research studies (Ruan et al., 2006; Akratos and Tsihrintzis, 2007) also documented the negative impact of lower temperature, on nitrogen and organics removal routes in treatment wetlands.

10.10 Design Guidelines for the Wetland Systems

According to Brix and Arias (2005), the design guidelines of HSSF and VSSF wetlands for treating domestic sewage are given below:

HSSF wetlands:

- Minimum area for a single household is 25 m^2.

- The minimum length of the system should be 10 m.
- The bottom of the bed should have a slope of 10% from inlet to outlet.
- Inlet depth should be 0.6 m and should be deeper towards the outlet.
- Inlet and outlet zones should be filled with stones to prevent exposure of wastewater.
- 0.5 mm thick membrane should enclose the whole system, and the membrane must be protected by a geotextile or sand.
- The substrate must be uniform sand with d_{10} between 0.3-2 mm and d_{60} between 0.5-8 mm; the uniformity coefficient should be <4.
- *Phragmites australis* should be planted into the system.

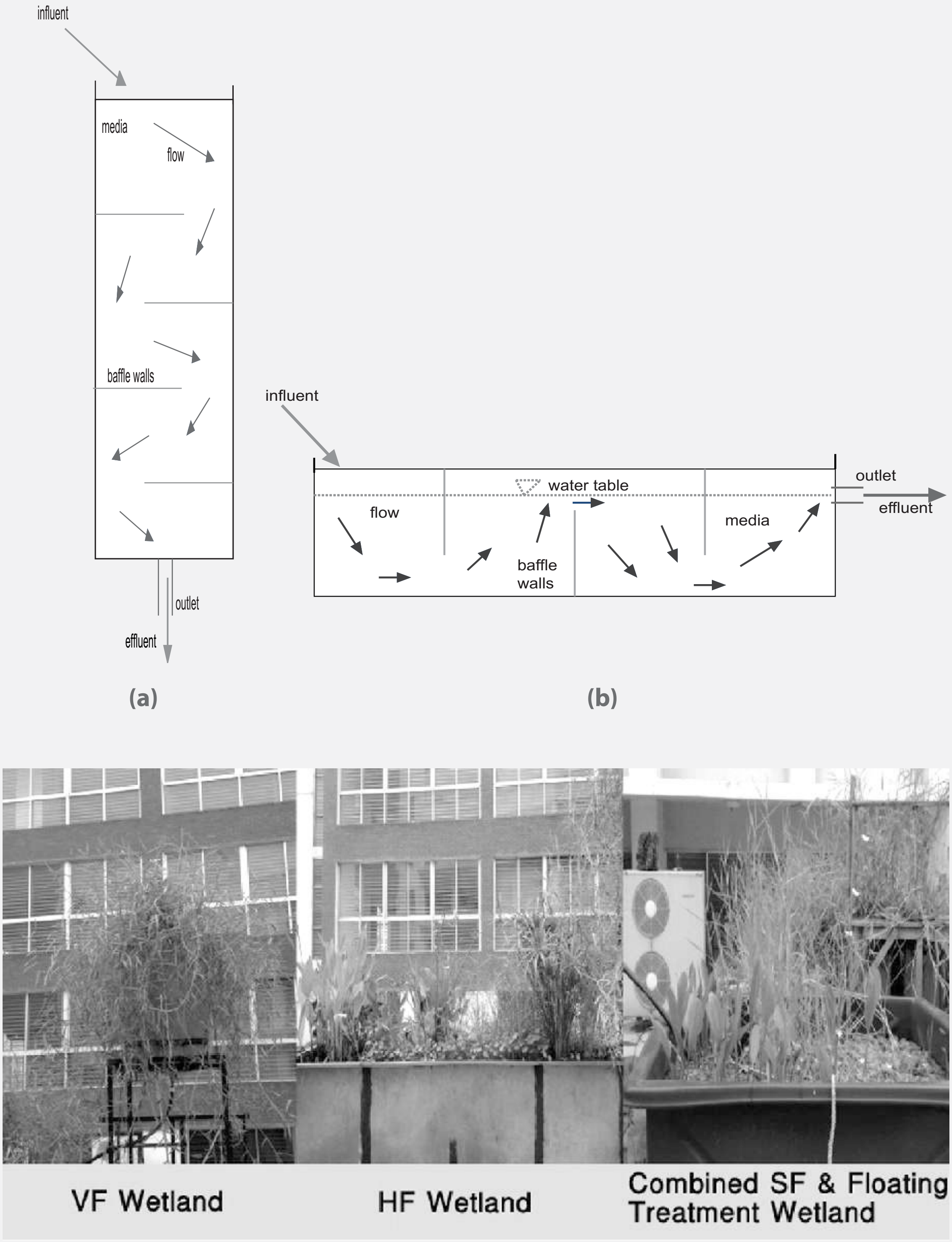
VSSF wetlands:

- The sewage must be pre-treated in a two or three chamber sedimentation tank (minimum volume 2 m^3 for a single household with up to 5 population equivalent).
- The necessary surface area of the root zone system is 3 m^2 per population equivalent (minimum area for a single household is 15 m^2).
- Filter depth should be 1.0 m with sand medium (d_{10} between 0.25-2 mm and d_{60} between 1-4 mm and the uniformity coefficient, $U = d_{60}/d_{10}$ should be <3.5), clay and silt contents (less than 0.125 mm) must be less than 0.5%.
- Minimum 0.5 mm thick membrane should enclose the whole system; the membrane must be protected by a geotextile or sand.
- *Phragmites australis* should be planted into the system to prevent filter clogging.
- Wastewater should be distributed evenly on the beds with distribution pipes insulated by a 0.2 m layer of coarse wood chips, sea shells on the surface of the filter to protect them against frost.
- Loading frequency should be 16-24 pulses per day, when half of the water is recirculated within the system.

Box 10.3

Baffled constructed wetlands for municipal wastewater treatment in Bangladesh (Saeed et al. 2014).

A three stage wetland system train was employed by Saeed et al. (2014) to provide treatment of municipal wastewater in Dhaka, Bangladesh. The wetland train included a baffled VF wetland as the first stage (Figure 10.11 a), followed by a second stage baffled HF wetland (Figure 10.11 b), and a final stage integrated SF- floating treatment wetland (Figure 10.11 c). Different types of organic and inorganic media were employed in the systems, that were planted with nineteen macrophyte types. The system was operated under hydraulic loading and seasonal variation. Wastewater sample analyses across inlet-outlet (of the whole system) indicated substantial nitrogen and organics removal performances. Incoming NH_4-N, NO_3-N, BOD_5 and COD concentrations were 107.5, 115.5, 1903.0, and 4048.0 mg/L respectively. Removal percentages of such pollutants were 82.3, 89.6, 97.0, and 94.4% respectively. The wetland train also achieved 99.9% E. Coli removal percentages from municipal wastewater. Such enhanced performances could be attributed to the flow direction by the baffle walls (integrated inside the systems), through aerobic-anaerobic pockets of the media stimulating biodegradation.



(a) (b)

(c)

Figure 10.11 Pilot scale (a) baffled VF wetland; (b) baffled HF wetland; and (c) pictorial plate of three stage wetland systems for the treatment of municipal wastewater in Bangladesh.

10.11 Modelling of Nitrogen and Organics Removal in VSSF and HSSF Systems

Currently, the kinetic modelling and system design of subsurface flow wetlands is primarily based on first order plug flow model, commonly referred as Kickuth equation (Cooper et al., 1996), according to Equation (10.10).

$$A = \frac{Q\,(\ln C_{in} - \ln C_{out})}{K} \qquad 10.10$$

Where, A = media area, m^2
K = first order rate constant, m/d
C_{in} *and* C_{out} = influent and effluent pollutant concentration, mg/L
Q = hydraulic loading, m^3/d

However, the accuracy of the Kickuth equation is now being questioned by the wetland researchers, due to the following reasons: (a) the first order models assume indefinite pollutant removals in a biological reactor, whereas maximum removal rates in such systems are often observed; and (b) the postulation of plug flow may not be appropriate, due to the packed media; it is likely that the flow is being diverged in all directions by the media when the wastewater flows towards outlet, particularly in the vertical flow wetlands.

In contrast to the Kickuth equation, complex wetland models (Mayo and Bigambo, 2005; Akratos et al., 2009) are often not practical for the design of treatment wetlands, due to the presence of many empirical relations. A complex model with many guessed parameters is susceptible to inaccurate design outcome. As such, further development of wetland models is desirable to overcome the inaccuracy of Kickuth equation, and the impracticality of complex models.

A balanced approach between the simplistic kinetic approach, and complex mechanistic models have been proposed by Saeed and Sun (2011b; c), for modelling nitrogen and organics removal in wetland systems. In these studies, the authors combined Monod kinetics with CSTR and plug flow pattern, for matching nitrogen and organics removal performances in VSSF and HSSF wetland systems respectively. The following sections describe such modeling approaches, to describe biodegradation routes of pollutants in wetland systems.

Modelling for VSSF systems. Monod kinetics, which contains the parameters of substrate concentration (C_{out}), half saturation constant of limiting substrate (C_{half}, mg/L) and maximum volumetric pollutant removal rates (K_{max}, g/m^3/d), can be expressed as:

$$\frac{dC}{dt} = -K_{max}\frac{C_{out}}{C_{half} + C_{out}} \qquad 10.11$$

CSTR flow pattern in a reactor can be expressed as:

$$\frac{dC}{dt} + \frac{1}{\tau}C_{in} = \frac{1}{\tau}C_{out} \qquad 10.12$$

Where, C_{in} = inlet pollutant concentration (mg/L)
τ = hydraulic retention time (days)

Combining volumetric maximum pollutant removal rates ($K_{max,}$ g/m^3/d) with media height (h) and porosity (ε) results areal maximum pollutant removal rates (K, g/m^2/d), as expressed in equation (10.13), to correlate inlet and outlet pollutant concentrations (Saeed and Sun, 2011b; 2011c).

$$K = \frac{q\left(C_{in} - C_{out}\right)\left(C_{half} + C_{out}\right)}{C_{out}} \qquad 10.13$$

Equation (10.13) can be used to predict nitrogen and organics degradation in VSSF systems. For the first step of NH_4-N transformation (i.e. from NH_4-N to NO_2-N) during nitrification, the half saturation constant value (C_{half} for *Nitrosomonas* degradation) has been measured as 0.05 mg/L (Verstraete and Vaerenbergh, 1986). As such, 0.05 mg/L can be used as the C_{half} value in Equation (10.13) for nitrification. For NO_3-N denitrification, half saturation nitrate constant in the Monod kinetics has been reported as 0.14 mg/L (Wiesmann, 1994). For heterotrophic biodegradable organics (BOD_5) removal, half saturation degradation constant is recommended as 60 mg/L for wastewater treatment (Metcalf and Eddy, 2003). For COD, half saturation COD constant can be taken as 20 mg/L, as suggested by Vaccari et al., (2006) for sewage treatment.

Figure 10.12 illustrates the correlation plot of Equation (10.13), for modelling nitrogen and organics removal in nine VSSF wetland systems (Saeed and Sun, 2011 b; c), evaluated by statistical parameter coefficient of determination (R^2). As observed in Figure 10.12, Monod kinetics with CSTR flow pattern showed closer interrelationship, for predicting nitrogen and organics biodegradation routes in VSSF systems. The *k* value indicates Monod kinetic rates (g/m^2/d) of such reactions, determined from the slope of the plots.

Modelling for HSSF systems. For HSSF systems, the combination of Monod kinetics, combined with plug flow pattern provided closer match, in terms of predicting nitrogen and organics removal (Saeed and Sun, 2011 b). As such, Equation (10.11) can be rearranged in the form of Equation (10.14), for developing Monod plug flow model.

$$\left(\frac{C_{half} + C_{out}}{C_{out}}\right) dC = -K_{max} dt \qquad 10.14$$

Applying the boundary conditions of idealized plug flow pattern in the above equation yields:

$$\int_{C_{in}}^{Cout} \left(\frac{C_{half} + C_{out}}{C_{out}}\right) dc = -K_{max} \int_{0}^{t} dt \qquad 10.15$$

Arranging Equation (10.15) in terms of areal maximum pollutant removal rates (K, g/m^2/d) yields equation (10.16), that expresses the combination of Monod kinetics with plug flow

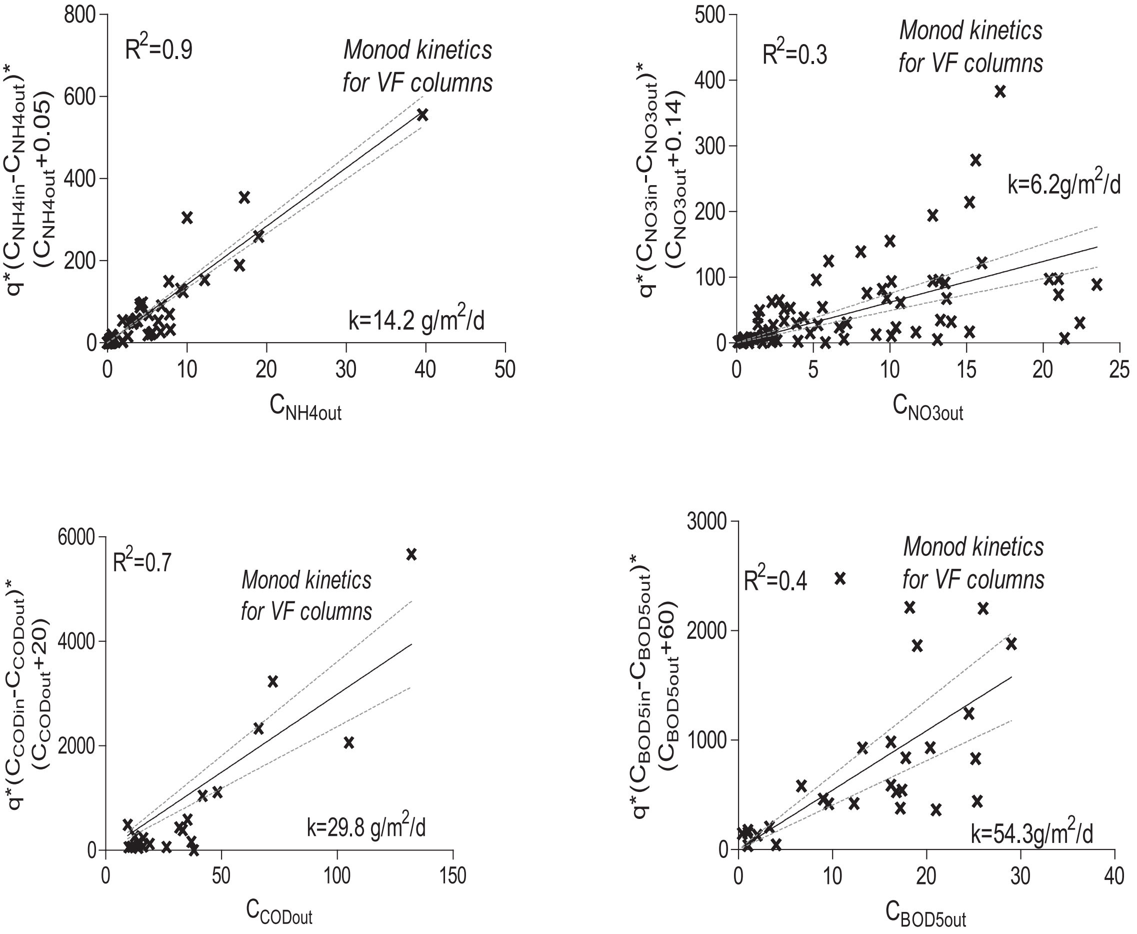


Figure 10.12 Correlation plot of Monod CSTR equation for predicting nitrogen and organics removal in VSSF systems (Saeed and Sun, 2011 b; c).

pattern, to correlate inlet and outlet pollutant values across HSSF wetlands (Sun and Saeed, 2009; Saeed and Sun, 2011 b).

$$K = \frac{C_{in} - C_{out} + C_{half} \ln \dfrac{C_{in}}{C_{out}}}{1/q} \qquad 10.16$$

Equation (10.16) may be used to correlate inlet and outlet NO_3-N, BOD_5 and COD values in HF wetlands. The half saturation constant for these target pollutants can be used as 0.14, 60 and 20 mg/L respectively, as described previously.

Figure 10.13 expresses the correlation plot of Equation (10.16) in terms of matching nitrogen and organics removal in three lab-scale HSSF systems, and 80 full-scale HSSF systems in the UK (Saeed and Sun, 2011 b). As observed in Figure 10.13, Equation (10.16) indicates closer match for predicting denitrification and organics removal (indicated by R^2) in HSSF systems.

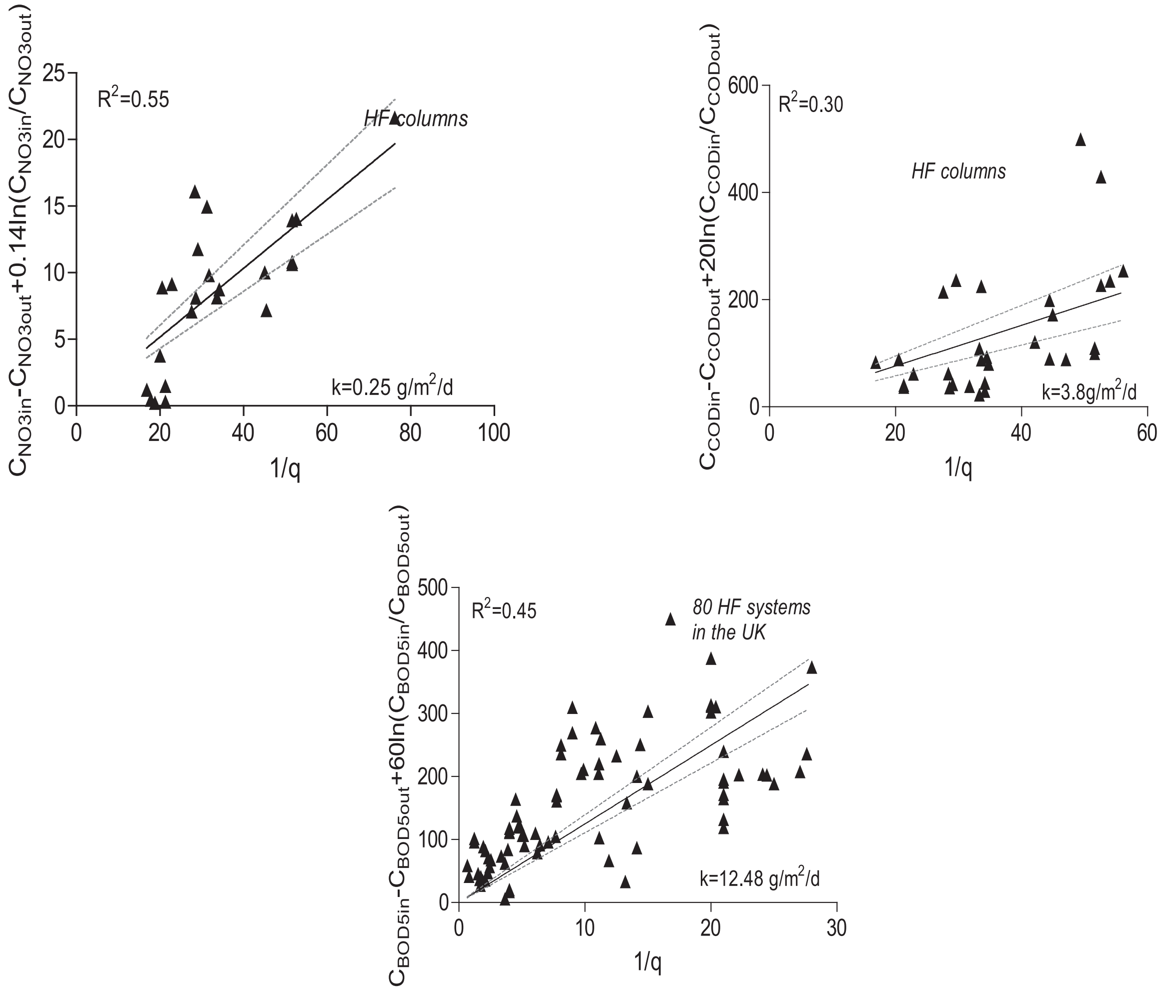


Figure 10.13 Correlation plot of Monod plug flow equation for predicting nitrogen and organics removal in HSSF systems (Saeed and Sun, 2011 b).

10.12 Floating Treatment Wetland: An Innovative Natural Treatment System

The sections as described earlier (in this chapter) illustrate pollutant removal mechanisms of subsurface flow and surface flow wetland systems. Apart from these natural reactors, a new natural treatment system has been employed recently which is referred as floating treatment wetland. Floating treatment wetland includes emergent macrophytes, supported by a buoyant mat structure (that floats on water column) with media to support the macrophytes. The stems of the macrophytes remain above the water column (Figure 10.14), whereas roots grow inside the water column extracting nutrients directly from water column. Microbiological population is attached on the roots, forming a complicated network of roots and biofilms. As wastewater passes through such network, biological removal of pollutants occurs (along with nutrient uptake by the roots).

Floating treatment wetlands provide coverage above the water column (via floating mats and macrophytes), thereby hindering the penetration of sunlight and growth of algae. A minimum water depth of approximately 0.8–1.0 m (Headley and Tanner, 2012) is recommended for employing floating treatment wetlands to encounter roots anchoring into the bottom sediments. If the roots are attached in the sediments floating mats will be submerged when the water level rises, which can damage the mat structure.

Different types of buoyant mats such as air tight frames with nets, integral buoyancy mats, suspended cables above water column supporting macrophytes, and rigid frames that are suspended close to water surface supporting plant growth can be designed.

Figure 10.14 Floating mat and hanging roots of a floating wetland.

A wide range of plants can grow on buoyant mats. A list of such plants has been provided in Table 10.11 (Headley and Tanner, 2012).

Table 10.11 List of macrophytes for floating wetlands.

Species	Source
Cyperus Giganteus	Wetland
Cyperus papyrus	Lake
Glyceria maxima	Rivers and wetlands
Miscanthidium violaceum	Lake
Panicum hemitomon	Lake
Phragmites australis	River delta
Pycreus nitidus	River delta
Rhynchospora asperula	Wetland
Schoenoplectus californicus	Wetland
Scirpus cubensis	Wetland
Thalia multiflora	Wetland
Typha latifolia	Wetland
Typha x glauca	Lake
Vossia cuspidata	Lake

The macrophytes available for floating treatment wetlands in Bangladesh are: Canna Indica, Thalia Dealbata, Sagittaria Trifolia, Hymenocallis Coronaria, Hymenocallis Caribaea, Centella Asiatica, Echinodorus v. Oriental, Ceratopteris Thalictroides, Cyperus Diandrus, Bulrush, Panicum Amarum, Phragmites, Glyceria Maxima, Pycreus Nitidus, Pennisetum Purpureum, Peltandra Virginica, Lobelia Cardinalis

10.13 Pollutant Removal Mechanisms in Floating Wetlands

Different types of wastewater such as: agricultural drainage, eutrophied lake water, storm runoff, aquaculture wastewater, swine wastewater, secondary treated wastewater can be treated employing floating treatment wetlands. Pollutants for example: solids, organics, nutrients, pathogens etc. can be removed from wastewater, when these natural bio-reactors are employed.

Solids. Floating mats and macrophytes reduce algae (due to absence of sunlight), which can be a significant source of suspended solids in open channels. The hanging roots inside water column reduce flow velocity, accelerating the settling of suspended solids. Finer solids (not removed by the settling process) are captured by the root-biofilm network. A study by Smith and Kalin (2000) reported 0.02 kg of captured solids/m^2 of root surface/ year, where Typha species were used as macrophyte to provide treatment of acid mine drainage pond.

Organics. Particulate organic matters are trapped by the biofilm- root network, and are subjected to biological degradation. A portion of the trapped solids settle in the bottom surface of water bodies, and is being degraded biologically. A number of studies (Revitt et al., 1997; Van Acker et al., 2005) reported COD removal rates between 31-68% employing floating treatment wetlands, that provided treatment of combined sewer flow and airport runoff.

Nutrients. Nitrification usually occurs in biofillms closer to the hanging roots. Subsequently, presence of plant derived carbon stimulates denitrification in the anaerobic pockets of floating wetland systems. In addition nitrogen and phosphorus uptake by plants also contributes to nutrients removal. Table 10.12 records nutrient removal rates by floating wetlands, as reported during different scientific studies.

Table 10.12 Nutrient removal rates by floating wetlands.

Parameter	Unit	Removal rates	Reference
NH_4-N	g/m^2/d	32.0–40.0	Wu et al. (2006)
NO_3-N	g/m^2/d	114.0	Stewart et al. (2008)
TN	g/m^2/d	1.1–2.5	Boonsong and Chansiri (2008)
TP	g/m^2/d	1.3–1.9	Kyambadde et al. (2005)

Pathogens. Pathogenic microorganisms are generally removed in floating wetlands via entrapment into the root- biofilm networks (beneath the mat). In addition these microorganisms may also be adsorbed by the organic matters, that are filtered by the root-biofilm networks. However, destruction of pathogens by UV radiation is inhibited, due to the shades provided by the floating mats and macrophytes.

Box 10.4

Application of floating treatment wetlands in Bangladesh.

A pilot-scale floating mat (Figure 10.14) was employed in Bangladesh by a research team headed by the lead author of this book. Such system was implemented to observe its pollutant removal performances from Gulshan lake. Water quality analyses indicated BOD_5 and E. Coli removal performances between 78.0-81.0% and 76.0-92.0% respectively. The system also achieved 95.0% turbidity removal performances.

After the pilot-scale research, real scale floating wetland systems were employed by the same research team at Sukandighi, Lalmonirhat district. Ten floating mats were employed; the floating mats were constructed with airtight UPVC pipes to achieve buoyancy. Fish nets were used to support media (i.e. straw and soil). Local macrophytes were collected from nearby water channels, and were planted. Within three weeks of implementation visual clarity of water had been observed. Figure 10.15 provides a pictorial view of the employed floating systems at Sukandighi.

Figure 10.15 Real scale floating wetlands at Sukandighi.

Questions

1. What are the main advantages of natural treatment systems, in terms of wastewater treatment?
2. What is the main reason of achieving higher nitrification rates in VSSF systems?
3. Why anaerobic conditions dominate in HSSF systems?
4. Enlist the major nitrogen removal routes in wetland systems.
5. How wetland macrophytes contribute to nitrogen removal from wastewater?
6. Why clogging is undesirable for wetland systems?
7. What is the relationship between bacteria and algae in stabilization ponds?
8. What are the principal mechanisms for faecal bacterial removal in maturation ponds?
9. Wastewater flow from an area averages 5000 m^3/d through out the year. The average yearly temperature is 28°C. The mean concentration of influent BOD_5 is 400mg/L with 70% being soluble. Reaction coefficient K is 0.23 d^{-1} at 20°C, and θ is 1.06. Prepare a facultative pond treatment system for the area to remove 85% of the soluble BOD.

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Chapter 11

Water Pollution: Analysis and Control

Water is considered to be polluted when it contains enough anthropogenic contaminants to render it unfit for a specific beneficial use; such as drinking, recreation or fish propagation. Water pollution occurs when pollutants are discharged directly or indirectly into the water bodies (such as lakes, streams or rivers, oceans, aquifers) without adequate treatment to remove harmful compounds. Water, because of its polar molecular structure, has a stronger tendency to dissolve other substances than any other common solvent and hence referred to as the universal solvent. Pure water is practically not found in natural conditions in water bodies, it usually has something dissolved or suspended in it. It is only when these dissolved or suspended substances go beyond a certain concentration in comparison with the water quality standards for a particular beneficial use, the water is considered to be contaminated. Although anthropogenic activity is the primary cause of water pollution, natural phenomena such as storms, cyclones, algal blooms, volcanic eruptions and earthquakes can also cause a major change in water quality and ecological status of water.

In many parts of the world, the introduction of pollutants from human activity has seriously degraded water quality even to the extent of turning pristine streams to foul open sewers with only a few life forms and fewer beneficial uses. Most of the rivers in urban areas in developing countries are considered as end points for effluent discharge from industries. In Bangladesh, industrial effluents from a cluster of industries in and around Dhaka in addition to untreated domestic sewage from DWASA sewerage lines are indiscriminately polluting the rivers surrounding the nation's capital. The most severe deterioration occurs in the dry season, when a decrease in river flow (sometimes tidally fluctuating) augments the effect of pollution bringing down the dissolved oxygen level close to zero at certain reaches of these rivers and thereby making the survival of fish almost impossible. The minimum requirement of dissolved oxygen for healthy fish population is 5 – 8 mg/L.

Figure 11.1 **The dumping of untreated industrial waste into the Turag River in Tongi. The dark colored water shows how heavily the river has been contaminated with chemical waste (*Inset*). (*Photo source: The Daily Star, August 2, 2011, www.thedailystar.net*).**

In today's world, the disease burden associated with polluted water has made it an issue of global concern. In the past, epidemics of waterborne diseases such as cholera have been responsible for the deaths of thousands of lives in the United States. Fortunately, epidemics of such diseases have been largely eliminated in the United States and other developed countries as a result of treating drinking water prior to consumption or through widespread vaccination. But this is certainly not the case worldwide; outbreaks of waterborne diseases continue to threat the developing countries. It has been suggested that water pollution is the leading worldwide cause of deaths and diseases, and that it accounts for the deaths of more than 14,000 people daily. In Bangladesh, where sanitation conditions are largely inadequate, contamination of water becomes a death trap. About 80% of the diseases are water-related that include cholera, typhoid, dysentery, hepatitis, diarrhea etc. claiming on an average one-tenth of each person's productive time and causing death in 20-30% cases. Providing water, safe for drinking and other purposes is essential for life and is one of the most important goals of development. Water quality management is concerned with the control of pollution from human activity so that the water is not degraded to the point that it is no longer suitable for its intended use.

Box 11.1

Water Pollution in historical times: "The Great Stink" in London

Figure: A caricature commenting on a letter from Faraday on the state of the river in the times magazine

Until the late 16th century, London citizens were reliant for their water supplies on water from shallow wells, the River Thames and its tributaries, or natural springs. In 1815 house waste was permitted to be carried to the Thames via the sewers. So for several years human waste was dumped into the Thames which was potentially pumped back to the same households for drinking, cooking and bathing. Around 1858, there were over 200,000 cesspits in London. Emptying one cesspit cost a shilling - a cost the average London citizen then could hardly afford. As a result, these cesspits turned their household environment severely odorous. During that time flush toilets were introduced, replacing the chamber-pots that most Londoners had used. These dramatically increased the volume of water and waste that was being poured into existing cesspits. These often overflowed into street drains designed originally to cope with rainwater, but now also being used to carry outfalls from factories, slaughterhouses and other activities, contaminating the city before emptying into the River Thames. The summer of 1858 was unusually hot. The Thames and many of its urban tributaries were overflowing with sewage; the warm weather encouraged bacteria to thrive and the resulting smell was so overwhelming that it affected the work of the House of Commons (countermeasures included draping curtains soaked in chloride of lime, while members considered relocating upstream to Hampton Court) and the law courts (plans were made to evacuate to Oxford and St Albans). This particular period in the summer of 1858 in London is known as the time of "The Great Stink" or "The Big Stink" referring to the noxious smell emanating from the untreated waste in Thames river and the cesspits in central London. Heavy rain finally ended the heat and humidity of summer and the immediate crisis ended. However, a House of Commons select committee was appointed to report on the Stink and recommend how to end the problem. Numerous caricatures emerged at that time to humorously portray the period of the Great Stink. One such caricature is shown in the figure to depict Faraday's comments on the state of the river Thames in the Times magazine of July, 1855.(*Source: www.wikipedia.org*)

To understand how much waste can be assimilated by a water body, we need to know about the type of pollutants and the manner in which they affect water quality. This chapter deals first with major types of pollutants and their sources, placing the emphasis on categories of pollutants found in domestic wastewaters. To understand the impact of human activities on water quality, the fate of contaminants particularly oxygen-demanding wastes on rivers and streams will be discussed in detail. This chapter will also focus on the characteristics of a typical lake environment and how water pollutants affect these conditions.

11.1 Water Pollutant Categories

To understand the effects of water pollution it is necessary to classify pollutants into various categories. Pollutants can be classified either by (a) nature of their origin or (b) groups of substances they contain. According to nature of their origin, a pollutant can be classified as either a *point source* or *a dispersed/non-point source*.

Point sources. A point source pollutant is one that reaches the water from a pipe, channel or any other confined and localized source. Domestic sewage and industrial wastes are called point sources because they are generally collected by a network of pipes or channels and conveyed to a single point of discharge into the receiving water. After applying proper wastewater treatment, the pollution from point sources can either be reduced or eliminated. Figure 11.2shows the point source discharge locations on Sitalakhya, Balu and Turag rivers; all these rivers surround Dhaka city, the nation's capital. Major pollutants are from the industries which discharge their effluents either directly on the rivers or on different channels (*khals*) which connect to these rivers.

Non-point sources. A dispersed or non-point source is a broad, unconfined area from which pollutants enter the water body. Surface runoff from agricultural areas may carry silt, pesticides, fertilizers or animal wastes to streams but not from a particular point. These materials can enter the water all along the stream as it flows through the area. Point source pollution can be easier to deal with compared to nonpoint source pollution; point sources can be readily identified and treatment plant may be installed to reduce pollution. The regulatory agencies can monitor the effluents from the treatment plant from time to time and may impose restrictions on effluent discharge. On the other hand, pollution from a non-point source is difficult to control; the most effective way is to set appropriate restrictions on landuse.

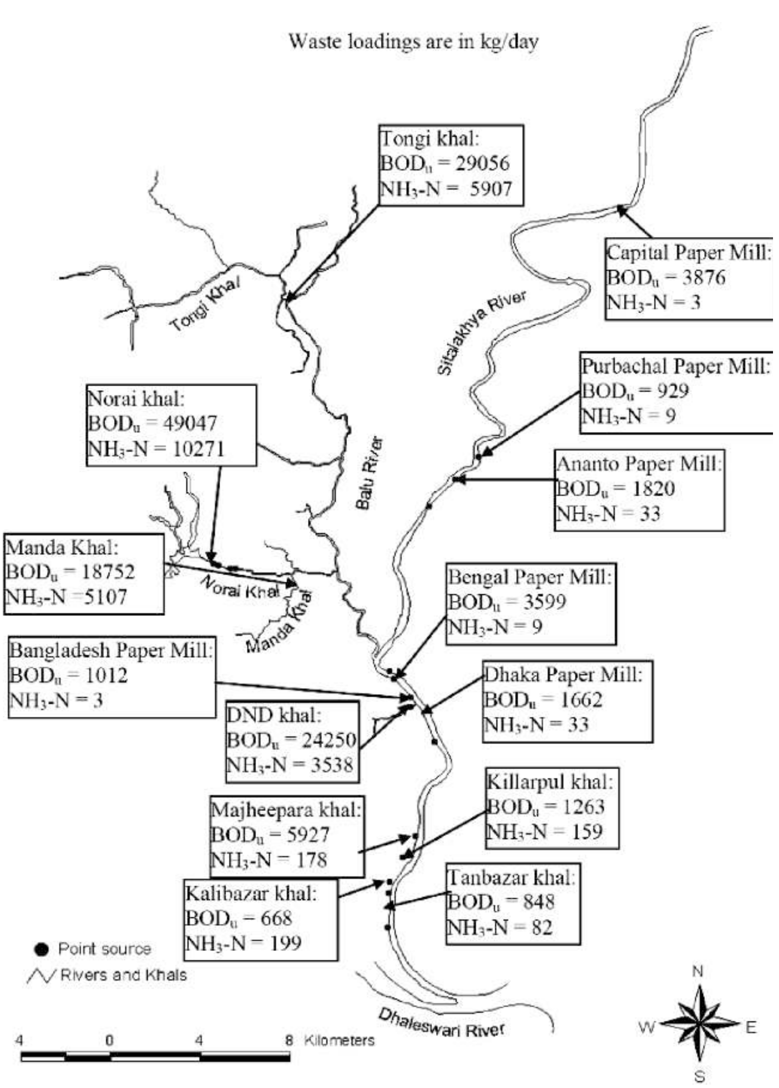


Figure 11.2 Point source loadings (in kg/day) in Sitalakhya and Balu rivers. BOD_u = Ultimate Biochemical Oxygen Demand, NH_3-N = Ammonia-Nitrogen (*Source: Alam et al. 2011*).

Figure 11.3 Non-point source pollution in Buriganga (*Image source: The Daily Star, www.thedailystar.net*).

Based on the groups of substance they contain, pollutants can be categorized by the following seven broad groups, also shown in Table 11.1, which also includes the principal sources of each group:

(1) Oxygen-demanding wastes
(2) Nutrients
(3) Pathogens
(4) Suspended solids
(5) Salts
(6) Toxic metals
(7) Toxic organic compounds
(8) Heat

Table 11.1 Major pollutant categories and principal sources of pollutants.

	Point sources		Non-point sources	
Pollutant category	**Domestic Sewage**	**Industrial wastes**	**Agricultural runoff**	**Urban runoff**
Oxygen-demanding wastes	X	X	X	X
Nutrients	X	X	X	X
Pathogens	X	X	X	X
Suspended solids/ sediments	X	X	X	X
Salts		X	X	X
Toxic metals		X		X
Toxic organic chemicals		X	X	
Heat		X		

Oxygen-demanding wastes. Anything that can be oxidized in the receiving water with the consumption of dissolved molecular oxygen is termed oxygen-demanding material. Oxygen demanding materials are primarily from human waste and food residue. Food processing and paper industries also produce oxygen-demanding wastes. As these wastes are oxidized, it reduces the amount of Dissolved Oxygen (DO) available threatening fish and other aquatic lives. In extreme cases as in anaerobic conditions, the higher forms of life (such as fish) may be killed or driven off. Also, as the DO is depleted, undesirable odors, tastes and colors not only reduce the acceptability of water for domestic supply but also reduce its attractiveness for recreational use. The oxidation of certain inorganic compounds may also contribute to the oxygen demand. There are several measures of oxygen demand. The two most commonly used are the Chemical Oxygen Demand (COD) and Biochemical Oxygen Demand (BOD) (see section 6.4). COD is the amount of oxygen needed to chemically oxidize the wastes while BOD is the amount of oxygen needed to biologically oxidize the waste. BOD has been traditionally used as the most important measure for the strength of organic pollution. Conventional wastewater treatment reduces the amount of BOD in the effluent but do not eliminate BOD completely.

Nutrients. Nutrients such as nitrogen and phosphorus are essential to the growth of living things. But if they are present in excessive amounts, it causes some organisms to proliferate at the expense of others and grossly disturb the food web. For example, these nutrients in excessive amounts can stimulate the growth of algae which eventually die and decompose in turn removing oxygen from water, potentially leading to levels in which aquatic species may become unsustainable. Algae and decaying organic matter add color, turbidity, odors and objectionable tastes to the water and diminish its acceptability for domestic use. The process of nutrient enrichment is called eutrophication and will be discussed in detail in later sections. Major sources of nitrogen and phosphorus include municipal wastewater discharges, runoff from animal feedlots, chemical fertilizers and also phosphorus-based detergents.

In addition to these anthropogenic sources of nutrients, there are also natural sources that may add nitrogen to the water bodies. Certain bacteria and blue-green algae can directly obtain nitrogen from the atmosphere. These life forms are abundant in lakes and have high biological productivity. Certain form of acid rains can also contribute nitrogen to the lakes.

Pathogenic organisms. Microorganisms found in wastewater include bacteria, viruses and protozoa excreted by diseased persons or animals. If wastes containing these microorganisms are discharged into surface waters, they render it unfit for drinking and if they enter the water supply, it can cause epidemic of enormous proportions. With the advent of chlorination, which began in 1908 in the United States, outbreak of waterborne diseases has become rare. However, developing countries today are still susceptible to high death rates from these diseases mostly because of the lack of adequate sanitation facilities. If the concentration of these pathogenic organisms in the surface water becomes very high, the water may also be unsafe for swimming, fishing or other recreational uses.

Box 11.2

The State of Buriganga river today

Figure *Left*: The Department of Environment collects samples of untreated waste being discharged into the Buriganga river by a factory in Keraniganj. *Right*: A DoE official finds 1.45mg oxygen in a litre of Buriganga water (*Image source: the Daily Star, July 5, 2011*).

In Dhaka, an average of 15,000 metric tons of human waste is generated daily. The only waste treatment plant for the city is located at Pagla in Narayanganj with a capacity of treating 1,20,000 m^3 of sewage. It was installed in 1973-74 and renovated in 1991-92 but can treat sewage at only one-third of its capacity. The rest of the waste dumped into rivers, canals and other water bodies. Residential areas including Uttara, Baridhara, Badda, part of Mirpur and Mohammadpur, Dhonia, Shyamoli, Kalyanpur, Banani, part of Gulshan and Jatrabari and a large area of the city's northern part altogether consisting of 80% area of Dhaka city remain out of WASA sewerage network. Due to unavailability of sewage connections, a large number of house-owners link sewage discharge connection directly to storm water drainage line. Some people illegally connect their sewage line to the drainage line even where sewage network is available to evade service charges. This results in dumping of untreated waste to rivers and water bodies in and around the city. Buriganga, once the lifeline of Dhaka city, is now severely polluted as it receives the major portion of this untreated sewage. Besides this, untreated industrial effluent is discharged from a cluster of industries along the river. Most industries do not have in-house treatment plants, and even if some of them had it, they would not use it perhaps to save operation cost in blatant violation of environmental rules. 500 tanneries in Hazaribagh including 200 large units discharge 4950 m^3 of obnoxious organic wastes every day and recent evidence points out that they are percolating into the subsurface and polluting the groundwater as well. Also, illegal grabbing of the river has obstructed the river's natural flow and exacerbated the situation. Previously the Department of Environment has filed cases against the factories for environmental pollution under the Bangladesh Environment Protection Act and fined some of those industries in various degrees, but the industries are still managing to operate unabatedly through various loopholes of the law. Grassroots movements are in progress initiated by various concerned groups in Bangladesh to stop pollution and save the Buriganga river.

Suspended solids. Organic or inorganic particles that are carried by the wastewater into receiving waters are termed suspended solids. When the speed of the water is reduced by flowing into a pool or a lake, many of these particles settle to the bottom as sediment. In common usage, the word sediment also includes eroded soil particles which are being carried by water even if they have not yet settled. Colloidal particles which do not settle readily are the cause of turbidity found in many surface waters. Organic suspended solids may also exert an oxygen demand. Inorganic suspended solids are discharged by some industries but result mostly from soil erosion. As excessive sediment loads are deposited into lakes and reservoirs, their usefulness is reduced. Even in rapidly flowing streams, sediments can disrupt ecological habitats of many aquatic species. For domestic water supply, excessive suspended solids necessitate water treatment at an increased cost.

Salts. All naturally occurring water contains some sort of salt; but the problem arises when the concentration of salt goes beyond such a level which makes it unsuitable for a particular purpose. Total Dissolved Solids (TDS) is a very common and simple measure of salinity in water. As a rough approximation, freshwater can be considered to be water with less than 1500 mg/L TDS; brackish waters may have TDS values up to 5000 mg/L; saline waters are those with concentrations above 5000 mg/L. According to Bangladesh Environmental Conservation Rules (1997), drinking water has a permissible maximum contaminant level of TDS of 1000 mg/L. High concentration of salts can also damage crops and cause soil pollution, therefore for irrigation water the permissible level is 2100 mg/L. Industrial discharge often consists of high amount of TDS. For the rivers which are used for irrigation, the concentration of salts progressively increase downstream as the amount of water available to dilute the salts becomes lower due to evaporation and diversions and as more and more salty irrigation drainage water is added to the flow.

Toxic metals. The list of toxic metals (often referred to as heavy metals) includes aluminum, arsenic, beryllium, bismuth, cadmium, chromium, cobalt, copper, iron, lead, manganese, mercury, nickel, selenium, strontium, thallium, tin, titanium and zinc. Some of these metals, such as iron, are essential nutrients in our diets, but in higher doses are extremely toxic. The adverse impacts include nervous system and kidney damage, creation of mutations and induction of tumors. Lead comes from exhaust of automobiles that use leaded gasoline, zinc comes from tire wear.

Toxic organic compounds. Many industrial discharges contain toxic metals or toxic organic substances. Pesticides, also a toxic organic chemical, may be carried into water in the surface runoff from agricultural areas. Perhaps the most dangerous among these are chlorinated organic hydrocarbons (organochlorines) (e.g. DDT, aldrin, endrin, heptachlor, methoxychlor). They not only possess toxic characteristics but also persistent, tend to

Box 11.3

Water Pollution from Textile Dyeing Industries in Bangladesh

Figure **Discharge of textile dyeing wastewater into a natural water body (*Source: Ahmed et al. 2012*).**

Textile is the top foreign exchange earning sector in Bangladesh, at the same time wastewater from textile dyeing has also become the top polluter of water and soil resources in the country. In Bangladesh, textile dying industries are grouped into red category that requires extensive Environmental Impact Assessment (EIA) and installation of treatment facilities to establish textile dyeing (GoB, 1997). But most of the industries do not have a treatment plants and discharge untreated effluent with very high pollution potential in water or on land. Some industries have physico-chemical and biological treatment plants which can reduce BOD, COD and colour to some extent but extended aeration and high dose of chemicals are required to reduce SS, BOD and COD to acceptable levels. On the other hand, the dissolved solids in the effluent remain very high. Some industries dilute the effluent by adding fresh water to meet the effluent quality standards but the total pollution loads discharged in the environment remain unchanged. Many of the treatment plants are not operated continuously for high cost of operation. As a result, severe pollution of water and soil resources around the industrial belt in Bangladesh due to indiscriminate disposal of wastewater has become a great environmental concern.

Textile dyeing requires huge quantity of water, which is contaminated with large quantities of salt, dye pigments and dissolved and suspended impurities either derived from fabrics or added for dyeing and discharged as wastewater. The wastewater from textile dyeing is characterized by high pH, turbidity, colour, dissolved and suspended solids, BOD and COD. The usual BOD of composite dyeing wastewater ranges from 300 to 600 mg/L, COD from 350 to 800 mg/L, Suspended Solids from 250 to 350 mg/L and Total Dissolved Solids as high as 3000 mg/L. Treatment of textile wastewater is relatively difficult and costly. Pollution and public health problems caused by textile dyeing wastewater have become the focus of the press and environmentalists for a number of years. The department of Environment as per provisions of ECA (1995) is trying to deal with the matter by setting deadlines for installing ETP and imposing fines for not operating ETPs but no improvement of situation is visible. About 100 units out of 300 knit processing industries within Bangladesh Knitwear Manufacturers & Export Association (BKMEA) may have installed ETPs. It has been reported in local news media that some of the dyeing industries are hiding their effluent by pumping it underground. The resulting pollution of groundwater will have far reaching consequences.

It is not possible to control environmental pollution from dyeing wastewater by the existing liberal effluent quality standards (EQS) of Bangladesh. The present effluent quality standards were formulated 18 years ago considering the abundance of effluent receiving water (surface water) in Bangladesh. In recent times, the flow of the existing streams have tremendously reduced in the dry season and on the other hand, composite dyeing industries have greatly flourished in the country during the last decade. Moreover, there are no suitable streams in the close vicinity of many factories for disposal by dilution. The improvement of quality of the industrial effluent is required to prevent pollution of the environment. The quality of the land around clusters of factories like Export Processing Zones (EPZs) in Bangladesh is fast deteriorating, even after largely complying with the EQS of Bangladesh (ECR, 1997). Discharge of excessive amount of salt with the effluent is mainly responsible for elevated water and soil salinity making the land around industrial clusters barren. The combined wastewater discharged from an industrial cluster is shown in Figure above.

Effective treatment of wastewater from dyeing industries and safe disposal of treated effluent and sludge are the only options to protect the environment. Chemical coagulation by an appropriate coagulant and oxidation by a strong oxidizing agent are required to remove colour and biological treatment can reduce BOD and COD. But these conventional methods cannot reduce salt content of the wastewater. Advanced treatment methods like Sequential Batch Reactor (SBR), Electro-chemical and Membrane Bio-filtration (MBR) processes are being used for the treatment of dyeing wastewater.

bioaccumulate. All organics having similar properties are termed POP, Persisting Organic Pollutants. POPs are prone to long-range transboundary atmospheric transport and deposition and are likely to cause significant adverse human health or environmental effects near to and distant from their sources. Government of Bangladesh has signed the Stockholm Convention on POP on 23 May 2001. As a Party and signatory to the Convention, Bangladesh is required to take actions to generate awareness of harmful consequences of POPs, to reduce their releases, and their ultimate elimination. The Stockholm Convention identifies 12 substances as POPs, which include (a) 9 substances used as pesticides, (b) Polychlorinated Biphenyl (PCB) and (c) Dioxins and Furans. The nine pesticides are Aldrin, Chlordane, Dieldrin, Endrin, Heptachlor, Mirex, Toxaphone, DDT and Hexachlorobenzene (HCB).

Heat (thermal pollution). Heat is considered as a pollutant because of the adverse effect it can have on the oxygen levels and aquatic life in a river. Typically the source of heat in natural rivers is the discharge of cooling water as it passes through the condensers collecting the waste heat in power plants. The cooling water, previously withdrawn from the river, may be subjected to a temperature increase of upto 15°C after it serves to condense the steam. The discharge of warm water into the river is usually called thermal pollution. Increase in

Box 11.4

Bioaccumulation of DDT

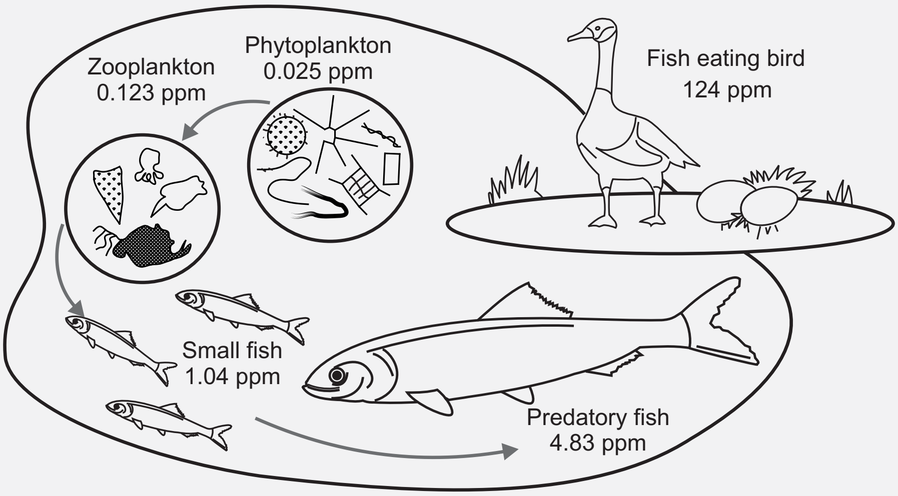


Figure Process of biological magnification. DDT concentrations increase in organisms higher in the food chain.

DDT (dichlorodiphenyltrichloroethane) is the most well-known organochlorine pesticide which has been widely used in the United States to control insects that carry diseases such as malaria, typhus and plague. It was a contributed to literally saving millions of lives and also, terms of human toxicity it was considered relatively safe. It was its impact on the food chain, rather than human toxicity, which led to its ban in the United States. It appeared that DDT had two properties which made it particularly harmful in food chains. Firstly, it was extremely *persistent*, that is, it lasts in the environment for a long time before it is broken down into other substances. Secondly, they are quite soluble in lipids which means they tend to accumulate in fatty tissue. The effect of this is that the organisms at higher trophic levels in the food chain will have progressively increasing concentrations of DDT accumulated in their tissue. The concentration in the body is the highest at the top of the food chain where the adverse effects will be manifested. It was the adverse effect of DDT on the reproductive system of birds that focused attention to this pesticide. DDT interfered with calcium metabolism of birds, resulting in eggs with shells that are too thin to support the weight of the parent. This phenomenon in which the concentration of a chemical increases at higher levels in the food chain is known as *biomagnification*.

temperature decreases the solubility of oxygen in water and increase the rate of metabolism of fish. This changes the ecological balance of fish. Several species of fish adapt well in warmer waters while several others prefer colder waters. In the outfall region, a certain species of fish may thrive due to warmer waters but problems may arise if there is a sudden shift in environmental conditions (e.g.decrease in temperature due to plant shutdown for repair). Sudden decrease in temperature may cause fish kills in significant proportions. Since thermal effluents also reduce the amount of DO available; a river that may accept a certain sewage load without much adverse effects, could have unacceptably low DO levels when a power plant is added. Thermal pollution may be controlled by passing the heated water through a cooling pond or a cooling tower after it leaves the condenser. The heat is dissipated into the air and the water can then be either discharged to the river or pumped back to the plant for reuse as cooling water (Figure 11.4 (b))

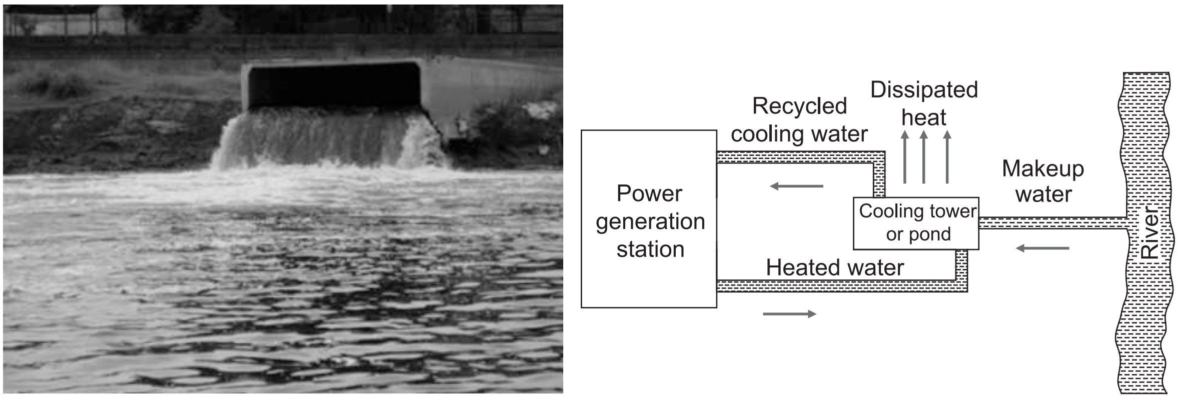


Figure 11.4 (a) Thermal effluent having a temperature 8.7°C in excess of the ambient water river water temperature being discharged in the Sitalakhya River by the Siddhirganj 210 MW Power Plant in Narayanganj. (b) Thermal pollution from power plants can be eliminated by using recirculation cooling towers or ponds.

11.2 Pollutant Discharge Limits

If pollutant discharge continues unabated, rivers and lakes will lose their acceptability for their intended beneficial use. That is why there are regulatory agencies, whose primary responsibility is to impose restrictions on the quality of domestic or industrial effluents on water bodies on the basis of certain guidelines or standards of a particular country. Water quality standards are therefore necessary to ensure that the appropriate quality of water is available for a particular use by the consumer. In Bangladesh, the Department of Environment (DoE) is the main regulatory agency which monitors effluent discharges by the industries and verify their compliance with the standards stated under the Environmental Conservation Rules (ECR), 1997. It has the authority to cancel permits or issue fines to the industries if they find them under violation of the standards. Table 11.2 and 11.3 show the standards for industrial and domestic effluent discharge to water bodies as stated in the ECR, 1997.

11.3 Water Quality Assessment Methods

Chemical assessment. Chemical assessment techniques are well known and involve regular sampling of water in the natural system and/or at some point in the abstraction and treatment processes and of most effluents before they are released back into the environment. The assessment involves regular testing for the presence and concentration of the major chemical parameters described in Tables11.2 and 11.3.Description of these tests are available in Standard Methods (2012) and will not be discussed in this chapter.

Table 11.2 Bangladesh Standards for sewage discharge into surface and inland water bodies.

Parameters	Unit	Values
BOD	mg/l	40
Nitrate	mg/l	250
Phosphate	mg/l	35
Suspended Solids (SS)	mg/l	100
Temperature	°C	30
Coliforms	number/100ml	1000

*Source:*Schedule- 9, Rule-13, Environment Conservation Rules, 1997.

Table 11.3 Bangladesh Standards for industrial effluent discharge.

Parameters	Unit	Values
Ammonia-nitrogen (as elementary N)	mg/l	50
Ammonia (as free ammonia)	mg/l	5
Arsenic (as As)	mg/l	0.2
BOD_5 at 20°C	mg/l	50
Boron	mg/l	2
Cadmium (as Cd)	mg/l	0.05
Chloride	mg/l	600
Chromium (as total Cr)	mg/l	0.5
COD	mg/l	200
Chromium (as hexavalent Cr)	mg/l	0.1
Copper (as Cu)	mg/l	0.5
Dissolved oxygen (DO)	mg/l	4.5-8
Electro-conductivity (EC)	μSiemens/cm	1200
Total dissolved solids	mg/l	2100
Fluoride (as F)	mg/l	2
Sulfide (as S)	mg/l	1
Iron (as Fe)	mg/l	2
Total kjeldahl nitrogen (as N)	mg/l	100
Lead (as Pb)	mg/l	0.1
Manganese (as Mn)	mg/l	5
Mercury (as Hg)	mg/l	0.01

Parameters	Unit	Values
Nickel (as Ni)	mg/l	1.0
Nitrate (as elementary N)	mg/l	10.0
Oil and grease	mg/l	10
Phenolic compounds (as C_6H_5OH)	mg/l	1.0
Dissolved phosphorus (as P)	mg/l	8
pH		6-9
Selenium (as Se)	mg/l	0.05
Zinc (as Zn)	mg/l	5
Total dissolved solids	mg/l	2100
Temperature (thermal effluent)	°C (summer) °C (winter)	40 45
Suspended solids	mg/l	150
Cyanide	mg/l	0.1

Source: Schedule –10, Rule-13, Environment Conservation Rules, 1997

Biological assessment. Environmental stresses alter the physico-chemical environment of the receiving water body which may disrupt the ecological balance of the system. Thus by measuring the extent of the ecological upset, the severity of the impact can be assessed. The sensitivity or tolerance to pollution may vary from species to species. For example, some species are very sensitive to the dissolved oxygen present in water and will not be found if the dissolved oxygen falls below a certain level. On the other hand, some species are more tolerant in this respect. The overall effects of an altered physico-chemical environment are changes in species composition, changes in dominant groups within species, changes in behavior, high mortality of sensitive life stages (e.g. eggs) while changes in physiology, metabolism and morphological deformities may occur. The water quality may be assessed through field inspection by noting these changes in various species from different trophic levels and comparing them with those expected in a clear, unpolluted water environment. There are several methods of assessing water quality using biological indicators which are described in several texts, e.g. Trivedi, 1981 and Spellerberg, 1991. The advantages and disadvantages of biological assessment compared to chemical assessment is shown in Table 11.4.

11.4 Oxygen Demand of Wastes

Oxygen-demanding wastes have been a pervasive surface-water problem throughout the world and its effect on water quality (typically dissolved oxygen) of lakes and streams has been an issue of particular interest to researchers and engineers. To understand the fate of oxygen-demanding wastes on rivers, we need to first understand the factors affecting oxygen consumption during a degradation process. This section describes the nature of different oxygen demands exerted by organic matter and how we can estimate them.

Oxygen demand of organic matter. When biodegradable organic matter is released into a body of water, microorganisms, especially bacteria, feed on the wastes, breaking it down into simpler organic and inorganic substances using up oxygen in the process:

Table 11.4 Advantages and disadvantages of different water quality monitoring techniques.

Realm	Performance of chemical monitoring	Performance of biological monitoring
Precision (i.e. pollutant concentration assessment)	Good	Poor
Discrimination (i.e. what kind of pollution)	Good	Poor
Reliability (how representative is a single or limited number of samples)	Poor	Good
Measure of ecological effects	No	Yes
Cost	Relatively high	Relatively low

Source: From Akolkar et al. 2008

Box 11.5

Macroinvertebrates as Bioindicators of River Water Quality

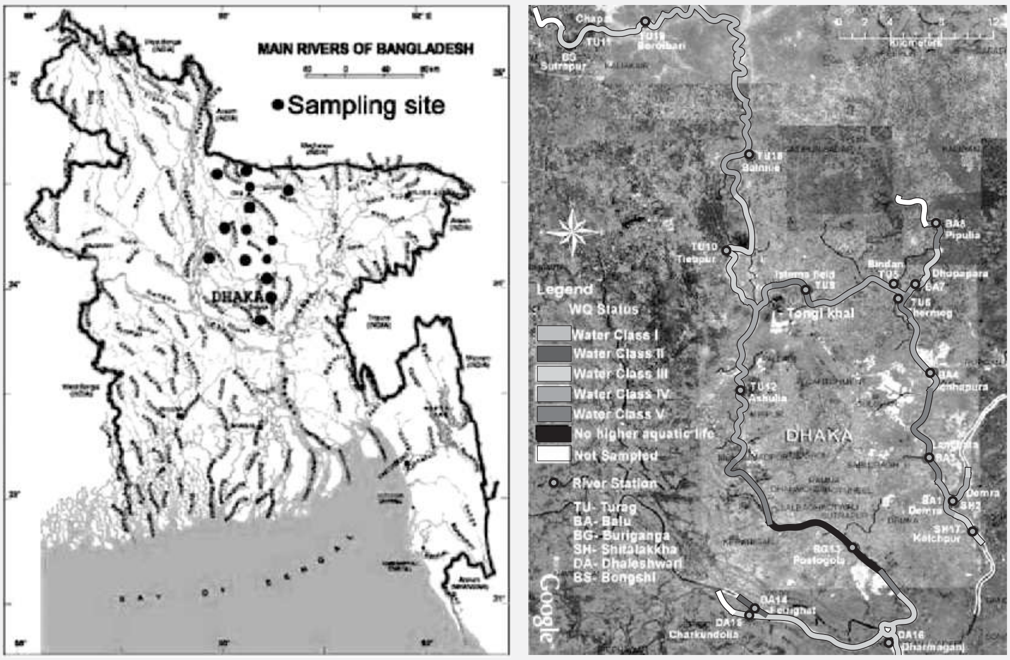


Figure: Left: Macroinvertebrate sampling sites in several rivers in Bangladesh to assess the biological environment. **Right:** Water quality map generated from the categories of biological indicators for the rivers around Dhaka city. The different stretches of the river are color-coded as water classes I to V, indicating good to worse, according to their water quality assessed from the biological indicators. (Source: Bari et al. 2008).

The species most commonly used for the investigation of river water quality are the larger and more easily visible invertebrate animals which colonize the substrate of all the rivers. Such animals are collectively referred as macroinvertebrates, of which the main constituents are young aquatic stage of certain insects. Within this bottom dwelling community, the sensitivity and tolerance to pollution varies considerably from species to species. In 2006, a macroinvertebrate sampling programme was undertaken in 34 sites in 25 rivers in the north-eastern region of Bangladesh in order to determine the ecological health of the rivers. This study used Multi-Habitat Sampling (MHS), Rapid field biomonitoring protocol and classified the rivers in Class I, II, III, IV and V based on ASSESS-HKHbios score index. The water quality map prepared as shown in the figure above provides a good visual picture of the pollution status of the river. The water quality map shows the state of the Buriganga river to be the worst due to effluents discharged from Hazaribagh tannery and other industrial installations along the bank of the river as wells as untreated sewage disposal from Dhaka city. On the eastern side, the Turag river has been seen to be extremely polluted. On the northern side, the Tongi khaal has been found to be extremely polluted mainly because of the wastewater discharged from Tongi industrial area. The upper reach of Balu river is extremely polluted (Water Quality Class V) due to agricultural runoff. The condition improves slightly (Water Quality Class III) further downstream before it gets worse again near its junction with Tongi khaal (Water Quality Class IV) which contributes heavy pollution to the river. Again in the downstream reach of Balu river, the pollution level increases and it becomes class V. Before the confluence with Sitalakhya river, the water quality of Balu river improves slightly to class IV and Sitalakhya river near Kanchpur, the water quality class is III. The Dhaleswari river has been found to be of water quality class IIIas there is no direct discharge of sewage or effluents. The water quality classes thus obtained through biomonitoring techniques fairly agree with the recorded physico-chemical parameters of the respective rivers. (Source: Bari et al. 2008)

$$Microorganisms\ (Organic\ matter) + O_2 \rightarrow CO_2 + H_2O + New\ cells + Stable\ products$$

The actual BOD (see section 6.4) is less than the theoretical BOD, which is the oxygen demand determined from the stoichiometric relationship between carbon in the organic matter and oxygen, due to the incorporation of some of the carbon into new bacterial cells. Theoretical BOD is of limited usefulness in practice since it considers a particular, single pollutant with a known chemical formula.

The total amount of oxygen that will be required for biodegradation is an important measure of the impact that a given waste stream will have on the receiving body of water. While we could imagine a test in which the oxygen required to completely degrade a sample of waste would be measured, such a test would require an extended period of time (several weeks), making it impractical. As a result, it has become standard practice simply to measure and report the oxygen demand over a shorter, restricted period of 5 days, realizing that the ultimate demand is considerably higher. The 5-day BOD, or BOD_5, is the total amount of oxygen consumed by microorganisms during the first 5 days of biodegradation. BOD_5 was chosen as the standard value for most purposes because the test was devised by sanitary engineers in England, where rivers have travel times to the sea of less than 5 days, so there was no need to consider oxygen demand at longer times. Since there is no other time which is any more justifiable than 5 days, this value has become firmly entrenched.

Laboratory measurement of BOD_5. In its simplest form, a BOD_5 test would involve putting a sample of waste into a stoppered bottle, measuring the concentration of dissolved oxygen in the sample at the beginning of the test and again 5 days later. The difference in DO would be the 5-day BOD. Light must be kept out of the bottle to keep algae from adding oxygen by photosynthesis and the stopper is used to keep air from replenishing DO that has been removed by biodegradation. To standardize the procedure, the test is run at a fixed temperature of 20°C. Since the oxygen demand of typical waste is several hundred milligrams per liter, and since the saturated value of DO for water at 20°C is only 9.1 mg/L, it is usually necessary to dilute the sample to keep final DO above zero. If during the 5 days the DO drops to zero, then the test is invalid, since more oxygen would have been removed had more been available.

The 5-day BOD of a diluted sample is given by

$$BOD_5 = \frac{DO_i - DO_f}{P} \tag{11.1}$$

where DO_i = the initial dissolved oxygen of the diluted wastewater
DO_f = the final DO of the diluted wastewater
P = the dilution fraction
= Volume of wastewater / (Volume of wastewater plus dilution water)

A standard BOD bottle holds 300 mL, so P is just the volume of wastewater divided by 300 mL.

In some cases it is necessary to seed the dilution water with microorganisms to assure that there is an adequate bacterial population to carry out the biodegradation. In such cases, to find the BOD of the waste itself, it is necessary to subtract the oxygen demand caused by the seed from the demand in the mixed sample of waste and dilution water.

To be able to sort out the effect of seeded dilution water from the waste itself, two BOD

Figure 11.5 Bottles specifically designed for incubation of water samples for BOD analysis. A flared mouth forms a water seal which prevents the drawing of air into the bottle during incubation. The interior shape of the bottle is such that the entrained air is sweeped out of the stopper opening. Glass stopper design uses a conical extension to displace excess sample which further insures no air entrapment in the sample. (Krackeler Scientific, Inc.).

bottles must be prepared, one containing just the seeded dilution water and the other containing the mixture of both the wastewater and seeded dilution water. The change in DO in the bottle containing just seeded dilution water (called the "blank"), as well as the change in DO in the mixture are then noted. The oxygen demand of the waste itself (BOD_w) can then be determined as follows:

$$BOD_m V_m = BOD_w V_w + BOD_d V_d \tag{11.2}$$

where BOD_m = BOD of the mixture of wastewater and seeded dilution water
BOD_w = BOD of the wastewater alone
BOD_d = BOD of the seeded dilution water alone
V_w = the volume of wastewater in the mixture
V_d = the volume of dilution water in the mixture
V_m = the volume of the mixture = $V_d + V_w$

As before, let P equal the fraction of the mixture that is wastewater = V_w/V_m so that $(1 - P)$ is the fraction of the mixture which is seeded dilution water = V_d/ V_m. Rearranging Eq. (11.2) gives

$$BOD_w = \frac{BOD_m}{V_w / V_m} - BOD_d \frac{V_d / V_m}{V_w / V_m} \tag{11.3}$$

Substituting the definitions of P and (1-P) into Eq. (11.3) gives

$$BOD_w = \frac{BOD_m - BOD_d (1-P)}{P} \tag{11.4}$$

Since $BOD_m = DO_i - DO_f$ and $BOD_d = B_i - B_f$

Where, B_i = initial DO in the seeded dilution water (blank)
B_f = final DO in the seeded dilution water

The final expression for the BOD of the wastewater itself is

$$BOD_w = \frac{(DO_i - DO_f) - (B_i - B_f)(1-P)}{P} \tag{11.5}$$

Example 11.1. Unseeded 5-Day BOD Test. A standard 5-day BOD test is run using a mix consisting of 3 parts distilled water and 1 part wastewater. The initial DO of the mix is 9.0 mg/L and the DO after 5 days is determined to be 1.0 mg/L. What is the BOD_5?

Solution.

The dilution fraction, $P = 1/4 = 0.25$. Using Eq. (11.1), the 5-day BOD,

$$BOD_5 = \frac{DO_i - DO_f}{P} = (9.0 - 1.0)/0.25 = 32 \text{ mg/L}$$

Example 11.2. A Seeded BOD Test. A mixture consisting of 30 mL of waste and 240 mL of seeded dilution water has an initial DO of 8.55 mg/L; after 5 days, it has a final DO of 2.40 mg/L. another bottle containing just the seeded dilution water has an initial DO of 8.75 mg/L and a final DO of 8.53 mg/L. What would be the 5-day BOD of the waste?

Solution

The dilution factor *P* is

$$P = 30/(240+30) = 0.11$$

Using Eq. (11.5)

$$BOD_5 = \frac{(DO_i - DO_f) - (B_i - B_f)(1-P)}{P} = \frac{(8.55-2.40)-(8.75-8.53)(1-0.11)}{0.11}$$

$$= 54\,\text{mg/L}$$

Factors affecting BOD rate constant. As discussed in section 6.4, the BOD kinetics can be described as a first-order reaction and the BOD reaction rate constant, *k*, is a factor that indicates the rate of biodegradation of wastes in streams. As *k* increases, the rate at which DO is consumed also increases. The magnitude of the reaction rate will depend on three factors: (1) nature of the waste, (2) ability of organisms to utilize the waste and (3) temperature.

Not all naturally occurring organic compounds are degraded at equal rates. The complexity of the hydrocarbon structure dictates the rate of degradation. Simple sugars and starches are easily degraded and will, therefore, have a large BOD rate constant. On the other hand cellulose degrades slowly and will have lower reaction rates. Table 11.5 shows a summary of some typical BOD rate constants. The lower rate constants for treated sewage compared to raw sewage result from the fact that easily degradable organics are more completely removed than less readily degradable organics during wastewater treatment. Again, a particular group of microorganism may be well suited to degrade certain kinds of organic matter. When such organic matters are discharged in the receiving streams, this group of microorganism generally thrives and dominates the microbial population. When BOD is determined in the laboratory, it is important to have the microorganisms which have adapted to that particular kind of waste so that rate constants become representative of actual conditions in the river. Finally, temperature speeds up reaction rates like most biological processes. Ideally, BOD rate constants should be experimentally determined for the temperature of the receiving water. But the temperature can vary both temporally (with changing seasons) and spatially (along different reaches of the river). Therefore, the standard practice is to determine BOD at a certain temperature (which is 20°C) and adjust the rate constant for the temperature of the receiving water using Eq. (6.17) (see section 6.4 for details).

Oxygen demand due to nitrification. Many organic compounds (e.g. proteins) contain not only carbon but also nitrogen that can be oxidized by microorganisms resulting in the consumption of molecular oxygen. The mechanisms and rate of this oxidation is different from those of carbon oxidation. Therefore, to separate these processes, the oxygen demand

Table 11.5 Typical values for the BOD rate constant.

sample	k (20°C)(day^{-1})
Raw sewage	0.35-0.70
Well-treated sewage	0.10-0.25
Polluted river water	0.10-0.25

Source: Davis and Cornwell (1985)

resulting from the oxidation of carbon is usually termed as Carbonaceous BOD (CBOD) and that resulting from the oxidation of nitrogen is termed as Nitrogenous BOD (NBOD).

When living things die or excrete waste products, nitrogen that was tied to the complex organic molecules gets released as ammonia by bacteria and fungi. In aerobic environments nitrite bacteria (*Nitrosomonas*) convert ammonia to nitrite (NO_2^-), and nitrate bacteria (*Nitrobacter*) convert nitrite to nitrate (NO_3^-) through nitrification process (see equations 7.2 and 7.3).

Nitrification is just one part of the biogeochemical cycle for nitrogen, which is shown in Figure 11.6. As is suggested there, nitrogen exists in many forms as it moves through the biosphere. In the atmosphere it is principally in the form of molecular nitrogen (N_2) and nitrous oxide (N_2O). Nitrogen is first transformed into either ammonia (NH_3) or nitrate (NO_3) in the process called nitrogen fixation. Nitrogen fixation occurs during electrical storms when N_2 oxidizes, combines with water, and is rained out as HNO_3. Certain bacteria and blue-green algae are also capable of fixing nitrogen. Under anaerobic conditions, certain denitrifying bacteria are capable of reducing NO_3 back into NO_2 and N_2, completing the nitrogen cycle.

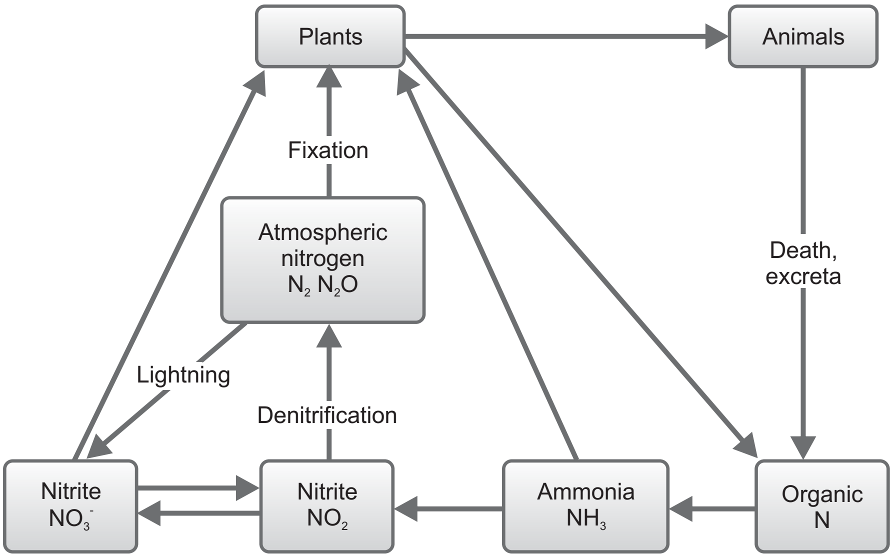


Figure 11.6 **Biogeochemical cycle of nitrogen.**

However, the process through which organic nitrogen is converted into Ammonia and then to nitrite and nitrate (nitrification) is the main concern in this section. It appears that this conversion happens sequentially and it is a matter of days before the rate of oxidation of ammonia is sufficient enough to create a significant oxygen demand (see Figure 11.7).

Figure 11.8 illustrates the two oxygen demands as they may be exerted in a BOD experiment. In typical municipal wastes, NBOD does not exert itself for at least 5-8 days, so most 5-day BOD tests are not affected by nitrification. This lag period is due to the fact that the bacterial population takes some time to reach a sufficient population for the amount of NBOD exertion to be significant. This is true for untreated sewage. However, if there is a sufficient number of nitrifying bacteria present as in treated sewage, NBOD may begin to be exerted from the very onset (Figure 11.8) and confound measurements of BOD if only CBOD is

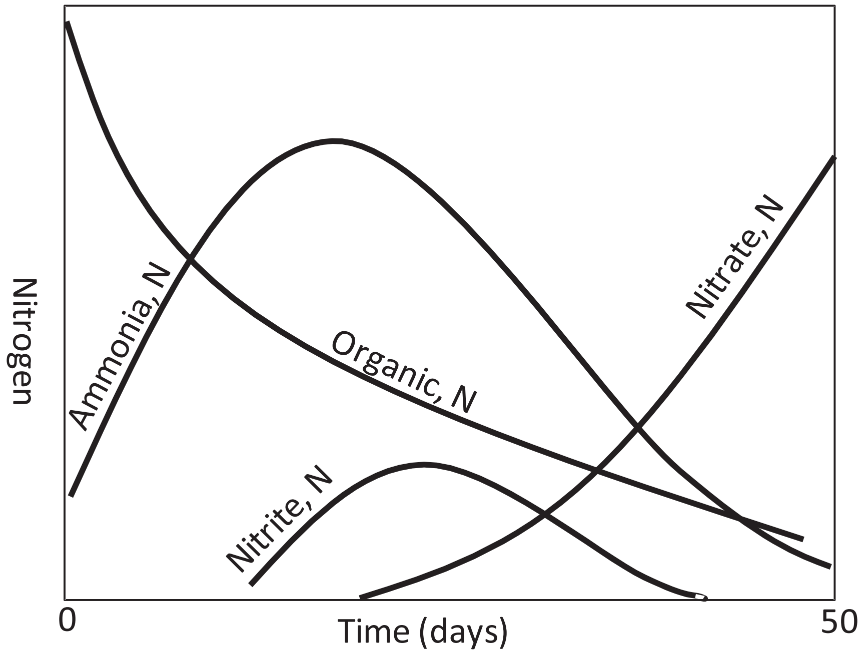


Figure 11.7 Changes in nitrogen species in polluted water under aerobic conditions (Adapted from Masters, 2001).

desired. Therefore, it is now an accepted practice to modify wastes in a way (using chemical inhibitors) that will inhibit nitrification during that 5-day period. The rate constant for nitrification is also affected by temperature and can be adjusted using Eq. (6.17).

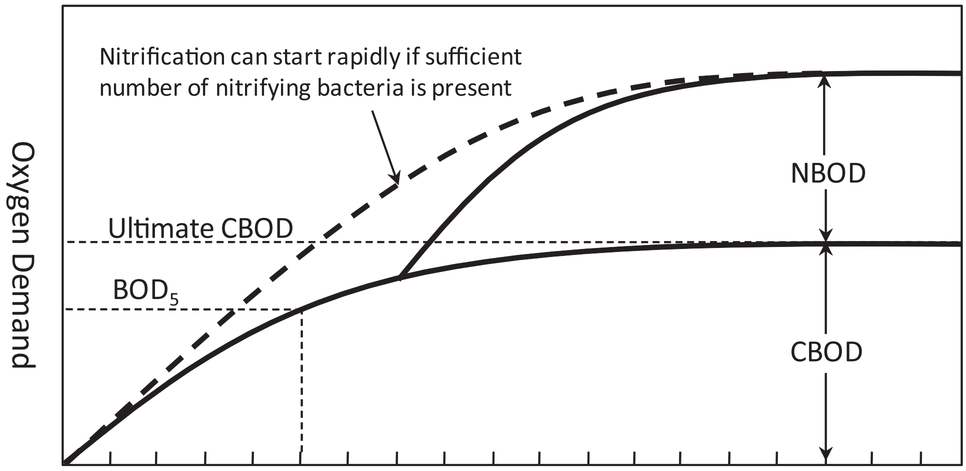


Figure 11.8 Illustrating the carbonaceous and nitrogenous biochemical oxygen demand.

Example 11.3. Quantifying Nitrogenous Oxygen Demand. Some domestic wastewater has 40 mg/L of nitrogen either in the form of organic nitrogen or ammonia. Assuming that very few new cells of bacteria are formed during the nitrification of the waste, find

a. The ultimate nitrogenous oxygen demand.
b. The ratio of the ultimate NBOD to the concentration of nitrogen in the waste.

Solution

a. The equation for the nitrification:

$$NH_3 + 2O_2 \xrightarrow{\qquad\qquad} NO_3^- + H^+ + H_2O$$

The molecular weight of NH_3 is 17 $(14 + 3 \times 1)$ and the molecular weight of O_2 is 32 (2×16). The above reaction indicates that one g-mole of NH_3 (17 g) requires two g-moles of O_2 $(2 \times 32 = 64$ g). Since 17 g of NH_3 contains 14 g of N, and the concentration of N is 40 mg/L, the final, or ultimate, NBOD:

$$\text{NBOD} = 40\,\text{mgN/L} \times (17\,\text{g}\,\text{NH}_3/\,14\,\text{g}\,\text{N}) \times (64\text{g}\;\text{O}_2/17\,\text{g}\,\text{NH}_3) = 183\,\text{mg}\,\text{O}_2/\text{L}$$

b. The oxygen demand due to nitrification divided by the concentration of nitrogen in the waste is

$$183\,\text{mg}\,\text{O}_2\,/\,40\,\text{mg}\,\text{N} = 4.57\,\text{mg}\,\text{O}_2/\text{mg}\,\text{N}$$

The total concentration of organic and ammonia nitrogen in wastewater is known as the total Kjeldahl nitrogen, or TKN. As was demonstrated in the above example, the nitrogenous oxygen demand can be estimated by multiplying the TKN by 4.57.

$$\text{Ultimate NBOD} \approx 4.6 \times \text{TKN} \qquad 11.6$$

Alternate measures of oxygen demand. Some organic materials, such as cellulose, phenols, benzene, and tannic acid, resist biodegradation. Others, such as pesticides and various industrial chemicals, are nonbiodegradable because they are toxic to microorganisms. The chemical oxygen demand, COD, is a measured quantity that does not depend either on the ability of microorganisms to degrade the waste or on knowledge of the particular substances in question (details in section 6.4). However, COD does not distinguish between the oxygen demand of the organic matter due to biodegradation, and the chemical oxidation of inert organic matter. Nor does it provide any information on the rate at which actual biodegradation will take place. The measured value of COD is higher than BOD, though for easily biodegradable matter the two will be quite similar. In fact, the COD test is sometimes used as a way to estimate the ultimate BOD. In municipal wastewaters, $\text{COD} \approx 1.6\,\text{BOD}_5$.

Although the 5-day BOD is chosen as the standard metric to characterize most wastewater, the ultimate BOD is actually a better indicator of the total waste strength. This is because different type of wastewaters having the same BOD_5 but different BOD reaction rates will exhibit a difference in ultimate BOD. This is illustrated in Figure 11.9 for a municipal and an industrial wastewater. Although both wastewaters have the same BOD_5, the industrial wastewater has a lower reaction rate and hence a higher ultimate BOD, and can be expected to have a greater impact on the dissolved oxygen in a river.

11.5 Fate of Oxygen Demanding Wastes in Rivers

The amount of dissolved oxygen in water is one of the most commonly used indicators of a river's health. As DO drops below 4 or 5 mg/L, the forms of life that can survive begin to be reduced. In nature, clean waters are saturated with oxygen or nearly so. When oxygen-demanding wastes are discharged into a stream, they undergo aerobic decomposition,

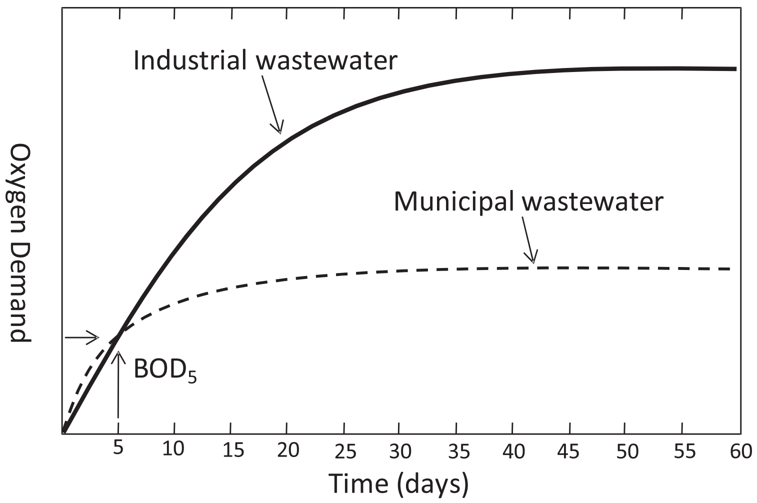


Figure 11.9 Comparing the oxygen demand of industrial and municipal wastewater.

Box 11.6

Oxygen-demanding wastes in Sitalakhya River

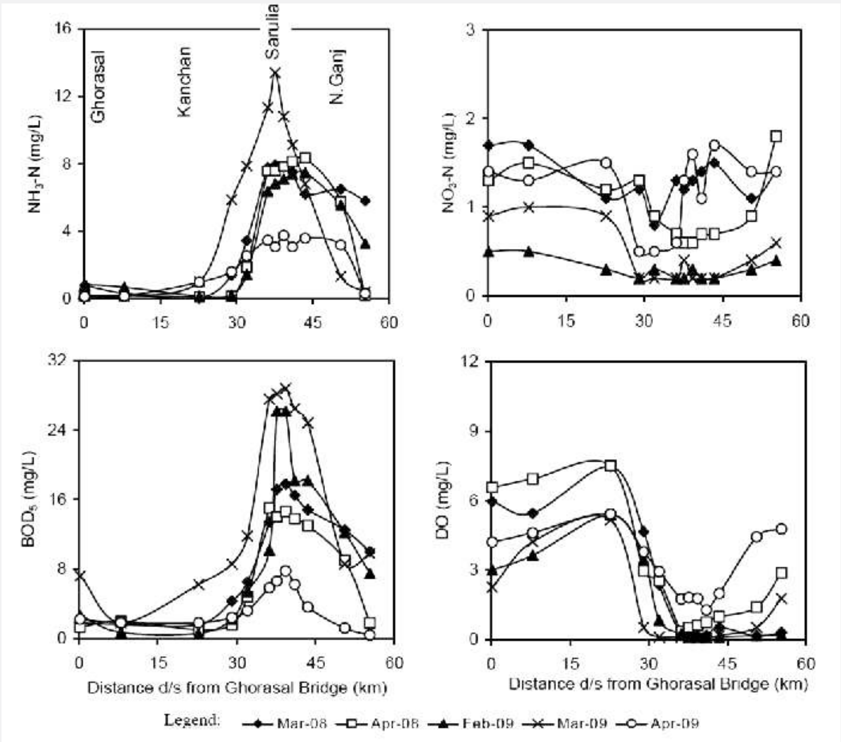


Figure Spatial variation of Ammonia-Nitrogen, Nitrate-Nitrogen,BOD5 and DO along Sitalakhya River (Alam et al, 2011).

The water quality of Sitalakhya river is of particular importance not only for ecological and commercial reasons but also for concerns regarding the supply of safe drinking water. The largest surface water treatment plant in Bangladesh located at Saidabad draws water from it through the intake at Sarulia about 400 m downstream of its confluence with BaluRiver to supply drinking water to the residents of the nation's capital. In recent years, the water quality of Sitalakhya has been widely studied particularly because of the high intensity of industries along its reach. A study conducted during 2008-2009 assessed the effect of industrial pollution on water quality of the Sitalakhya river. Figure shows the profile of some significant water quality parameters along the entire reach of the Sitalakhya river during the dry season when the water quality scenario is the most critical. These profiles indicate a deterioration of water quality parameters (increased BOD_5, NH_3 and depleted Dissolved Oxygen) starting from around 30 km downstream of Ghorashal Bridge. This is because there is a high density of industrial installations on both sides of the river as well as connections to polluted drainage canals. It may be noted here that, the increase in BOD_5 starting from 30 km downstream of Ghorashal bridge coincides with the starting point of the depletion of DO levels in the river (i.e. the *DO sag curve*, described later). The BOD_5 levels eventually come down further downstream near the confluence of Dhaleshwari river and consequently the DO levels also tend to recover to some extent. Also, high ammonia concentration in the river (highest near Sarulia, where it threatens the operation of the Saidabad water treatment plant) eventually is decreased downstream mostly due to dilution. Some conversion from Ammonia to nitrate takes place as there is an increasing trend in nitrate levels. Since Sitalakhya is not a long river, the residence time of these pollutants in this river is unlikely to exceed the typical time required for NBOD to fully exert.

depleting the dissolved oxygen resources in the process. This process is called deoxygenation. Streams and rivers can naturally assimilate biodegradable wastes to some extent and thus recovering from the effect of pollution without significant or permanent environmental damage. This capacity for self-purification depends on the strength and volume of pollutants as wells as the discharge or flowrate of the rivers. It is commonly said that "the solution to pollution is dilution". The effects of dilution and the constant flushing action of the flowing water are obvious factors involved in the waste assimilative capacity of a stream. But not as obvious, but equally important, is the effect of oxygen transfer between the air-water interface, a process termed as reaeration. Atmospheric oxygen is constantly being dissolved through the air-water interface replenishing the DO in the water. Fast-flowing, shallow, turbulent streams are reaerated more effectively than slow, deep, meandering streams. This is because in turbulent, shallow streams, the water becomes well-mixed throughout its depth and there is a constant renewal of air-water interface.

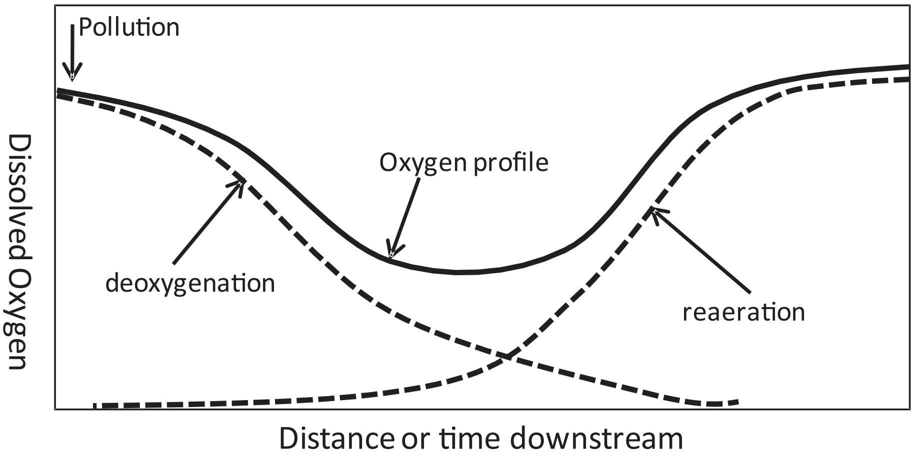


Figure 11.10 The dissolved oxygen sag curve showing the effect of oxygen-demanding wastes on the DO levels in a stream or river.

The rate of deoxygenation is proportional to the available BOD. Since the BOD immediately after the discharge is the highest and decreases gradually over time, the slope of the deoxygenation curve will be steep initially and will become gradually milder (see Figure 11.10). On the other hand, the rate of oxygen transfer from the air into the water depends on the difference between the saturation DO and the actual DO of the water. The larger the difference is, the faster will be the rate of reaeration. The slope of the reaeration curve gradually increases as the deoxygenation curve falls (see Figure 11.10). At any given time, the DO level in the stream is a combined effect of deoxygenation and reaeration. In other words, the actual DO is equal to the sum of the DO on the deoxygenation curve and the DO on the reaeration curve. The resultant DO curve will have a dip where the DO is minimum and is also known as the *DO sag curve*. Since the product of velocity and time is distance, the horizontal or x-axis in Figure 11.10 can also be labeled as distance for a given reach of the stream and the DO curve is practically the profile view of the DO concentrations along the length of the stream and also called the *dissolved oxygen profile*.

The purpose of this section is to develop a simplified mathematical tool to predict the spatial DO concentration in a river system. This is accomplished by performing simple analytical treatment to the equations describing deoxygenation and reaeration, the two key processes governing the DO balance in a river. For the sake of simplicity, it is assumed that there is a continuous discharge of waste at a given location on the river, the waste is uniformly mixed at any given cross section of river, and also there is no dispersion of wastes in the direction of flow.

Mixing and dilution. When a point discharge of wastewater enters a flowing stream, the physical process of mixing and dilution begins immediately. But with the exception of small turbulent streams, it is unlikely that the pollutants will be thoroughly mixed in the stream flow at or near the point of discharge. Instead, a waste plume forms, as illustrated in Figure 11.11. The length of this gradually widening mixing zone depends on the channel geometry, the flow velocity, and the design of the discharge pipe.

In water pollution control, it is often necessary to predict the BOD concentrations and DO levels downstream from a sewage discharge point. One of the first computations needed for

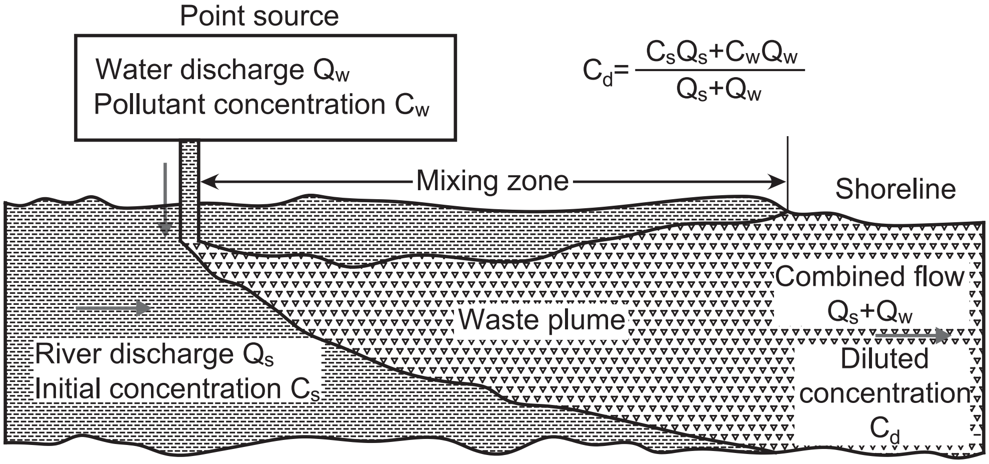


Figure 11.11 Dilution of pollutants from a point source, such as a sewage treatment plant, occurs within the mixing zone of the stream.

this involves the effect of dilution. Assuming that the pollutant is completely mixed in the streamflow (at a point just below the end of the mixing zone), the diluted concentration of any water quality parameter can be calculated using the following mass balance equation:

$$C_d = \frac{C_s Q_s + C_w Q_w}{Q_s + Q_w} \qquad 11.7$$

Where, C_d = diluted concentration
C_s = original concentration
C_w = concentration in the wastewater
Q_s = stream discharge
Q_w = waste discharge

Deoxygenation. As mentioned earlier, the rate of deoxygenation at any point in the river is assumed to be proportional to the BOD remaining at that point. That is,

$$\text{Rate of deoxygenation} = k_d L_t \qquad 11.8$$

where k_d = the deoxygenation rate constant (day^{-1})
L_t = the BOD remaining t (days) after the wastes enter the river (mg/L)

Substituting Eq. (6.14), which gives BOD remaining at any time *t*, into Eq. (11.8) gives

$$\text{Rate of deoxygenation} = k_d L_0 e^{-k_d t} \qquad 11.9$$

where L_0 is the BOD of the mixture of streamwater and wastewater at the point of discharge estimated using Eq.(11.7)

The deoxygenation rate constant k_d is often assumed to be the same as the (temperature adjusted) BOD rate constant *k* obtained in a standard, laboratory BOD test. For deep, slowly moving rivers, this seems to be a reasonable approximation, but for turbulent, shallow, rapidly moving streams, the approximation is less valid. Such streams have deoxygenation constants that can be significantly higher than the values determined in the laboratory.

In general, BOD is exerted more rapidly in a river because of turbulent mixing, larger number of "seed" organisms, and BOD removal by organisms in the stream bed as well as by those suspended in the water. A method has been developed to estimate k_d from *k* using the characteristics of the stream (Bosko, 1966):

$$k_d = k + \frac{u}{H}\eta \qquad 11.10$$

where k_d = deoxygenation coefficient at 20 °C (day^{-1})
k = BOD rate constant at 20 °C (day^{-1})
u = average stream velocity (m/s)
H = average stream depth (m)
η = bed activity coefficient

The bed activity coefficient may vary from 0.1 for stagnant or deep water to 0.6 or more for rapidly flowing streams. Adjustments to the deoxygenation rate constant for temperatures other than 20°C can be made using Eq. (6.17).

Example 11.4 Downstream BOD. A municipal wastewater treatment plant discharges 1.2 m^3/s of treated effluent having an ultimate BOD of 60.0 mg/L into a stream that has a flow of 9.3 m^3/s and a BOD of its own equal to 6.0 mg/L. The deoxygenation constant k_d is 0.20/day.

a. Assuming complete and instantaneous mixing, estimate the ultimate BOD of the river just downstream from the outfall.
b. If the stream has constant cross section so that it flows at a fixed speed equal to 0.30 m/s, estimate the BOD of the stream at a distance 40,000 m downstream.

Solution

a. *The BOD of the mixture of effluent and stream water can be found using Eq. (11.7):*

$$L_0 = \frac{1.2 \times 60 + 9.3 \times 6.0}{1.2 + 9.3} = 12.2 \text{ mg/L}$$

b. *At a speed of 0.30 m/s, the time required for the waste to reach a distance 30,000 m downstream would be*

$$t = \frac{40000}{0.30} \text{ s} = 1.54 \text{ days}$$

So, the BOD remaining at that point, 40 km downstream, would be

$$L_t = L_0 e^{-k_d t} = 12.2 e^{-(0.2 \times 1.54)} = 8.96 \text{ mg/L}$$

Reaeration. The rate at which oxygen is replenished is proportional to the difference between the actual DO in the river at any given location, and the saturated value of dissolved oxygen:

$$\text{Rate of reaeration} = k_r (DO_s - DO) \qquad 11.11$$

where k_r = reaeration constant (time^{-1})
DO_s = saturated value of dissolved oxygen
DO = actual dissolved oxygen at a given location in the river

The reaeration constant k_r is very much dependent on the particular conditions in the river. A fast moving, shallow stream will have a much higher reaeration constant than a sluggish stream or a pond. Many attempts have been made to empirically relate key stream parameters to the reaeration constant, with the most commonly used formulation being the following (O'Connor and Dobbins, 1958):

$$k_r = \frac{3.9\sqrt{u}}{H^{3/2}} \qquad 11.12$$

where k_r = reaeration coefficient at 20 °C (day^{-1})
u = average stream velocity (m/s)
H = average stream depth (m)

Typical values of the reaeration constant k_r for various bodies of water are given in Table 11.6. Adjustments to the reaeration rate constant for temperatures other than 20°C can be made using Eq. (6.17) but with a temperature coefficient θ equal to 1.024

The solubility of oxygen in water (DO_s) varies with temperature, barometric pressure and salinity. Table 11.7 gives representative values of the solubility of oxygen in water at various temperatures and chloride concentrations.

Table 11.6 **Typical reaeration constants for various water bodies.**

Water body	**Range of k_r at 20°C (base e) (day^{-1})**
Small ponds and backwaters	0.10-0.23
sluggish streams and large lakes	0.23-0.35
Large streams of low velocity	0.35-0.46
Large streams of normal velocity	0.46-0.69
Swift streams	0.69-1.15
Rapids and waterfalls	>1.15

Source : Tchobanoglous and Schroeder (1985)

DO sag curve. As illustrated in Figure 11.10, the DO sag curve represents the resultant effect of deoxygenation and reaeration. Therefore the equation describing the DO sag curve (or the *DO sag equation*) is derived from the superposition of the equations describing deoxygenation and reaeration. In practice, the DO sag equation is described using oxygen deficit rather than dissolved oxygen concentration to make it easier to solve the integral that results from the mass balance. The oxygen deficit (D) is defined by the amount by which the actual dissolved oxygen concentration is less than the saturation value:

$$D = DO_s - DO \qquad 11.13$$

The saturation value of oxygen for a particular temperature and salinity is derived from Table 11.7. The term *initial DO deficit* is used to define the DO at the beginning of the sag curve where the waste discharge mixes with the river. The initial deficit is calculated as the difference between the saturated DO and the concentration of DO after mixing (Equation 11.7)

Table 11.7 **Solubility of oxygen in water (mg/L) at 1 atm pressure.**

Temperture(°C)	**Chloride concentration in water (mg/L)**			
	0	5000	10000	15000
0	14.62	13.73	12.89	12.10
5	12.77	12.02	11.32	10.66
10	11.29	10.66	10.06	9.49
15	10.08	9.54	9.03	8.54
20	9.09	8.62	8.17	7.75
25	8.26	7.85	7.46	7.08
30	7.56	7.19	6.85	6.51

Source: Thomann and Mueller (1987)

Therefore, combining the two equations (11.9) and (11.11) yields the following mass balance equation which describes the rate of increase of the oxygen deficit:

Rate of increase of the deficit = Rate of deoxygenation - Rate of oxygenation (reaeration)

$$\frac{dD}{dt} = k_d L_0 e^{-k_d t} - k_r D \qquad 11.14$$

Integrating Eq. (11.14), the equation for the DO sag curve can be obtained (graphically shown in Figure 11.12)

$$D = \frac{k_d L_0}{k_r - k_d}\left(e^{-k_d t} - e^{-k_r t}\right) + D_0 e^{-k_r t} \qquad 11.15$$

Eq. (11.15) is the classic Streeter-Phelps oxygen-sag equation first described in 1925. In this equation, D_0 is the initial DO deficit and t represents the time of travel in the stream from the point of discharge to the point in question downstream.

If the stream has constant cross-sectional area, and it is traveling at a speed u, then time and distance downstream are related by

$$x = ut \qquad 11.16$$

where x = distance downstream
u = stream velocity
t = elapsed time between discharge point and distance x downstream

And Eq. (11.15) can be rewritten as:

$$D = \frac{k_d L_0}{k_r - k_d}\left(e^{-k_d x/u} - e^{-k_r x/u}\right) + D_0 e^{-k_r x/u} \qquad 11.17$$

Subtracting the oxygen deficit, given by (11.15) or (11.17), from the saturation value DO, gives DO as a function of time or distance downstream. A plot of this DO is given in Figure 11.12. As can be seen in the figure, the stretch of the river immediately downstream of the discharge point ($x = 0$ or $t = 0$) oxygen is depleted at a faster rate than aeration can replace

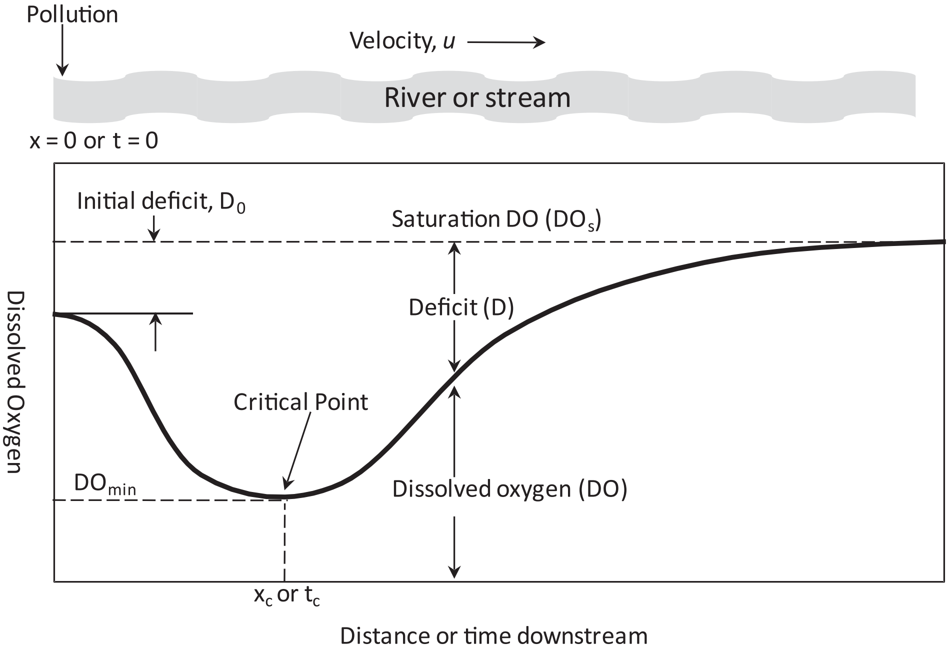


Figure 11.12 The Streeter-Phelps oxygen sag curve.

causing a dip in the DO curve. After a certain point in time or distance, the DO reaches a minimum. At this point, which is called the critical point, the rate of deoxygenation equals the rate of reaeration. Beyond the critical point, reaeration exceeds deoxygenation and the stream naturally recovers.

Zones of pollution. The DO sag curve resulting from a point source of pollution divides the stream into four relatively distinct zones which are illustrated in Figure 11.13. The first is the zone of degradation which occurs just below the location where the outfall sewer meets the river. In this zone fish population shows an increasing trend with the increase in the number of protozoa which feed on bacteria but gradually declines as the dissolved oxygen decreases. The river becomes turbid and dark in color with floating solids and other visual evidences of pollution. Dissolved oxygen drops rapidly and anaerobic decomposition starts at the bottom deposits of settable organics. In the bottom, sludge forms, which will contain redish worms and earth worms etc. In the case of extreme pollution, the zone may become devoid of oxygen and anaerobic decomposition starts. This is called the *zone of active decomposition* or *septic zone*. Sludge decomposition starts forming scum at the surface, objectionable odor by hydrogen sulphide becomes prominent and color of water becomes darkish. Fungi will disappear giving way to anaerobic bacteria. Higher forms of life is confined to sludge worms, rat tailed maggots, psychoda etc. Other higher forms of aquatic lives or desirable species either die or migrate out of the area. After most of the organics have been decomposed by the microbes in the water, the rate of reaeration will exceed the rate of deoxygenation, the dissolved oxygen increases towards its saturation value and the *zone of recovery* begins. This zone is characterized by gradually clearing water with no offensive odors with desirable aquatic species reappearing. The number of bacteria is diminished while protozoa, rotifer and crustacea populations are increased. When the organic loading is low or there is enough dilution, the zone of recovery may progress immediately after the zone of degradation without a zone of active decomposition. Following the zone of recovery is the *zone of clear water*. In this zone the natural aquatic ecosystem of the stream is restored. Dissolved oxygen content will be approaching closer to the saturation values, diverse species of aquatic

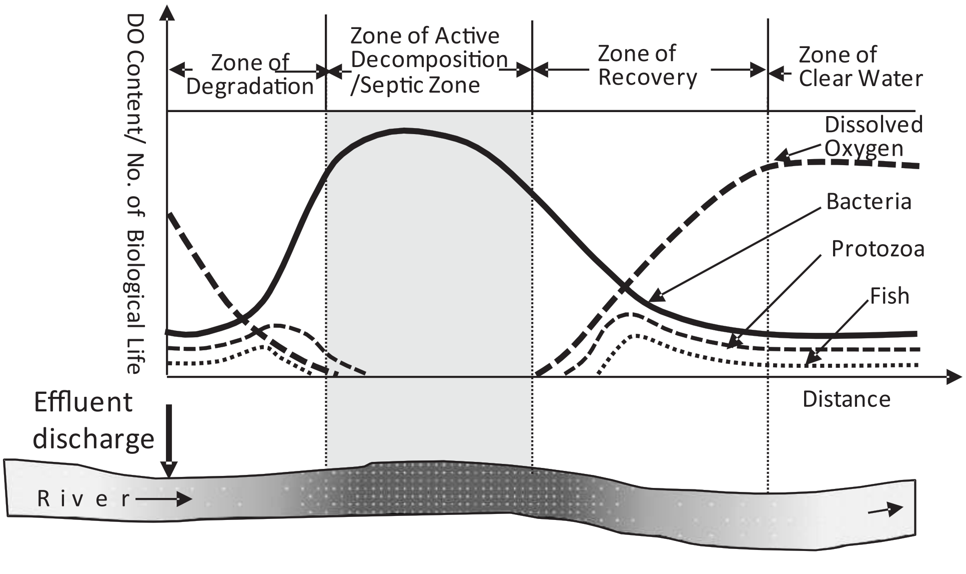


Figure 11.13 Pollution and self-purification of stream and changes in the aquatic ecology by the disposal of sewage/industrial wastewater.

organisms such as clear water algae will thrive which will use the stable inorganic nutrients remaining in the water. Higher form of aquatic lives will also be present. In other words, the stream has recovered its original quality through the process of self-purification. Additional point discharges along the reach will alter this model of pollution zones but nevertheless, this pollution model remains invaluable in understanding stream pollution and proposing technical solutions to the problem.

Computation of minimum DO. It is important to be able to predict the minimum dissolved oxygen level in a polluted stream or river and where it occurs. For example, if a new sewage treatment plant is to discharge its effluent into a stream, it is possible that conventional (secondary) treatment levels will not remove enough BOD to prevent excessively low DO downstream. To determine if some form of advanced treatment is required to preserve the water quality of the river, it is necessary to compute the minimum DO caused by the sewage effluent and to compare it to the allowable limits for a particular beneficial use.

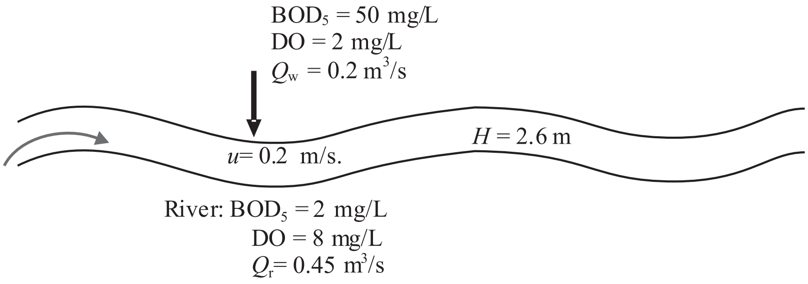
The time to critical point (t_c)can be found by differentiating Eq. (11.7), setting it equal to zero and solving it for t:

$$t_c = \frac{1}{k_r - k_d} \ln\left\{ \frac{k_r}{k_d} \left[1 - \frac{D_0 (k_r - k_d)}{k_d L_0} \right] \right\} \qquad 11.18$$

The maximum deficit can then be found by substituting the value obtained for the critical time in Eq. (11.15).

Example 11.5 Oxygen Sag Curve. A municipal wastewater treatment plant discharges 0.2 m^3/s of treated effluent having BOD_5 of 50.0 mg/L and DO of 2 mg/L into a stream that has a flow of 0.45 m^3/s and a BOD_5 of 2 mg/L and DO of 8 mg/L. The temperature of the river is 25°C. The deoxygenation constant k_d is 0.23/day at 20°C. The stream has a depth of 2.6 m and the average stream velocity is 0.2 m/s.

a. Find the critical distance downstream at which DO is a minimum.
b. Find the minimum DO.



Solution:

The BOD and DO of effluent and stream can be found using Eq. (11.7)

$$BOD_{5,mix} = \frac{2 \times 0.45 + 50 \times 0.2}{0.45 + 0.2} = 16.8\,\text{mg/L}$$

$$DO_{mix} = \frac{8 \times 0.45 + 2 \times 0.2}{0.45 + 0.2} = 6.2 \text{ mg/L}$$

Ultimate BOD of the stream, $L_0 = \frac{BOD_{5,mix}}{1 - e^{-kt}} = \frac{16.8}{1 - e^{-0.23 \times 5}} = 24.6 \text{ mg/L}$

The saturation value of DO at 25°C is given as 8.26 mg/L (from Table 11.6), so the initial deficit would be

$$D_0 = 8.26 - 6.2 = 2.06 \text{ mg/L}$$

To estimate the reaeration constant we can use the O'Connor and Dobbin's relationship given in Eq. (11.12)

$$k_r = \frac{3.9\sqrt{u}}{H^{3/2}} = \frac{3.9\sqrt{0.20}}{2.6^{3/2}} = 0.4 \text{ day}^{-1}$$

Adjusting the reaeration and deoxygenation constants for stream temperature (using Eq 6.17):

$$k_r(25°C) = 0.4\,(1.024)^{25-20} = 0.45 \text{ day}^{-1}$$

$$k_d(25°C) = 0.23\,(1.047)^{25-20} = 0.29 \text{ day}^{-1}$$

a. Using Eq. (11.18) we can find the time at which the deficit is maximum

$$t_c = \frac{1}{k_r - k_d} \ln\left\{\frac{k_r}{k_d}\left[1 - \frac{D_0(k_r - k_d)}{k_d L_0}\right]\right\}$$

$$= \frac{1}{0.45 - 0.29} \ln\left\{\frac{0.45}{0.29}\left[1 - \frac{2.06\,(0.45 - 0.29)}{0.29 \times 24.6}\right]\right\} = 2.45 \text{ days}$$

The critical distance downstream would be

$$x = ut_c = 0.2 \text{ m/s} \times 3600 \text{ s/hr} \times 24 \text{ hr/day} \times 2.45 \text{ days} = 42{,}336 \text{ m}$$

b. The maximum deficit can be found from Eq. (11.15)

$$D = \frac{k_d L_0}{k_r - k_d}\left(e^{-k_d t} - e^{-k_r t}\right) + D_0 e^{-k_r t} = \frac{0.29 \times 24.6}{0.45 - 0.29}\left(e^{-0.29 \times 2.45} - e^{-0.45 \times 2.45}\right) + 2.06 e^{-0.45 \times 2.45}$$

$$= 7.78 \text{ mg/L}$$

So the minimum value of DO will be the saturation value minus this maximum deficit:

$$DO_{min} = 8.26 - 7.78 = 0.48 \text{ mg/L}$$

Effect of temperature on DO sag curve. The effect of temperature on DO sag curve is important particularly in cases where thermal effluents are involved. The rate of deoxygenation increases while the solubility of oxygen decreases with increase in temperature. The combined effects of these are two folds: (1) the critical DO reduces and (2) the critical DO is reached at an earlier time (or distance). Thus a stream that may have sufficient DO in a colder period may have unacceptably low DO in a warmer season. Figure 11.14 shows this potential adverse impact associated with a change in seasonal temperature.

Effect of NBOD. So far the effect of carbonaceous BOD has been considered in the DO sag curve. However in some cases the nitrogenous BOD may have similar impact on DO levels. Nitrogenous BOD can be incorporated into the DO sag curve by adding an additional term to Eq. (11.15):

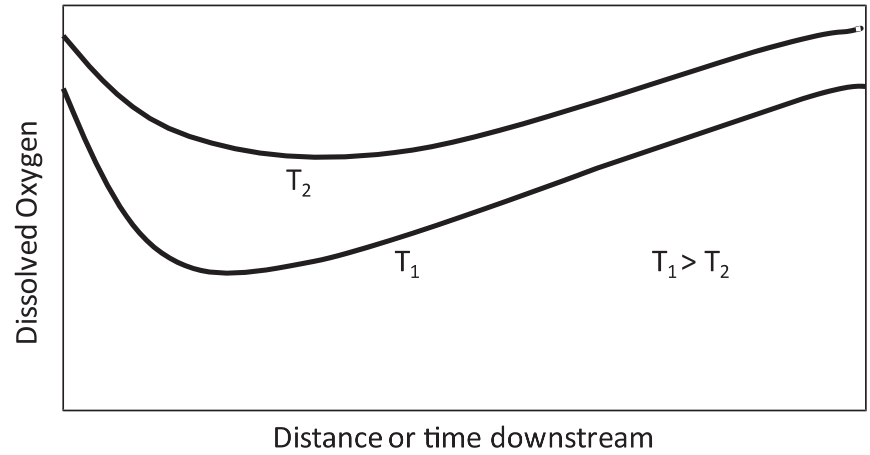


Figure 11.14 At higher temperature the minimum DO is lowered and occurs at a location closer to the discharge source.

$$D = \frac{k_d L_0}{k_r - k_d}\left(e^{-k_d t} - e^{-k_r t}\right) + D_0 e^{-k_r t} + \frac{k_n L_n}{k_r - k_n}\left(e^{-k_n t} - e^{-k_r t}\right) \qquad 11.19$$

where k_n = the nitrogenous deoxygenation rate constant (day^{-1})
L_n = ultimate NBOD after waste and river have mixed (mg/L)

With the additional term for NBOD, it is not possible to find the critical point analytically as in Eq. (11.18). It must be found by trial and error solution of Eq. (11.19).

Multiple point sources. The classical DO sag curve assumes that there is only one point source discharge of waste into the river. In reality, there may be multiple point sources. Multiple point sources can be handled by dividing the river into multiple reaches, with each of the point sources located at the beginning of each reach. The oxygen level and residual BOD can be calculated at the end of each reach which will be subsequently used to determine the initial conditions (parameters D_0 and L_0 of Eq. (11.15)) of the following reach. Dividing the river into reaches is also necessary if there are changes in flow regime (e.g. changes in aeration due to depth changes due to dredging, installation of dams etc.). Figure 11.15 shows an application of the DO sag equation for Buriganga river for multiple point sources.

Other factors affecting DO levels. The oxygen-sag curve is affected by a number of other factors besides those already discussed. If there are large deposits of organic matter in river sediments, they shall exert their own oxygen demand. These organic matters can be natural deposits of leaves, dead aquatic plants or can be sludge deposits from wastewaters receiving little or no treatment. Decomposition of these organic matters will utilize the oxygen resources in the overlying water column and if their oxygen demand becomes significant,

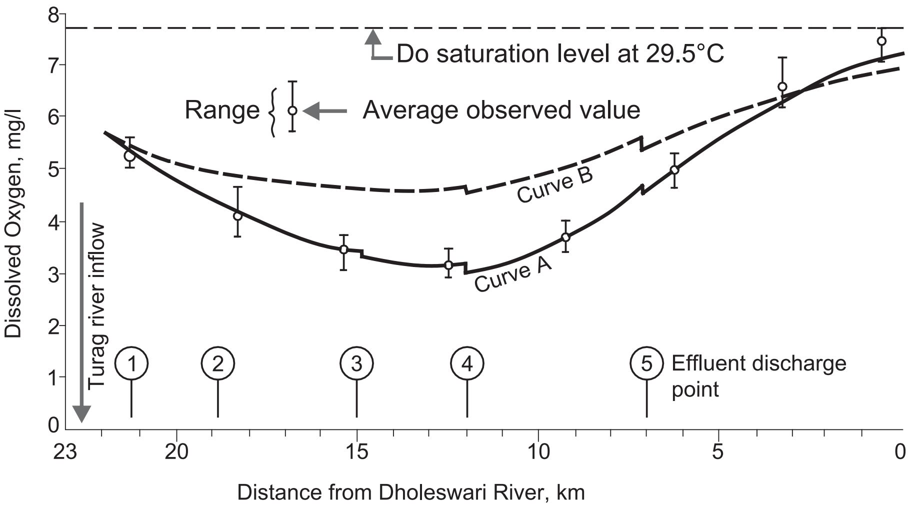


Figure 11.15 **Application of the Streeter-Phelps DO sag equation in computing the DO of Buriganga river for multiple point sources. Curve A is the present predicted dry flow profile with observed river sampling results. Curve B is the expected dry flow DO profile after implementation of pollution control measures (Ahmed and Mohammed, 1988).**

they must be included in the water quality model. Algae and aquatic plants can add DO during the daytime hours while photosynthesis is occurring, but deplete the DO at night for respiration. The net effect is a diurnal variation in DO that can lead to elevated levels of DO in the late afternoon and lower levels at night. Plant growth is usually highest in the summer when flows are low and temperatures are high, so that large nighttime respiration requirements can coincide with the worst cases of oxygen depletion from BOD exertion. To properly model all of these effects and their interactions is an exceedingly difficult task. Sophisticated water quality models, available commercially as software packages, rely on computers to simulate all these interactions and can predict water quality parameters relatively accurately. However, the simplified model presented above can serve as a first approximation to reality.

Application of the DO curve in water quality management. The DO sag curve can be used to assess the adequacy of water treatment facilities or to determine the location of an effluent discharge outfall. DO standards are generally set to protect the aquatic species in a river or to make the river useful for a particular purpose. For a known waste discharge and a known set of river hydromorphological characteristics, the DO sag equation can be applied to find the DO at the critical point. If this value is higher than the standard, then the river has sufficient assimilation capacity. If this value goes below the standard then additional management measures are required. Usually the environmental engineer operating the treatment plant has control over two parameters, L_0 and D_0. By increasing the efficiency of the treatment process or by adding additional treatment steps, the BOD of the wastewater can be reduced and thereby reducing L_0. Often, the stream quality can be improved by aerating the wastewater and bringing it close to saturation prior to discharge and thereby reducing the initial DO deficit of the mixed stream (D_0). If none of these work to meet the standards, alternate options are to be sought and these include building structures across rivers to promote aeration, looking for alternate outfall locations where dissolved oxygen levels are higher etc.

When using the DO sag equation, it is important to determine the adequacy of management strategies under the worst case scenario (using the conditions that will cause the lowest DO concentration). Usually these conditions occur when the flow of the river is low and temperatures are high. A frequently used criterion is the "10-year, 7-day low flow," which is the recurrence interval of the average low flow for a 7-day period estimated using the partial duration series technique. Low river flows reduce dilution of wastewater, thereby increasing L_0 and D_0. The value of k_r is usually reduced by low flow because of smaller flow velocities. Also in higher temperatures, k_d is increased and DO saturation is decreased. The simulation of DO under these circumstances will be the worst possible condition of the stream.

11.6 Water Quality in Lakes and Reservoirs

The pollution of natural lakes or conservation reservoirs poses problems that are somewhat different from the problems caused by pollution of streams or rivers. This is primarily because of the difference in physical characteristics between lakes and streams. Water in a stream is constantly in motion and providing a flushing action for incoming pollutants. But in lakes, apart from its internal circulations, water is relatively still and retained for longer periods of time. In some cases, pollutants discharged into a lake can remain there for many years. Seasonal temperature changes also affect the water quality in lakes and reservoirs.

In streams, organic pollutants affect the oxygen profile. In lakes, water quality may be more dependent on plant nutrients than on organics from sewage. Phosphorus and nitrogen are the most critical plant nutrients. When pollutants containing phosphorus and nitrogen compounds accumulate in a lake, rooted aquatic plants and free-floating algae may grow profusely. This excessive growth, known as *algal blooms*, forms slimy mats that float on the lake surface(Figure 11.16 (a)). They are unsightly and, along with the thick growths of weeds that develop along the shore, they interfere with boating, swimming, and fishing and diminish the quality of the lake for recreational purpose. Although algae tend to grow very quickly under high nutrient availability, they are short-lived and their death results in high concentration of organic matter which starts to decay. The decay process consumes dissolved oxygen in the water. Without sufficient dissolved oxygen in the water, aquatic lives may die off in large numbers. Some algae can release toxins which can also kill fish in massive numbers (Figure 11.16 (b)). If the lake water is used for water supply, the algae may clog the filters in the treatment facility and require additional measures for cleaning. This may increase the cost of treatment. Furthermore, algae impart undesirable taste and odor to the water and additional chemicals may be required to make the water more palatable.

The algae and aquatic weeds eventually die and settle to the bottom of the lake, where they are decomposed by bacteria and protozoa. This exerts an oxygen demand on the water column and may deplete the DO in parts of the lake. Decaying plants, along with silt carried into the lake by overland runoff and feeder streams, gradually accumulate in significant amounts as sediment at the lake bottom. This makes the lake shallower and can alter the ecological environment of the lake including a shift in aquatic species.

A knowledge of the lake systems is essential to understand the role of nutrients in determining the lake water quality. This section describes the natural life-cycle of a lake and the characteristics of a typical lake environment and how seasonal variations and nutrient

Figure 11.16 (a) A spectacular “red tide” bloom (non-toxic) of Noctilucascintillans in New Zealand. (Photo by M. Godfrey) (b) Dead fish from a Kareniabrevis bloom in Texas. At high concentrations, toxins produced by this organism can cause massive fish kills. (Photo by Brazosports) (Image source: Woods Hole Oceanographic Institute).

Box 11.7

Algal bloom in the Bay of Bengal and its human health implications

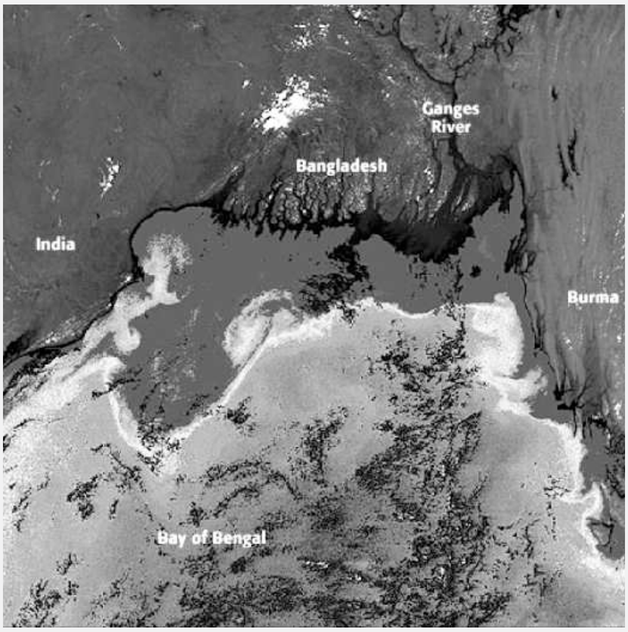


Figure: A CZCS image showing the ocean color of Bay of Bengal

Using satellite measurements, such as the Coastal Zone Color Scanner (CZCS), researchers can identify algal blooms in the ocean all over the world and quantify algal biomass from the color intensity. The figure shows a CZCS image for the Bay of Bengal taken in October, 1982 showing algal blooms along the coast triggered by the nutrient loading brought to the bay by the major rivers and their tributaries. These annual algal blooms are characteristic of coastal areas near densely populated cities. Recent investigations revealed that the emergence of phytoplankton in the Bay of Bengal is correlated with the occurrence of cholera outbreaks in coastal regions and other cities in Bangladesh. Vibrio cholerae, the bacteria responsible for cholera disease has been known to associate with phytoplankton and it is likely that the bacterial populations in inland and coastal waters also thrive during the time of algal blooms. The cholera outbreaks in this region might also include other factors which are complex and difficult to quantify. Therefore, although the relationship of several climatic factors as well as algal blooms and cholera have been found to be significant, developing the most precise predictive model for cholera outbreak in this region is still a challenge and is a field of active research. (*Source: www.nasa.gov*)

loading can affect these characteristics. Emphasis will be given on the effect of phosphorus in lakes and phosphorus loading management strategies because phosphorus is usually the limiting nutrient in such environments.

Eutrophication and lake productivity. Eutrophication is a natural process in which lakes gradually become shallower and more productive through the introduction and cycling of nutrients. A lake's productivity may be determined by measuring the amount of algal growth that can be sustained by the available nutrients. Most lakes start out geologically as deep, cold, clear bodies of water. At this stage, they are called *oligotrophic* lakes. They usually have a sand or rock bottom, very few nutrients, a very low level of productivity due to the scarcity of nutrients and high levels of oxygen. Over the years, nutrients slowly accumulate and various forms of aquatic lives appear. Silty sediments begin to form at the bottom as the lake passes through a *mesotrophic* stage of existence. The *eutrophic* stage of a lake's life cycle is characterized by a relatively shallow and warmer body of water, with enough nutrients to support large populations of plants and diverse aquatic lives and relatively low levels of oxygen. In a eutrophic lake, there are frequent algal blooms, as previously described, and at certain times of the year the water at the bottom may be devoid of dissolved oxygen. Although a more productive lake usually will have a higher fish population, the number of the most desirable fish may decline. Table 11.8 summarizes the differences between oligotrophic and eutrophic lakes. Further aging or eutrophication leads to what is called a senescent lake, characterized by thick deposits of organic silts and very high nutrient levels. Senescent lakes are very shallow, with much rooted emergent vegetation growing

Table 11.8 General characteristics of eutrophic and oligotrophic lakes.

Character	Eutrophic	Oligotrophic
Basin shape	Broad and shallow	Narrow and deep
Lake substrate	Fine organic salt	Stones and inorganic silt
Lake shoreline	Weedy	Stony
Light penetration	low	high
Water color	Yellow or green	Green or blue
Chlorophyll-a concentration (μg/L)	>15	0.3 – 2.5
Total P (ppb)	10 - 30	<1 – 5
Total N (ppb)	300 - 650	<1 – 200
Oxygen	High at the surface, low under the thermocline	High
Phytoplankton	Few species, high numbers	Many species, low numbers
Zooplankton	Few species, high numbers	Many species, low numbers
Macroinvertebrates	Many species, high numbers	Moderate species, low numbers
Fish	Many species	Few species

Source: Adapted from Kiely, 1998

throughout the lake. Eventually, what was once a lake will become a marsh as natural geological and ecological processes continue. The aging of a lake is illustrated in Figure 11.17.

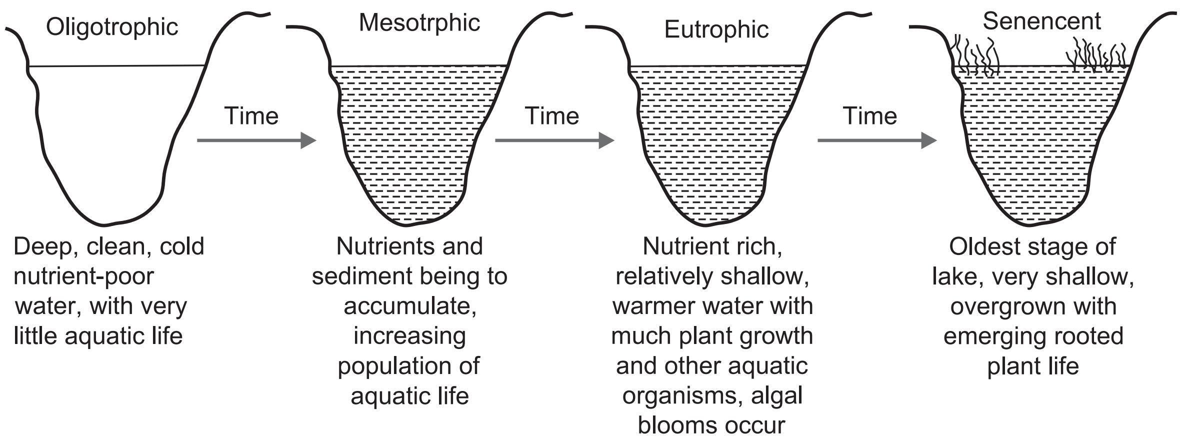


Figure 11.17 The different stages of the life cycle of a lake.

The aging process of the lake is natural and inevitable and lake eutrophication, from the oligotrophic through the senescent stages, takes many thousands of years. It is an exceedingly slow process. But many people use the term *eutrophication* synonymously with pollution in reference to lakes. To eliminate the ambiguity, the term cultural eutrophication is often used in this regard. *Cultural eutrophication* is caused when human activity accelerates these naturally occurring processes where excess nutrients are added from fertilizers or sewage input leads to pollution. Similarly, damming a river leads to concentration of organic matter upstream of the dam and can lead to enriched, lake-like conditions with increased primary production. The Kaptai Lake located upstream of the Karnaphuli river in Bangladesh is such a manmade lake.

Eutrophic lakes are necessarily polluted, but pollution contributes to eutrophication. Water quality management in lakes is primarily concerned with slowing eutrophication to at least the natural rate. This is generally accomplished by controlling phosphorus. To understand why this is so, it is necessary to understand the factors contributing to algal growth and why phosphorus is the limiting nutrient.

The role of phosphorus in eutrophication. There are many factors that control the rate of primary production in a lake, including the availability of sunlight to power the photosynthetic reactions and the concentration of nutrients required for growth.The amount of light that can penetrate through the water to aid the photosynthesis reactions is related to the transparency of the water, which is in turn a function of the level of eutrophication. An oligotrophic lake may allow sunlight to penetrate to considerable depths (~ 100 m or more), while eutrophic lakes may be so murky that photosynthesis is limited to a thin layer of water very near the surface. While the amount of sunlight available can be a limiting factor in algal growth, it is not something that one would imagine controlling as a way to slow eutrophication. Since nutrient stimulation by human activity is the prime cause of cultural eutrophication, restricting the available nutrients to control cultural eutrophication is a more feasible option. All algae require macronutrients such as carbon, nitrogen, and phosphorus, and micronutrients such as trace elements and the lack of any one nutrient will essentially limit the total algal population.

Among the nutrients required for algal growth, nitrogen and carbon can be derived mostly from natural sources. For example, carbon is derived from carbon dioxide dissolved in water which is pretty much always available and since the atmosphere is virtually an inexhaustible source of this gas. Although nitrogen in lakes is usually in the form of nitrate (NO_3^-) which comes from external sources by way of inflowing streams or groundwater as a result of human activities, the atmosphere itself is an unending repository of nitrogen gas. Some photosynthetic microorganisms can also fix nitrogen gas from the atmosphere directly by converting it to organic nitrogen. In lakes the most important nitrogen-fixing microorganisms are photosynthetic bacteria called cyanobacteria, formerly known as blue-green algae because of the pigments they contain. Because of their nitrogen-fixing ability, cyanobacteria have a competitive advantage over green algae when nitrate and ammonium concentrations are low but other nutrients are sufficiently abundant. Therefore, controlling the Carbon and Nitrogen inputs is practically not feasible since algae can find a way to proliferate even without the inputs generated from human activities. On the other hand, Phosphorus in lakes originates from external sources only and is taken up by algae in the inorganic form (PO_4^{3-}) and incorporated into organic compounds. The relative amounts of nitrogen and phosphorus that are required for algal growth can be estimated from stoichiometric considerations using the general algal photosynthesis equation:

$$106\,CO_2 + 16\,NO_3^- + HPO_4^{2-} + 122\,H_2O + 18\,H^+ \rightarrow C_{106}H_{263}O_{110}N_{16}P + 138\,O_2$$

The ratio of the weights of nitrogen to phosphorus in this algae would be

$$\frac{N}{P} = \frac{16 \times 14}{1 \times 31} = 7.2$$

For a first approximation, it takes about 7 times more nitrogen than phosphorus to produce a given amount of algae. Accounting for variations in plant stoichiometry, however, N/P ratios in a body of water over 20 generally indicate that phosphorus is the limiting nutrient, whereas N/P ratios of 5 or less reflect nitrogen limited systems.

Of all the nutrients, only phosphorus is not readily available from the atmosphere or the natural water supply. For this reason, phosphorus is deemed the limiting nutrient in lakes. The amount of phosphorus controls the quantity of algal growth and therefore the productivity of lakes. It typically takes only a concentration of about 0.02 mg/L of inorganic phosphorus to cause algal blooms in a lake; the inorganic nitrogen concentration can be more than 10 times that level. On the other hand, even with very high nitrogen levels, if phosphorus concentrations are kept below 0.02 mg/L, excessive growths of algae usually do not occur.

There is a direct correlation between the phosphorus concentration and chlorophyll a, one of the green pigments involved in photosynthesis and generally used as a proxy to algal concentration in the water body. The empirical relationship is given by:

$$\log(Chlorophyll\ a) = -1.09 + 1.46 \log P_T \qquad 11.20$$

where, *Chlorophyll a* = concentration of chlorophyll a, mg/m^3

P_T = total phosphorus concentration, mg/m^3

A simple phosphorus balance in a lake. Since phosphorus is the most common limiting nutrients for lake eutrophication, research in the past few decades mostly centred around its availability and significance in the lake environment. An idealized lake model is shown in Figure 11.18. Phosphorus can enter the lake from a variety of sources, including the incoming stream flow, runoff from adjacent lands, and industrial or municipal point sources. It is removed by both settling into sediments and by flowing out with the stream flow leaving the lake.

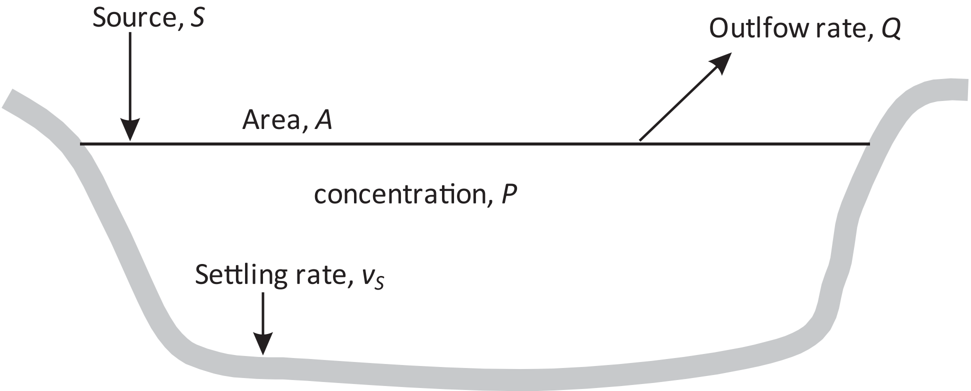


Figure 11.18 A simplified phosphorus model for a lake.

If we assume that the lake is well mixed, and that steady-state conditions prevail, we can write the following mass balance equation for phosphorus incorporating the above mentioned two competing processes:

Rate of addition of P = Rate of removal of P

$$S = QP + V_s AP \qquad 11.21$$

where S = the rate of addition of phosphorus from all sources (g/s)

P = the concentration of phosphorus (g/m^3)

Q = the stream outflow rate (m^3/s)

v_s = the phosphorus settling rate (m/s)

A = the surface area of the lake (m^2)

The steady-state concentration of Phosphorus is therefore,

$$P = \frac{S}{Q + v_s A} \qquad 11.22$$

The settling rate v_s is an empirically determined quantity that is difficult to predict with any confidence. Thomann and Mueller (1987) suggest that lakes have a settling rate of approximately 3-30 m/year.

Example 11.6. **Phosphorus loading in a lake.** Consider a lake with 200×10^6 m^2 of surface area for which the only source of phosphorus is the effluent from a wastewater treatment plant. The effluent flow rate is 0.45 m^3/s and its phosphorus concentration is 10.0 mg/L (=

10.0 g/m^3). The lake is also fed by a stream having 30 m^3/s of flow with no phosphorus. If the phosphorus settling rate is estimated to be 10 m/year.

(a) Estimate the average phosphorus concentration in the lake.
(b) What level of phosphorus removal at the treatment plant would be required to keep the average lake concentration below 0.010 mg/L?

Solution

(a) The phosphorus loading from the wastewater treatment plant

$S = (10 \times 0.5) = 5$ g/s

This is the total phosphorus loading in the lake.

The flow rate out of the lake would be the sum of the inlet stream flow and the effluent flow.

$Q = (30 + 0.5) = 30.5\ \text{m}^3/\text{s}$

The estimated settling rate is: $\text{vs} = \dfrac{10}{365 \times 24 \times 3600} = 3.17 \times 10^{-7}$ m/s

Using Eq. (11.22), the steady-state phosphorus concentration would be

$$P = \frac{S}{Q + v_s A} = \frac{5}{30.5 + 3.17 \times 10^{-7} \times 200 \times 10^6} = 0.053\ \text{mg/L}$$

This is above the 0.01 mg/L limit which is suggested as an acceptable concentration.

(b) To reach 0.01 mg/L, the total phosphorus loading must be

$$S = P(Q + v_s A) = 0.010\left(30.5 + 3.17 \times 10^{-7} \times 200 \times 10^6\right) = 0.94\ \text{g/s}$$

The wastewater is currently contributing 5 g/s of phosphorus. There is need for 81 per cent phosphorus removal.

Thermal stratification in lakes. Nutrients stimulate algal growth, and the subsequent death and decay of that algae can lead to oxygen depletion. This oxygen depletion problem can be exacerbated by certain physical characteristics of the lake such as thermal stratification.

An important property of water is that density is greatest at approximately 4°C – water above or below this temperature floats on water at 4°C. Also, warmer water floats on cooler water. A lake warmed by the sun during the summer will tend to have a layer of warm water floating on top of the denser, colder water below. Conversely, in the winter, if the lake's surface drops below 4°C, it will create a layer of cold water that floats on top of the more dense, 4°C water below. During the year, as the water body warms and cools seasonally, a changing temperature profile along the depth emerges. In winter, temperatures are relatively uniform throughout the lake and any wind action can practically mix the lake water from top to bottom. Progressing through Spring and Summer, the surface of the water starts to get heated up and the density differences between surface water and the water nearer to the bottom inhibit vertical mixing in the lake. As temperature increases, discontinuity becomes more pronounced until the water body is fully stratified into three parts: a warm *epilimnion* on top, a cold *hypolimnion* at the bottom and a narrow region in between, called the *thermocline*, where the temperature changes rather sharply with depth. Figure 11.19 shows the stratification that typically occurs in a deep lake, in the temperate zone, during the summer. Thermocline creates a barrier preventing the two water layers above and below from being mixed up by the surface wind action.

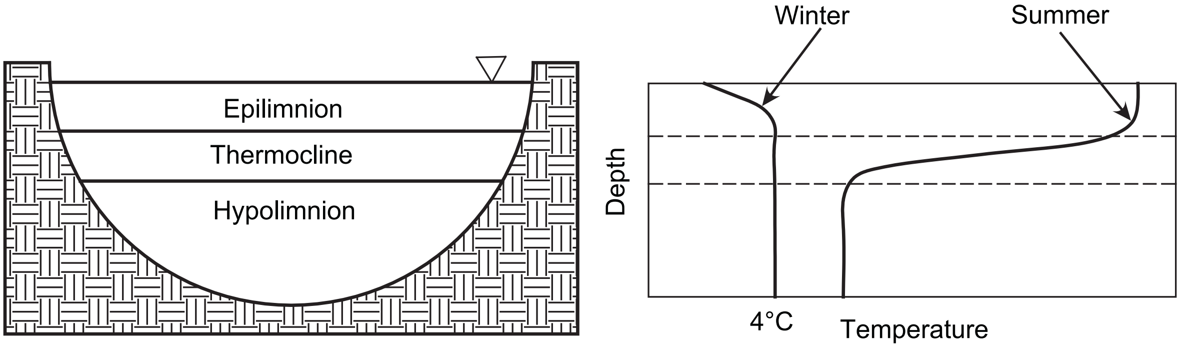


Figure 11.19 Thermal stratification of a lake showing winter and summer stratification temperature profiles.

The extent of stratification in lakes can be quantified by their densiometric Froude number (F):

$$F_D = \frac{V}{\sqrt{\frac{\Delta\rho}{\rho_0} Dg}} = \frac{\text{inertial force}}{\text{gravitational force}} \qquad 11.23$$

Where, ρ_0 = reference density

$\Delta\rho$ = density change over depth D

If $F_D > 0.32 \rightarrow$ no stratification

$0.01 < F_D < 0.32 \rightarrow$ moderately stratified

$F_D < 0.01 \rightarrow$ strongly stratified

Example 11.7. Stratification of a lake. Determine the stratification category for a lake if its length by width by depth is 10 km × 2 km × 25 m. The summer discharge is 10 m^3/s. The surface temperature in the summer is 25°C.

Solution

$$V = \frac{Q}{BD} = \frac{10}{2000 \times 25} = 2 \times 10^{-4}\ \text{m/s}$$

$\rho_{surface} = 997\ \text{kg/m}^3$

$\rho_0 = 1000\ \text{kg/m}^3$

$$F_D = \frac{V}{\sqrt{(\Delta\rho/\rho_0)Dg}} = \frac{2 \times 10^{-4}}{\sqrt{[(1000-997)/1000]\,25 \times 9.81}} = 2.3 \times 10^{-4} << 0.01$$

Therefore, the lake is strongly stratified.

Thermal stratification and lake water quality. Thermal stratification in lakes has major effects on both oxygen concentration and nutrient availability. Because of good mixing (reaeration) and photosynthesis the epilimnion will be rich in DO. But the hypolimnion, which is practically cut off from the overlying DO-rich waters, will have a lower DO and may become anaerobic (devoid of oxygen). The only source of oxygen in the hypolimnion will be the result of photosynthesis that will occur only if the water is clear enough to allow the euphotic zone (the upper layer of water through which sunlight can penetrate) to extend below the thermocline. That is, the hypolimnion of the clear, oligotrophic lake at least has the possibility of having a source of oxygen, while that of the eutrophic lake does not. In addition, the eutrophic lake is rich in nutrients and organic matter. The settling organic debris into the hypolimnion leads to increased oxygen demands due to decomposition. During the course of summer, the stratification becomes more and more stable as the epilimnion is further heated creating more pronounced density differences in the water column. In the extreme case, the hypolimnion of a eutrophic lake can become anaerobic during the summer, as is suggested in Figure 11.20. On the other hand, the epilimnion, where the plants are, receives no dissolved nutrients from the bottom, where the decomposition occurs, so primary productivity becomes nutrient limited and declines over the summer.

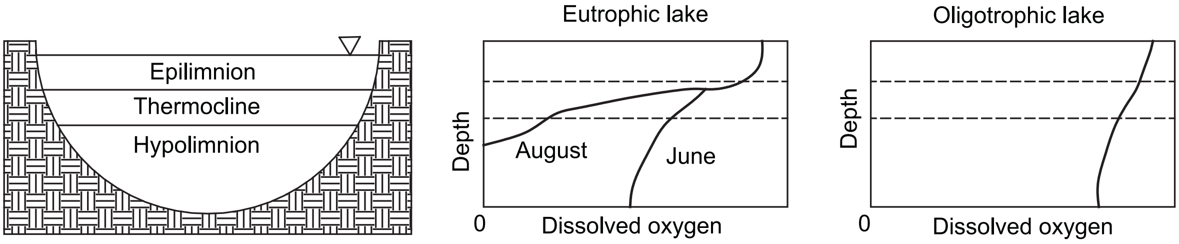


Figure 11.20 Dissolved oxygen profiles under the conditions of the summer thermal stratification for eutrophic and oligotrophic lakes.

As the seasons progress and winter approaches, the temperature of the epilimnion begins to drop and the marked stratification of summer begins to disappear. Sometime in the fall, the stratification will totally disappear, the temperature will become uniform with depth, and wind action can cause complete mixing of the lake. Oxygen from the epilimnion becomes mixed with the oxygen-poor hypolimnion and nutrients from the bottom are also get evenly distributed throughout the lake. This phenomenon is termed as the *fall overturn*. Similarly, in climates that are cold enough for the surface to drop below 4 °C, there will be a winter stratification, followed by a *spring overturn* when the surface warms up enough to allow complete mixing once again. Thus, temperate climate lakes have at least one, if not two cycles of stratification and turnover every year. In winter, demands for oxygen decrease as metabolic rates decrease, while at the same time the capacity of water to hold oxygen increases. Thus, even though winter stratification may occur, its effects tend not to be as severe as those in the summer. The end results of such seasonal overturns are seasonal blooms of phytoplankton, due to the replenishment of nutrients in autumn and increasing temperatures and light levels in spring.

Water quality management in lakes. Since phosphorus is the limiting nutrient, the primary objective of controlling the cultural eutrophication should be controlling the release of phosphorus into the lakes. Once the input is reduced, the phosphorus concentration will gradually fall as phosphorus is incorporated in the sediment or flushed from the lake. Other strategies for reversing or slowing the eutrophication process, such as precipitating phosphorus with additions of aluminum (alum), removing phosphorus-rich sediments by dredging or even sealing the bed or bottom of the lake using perforated membrane-like materials in order to prevent exchange of phosphorus between water and sediment, have been proposed. However, if the input of phosphorus is not also curtailed, the eutrophication process will continue.

The nuisances caused by excessive algal bloom in lakes and reservoirs may be alleviated temporarily by the application of copper sulfate. The copper sulfate kills the algae, but its dose must be carefully controlled to prevent killing of fish. Ferric Sulphate solutions have been successfully used to reduce phosphate and Chlorophyll-a concentrations (Mason, 1991). Underwater weed cutters mounted on boats, can be used to remove root of aquatic plants and dredges can be used to remove sediments, but these are not very practical measures if the water body is very large. These measures can help speed up the removal of phosphorus already in the lake system in association with other phosphorus reducing options. Of course, the need to speed the recovery process must be weighed against the potential damage from inundating shoreline areas with sludge and stirring up toxic compounds buried in the sediment further exacerbating the situation.

The most prominent sources of phosphorus are municipal and industrial wastewaters, seepage from septic tanks, and agricultural runoff that carries phosphorus fertilizers into the water. Tertiary treatment of sewage can effectively remove much of the phosphorus, as well as the nitrogen, from wastewater, but this is a very costly means of control. Another option would be to divert the wastewater effluents around the lake into some other body of water that are relatively less sensitive to nutrient inputs, such as a streams and rivers. But this measure would be of less effective if most of the nutrient loading is being contributed by dispersed (non-point) sources such as runoff from agricultural areas. In these cases, waste minimization can be applied to the control of phosphorus loading to lakes from agricultural fertilization by encouraging farmers to fertilize more often with smaller amounts and to take effective action to stop soil erosion. On a household level, avoiding the use of phosphate-based detergents or even in the dairying industry, using cleaning agents other than phosphoric acid can reduce the amount of phosphorus entering into surface waters.

Questions

1. Why is surface water pollution a cause for concern in developing countries of the world today?
2. What are point and non-point sources of pollution? Why are non-point sources of pollution difficult to control?
3. Describe the different categories of pollutants by composition.
4. What is cultural eutrophication? How does thermal stratification affect the DO levels in the lake?
5. Derive the Streeter-Phelps DO sag curve equation. What are the factors affecting DO levels in rivers?
6. As an engineer, what are your options in managing river and lake water quality?
7. In a standard 5-day BOD test:
    a. Why is the BOD bottle stoppered?
    b. Why is the test run in the dark (or in a black bottle)?
    c. Why is it usually necessary to dilute the sample?
    d. Why is it sometimes necessary to seed the sample?
    e. Why isn't ultimate BOD measured?
8. A BOD test is run using 100 mL of treated wastewater mixed with 200 mL of pure water. The initial DO of the mix is 9.0 mg/L. After 5 days, the DO is 4.0 mg/L. After a long period of time, the DO is 2.0 mg/L and it no longer seems to be dropping. Assuming that nitrification has been inhibited so that the only BOD being measured is carbonaceous:
    a. What is the 5-day BOD of the wastewater?
    b. Assuming no nitrification effects, estimate the ultimate carbonaceous BOD.
    c. What would be the remaining BOD after 5 days have elapsed?
    d. Estimate the reaction rate constant k.
9. For a solution containing 200 mg/L of glycine [$CH_2(NH_2)COOH$], whose oxidation can be represented as

    $$CH_2(NH_2)COOH + 3O_2 \rightarrow 4CO_2 + 2H_2O + 2NH_3$$

    $$NH_3 + 2O_2 \rightarrow NO_3\text{-} + H^+ + H_2O$$

    a. Find the theoretical CBOD.
    b. Find the ultimate NBOD.
    c. Find the total theoretical BOD.
10. The ultimate BOD of a river just below a sewage outfall is 50.0 mg/L and the DO is at the saturation value of 10.0 mg/L. The deoxygenation rate coefficient k_d is 0.30/day and the reaeration rate coefficient k_r is 0.90/day. The river is flowing at the speed of 48.0 miles per day. The only source of BOD on this river is this single outfall.
    c. Find the critical distance downstream at which DO is a minimum.
    d. Find the minimum DO.
    e. If a wastewater treatment plant is to be built, what fraction of the BOD would

have to be removed from the sewage to assure a minimum of 5.0 mg/L every every- where downstream?

11. A lake with surface area equal to 80 × 10^6 m^2 is fed by a stream having an average flow of 15.0 m^3/s and an average total phosphorus concentration of 0.010 mg/L. In addition, treated effluent from a wastewater treatment plant adds 0.20 m^3/s of flow having 5.0 mg/L total phosphorus. The phosphorus settling rate is estimated at 10 m/year. Estimate the average total phosphorus concentration. What rate of phosphorus removal at the wastewater treatment plant would be required to keep the concentration of phosphorus in the lake at an acceptable level of 0.010 mg/L?

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Chapter 12

Decentralized Wastewater Treatment

As observed in Chapter 3 of this book, most of the areas in and around Dhaka City do not have sewerage networks, for transporting wastewater into Pagla treatment plant. In such cases decentralized wastewater treatment (DWWT) system can be an attractive option for wastewater disposal and treatment. DWWT systems provide on-site treatment of wastewater, generated from individual domestic houses, residential areas, isolated communities and industrial or institutional facilities/clusters. DWWT systems can be managed individually, or be integrated with centralized sewage treatment systems.

Design and management of DWWT systems can be achieved at a fraction of cost, required for typical centralized treatment plants (Chapters 4-9). Table 12.1 summarizes some salient features of DWWT systems, over conventional centralized treatment technologies.

Table 12.1 **Comparison between centralized and DWWT systems.**

	Centralized systems	DWWT systems
Reliability	Require complex operation and maintenance schedules to ensure optimal performances.	Do not require intensive maintenance for better performances.
Environmental sustainability	May generate partially treated or untreated wastewater, that may not meet discharge standards. Also require higher energy supply.	Treated wastewater can be used locally, or can be safely disposed into local water channels. Energy requirement is low.
Financial sustainability	Substantial grants, government funding and subsidies are required for construction, operation and maintenance.	Require less capital cost when compared with centralized sewerage systems.
Affordability	Score low on affordability due to substantial cost of installation, sewerage network, operational and maintenance costs.	Affordable due to lower costs when compared with centralized systems. Require locally available materials, as a majority portion of such systems is based on natural technologies.

Source: IITs 2011.

The main advantages of DWWT systems are summarized below:

- DWWT systems can be constructed according to influent wastewater characteristics.
- It can treat wastewaters from different sources such as: residential structures, hospitals, schools, markets, hotels etc.
- Demand lower primary investment costs as no imports are needed.
- Require lower maintenance costs.
- Can provide efficient treatment of wastewater flow up to 1000 m^3/d.
- Tolerant towards inflow fluctuations.
- Reliable and long-lasting construction design.
- If properly managed, these systems can meet discharge criteria.
- Provide effective solution for ecologically sensitive areas.

12.1 Components of DWWT Systems

Wastewater treatment in a DWWT system is achieved via: (a) primary treatment (settlers or septic tanks); (b) secondary treatment (anaerobic baffled reactors); (c) tertiary treatment (subsurface vertical flow and/or horizontal flow wetland systems); and (d) additional tertiary treatment (polishing ponds or surface flow wetlands).

A brief description about the pollutant removal mechanisms in septic tanks, wetland systems and ponds have been described in the previous chapters of this book. Figure 12.1 illustrates an operational diagram of a typical DWWT train.

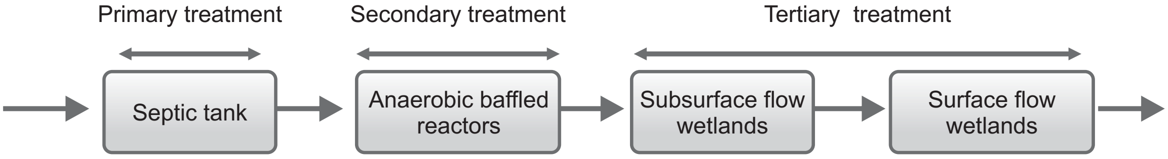


Figure 12.1 Typical DWWT configuration.

Box 12.1

Anaerobic baffled reactor for DWWT system.

Anaerobic baffled reactor (ABR) was developed by McCarty and co-workers at Stanford University, and consists of several compartments (i.e. ranging between 3-16). The vertical baffles of a typical ABR system direct the wastewater under and over the baffles, as it passes from the inlet to the outlet. Such forced direction enhances contact between the wastewater and microorganisms of the sludge materials, that settle on the bottom (of an ABR system). The over and underflow of the liquid also reduces bacteria washout, allowing the ABR to retain active biological mass.

In an ABR system pollutants are generally removed from wastewater via anaerobic digestion (Figure 9.9- Chapter 9). As such, ABR is efficient in terms of organic and solids removal; however, the effluent from an ABR may contain nitrogen, phosphorus and pathogenic microorganisms. To facilitate the removal of these parameters, subsurface flow and surface flow wetlands should follow an ABR system in a DWWT train (Figure 12.1). Figure 12.2 gives an operational diagram of a typical ABR unit.

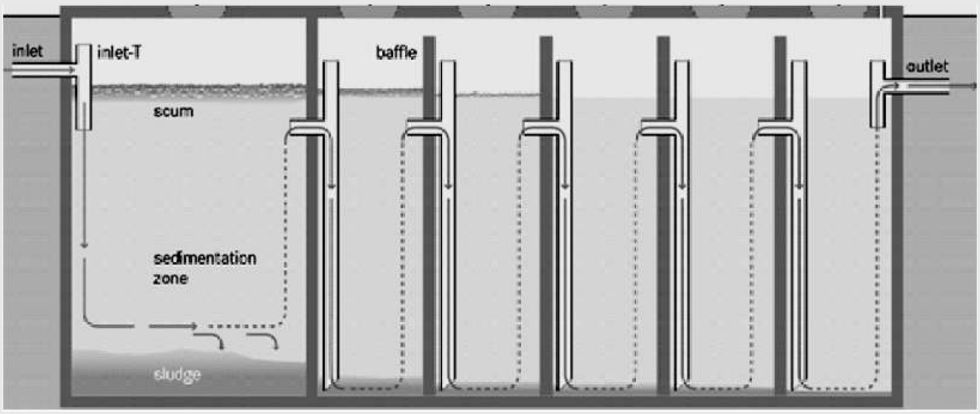


Figure 12.2 Anaerobic baffled reactor.

12.2 Planning and Designing of DWWT Systems

Appropriate planning and designing are critical prior to the construction of a DWWT system. The planning for employing a DWWT system should include (CSE, 2013):

- Identification of a site.
- Land availability of the proposed site.
- Topography of the land. Detailed examination of soil, slopes, hydrological parameters are critical.
- Location of groundwater.
- Source and volume of generated wastewater.
- Local climate conditions.
- Socio-economical environment of the local area and target population.
- Identification of reuse options (if required).
- Rigorous analyses of construction, operational and maintenance costs.

The main objective of a DWWT system is to remove pollutants from wastewater, so that it can either be disposed into the environment safely, or be reused. To meet such objectives an engineer must consider the following factors, prior to the design procedures (CSE, 2013).

- The volume of incoming wastewater. Daily and peak hourly incoming flow should be identified.
- Parameters of incoming wastewater, for example: BOD, COD, suspended solids, pH and temperature should be measured.
- Local discharge standards should be examined carefully.

12.3 DWWT Design Factors

Wastewater generation. Typical water consumption rate is considered to be 100L/person (P) /d. For domestic purposes, wastewater generation rate is around 80 % of the consumed water. Wastewater generation rate (i.e. flow rate) is calculated in terms of volume per day. Flow rate can be measured via: (a) employing a flow meter; (b) monitoring the rise of water level in a closed chamber for a specific time period; and (c) calculating the required time during initial filling of the first unit (of a treatment train) to overflow. It should be mentioned that design of a DWWT system is dependent directly on the incoming flow rate; hence such parameter must be measured accurately.

Sludge production rate. Sludge production in DWWT systems (i.e. in septic tanks and ABR modules) is related with organic material (BOD/COD) removal from wastewater. Such organic removal occurs through aerobic/ anaerobic process (see Chapters 6 and 9 of this book); aerobic decomposition produces more sludge than anaerobic degradation. For DWWT systems treating domestic sewage, sludge retention period (inside a septic tank) is considered to be one year to allow anaerobic degradation. Desludging of the accumulated sludge is necessary for efficient performances. If sludge removal frequency is 1 year then approximately 0.1 liter of sludge per person (0.1 L/P/d) may be considered as the generated sludge rate. If desludging intervals are greater than two years, sludge volume can be considered to be 0.08 L/P/d as sludge becomes compacted over time span.

Area requirement. The area required by the typical four units of a DWWT system (Figure 12.1) is dependent on wastewater volume. For one m^3 wastewater, the required area has been illustrated in Table 12.2. Please note that Table 12.2 indicates a series arrangement of a VF wetland (after an ABR) followed by a HF wetland (prior to a SF wetland). It should also be noted that integration of such hybrid wetland system (in a DWWT train) allows nitrification in the aerobic VF wetland, followed by denitrification in the latter anaerobic HF wetland. The last stage SF wetland can remove residual pollutants, that are not removed by the previous units.

Table 12.2 Area requirement of different modules of a DWWT train.

Component	Wastewater volume m^3	Minimum recommended area (m^2)
Septic tank	1.0	0.5
ABR		1.0
VF constructed wetland		3.75
HF constructed wetland		6.5
Polishing pond or SF wetland		1.2
		Total area12.95 m^2

Depending on wastewater characteristics and land availability, a single VF or a HF wetland can also be employed in a DWWT train, instead of a VF-HF hybrid system. In such cases, total area requirement employing a VF (Table 12.3) or a HF (Table 12.4) wetland will be lower when compared with Table 12.2 (employing combined VF-HF wetlands).

Table 12.3 Area requirement of a DWWT train with VF system as tertiary unit.

Component	Wastewater volume m^3	Minimum recommended area (m^2)
Septic tank	1.0	0.5
ABR		1.0
VF constructed wetland		3.75
Polishing pond or SF wetland		1.2
		Total area 6.45 m^2

Table 12.4 Area requirement of a DWWT train with VF system as tertiary unit.

Component	Wastewater volume m^3	Minimum recommended area (m^2)
Septic tank	1.0	0.5
ABR		1.0
HF constructed wetland		6.5
Polishing pond or SF wetland		1.2
		Total area 9.2 m^2

It should be mentioned that mathematical equations such as Kickuth equation or Monod kinetics combined with CSTR/ plug flow pattern (Chapter 10) may calculate higher area requirements for VF/ HF wetlands, to provide treatment of wastewater volume indicated in the Tables 12.2-12.4. As such, the proposed area values in Tables 12.2-12.4 are conservative, as these numbers have been estimated from practical experiences. It is recommended to provide greater area values (as indicated in the tables) for VF/ HF systems, if sufficient land is available.

Dimensions of the units. The septic tank should be rectangular in shape, with length and width ratio 3:1. All chambers should have equal depth; however, the depth of the first chamber can be deeper since most of the sludge accumulates in this zone. Detailed design of a septic tank has been provided in Chapter 3 of this book.

For an ABR system, the HRT should be designed as one day (24 hours) to ensure 70-90% organic removal (CSE, 2013). HRT can be calculated as the ratio of reactor volume and wastewater generation rate. The upflow velocity should be ≤2m/hr.

The media depth of VF and HF wetlands should range between 1.0-2.0 m, and 0.9- 1.5 m respectively. A clear space (30-40 cm) above the media should be provided to accommodate excess incoming flow (i.e. during rainy season). For VF and HF systems, greater depth and length should be provided respectively to increase the retention time of wastewater inside the system, enhancing removal performances. The bottom slope of VF and HF systems should be 1%. Steeper slope will promote quick movement of wastewater from inlet to outlet, resulting inadequate contact between plants, media, attached bacteria and wastewater, that may lead to inefficient treatment performances. Different types of media (Chapter 10) can be employed in wetland reactors. In DWWT wetland systems special media for example cupola slag (size 25-50 mm) can be employed, to enhance phosphorus removal. The chosen media should have a porosity ranging between 40-60%. Detail information on wetland media is available in Chapter 10 of this book. A list of plants that can be employed in the wetland systems of Bangladesh is also available in Chapter 10 of this book. For surface flow wetland a shallow water depth between 0.5-0.6 m is recommended to maintain aerobic conditions.

Other design factors. For septic tanks, the inlet pipe can be placed either: (a) below the lowest scum level; or (b) above water level to evacuate gas. The chambers should be designed to reduce turbulent flow, enhancing sedimentation of solids. The chambers should be water sealed. Manholes should be provided above each chamber for regular inspection and maintenance.

For ABR systems, the inlet pipes of every chamber should be placed at a certain height from the bottom (of the reactor). Such arrangement allows the mixing of the incoming wastewater with the stored sludge to foster anaerobic degradation, due to presence of microbes inside the sludge mass. The chambers should be in series; the number of chambers is dependent on peak hourly flow of wastewater. Last two chambers are often packed with media to allow solid filtration and biological degradation by attached growth process. Similar to septic tanks, the chambers of an ABR system must be water tight, with manholes above each chamber for maintenance purposes.

The inlet perforated pipe for a VF system should be placed parallel to the length of the bed, so that wastewater can be distributed evenly over the media. Large stones may be placed on the top of the media to allow even distribution of wastewater (inside the media), and to prevent clogging. The outlet valve is generally placed at the bottom of the bed; large stones are placed at the bottom to achieve homogenous wastewater distribution towards outlet. For HF systems, the inlet perforated pipe should be placed parallel to the width of the bed, above the inlet zone. Large stones should be placed in the inlet and outlet zones to prevent clogging. The outlet valve is placed at a certain distance (0.6-1.2 m) from the bottom of the reactor, to maintain constant water depth inside the media. Selected wetland site should not contaminate drinking water source. To achieve such objective, bottom and side walls of the wetland systems should be sealed to avoid groundwater contamination. Different materials for example bricks, cement-sand plasters or low density poly ethylene sheets (LDPE) with a thickness ≥ 0.5 mm can be utilized. Polyethylene sheets should have root resistance properties; in addition these sheets should be UV resistant (if exposed to the sun). After sealing, leakage tests should be performed by filling the bed with water and be left overnight. If water loss is less than 2 mm, sealing is considered to be satisfactory.

Box 12.2

Design of a DWWT system for a residential building of 200 population.

Step 1. Determine water consumption rate

Assuming typical water consumption rate to be 100L/P/d, total water consumption rate by 200 people can be calculated as: $200 \times 100 = 20000 L/d = 20 \text{m}^3/d$

Step 2. Determine wastewater generation rate

Wastewater generation is around 80% of the consumed water. As such, total wastewater generation from the residential building can be calculated as: $20000 L/d \times 0.8 = 16000\ L/d = 16 \text{m}^3/d$

Step 3. Area of the septic tank and ABR for generated wastewater

The area of the septic tank can be calculated as (Table 12.2): $16 \times 0.5 \text{m}^2 = 8 \text{m}^2$

From Table 12.2, the required area of the ABR can be calculated as: $16 \text{x} \times 1.0 \text{m}^2 = 16 \text{m}^2$

To achieve 24 hrs retention time, the volume of the ABR unit can be calculated as: $Q \times t$

$$= 16 m^3/d \times 1 d = 16 m^3$$

Step 4. Determine sludge generation rate

Approximately 0.1 L/P/d may be considered as the generated sludge rate, for desludging interval of one year. As such, sludge production volume can be calculated as: $200 \times 0.1 L/P/d = 20 L/d$

Sludge production in a year can be calculated as: $20 L/d \times 365 d = 7300 L = 7.3 m^3$

Step 5. Calculating the area of the wetlands for generated wastewater

From Table 12.2 the required area of VF, HF and SF wetlands can be calculated as:

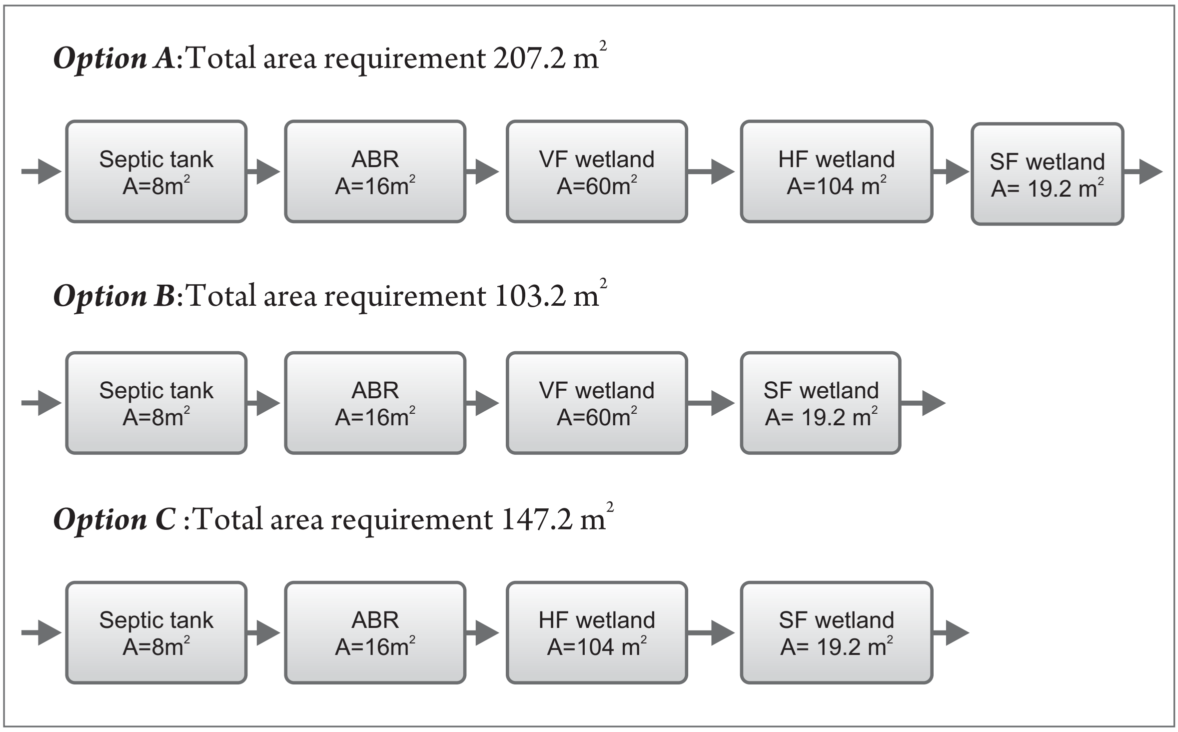
Area of the VF wetland: $16 \times 3.75 m^2 = 60 m^2$

Area of the HF wetland: $16 \times 6.5 m^2 = 104 m^2$

Area of the SF wetland: $16 \text{x} 1.2 m^2 = 19.2 m^2$

Step 6. Arrangement choices

The choice of the arrangements can be expanded between three options as illustrated below. Option A is the best choice for achieving better pollutant removal performances, due to sequential aerobic-anaerobic zones of VF and HF wetlands respectively (Chapter 10). However, if sufficient land is not available for choosing option A, the engineer may restrict his/ her choice between options B and C, depending on the influent wastewater characteristics.



12.4 Maintenance of DWWT Systems

The treatment performance of a DWWT system can be enhanced by regular maintenance of DWWT system modules. Such schedules can be divided into three categories: (a) general maintenance; (b) maintenance of settler and ABR; and (c) maintenance of constructed wetland systems.

(a) General maintenance: General maintenance schedules include overall maintenance of the whole system, as illustrated below.

- Flow monitoring should be done during peak hours, when maximum wastewater volumes are generated.
- For DWWT systems at hotels, canteens, hostels, and domestic premises grease trap is required prior to the entrance of wastewater into the settler unit. The grease trap should be cleaned bi-monthly, to prevent clogging. The pipes before and after the grease trap should also be cleaned regularly.
- Skimming process is required to remove oil and grease, that float on the surface.
- The inlet and outlet pipes of all modules should be inspected monthly to prevent overflow/backflow into the treatment units.

(b) Maintenance of settler and ABR: Maintenance schedules of settler and ABR units are summarized below.

- For settler desludging should be carried out yearly, or once in two years (depending on incoming wastewater quality). For ABR unit desludging should be done after 2/3 years.
- During desludging from settler and ABR units, some sludge should be left inside the system for the continuation of biological removal mechanisms.

- After desludging the removed sludge should be further treated prior to disposal.
- The filter materials of the ABR unit should be cleaned every 5 years to prevent clogging.
- Growth of plants around the settler and ABR units should be controlled to prevent the penetration of roots into pipes and chambers, which can damage structural configurations.

(c) Maintenance of the wetland systems: The maintenance schedules of the wetland systems are listed below.

- The inlet structure should be designed in such a way so that it can distribute incoming flow homogeneously throughout the treatment zone.
- A drainage valve should be constructed at the bottom of the VF/HF wetlands to allow drainage of wastewater (from the system) if required.
- For HF wetlands, macrophytes should be planted beneath water table.
- After planting the wetland systems should be water logged (with a mixture of wastewater and fresh water), until the planted macrophytes are established. Such mixture will also promote the growth of bacteria inside the systems.

Box 12.3

Baffled HF wetland in a DWWT train of WaterAid, Bangladesh.

The wetland research team at Ahsanullah University of Science and Technology previously conducted research on baffled wetland systems, to provide treatment of municipal wastewater (Box 10.3 - Chapter 10). Enhanced pollutant removal performances were observed in such systems (Saeed et al. 2014), due to flow direction by the baffles throughout the media. The output of this research (i.e. baffled wetland) has been implemented in a DWWT system train of WaterAid Bangladesh at Pachtola Colony, Khulna. The DWWT system has been designed to serve 285 people, with a design incoming flow rate of 28.5 m^3/d. The system train includes a settler as the first stage, followed by an ABR unit, a baffled HF wetland and a SF wetland as the polishing stage. The baffled wetland has been employed to utilize the effect of the total media (of HF system), and attached biofilms in a compact land area. The baffled HF wetland includes two baffle walls (inside the system- Figure 12.3), to direct the flow downwards (i.e. anaerobic zone) and upwards (i.e. aerobic zone) enhancing biodegradation.

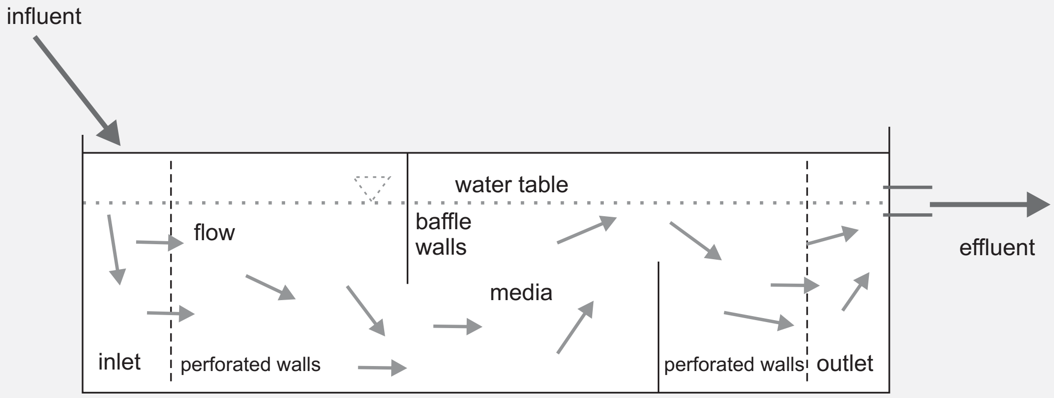


Figure 12.3 Baffled HF system in DWWT train of WaterAid, Bangladesh.

- Macrophytes should be harvested regularly to promote growth. Weak species should be removed from the system.
- If clogging occurs, the wetland systems should be kept in resting mode (i.e. without dosing of wastewater) for 21-30 days. After the specified time period, passing of wastewater (from the previous treatment unit) can be commenced. However, if the resting period doesnot encounter clogging, the media should be cleaned and refilled into the wetland rectors.

12.5 Reuse of Wastewater

Reuse of treated effluent (particularly treated domestic effluent) can reduce the pressure on fresh water sources (i.e. surface/ground water), as well as wastewater discharge problems into the environment. Wastewater reuse/recycling/reclamation can be an excellent option for DWWT systems, as these systems are generally employed in decentralized areas (i.e. urban/rural), where source of fresh water can be scarce. Wastewater reuse can be divided into two classes: indirect and direct reuse (WHO, 1973). When water is extracted from lakes, rivers and aquifers containing sewage it is termed as "indirect reuse" while planned reuse of treated wastewater for beneficial purposes is known as "direct reuse". According to Madungweet and Sakuringwa (2007), there are many benefits of wastewater reuse including:

Reduced costs: Recycled wastewater can reduce pressure on municipal supplies, thereby saving drinking water used for such purposes.

Low environmental risks: Soil and plants can uptake many contaminants (i.e. salts, nutrients and sediments) of the recycled wastewater. As such, groundwater pollution due to the application of recycled wastewater is minimal, if proper wastewater treatment technologies are employed.

Plant growth: Recycled wastewater contains nutrients which can supplement the growth of plants, and maintaining soil fertility. The presence of nutrients in recycled wastewater also reduces the necessity of artificial fertilizer addition.

Table 12.5 lists reuse categories and associated constraints. Subsequently, Table 12.6 illustrates the required guidelines for wastewater reuse.

Table 12.5 Wastewater reuse categories.

Reuse categories	Constraints
Agricultural irrigation crop irrigation, commercial nurseries	• Surface and ground water contamination if not managed properly. • Crops marketability, public acceptance. • Public health concerns due to presence of virus, bacteria and parasites.
Landscape irrigation parks, school yards, golf courses, cemeteries, residential	• Effect of pathogens on public health.
Industrial cooling water, boiler feed, process water	• Scaling, corrosion, biological growth, fouling. • Pathogen transmission in cooling water. • Cross connection of potable and reclaimed water.

Reuse categories	Constraints
Groundwater recharge groundwater replenishment	• Contamination of groundwater aquifer. • Organic chemicals in reclaimed water and their toxic effects. • Total dissolved solids, nitrates, pathogens in reclaimed water.
Recreational uses habitat wetlands, lakes, ponds, marsh enhancement, fisheries, snow making	• Health concerns due to presence of bacteria, viruses. • Eutrophication due to presence of nutrients. • Toxicity to aquatic life.
Aquaculture	• Trace organics and their toxic impact. • Public acceptance.
Other uses fire protection, air conditioning, toilet flush	• Public health concern by pathogens due to transmission by aerosols. • Scale corrosion and biological growth

Source: Tchobanoglous and Angelakis (1996).

Table 12.6 Wastewater reclamation guidelines.

Treatment level	Reuse types	Reclaimed wastewater quality	Reclaimed wastewater monitoring	Setback distances
Secondary[1] Filtration[2] Disinfection[3]	*Urban reuse* Landscape irrigation, vehicle washing, toilet flushing, fire protection, commercial air conditioners, and other uses with similar access or exposure to the water.	pH = 6–9 ≤10 mg/L biochemical oxygen demand (BOD) ≤ 2 turbidity units (NTU) No detectable fecal coliform/100 mL[4] 1 mg/L chlorine (Cl_2) residual (min.)	pH – weekly BOD – weekly Turbidity – continuous Coliform – daily Cl_2 residual – continuous	50 ft (15 m) to potable water supply wells
Secondary Disinfection	*Agricultural reuse for non-food crops* Pasture for milking animals, fodder, fiber and seed crops.	pH = 6–9 ≤ 30 mg/L BOD ≤ 30 mg/L total suspended solids (TSS) ≤ 200 fecal coliform/100 mL[5] 1 mg/L Cl_2 residual (min.)	pH – weekly BOD – weekly TSS – daily Coliform – daily Cl_2 residual –continuous	300 feet (90 m) to potable water supply wells
Site specific Secondary and Disinfection (min.) May also need	Indirect potable reuse Groundwater recharge by spreading into	Site specific. Meet drinking water standards after percolation through vadose zone.	pH – daily Turbidity – continuous Coliform – daily Cl_2 residual –	100 ft (30 m) to areas accessible to the public (if spray irrigation) site specific

Treatment level	Reuse types	Reclaimed wastewater quality	Reclaimed wastewater monitoring	Setback distances
filtration and/or advanced wastewater treatment	potable aquifers.		continuous Drinking water standards– quarterly Other – depends on constituent	

Source: US EPA, (1992).

Notes:

[1] Secondary treatment processes include activated sludge processes, trickling filters, rotating biological contactors, and many stabilization pond systems. Secondary treatment should produce effluent in which both the BOD and TSS do not exceed 30 mg/L.

[2] Filtration means passing the effluent through natural undisturbed soil or filter media such as sand and/or anthracite.

[3] Disinfection means the destruction, inactivation or removal of pathogenic microorganisms. It may be accomplished by chlorination, or other chemical disinfectants, UV radiation or other processes.

[4] The number of fecal coliform organisms should not exceed 14/100 mL in any sample.

[5] The number of fecal coliform organisms should not exceed 800/100 mL in any sample.

Box 12.4

DWWT system at Bankers colony, Bhuj, Gujarat, India (CSE, 2013).

A DWWT system was employed at Bankers Colony, Bhuj, Gujarat by Hunnarshala Foundation in collaboration with American India Foundation and Care today group in the year 2006. The system was built on an area of 300 m^2, with a design capacity of 300 kilo liters per day. The capital construction cost was Indian Rs 14-15 lakhs; current maintenance cost is Rs 1-1.5 lakhs/year. The main objective of this project was: (a) to provide wastewater treatment locally; and (b) to reuse treated effluent for horticultural reuse.

The DWWT system includes a two chambered settler, an ABR system with nine chambers (last two chambers are anaerobic filters), a planted filter and a polishing pond, arranged in series to form a DWWT train. The effluent from the polishing pond is being reused; the remaining treated wastewater is discharged into a storm water drain, that is linked with Hamirsar Lake.

Table 12.7 represents performance of the individual treatment unit of the DWWT train. As observed in Table 12.7pollutant removal performances across the DWWT system is higher, with overall BOD and COD reduction of 91% and 81% respectively. Subsequently, Figure 12.4 represents a pictorial view of DWWT system at Bankers colony.

Table 12.7 Treatment performances of different units of DWWT train at Bankers colony.

	Unit	Settler inlet	ABR inlet	Planted filter inlet	Polishing pond
pH	mg/L	7.2	7.1	7	7
Suspended solids		1056	142	60	19
BOD		182	52	40	15
COD		260	101	90	49

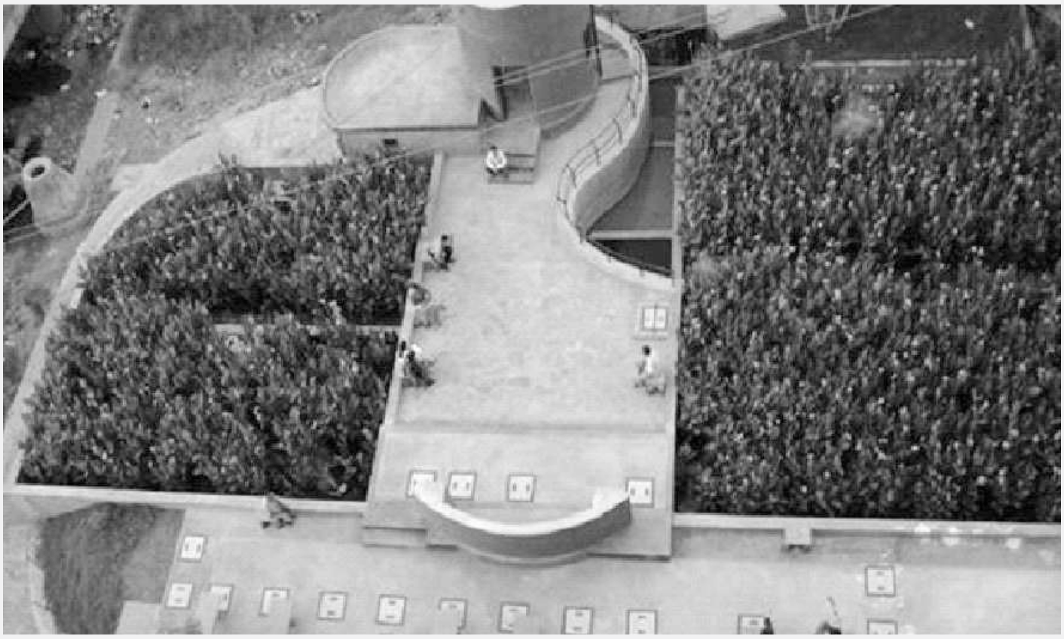


Figure:12.4 Pictorial view of DWWT system at Bankers colony.

Questions

1. What are the main advantages of DWWT systems when compared with conventional centralized treatment processes?
2. Why integration of VF-HF hybrid wetland systems into a DWWT train enhances removal performances?
3. What are the benefits of wastewater reuse from DWWT systems in a decentralized area?
4. Design a DWWT system for a residential building (of a decentralized area) with 300 people. Provide a block diagram of the proposed DWWT system.

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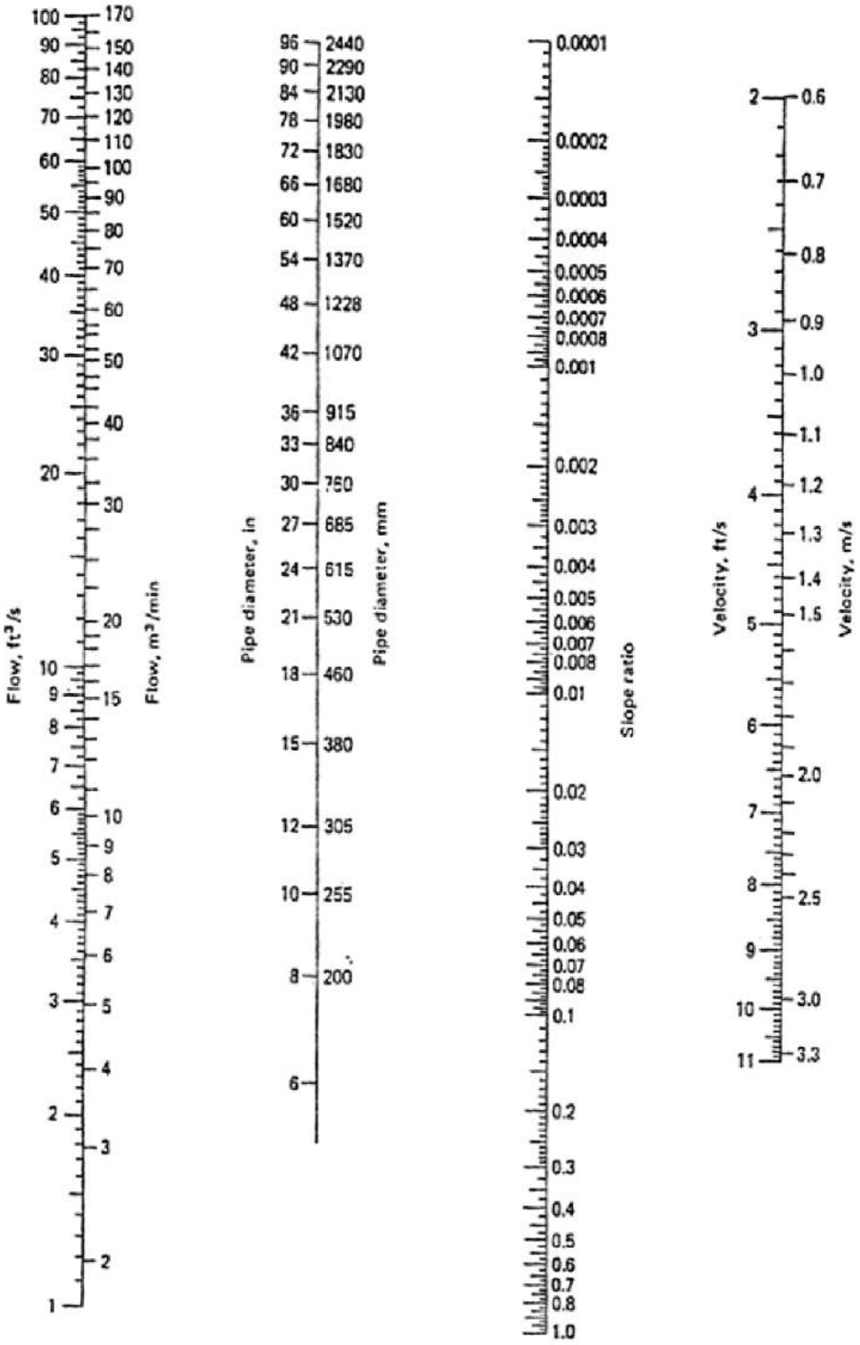
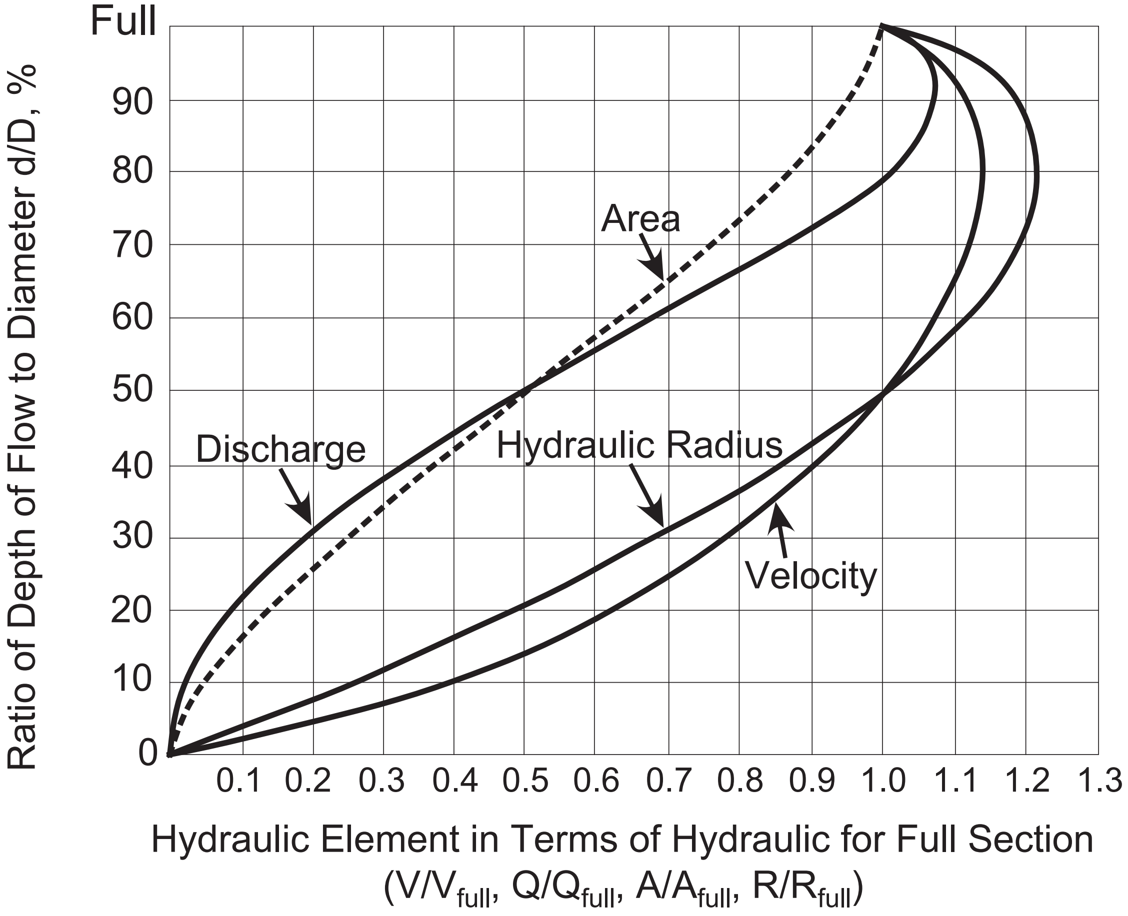
Appendix

Appendix A

Nomograph for the solution of Manning's equation, for full flowing circular pipes.

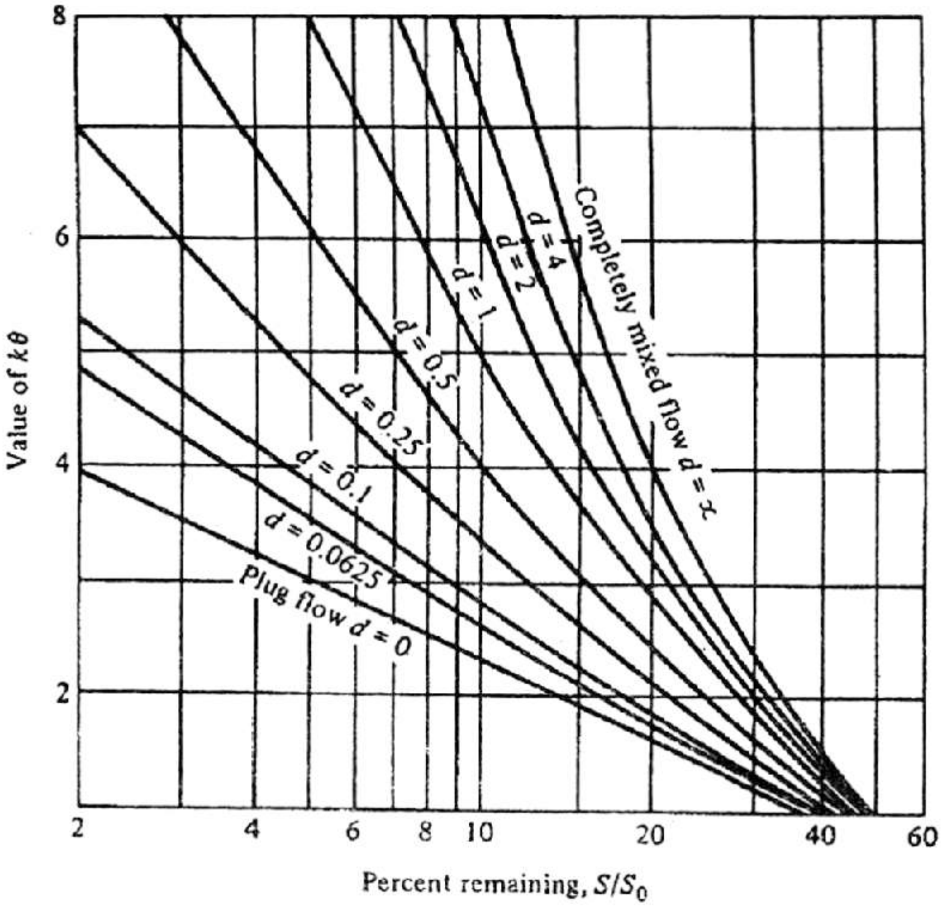


Appendix B
Ratios of sewer hydraulic elements.



Appendix C

Graphical plot of the Thirumurthi equation.



About the Book

This text book has been designed to cover the curricula of undergraduate and postgraduate environmental engineering and related courses, that are generally offered by the Civil and Environmental Engineering Departments of the technological universities in Bangladesh. The book is comprised of 12 chapters covering a wide range of topics related to sanitation, wastewater treatment, disposal and water pollution. Chapter 1 gives a brief description on the importance of sanitation, wastewater treatment and safe disposal, with an aim to provide the reader a scope to think about the necessity of sustainable environmental solutions for the country. Chapter 2 describes engineered sanitation technologies in context of Bangladesh. Chapters 3-7 describe municipal wastewater transportation into treatment plants, and different treatment stages for achieving safe disposal criteria. Chapter 8 describes different approaches to provide treatment of industrial wastewater. Chapter 9 includes a brief description on sludge treatment (for safe disposal), generated from wastewater treatment plants. Chapter 10 focuses on low-cost natural treatment technologies, such as ponds and constructed wetlands. Chapter 11 gives a brief description on the factors associated with water pollution. Finally Chapter 12 gives brief information and required guidelines for the construction, operation and maintenance of decentralized wastewater treatment systems.